AD-A277 852

(1)

Corps of Engineers

U.S. Army

ROCK TESTING HANDBOOK

(Test Standards 1993)

Prepared by:
Geotechnical Laboratory
Rock Mechanics Branch
U.S. Army Engineer Waterways Experiment Station
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199

REPLY TO
ATTENTION OF

CEWES-GS-R (1110-1-1150a)

MEMORANDUM FOR SEE DISTRIBUTION

SUBJECT: Transmittal of Rock Testing Handbook - Test Standards 1993, Part I - Laboratory Test Methods

1. The subject Part I - Laboratory Test Methods (encl 1) is a compilation of standards and recommended rock testing methods and has been prepared for use in laboratory offices of the Corps of Engineers. Revision of Part I - Laboratory Test Methods was authorized and funded by the Office, Chief of Engineers, FY 93.

2. The subject Part I - Laboratory Test Methods, supersedes the previous Part I of the "Rock Testing Handbook (Standard and Recommended Methods) March 1990". The current Part I includes: two new methods RTH-107a and RTH 101a-93; small modifications to RTH-102, 104, 106, 108, 109, and 114; replacement of RTH-101, 107, 110, 112, 113, 115, and 205 with appropriate American Society for Testing and Materials standards; and no changes to RTH-105, 111, 201, 203, and 207.

3. Correspondence concerning the subject Part I - Laboratory Test Methods or any part of the Rock Testing Handbook should be addressed to Director, U.S. Army Engineer Waterways Experiment Station, ATTN: CEWES-GS-R, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

for

Encl

ROBERT W. WHALIN, PhD, PE
Director

DISTRIBUTION
For all Commanders, Divisions and Districts

94 4 4 058

PREFACE

This handbook is a compilation of standard and recommended rock testing methods and has been prepared for use in both the laboratory and the field.

Preparation of the handbook was authorized and funded by the Office, Chief of Engineers, U.S. Army. The cooperation of the American Society for Testing and Materials, the International Society for Rock Mechanics, and the U.S. Bureau of Reclamation in permitting the use of a number of their standards is appreciated.

Suggestions for revisions, corrections, and additions are welcomed. Correspondence concerning such matters should be addressed either to the Commander, U.S. Army Engineer Waterways Experiment Station (ATTN: CEWES-GS-R), 3909 Halls Ferry Road, Vicksburg, MS 39180-6199; or to the Office, Chief of Engineers, U.S. Army (ATTN: Engineering Division, Civil Works), Washington, DC 20314-1000.

In order that this handbook may be of greatest service, it is intended that it be kept up to date by the issuance of supplementary items and the revision of existing items whenever necessary. To facilitate such revisions, the handbook has been issued in loose-leaf form.

The handbook has been prepared in the Geotechnical Laboratory (GL) and Structures Laboratory (SL) of the U.S. Army Engineer Waterways Experiment Station (WES). It was compiled by Ms. M. Eileen Glynn and Mr. Willie E. McDonald under the direct supervision of Messrs. Jerry S. Huie, Chief, Rock Mechanics Branch, GL; and Kenneth L. Saucier, Chief, Concrete Technology Division, SL; and the general guidance of Dr. Don C. Banks, Chief, Soil and Rock Mechanics Division, GL. Dr. W. F. Marcuson III was Chief of GL; Mr. Bryant Mather was Chief of SL. Commander of WES during publication of this handbook was COL Bruce K. Howard, EN. Director was Dr. Robert W. Whalin.

CONTENTS
June 1991

CONTENTS
June 1991

CONTENTS
June 1991

CONTENTS
June 1991

PART I. LABORATORY TEST METHODS

A. Characterization Methods

Designation: D 653 – 90

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Terminology Relating to Soil, Rock, and Contained Fluids[1]

These definitions were prepared jointly by the American Society of Civil Engineers and the American Society for Testing and Materials.

INTRODUCTION

A number of the definitions include symbols and indicate the units of measurement. The symbols appear in italics immediately after the name of the term, followed by the unit in parentheses. No significance should be placed on the order in which the symbols are presented where two or more are given for an individual term. The applicable units are indicated by capital letters, as follows:

F—Force, such as pound-force, ton-force, newton
L—Length, such as inch, foot, centimetre
T—Time, such as second, minute
D—Dimensionless

Positive exponents designate multiples in the numerator. Negative exponents designate multiples in the denominator. Degrees of angle are indicated as "degrees."

Expressing the units either in SI or the inch-pound system has been purposely omitted in order to leave the choice of the system and specific unit to the engineer and the particular application, for example:

FL^{-2}—may be expressed in pounds-force per square inch, kilopascals, tons per square foot, etc.
LT^{-1}—may be expressed in feet per minute, centimetres per second, etc.

Where synonymous terms are cross-referenced, the definition is usually included with the earlier term alphabetically. Where this is not the case, the later term is the more significant.

Definitions marked with (ISRM) are taken directly from the publication in Ref 42 and are included for the convenience of the user.

For a list of ISRM symbols relating to soil and rock mechanics, refer to Appendix X1.

A list of references used in the preparation of these definitions appears at the end.

AASHTO compaction—see **compaction test**.

"A" Horizon—see **horizon**.

abrasion—a rubbing and wearing away. (ISRM)

abrasion—the mechanical wearing, grinding, scraping or rubbing away (or down) of rock surfaces by friction or impact, or both.

abrasive—any rock, mineral, or other substance that, owing to its superior hardness, toughness, consistency, or other properties, is suitable for grinding, cutting, polishing, scouring, or similar use.

abrasiveness—the property of a material to remove matter when scratching and grinding another material. (ISRM)

absorbed water—water held mechanically in a soil or rock mass and having physical properties not substantially different from ordinary water at the same temperature and pressure.

absorption—the assimilation of fluids into interstices.

absorption loss—that part of transmitted energy (mechanical) lost due to dissipation or conversion into other forms (heat, etc.).

accelerator—a material that increases the rate at which chemical reactions would otherwise occur.

activator—a material that causes a catalyst to begin its function.

active earth pressure—see **earth pressure**.

active state of plastic equilibrium—see **plastic equilibrium**.

additive—any material other than the basic components of a grout system.

adhesion—shearing resistance between soil and another material under zero externally applied pressure.

	Symbol	Unit
Unit Adhesion	c_a	FL^{-2}
Total Adhesion	C_a	F or FL^{-1}

adhesion—shearing resistance between two unlike materials under zero externally applied pressure.

admixture—a material other than water, aggregates, or cementitious material, used as a grout ingredient for cement-based grouts.

adsorbed water—water in a soil or rock mass attracted to the particle surfaces by physiochemical forces, having properties that may differ from those of pore water at the same temperature and pressure due to altered molecular ar-

[1] This terminology is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.93 on Terminology for Soil, Rock, and Contained Fluids.

Current edition approved Oct. 27, 1989, and Feb. 1, 1990. Published March 1990. Originally published as D 653 – 42 T. Last previous edition D 653 – 89.

This extensive list of definitions represents the joint efforts of Subcommittee D18.93 on Terminology for Soil, Rock, and Contained Fluids of ASTM Committee D-18 on Soil and Rock, and the Committee on Definitions and Standards of the Geotechnical Engineering Division of the American Society of Civil Engineers. These two groups function together as the Joint ASCE/ASTM Committee on Nomenclature in Soil and Rock Mechanics. This list incorporates some terms from ASTM Definitions D 1707, Terms Relating to Soil Dynamics, which were discontinued in 1967.

rangement; adsorbed water does not include water that is chemically combined within the clay minerals.

adsorption—the attachment of water molecules or ions to the surfaces of soil particles.

advancing slope grouting—a method of grouting by which the front of a mass of grout is caused to move horizontally by use of a suitable grout injection sequence.

aeolian deposits—wind-deposited material such as dune sands and loess deposits.

aggregate—*as a grouting material*, relatively inert granular mineral material, such as sand, gravel, slag, crushed stone, etc. "Fine aggregate" is material that will pass a No. 4 (6.4-mm) screen,

"Coarse aggregate" is material that will not pass a No. 4 (6.4-mm) screen. Aggregate is mixed with a cementing agent (such as Portland cement and water) to form a grout material.

agitator tank—a tank, usually vertical and with open top, with rotation paddles used to prevent segregation of grout after mixing.

air-space ratio, G_a (D)—ratio of: (*1*) volume of water that can be drained from a saturated soil or rock under the action of force of gravity, to (*2*) total volume of voids.

air-void ratio, G_v (D)—the ratio of: (*1*) the volume of air space, to (*2*) the total volume of voids in a soil or rock mass.

alkali aggregate reaction—a chemical reaction between Na_2O and K_2O in the cement and certain silicate minerals in the cement and certain silicate minerals in the aggregate, which causes expansion resulting in weakening and cracking of Portland cement grout. See **reactive aggregate**.

allowable bearing value (allowable soil pressure), q_a, p_a (FL^{-2})—the maximum pressure that can be permitted on foundation soil, giving consideration to all pertinent factors, with adequate safety against rupture of the soil mass or movement of the foundation of such magnitude that the structure is impaired.

allowable pile bearing load, Q_a, P_a (F)—the maximum load that can be permitted on a pile with adequate safety against movement of such magnitude that the structure is endangered.

alluvium—soil, the constituents of which have been transported in suspension by flowing water and subsequently deposited by sedimentation.

amplification factor—ratio of dynamic to static displacement.

amorphous peat—see **sapric peat**.

angle of external friction (angle of wall friction), δ (degrees)—angle between the abscissa and the tangent of the curve representing the relationship of shearing resistance to normal stress acting between soil and surface of another material.

angle of friction (angle of friction between solid bodies), ϕs (degrees)—angle whose tangent is the ratio between the maximum value of shear stress that resists slippage between two solid bodies at rest with respect to each other, and the normal stress across the contact surfaces.

angle of internal friction (angle of shear resistance), ϕ (degrees)—angle between the axis of normal stress and the tangent to the Mohr envelope at a point representing a given failure-stress condition for solid material.

angle of obliquity, α, β, ϕ, Ψ (degrees)—the angle between the direction of the resultant stress or force acting on a given plane and the normal to that plane.

angle of repose, α (degrees)—angle between the horizontal and the maximum slope that a soil assumes through natural processes. For dry granular soils the effect of the height of slope is negligible; for cohesive soils the effect of height of slope is so great that the angle of repose is meaningless.

angle of shear resistance—see **angle of internal friction**.

angle of wall friction—see **angle of external friction**.

angular aggregate—aggregate, the particles of which possess well-defined edges formed at the intersection of roughly planar faces.

anisotropic mass—a mass having different properties in different directions at any given point.

anisotropy—having different properties in different directions. (ISRM)

apparent cohesion—see **cohesion**.

aquiclude—a relatively impervious formation capable of absorbing water slowly but will not transmit it fast enough to furnish an appreciable supply for a well or spring.

aquifer—a water-bearing formation that provides a ground water reservoir.

aquitard—a confining bed that retards but does not prevent the flow of water to or from an adjacent aquifer; a leaky confining bed.

arching—the transfer of stress from a yielding part of a soil or rock mass to adjoining less-yielding or restrained parts of the mass.

area grouting—grouting a shallow zone in a particular area utilizing holes arranged in a pattern or grid.

Discussion—This type of grouting is sometimes referred to as blanket or consolidation grouting.

area of influence of a well, α (L^2)—area surrounding a well within which the piezometric surface has been lowered when pumping has produced the maximum steady rate of flow.

area ratio of a sampling spoon, sampler, or sampling tube, A_r (D)—the area ratio is an indication of the volume of soil displaced by the sampling spoon (tube), calculated as follows:

$$A_r = [(D_e^2 - D_i^2/D_i^2] \times 100$$

where:

D_e = maximum external diameter of the sampling spoon, and

D_i = minimum internal diameter of the sampling spoon at the cutting edge.

armor—the artificial surfacing of bed, banks, shore, or embankment to resist erosion or scour.

armor stone—(generally one ton to three tons in weight) stone resulting from blasting, cutting, or by other methods to obtain rock heavy enough to require handling two individual pieces by mechanical means.

ash content—the percentage by dry weight of material remaining after an oven dry organic soil or peat is burned by a prescribed method.

attenuation—reduction of amplitude with time or distance.

"B" horizon—see **horizon**.

backpack grouting—the filling with grout of the annular space between a permanent tunnel lining and the surrounding formation.

Discussion—Same as crown grouting and backfill grouting.

back-packing—any material (usually granular) that is used to fill the empty space between the lagging and the rock surface. (ISRM)

baffle—a pier, weir, sill, fence, wall, or mound built on the bed of a stream to parry, deflect, check, or regulate the flow or to float on the surface to dampen the wave action.

base—*in grouting*, main component in a grout system.

base course (base)—a layer of specified or selected material of planned thickness constructed on the subgrade or subbase for the purpose of serving one or more functions such as distributing load, providing drainage, minimizing frost action, etc.

base exchange—the physicochemical process whereby one species of ions adsorbed on soil particles is replaced by another species.

batch—*in grouting*, quantity of grout mixed at one time.

batch method—*in grouting*, a quantity of grout materials are mixed or catalyzed at one time prior to injection.

batch mixer—*in grouting*, a machine that mixes batches of grout, in contrast to a continuous mixer.

bearing capacity—see **ultimate bearing capacity.**

bearing capacity (of a pile), Q_p, P_p (F)—the load per pile required to produce a condition of failure.

bedding—applies to rocks resulting from consolidation of sediments and exhibiting surfaces of separation (bedding planes) between layers of the same or different materials, that is, shale, siltstone, sandstone, limestone, etc. (ISRM)

bedding—collective term signifying the existence of layers of beds. Planes or other surfaces dividing sedimentary rocks of the same or different lithology.

bedrock—the more or less continuous body of rock which underlies the overburden soils. (ISRM)

bedrock (ledge)—rock of relatively great thickness and extent in its native location.

bench—(*1*) the unexcavated rock having a nearly horizontal surface which remains after a top heading has been excavated, or (*2*) step in a slope; formed by a horizontal surface and a surface inclined at a steeper angle than that of the entire slope. (ISRM)

bending—process of deformation normal to the axis of an elongated structural member when a moment is applied normal to its long axis. (ISRM)

bentonitic clay—a clay with a high content of the mineral montmorillonite, usually characterized by high swelling on wetting.

berm—a shelf that breaks the continuity of a slope.

biaxial compression—compression caused by the application of normal stresses in two perpendicular directions. (ISRM)

biaxial state of stress—state of stress in which one of the three principal stresses is zero. (ISRM)

binder (soil binder)—portion of soil passing No. 40 (425-μm) U.S. standard sieve,

binder—anything that causes cohesion in loosely assembled substances, such as clay or cement.

bit—any device that may be attached to or is an integral part of a drill string and is used as a cutting tool to bore into or penetrate rock or other materials.

blaine fineness—the fineness of powdered materials, such as cement and pozzolans, expressed as surface area usually in square centimetres per gram.

blanket grouting—a method in which relatively closely spaced shallow holes are drilled and grouted on a grid pattern over an area, for the purpose of making the upper portions of the bedrock stronger and less pervious.

blastibility—index value of the resistance of a rock formation to blasting. (ISRM)

blasting cap (detonator, initiator)—a small tube containing a flashing mixture for firing explosives. (ISRM)

bleeding—*in grouting*, the autogeneous flow of mixing water within, or its emergence from, newly placed grout caused by the settlement of the solid materials within the mass.

bleeding rate—*in grouting*, the rate at which water is released from grout by bleeding.

blocking—wood blocks placed between the excavated surface of a tunnel or shaft and the main bracing system. (ISRM)

body force—a force such as gravity whose effect is distributed throughout a material body by direct action on each elementary part of the body independent of the others. (ISRM)

bog—a peat covered area with a high water table and a surface dominated by a carpet of mosses, chiefly sphagnum. It is generally nutrient poor and acidic. It may be treed or treeless.

bond strength—*in grouting*, resistance to separation of set grout from other materials with which it is in contact; a collective expression for all forces such as adhesion, friction, and longitudinal shear.

bottom charge—concentrated explosive charge at the bottom of a blast hole. (ISRM)

boulder clay—a geological term used to designate glacial drift that has not been subjected to the sorting action of water and therefore contains particles from boulders to clay sizes.

boulders—a rock fragment, usually rounded by weathering or abrasion, with an average dimension of 12 in. (305 mm) or more.

breakwater stone—(generally three tons to twenty tons in weight) stone resulting from blasting, cutting, or other means to obtain rock heavy enough to require handling individual pieces by mechanical means.

buckling—a bulge, bend, bow, kink, or wavy condition produced in sheets, plates, columns, or beams by compressive stresses.

bulb of pressure—see **pressure bulb**.

bulkhead—a steep or vertical structure supporting natural or artificial embankment.

bulking—the increase in volume of a material due to manipulation. Rock bulks upon being excavated; damp sand bulks if loosely deposited, as by dumping, because the apparent cohesion prevents movement of the soil particles to form a reduced volume.

buoyant unit weight (submerged unit weight)—see **unit weight.**

burden—*in an explosive blasting*, the distance between the charge and the free face of the material to be blasted.

burden—distance between charge and free surface in direction of throw. (ISRM)

"C" Horizon—see **horizon.**

California bearing ratio, *CBR* (D)—the ratio of: (*1*) the force per unit area required to penetrate a soil mass with a 3-in.^{2} (19-cm)^2 circular piston (approximately 2-in. (51-mm) diameter) at the rate of 0.05 in. (1.3 mm)/min, to (*2*) that required for corresponding penetration of a standard material. The ratio is usually determined at 0.1-in. (2.5-mm) penetration, although other penetrations are sometimes used. Original California procedures required determination of the ratio at 0.1-in. intervals to 0.5 in. (12.7 mm). Corps of Engineers' procedures require determination of the ratio at 0.1 in. and 0.2 in. (5.1 mm). Where the ratio at 0.2 in. is consistently higher than at 0.1 in., the ratio at 0.2 in. is used.

camouflet—the underground cavity created by a fully contained explosive. (ISRM)

capillary action (capillarity)—the rise or movement of water in the interstices of a soil or rock due to capillary forces.

capillary flow—see **capillary action.**

capillary fringe zone—the zone above the free water elevation in which water is held by capillary action.

capillary head, h (L)—the potential, expressed in head of water, that causes the water to flow by capillary action.

capillary migration—see **capillary action.**

capillary rise (height of capillary rise), h_c (L)—the height above a free water elevation to which water will rise by capillary action.

capillary water—water subject to the influence of capillary action.

catalyst—a material that causes chemical reactions to begin.

catalyst system—those materials that, in combination, cause chemical reactions to begin; catalyst systems normally consist of an initiator (catalyst) and an activator.

cation—an ion that moves, or would move toward a cathode; thus nearly always synonymous with positive ion.

cation exchange—see **base exchange.**

cavity—a natural underground opening that may be small or large.

cavity—underground opening created by a fully contained explosive. (ISRM)

cement factor—quantity of cement contained in a unit volume of concrete or grout, expressed as weight, or volume (specify which).

cement grout—a grout in which the primary cementing agent is Portland cement.

cementitious factor—quantity of cement and other cementitious materials contained in a unit volume of concrete or grout, expressed as weight or volume (specify which).

centrifuge moisture equivalent—see **moisture equivalent.**

chamber—a large room excavated underground, for example, for a powerhouse, pump station, or for storage. (ISRM)

chamber blasting (coyotehole blasting)—a method of quarry blasting in which large explosive charges are confined in small tunnel chambers inside the quarry face. (ISRM)

chemical grout—any grouting material characterized by being a true solution; no particles in suspension. See also **particulate grout.**

chemical grout system—any mixture of materials used for grouting purposes in which all elements of the system are true solutions (no particles in suspension).

chip—crushed angular rock fragment of a size smaller than a few centimetres. (ISRM)

chisel—the steel cutting tool used in percussion drilling. (ISRM)

circuit grouting—a grouting method by which grout is circulated through a pipe extending to the bottom of the hole and back up the hole via the annular space outside the pipe. Then the excess grout is diverted back over a screen to the agitator tank by means of a packing gland at the top of the hole. The method is used where holes tend to cave and sloughing material might otherwise clog openings to be grouted.

clay (clay soil)—fine-grained soil or the fine-grained portion of soil that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength when air-dry. The term has been used to designate the percentage finer than 0.002 mm (0.005 mm in some cases), but it is strongly recommended that this usage be discontinued, since there is ample evidence from an engineering standpoint that the properties described in the above definition are many times more important.

clay size—that portion of the soil finer than 0.002 mm (0.005 mm in some cases) (see also **clay**).

clay soil—see **clay.**

cleavage—*in crystallography*, the splitting, or tendency to split, along planes determined by the crystal structure. *In petrology*, a tendency to cleave or split along definite, parallel, closely spaced planes. It is a secondary structure, commonly confined to bedded rocks.

cleavage—the tendency to cleave or split along definite parallel planes, which may be highly inclined to the bedding. It is a secondary structure and is ordinarily accompanied by at least some recrystallization of the rock. (ISRM)

cleavage planes—the parallel surfaces along which a rock or mineral cleaves or separates; the planes of least cohesion, usually parallel to a certain face of the mineral or crystal.

cleft water—water that exists in or circulates along the geological discontinuities in a rock mass.

closure—the opening is reduced in dimension to the extent that it cannot be used for its intended purpose. (ISRM)

closure—*in grouting*, closure refers to achieving the desired reduction in grout take by splitting the hole spacing. If closure is being achieved, there will be a progressive decrease in grout take as primary, secondary, tertiary, and quanternary holes are grouted.

cobble (cobblestone)—a rock fragment, usually rounded or semirounded, with an average dimension between 3 and 12 in. (75 and 305 mm).

coefficient of absolute viscosity—see **coefficient of viscosity.**

coefficient of active earth pressure—see **coefficient of earth pressure.**

coefficient of compressibility (coefficient of compression), a_v (L^2F^{-1})—the secant slope, for a given pressure increment,

of the pressure-void ratio curve. Where a stress-strain curve is used, the slope of this curve is equal to $a_v/(1 + e)$.

coefficient of consolidation, c_v (L^2T^{-1})—a coefficient utilized in the theory of consolidation, containing the physical constants of a soil affecting its rate of volume change.

$$c_v = k(1 + e)/a_v\gamma_w$$

where:

k = coefficient of permeability, LT^{-1},
e = void ratio, D,
a_v = coefficient of compressibility, L^2F^{-1}, and
γ_w = unit weight of water, FL^{-3}.

Discussion—In the literature published prior to 1935, the coefficient of consolidation, usually designated c, was defined by the equation:

$$c = k/a_v\gamma_w(1 + e)$$

This original definition of the coefficient of consolidation may be found in some more recent papers and care should be taken to avoid confusion.

coefficient of earth pressure, K (D)—the principal stress ratio at a point in a soil mass.

coefficient of earth pressure, active, K_A (D)—the minimum ratio of: (*1*) the minor principal stress, to (*2*) the major principal stress. This is applicable where the soil has yielded sufficiently to develop a lower limiting value of the minor principal stress.

coefficient of earth pressure, at rest, K_O (D)—the ratio of: (*1*) the minor principal stress, to (*2*) the major principal stress. This is applicable where the soil mass is in its natural state without having been permitted to yield or without having been compressed.

coefficient of earth pressure, passive, K_P (D)—the maximum ratio of: (*1*) the major principal stress, to (*2*) the minor principal stress. This is applicable where the soil has been compressed sufficiently to develop an upper limiting value of the major principal stress.

coefficient of friction (coefficient of friction between solid bodies), f (D)—the ratio between the maximum value of shear stress that resists slippage between two solid bodies with respect to each other, and the normal stress across the contact surfaces. The tangent of the angle of friction is ϕs.

coefficient of friction, f—a constant proportionality factor, μ, relating normal stress and the corresponding critical shear stress at which sliding starts between two surfaces: $T = \mu \cdot \sigma$. (ISRM)

coefficient of internal friction, μ (D)—the tangent of the angle of internal friction (angle of shear resistance) (see internal friction).

coefficient of permeability (permeability), k (LT^{-1})—the rate of discharge of water under laminar flow conditions through a unit cross-sectional area of a porous medium under a unit hydraulic gradient and standard temperature conditions (usually 20°C).

coefficient of shear resistance—see coefficient of internal friction, μ (D).

coefficient of subgrade reaction (modulus of subgrade reaction), k, k_s (FL^{-3})—ratio of: (*1*) load per unit area of horizontal surface of a mass of soil, to (*2*) corresponding settlement of the surface. It is determined as the slope of the secant, drawn between the point corresponding to zero settlement and the point of 0.05-in. (1.3-mm) settlement, of a load-settlement curve obtained from a plate load test on a soil using a 30-in. (762-mm) or greater diameter loading plate. It is used in the design of concrete pavements by the Westergaard method.

coefficient of transmissibility—the rate of flow of water in gallons per day through a vertical strip of the aquifer 1 ft (0.3 m) wide, under a unit hydraulic gradient.

coefficient of uniformity, C_u (D)—the ratio D_{60}/D_{10}, where D_{60} is the particle diameter corresponding to 60 % finer on the cumulative particle-size distribution curve, and D_{10} is the particle diameter corresponding to 10 % finer on the cumulative particle-size distribution curve.

coefficient of viscosity (coefficient of absolute viscosity), η (FTL^{-2})—the shearing force per unit area required to maintain a unit difference in velocity between two parallel layers of a fluid a unit distance apart.

coefficient of volume compressibility (modulus of volume change), m_v (L^2F^{-1})—the compression of a soil layer per unit of original thickness due to a given unit increase in pressure. It is numerically equal to the coefficient of compressibility divided by one plus the original void ratio, or $a_v/(1 + e)$.

cohesion—shear resistance at zero normal stress (an equivalent term in rock mechanics is intrinsic shear strength). (ISRM)

cohesion, c (FL^{-2})—the portion of the shear strength of a soil indicated by the term c, in Coulomb's equation, $s = c + p \tan \phi$. See **intrinsic shear strength**.

apparent cohesion—cohesion in granular soils due to capillary forces.

cohesionless soil—a soil that when unconfined has little or no strength when air-dried and that has little or no cohesion when submerged.

cohesive soil—a soil that when unconfined has considerable strength when air-dried and that has significant cohesion when submerged.

collar—*in grouting*, the surface opening of a borehole.

colloidal grout—*in grouting*, a grout in which the dispersed solid particles remain in suspension (colloids).

colloidal mixer—*in grouting*, a mixer designed to produce colloidal grout.

colloidal particles—particles that are so small that the surface activity has an appreciable influence on the properties of the aggregate.

communication—*in grouting*, subsurface movement of grout from an injection hole to another hole or opening.

compaction—the densification of a soil by means of mechanical manipulation.

compaction curve (Proctor curve) (moisture-density curve)—the curve showing the relationship between the dry unit weight (density) and the water content of a soil for a given compactive effort.

compaction test (moisture-density test)—a laboratory compacting procedure whereby a soil at a known water content is placed in a specified manner into a mold of given dimensions, subjected to a compactive effort of controlled magnitude, and the resulting unit weight determined. The procedure is repeated for various water contents sufficient to establish a relation between water content and unit weight.

compressibility—property of a soil or rock pertaining to its susceptibility to decrease in volume when subjected to load.

compression curve—see **pressure-void ratio curve.**

compression index, C_c (D)—the slope of the linear portion of the pressure-void ratio curve on a semi-log plot.

compression wave (irrotational)—wave in which element of medium changes volume without rotation.

compressive strength (unconfined or uniaxial compressive strength), p_c, q_u, C_o (FL^{-2})—the load per unit area at which an unconfined cylindrical specimen of soil or rock will fail in a simple compression test. Commonly the failure load is the maximum that the specimen can withstand in the test.

compressive stress—normal stress tending to shorten the body in the direction in which it acts. (ISRM)

concentration factor, n (D)—a parameter used in modifying the Boussinesq equations to describe various distributions of vertical stress.

conjugate joints (faults)—two sets of joints (faults) that formed under the same stress conditions (usually shear pairs). (ISRM)

consistency—the relative ease with which a soil can be deformed.

consistency—*in grouting,* the relative mobility or ability of freshly mixed mortar or grout to flow; the usual measurements are slump for stiff mixtures and flow for more fluid grouts.

consistency index—see **relative consistency.**

consolidated-drained test (slow test)—a soil test in which essentially complete consolidation under the confining pressure is followed by additional axial (or shearing) stress applied in such a manner that even a fully saturated soil of low permeability can adapt itself completely (fully consolidate) to the changes in stress due to the additional axial (or shearing) stress.

consolidated-undrained test (consolidated quick test)—a soil test in which essentially complete consolidation under the vertical load (in a direct shear test) or under the confining pressure (in a triaxial test) is followed by a shear at constant water content.

consolidation—the gradual reduction in volume of a soil mass resulting from an increase in compressive stress.

initial consolidation (*initial compression*)—a comparatively sudden reduction in volume of a soil mass under an applied load due principally to expulsion and compression of gas in the soil voids preceding primary consolidation.

primary consolidation (*primary compression*) (*primary time effect*)—the reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to a squeezing out of water from the void spaces of the mass and accompanied by a transfer of the load from the soil water to the soil solids.

secondary consolidation (*secondary compression*) (*secondary time effect*)—the reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to the adjustment of the internal structure of the soil mass after most of the load has been transferred from the soil water to the soil solids.

consolidation curve—see **consolidation time curve.**

consolidation grouting—injection of a fluid grout, usually sand and Portland cement, into a compressible soil mass in order to displace it and form a lenticular grout structure for support.

Discussion—In rock, grouting is performed for the purpose of strengthening the rock mass by filling open fractures and thus eliminating a source of settlement.

consolidation ratio, U_z (D)—the ratio of: (*1*) the amount of consolidation at a given distance from a drainage surface and at a given time, to (*2*) the total amount of consolidation obtainable at that point under a given stress increment.

consolidation test—a test in which the specimen is laterally confined in a ring and is compressed between porous plates.

consolidation-time curve (time curve) (consolidation curve) (theoretical time curve)—a curve that shows the relation between: (*1*) the degree of consolidation, and (*2*) the elapsed time after the application of a given increment of load.

constitutive equation—force deformation function for a particular material. (ISRM)

contact grouting—see **backpack grouting.**

contact pressure, p (FL^{-2})—the unit of pressure that acts at the surface of contact between a structure and the underlying soil or rock mass.

continuous mixer—a mixer into which the ingredients of the mixture are fed without stopping, and from which the mixed product is discharged in a continuous stream.

contraction—linear strain associated with a decrease in length. (ISRM)

controlled blasting—includes all forms of blasting designed to preserve the integrity of the remaining rocks, that is, smooth blasting or pre-splitting. (ISRM)

controlled-strain test—a test in which the load is so applied that a controlled rate of strain results.

controlled-stress test—a test in which the stress to which a specimen is subjected is applied at a controlled rate.

convergence—generally refers to a shortening of the distance between the floor and roof of an opening, for example, in the bedded sedimentary rocks of the coal measures where the roof sags and the floor heaves. Can also apply to the convergence of the walls toward each other. (ISRM)

core—a cylindrical sample of hardened grout, concrete, rock, or grouted deposits, usually obtained by means of a core drill.

core drilling; diamond drilling—a rotary drilling technique, using diamonds in the cutting bit, that cuts out cylindrical rock samples. (ISRM)

core recovery—ratio of the length of core recovered to the length of hole drilled, usually expressed as a percentage.

cover—the perpendicular distance from any point in the roof of an underground opening to the ground surface. (ISRM)

cover—*in grouting,* the thickness of rock and soil material overlying the stage of the hole being grouted.

crack—a small fracture, that is, small with respect to the scale of the feature in which it occurs. (ISRM)

crater—excavation (generally of conical shape) generated by an explosive charge. (ISRM)

creep—slow movement of rock debris or soil usually imperceptible except to observations of long duration. Time-dependent strain or deformation, for example, continuing strain with sustained stress.

critical circle (critical surface)—the sliding surface assumed in a theoretical analysis of a soil mass for which the factor of safety is a minimum.

critical damping—the minimum viscous damping that will allow a displaced system to return to its initial position without oscillation.

critical density—the unit weight of a saturated granular material below which it will lose strength and above which it will gain strength when subjected to rapid deformation. The critical density of a given material is dependent on many factors.

critical frequency, f_c—frequency at which maximum or minimum amplitudes of excited waves occur.

critical height, H_c (L)—the maximum height at which a vertical or sloped bank of soil or rock will stand unsupported under a given set of conditions.

critical hydraulic gradient—see **hydraulic gradient**.

critical slope—the maximum angle with the horizontal at which a sloped bank of soil or rock of given height will stand unsupported.

critical surface—see **critical circle**.

critical void ratio—see **void ratio**.

crown—also roof or back, that is, the highest point of the cross section. *In tunnel linings*, the term is used to designate either the arched roof above spring lines or all of the lining except the floor or invert. (ISRM)

cryology—the study of the properties of snow, ice, and frozen ground.

cure—*in grouting*, the change in properties of a grout with time.

cure time—*in grouting*, the interval between combining all grout ingredients or the formation of a gel and substantial development of its potential properties.

curtain grouting—injection of grout into a sub-surface formation in such a way as to create a barrier of grouted material transverse to the direction of the anticipated water flow.

cuttings—small-sized rock fragments produced by a rock drill. (ISRM)

damping—reduction in the amplitude of vibration of a body or system due to dissipation of energy internally or by radiation. (ISRM)

damping ratio—*for a system with viscous damping*, the ratio of actual damping coefficient to the critical damping coefficient.

decay time—the interval of time required for a pulse to decay from its maximum value to some specified fraction of that value. (ISRM)

decomposition—*for peats and organic soils*, see **humification**.

decoupling—the ratio of the radius of the blasthole to the radius of the charge. In general, a reducing of the strain wave amplitude by increasing the spacing between charge and blasthole wall. (ISRM)

deflocculating agent (deflocculant) (dispersing agent)—an agent that prevents fine soil particles in suspension from coalescing to form flocs.

deformability—*in grouting*, a measure of the elasticity of the grout to distort in the interstitial spaces as the sediments move.

deformation—change in shape or size.

deformation—a change in the shape or size of a solid body. (ISRM)

deformation resolution (deformation sensitivity), R_d (L)—ratio of the smallest subdivision of the indicating scale of a deformation-measuring device to the sensitivity of the device.

degree-days—the difference between the average temperature each day and 32°F (0°C). In common usage degree-days are positive for daily average temperatures above 32°F and negative for those below 32°F (see **freezing index**).

degree of consolidation (percent consolidation), U (D)—the ratio, expressed as a percentage, of: (*1*) the amount of consolidation at a given time within a soil mass, to (*2*) the total amount of consolidation obtainable under a given stress condition.

degrees-of-freedom—the minimum number of independent coordinates required in a mechanical system to define completely the positions of all parts of the system at any instant of time. In general, it is equal to the number of independent displacements that are possible.

degree of saturation—see **percent saturation**.

degree of saturation—the extent or degree to which the voids in rock contain fluid (water, gas, or oil). Usually expressed in percent related to total void or pore space. (ISRM)

degree of sensitivity (*sensitivity ratio*)—see **remolding index**.

delay—time interval (fraction of a second) between detonation of explosive charges. (ISRM)

density—the mass per unit, ρ (ML^{-3}) kg/m³.

density of dry soil or rock, ρ_d (ML^{-3}) kg/m³—the mass of solid particles per the total volume of soil or rock.

density of saturated soil or rock, ρ_{sat} (ML^{-3}) kg/m³—the total mass per total volume of completely saturated soil or rock.

density of soil or rock (*bulk density*), ρ (ML^{-3}) kg/m³—the total mass (solids plus water) per total volume.

density of solid particles, ρ_s (ML^{-3}) kg/m³—the mass per volume of solid particles.

density of submerged soil or rock, ρ_{sub} (ML^{-3}) kg/m³—the difference between the density of saturated soil or rock, and the density of water.

density of water, ρ_w (ML^{-3}) kg/m³—the mass per volume of water.

detonation—an extremely rapid and violent chemical reaction causing the production of a large volume of gas. (ISRM)

deviator stress, Δ, σ (FL^{-2})—the difference between the major and minor principal stresses in a triaxial test.

deviator of stress (strain)—the stress (strain) tensor obtained by subtracting the mean of the normal stress (strain) components of a stress (strain) tensor from each normal stress (strain) component. (ISRM)

differential settlement—settlement that varies in rate or amount, or both, from place to place across a structure.

dilatancy—property of volume increase under loading. (ISRM)

dilatancy—the expansion of cohesionless soils when subject to shearing deformation.

direct shear test—a shear test in which soil or rock under an applied normal load is stressed to failure by moving one section of the sample or sample container (shear box) relative to the other section.

discharge velocity, v, q (LT^{-1})—rate of discharge of water through a porous medium per unit of total area perpendicular to the direction of flow.

discontinuity surface—any surface across which some property of a rock mass is discontinuous. This includes fracture surfaces, weakness planes, and bedding planes, but the term should not be restricted only to mechanical continuity. (ISRM)

dispersing agent—*in grouting*, an addition or admixture that promotes dispersion of particulate grout ingredients by reduction of interparticle attraction.

dispersing agent—see **deflocculating agent**.

dispersion—the phenomenon of varying speed of transmission of waves, depending on their frequency. (ISRM)

displacement—a change in position of a material point. (ISRM)

displacement grouting—injection of grout into a formation in such a manner as to move the formation; it may be controlled or uncontrolled. See also **penetration grouting**.

distortion—a change in shape of a solid body. (ISRM)

divergence loss—that part of transmitted energy lost due to spreading of wave rays in accordance with the geometry of the system.

double amplitude—total or peak to peak excursion.

drag bit—a noncoring or full-hole boring bit, which scrapes its way through relatively soft strata. (ISRM)

drain—a means for intercepting, conveying, and removing water.

drainage curtain—*in grouting*, a row of open holes drilled parallel to and downstream from the grout curtain of a dam for the purpose of reducing uplift pressures.

Discussion—Depth is ordinarily approximately one-third to one-half that of the grout curtain.

drainage gallery—*in grouting*, an opening or passageway from which grout holes or drainage curtain holes, or both, are drilled. See also **grout gallery**.

drawdown (L)—vertical distance the free water elevation is lowered or the pressure head is reduced due to the removal of free water.

drill—a machine or piece of equipment designed to penetrate earth or rock formations, or both.

drillability—index value of the resistance of a rock to drilling. (ISRM)

drill carriage; jumbo—a movable platform, stage, or frame that incorporates several rock drills and usually travels on the tunnel track; used for heavy drilling work in large tunnels. (ISRM)

drilling pattern—the number, position, depth, and angle of the blastholes forming the complete round in the face of a tunnel or sinking pit. (ISRM)

drill mud—*in grouting*, a dense fluid or slurry used in rotary drilling; to prevent caving of the bore hole walls, as a circulation medium to carry cuttings away from the bit and out of the hole, and to seal fractures or permeable formations, or both, preventing loss of circulation fluid.

Discussion—The most common drill mud is a water-bentonite mixture, however, many other materials may be added or substituted to increase density or decrease viscosity.

dry pack—a cement-sand mix with minimal water content used to fill small openings or repair imperfections in concrete.

dry unit weight (*dry density*)—see **unit weight**.

ductility—condition in which material can sustain permanent deformation without losing its ability to resist load. (ISRM)

dye tracer—*in grouting*, an additive whose primary purpose is to change the color of the grout or water.

earth—see **soil**.

earth pressure—the pressure or force exerted by soil on any boundary.

	Symbol	Unit
Pressure	p	FL^{-2}
Force	P	F or FL^{-1}

active earth pressure, P_A, p_A—the minimum value of earth pressure. This condition exists when a soil mass is permitted to yield sufficiently to cause its internal shearing resistance along a potential failure surface to be completely mobilized.

earth pressure at rest, P_o, p_o—the value of the earth pressure when the soil mass is in its natural state without having been permitted to yield or without having been compressed.

passive earth pressure, P_p, p_p—the maximum value of earth pressure. This condition exists when a soil mass is compressed sufficiently to cause its internal shearing resistance along a potential failure surface to be completely mobilized.

effect diameter (effective size), D_{10}, D_e (L)—particle diameter corresponding to 10 % finer on the grain-size curve.

effective drainage porosity—see **effective porosity**.

effective force, $\bar{F}$ (F)—the force transmitted through a soil or rock mass by intergranular pressures.

effective porosity (effective drainage porosity), n_e (D)—the ratio of: (*1*) the volume of the voids of a soil or rock mass that can be drained by gravity, to (*2*) the total volume of the mass.

effective pressure—see **stress**.

effective size—see **effective diameter**.

effective stress—see **stress**.

effective unit weight—see **unit weight**.

efflux time—time required for all grout to flow from a flow cone.

elasticity—property of material that returns to its original form or condition after the applied force is removed. (ISRM)

elastic limit—point on stress strain curve at which transition from elastic to inelastic behavior takes place. (ISRM)

elastic state of equilibrium—state of stress within a soil mass when the internal resistance of the mass is not fully mobilized.

elastic strain energy—potential energy stored in a strained solid and equal to the work done in deforming the solid from its unstrained state less any energy dissipated by inelastic deformation. (ISRM)

electric log—a record or log of a borehole obtained by lowering electrodes into the hole and measuring any of the various electrical properties of the rock formations or materials traversed.

electrokinetics—involves the application of an electric field to soil for the purpose of dewatering materials of very low permeability to enhance stability. The electric field produces negative pore pressures near a grout pipe that facilitates grout injection.

emulsifier—a substance that modifies the surface tension of colloidal droplets, keeping them from coalescing, and keeping them suspended.

emulsion—a system containing dispersed colloidal droplets.

endothermic—pertaining to a reaction that occurs with the adsorption of heat.

envelope grouting—grouting of rock surrounding a hydraulic pressure tunnel for purpose of consolidation, and primarily, reduction of permeability.

epoxy—a multicomponent resin grout that usually provides very high, tensile, compressive, and bond strengths.

equipotential line—see **piezometric line.**

equivalent diameter (equivalent size), D **(L)**—the diameter of a hypothetical sphere composed of material having the same specific gravity as that of the actual soil particle and of such size that it will settle in a given liquid at the same terminal velocity as the actual soil particle.

equivalent fluid—a hypothetical fluid having a unit weight such that it will produce a pressure against a lateral support presumed to be equivalent to that produced by the actual soil. This simplified approach is valid only when deformation conditions are such that the pressure increases linearly with depth and the wall friction is neglected.

excess hydrostatic pressure—see **hydrostatic pressure.**

exchange capacity—the capacity to exchange ions as measured by the quantity of exchangeable ions in a soil or rock.

excitation (stimulus)—an external force (or other input) applied to a system that causes the system to respond in some way.

exothermic—pertaining to a reaction that occurs with the evolution of heat.

expansive cement—a cement that tends to increase in volume after it is mixed with water.

extender—an additive whose primary purpose is to increase total grout volume.

extension—linear strain associated with an increase in length. (ISRM)

external force—a force that acts across external surface elements of a material body. (ISRM)

extrados—the exterior curved surface of an arch, as opposed to intrados, which is the interior curved surface of an arch. (ISRM)

fabric—*for rock or soil*, the spatial configuration of all textural and structural features as manifested by every recognizable material unit from crystal lattices to large scale features requiring field studies.

fabric—the orientation in space of the elements composing the rock substance. (ISRM)

face (heading)—the advanced end of a tunnel, drift, or excavation at which work is progressing. (ISRM)

facing—the outer layer of revetment.

failure (in rocks)—exceeding the maximum strength of the rock or exceeding the stress or strain requirement of a specific design. (ISRM)

failure by rupture—see **shear failure.**

failure criterion—specification of the mechanical condition under which solid materials fail by fracturing or by deforming beyond some specified limit. This specification may be in terms of the stresses, strains, rate-of-change of stresses, rate-of-change of strains, or some combination of these quantities, in the materials.

failure criterion—theoretically or empirically derived stress or strain relationship characterizing the occurrence of failure in the rock. (ISRM)

false set—*in grouting*, the rapid development of rigidity in a freshly mixed grout without the evolution of much heat.

Discussion—Such rigidity can be dispelled and plasticity regained by further mixing without the addition of water; premature stiffening, hesitation set, early stiffening, and rubber set are other much used terms referring to the same phenomenon.

fatigue—the process of progressive localized permanent structural change occurring in a material subjected to conditions that produce fluctuating stresses and strains at some point or points and that may culminate in cracks or complete fracture after a sufficient number of fluctuations.

fatigue—decrease of strength by repetitive loading. (ISRM)

fatigue limit—point on stress-strain curve below which no fatigue can be obtained regardless of number of loading cycles. (ISRM)

fault—a fracture or fracture zone along which there has been displacement of the two sides relative to one another parallel to the fracture (this displacement may be a few centimetres or many kilometres). (See also **joint fault set** and **joint fault system.** (ISRM)

fault breccia—the assemblage of broken rock fragments frequently found along faults. The fragments may vary in size from inches to feet. (ISRM)

fault gouge—a clay-like material occurring between the walls of a fault as a result of the movement along the fault surfaces. (ISRM)

fiber—*for peats and organic soils*, a fragment or piece of plant tissue that retains a recognizable cellular structure and is large enough to be retained after wet sieving on a 100-mesh sieve (openings 0.15 mm).

fibric peat—peat in which the original plant fibers are slightly decomposed (greater than 67 % fibers).

fibrous peat—see **fibric peat.**

field moisture equivalent—see **moisture equivalent.**

fill—man-made deposits of natural soils or rock products and waste materials.

filling—generally, the material occupying the space between joint surfaces, faults, and other rock discontinuities. The filling material may be clay, gouge, various natural cementing agents, or alteration products of the adjacent rock. (ISRM)

filter bedding stone—(generally 6-in. minus material) stone placed under graded riprap stone or armor stone in a layer or combination of layers designed and installed in such a manner as to prevent the loss of underlying soil or finer bedding materials due to moving water.

filter (protective filter)—a layer or combination of layers of pervious materials designed and installed in such a manner as to provide drainage, yet prevent the movement of soil particles due to flowing water.

final set—*in grouting*, a degree of stiffening of a grout mixture greater than initial set, generally stated as an empirical value indicating the time in hours and minutes that is required for cement paste to stiffen sufficiently to resist the penetration of a weighted test needle.

fineness—a measure of particle-size.

fineness modulus—an empirical factor obtained by adding the total percentages of an aggregate sample retained on each of a specified series of sieves, and dividing the sum by 100; in the United States, the U.S. Standard sieve sizes are: No. 100 (149 µm), No. 50 (297 µm), No. 30 (590 µm), No. 16 (1,190 µm), No. 8 (2,380 µm), and No. 4 (4,760 µm) and ⅜ in. (9.5 mm), ¾ in. (19 mm), 1½ in. (38 mm), 3 in. (76 mm), and 6 in. (150 mm).

fines—portion of a soil finer than a No. 200 (75-µm) U.S. standard sieve.

finite element—one of the regular geometrical shapes into which a figure is subdivided for the purpose of numerical stress analysis. (ISRM)

fishing tool—*in grouting*, a device used to retrieve drilling equipment lost or dropped in the hole.

fissure—a gapped fracture. (ISRM)

flash set—*in grouting*, the rapid development of rigidity in a freshly mixed grout, usually with the evolution of considerable heat; this rigidity cannot be dispelled nor can the plasticity be regained by further mixing without addition of water; also referred to as quick set or grab set.

floc—loose, open-structured mass formed in a suspension by the aggregation of minute particles.

flocculation—the process of forming flocs.

flocculent structure—see soil structure.

floor—bottom of near horizontal surface of an excavation, approximately parallel and opposite to the roof. (ISRM)

flow channel—the portion of a flow net bounded by two adjacent flow lines.

flow cone—*in grouting*, a device for measurement of grout consistency in which a predetermined volume of grout is permitted to escape through a precisely sized orifice, the time of efflux (flow factor) being used as the indication of consistency.

flow curve—the locus of points obtained from a standard liquid limit test and plotted on a graph representing water content as ordinate on an arithmetic scale and the number of blows as abscissa on a logarithmic scale.

flow failure—failure in which a soil mass moves over relatively long distances in a fluid-like manner.

flow index, F_w, I_f(D)—the slope of the flow curve obtained from a liquid limit test, expressed as the difference in water contents at 10 blows and at 100 blows.

flow line—the path that a particle of water follows in its course of seepage under laminar flow conditions.

flow net—a graphical representation of flow lines and equipotential (piezometric) lines used in the study of seepage phenomena.

flow slide—the failure of a sloped bank of soil in which the movement of the soil mass does not take place along a well-defined surface of sliding.

flow value, N_ϕ (degrees)—a quantity equal to tan [45 deg + (ϕ/2)].

fluidifier—*in grouting*, an admixture employed in grout to increase flowability without changing water content.

fly ash—the finely divided residue resulting from the combustion of ground or powdered coal and which is transported from the firebox through the boiler by flue gases.

fold—a bend in the strata or other planar structure within the rock mass. (ISRM)

foliation—the somewhat laminated structure resulting from segregation of different minerals into layers parallel to the schistosity. (ISRM)

footing—portion of the foundation of a structure that transmits loads directly to the soil.

footwall—the mass of rock beneath a discontinuity surface. (ISRM)

forced vibration (forced oscillation)—vibration that occurs if the response is imposed by the excitation. If the excitation is periodic and continuing, the oscillation is steady-state.

forepoling—driving forepoles (pointed boards or steel rods) ahead of the excavation, usually over the last set erected, to furnish temporary overhead protection while installing the next set. (ISRM)

foundation—lower part of a structure that transmits the load to the soil or rock.

foundation soil—upper part of the earth mass carrying the load of the structure.

fracture—the general term for any mechanical discontinuity in the rock; it therefore is the collective term for joints, faults, cracks, etc. (ISRM)

fracture—a break in the mechanical continuity of a body of rock caused by stress exceeding the strength of the rock. Includes joints and faults.

fracture frequency—the number of natural discontinuities in a rock or soil mass per unit length, measured along a core or as exposed in a planar section such as the wall of a tunnel.

fracture pattern—spatial arrangement of a group of fracture surfaces. (ISRM)

fracturing—*in grouting*, intrusion of grout fingers, sheets, and lenses along joints, planes of weakness, or between the strata of a formation at sufficient pressure to cause the strata to move away from the grout.

fragmentation—the breaking of rock in such a way that the bulk of the material is of a convenient size for handling. (ISRM)

free water (gravitational water) (ground water) (phreatic water)—water that is free to move through a soil or rock mass under the influence of gravity.

free water elevation (water table) (ground water surface) (free water surface) (ground water elevation)—elevations at which the pressure in the water is zero with respect to the atmospheric pressure.

freezing index, F (degree-days)—the number of degree-days between the highest and lowest points on the cumulative degree-days—time curve for one freezing season. It is used as a measure of the combined duration and magnitude of below-freezing temperature occurring during any given freezing season. The index determined for air temperatures at 4.5 ft (1.4 m) above the ground is commonly

designated as the air freezing index, while that determined for temperatures immediately below a surface is known as the surface freezing index.

free vibration—vibration that occurs in the absence of forced vibration.

frequency, f (T^{-1})—number of cycles occurring in unit time.

frost action—freezing and thawing of moisture in materials and the resultant effects on these materials and on structures of which they are a part or with which they are in contact.

frost boil—(*a*) softening of soil occurring during a thawing period due to the liberation of water form ice lenses or layers.

(*b*) the hole formed in flexible pavements by the extrusion of soft soil and melt waters under the action of wheel loads.

(*c*) breaking of a highway or airfield pavement under traffic and the ejection of subgrade soil in a soft and soupy condition caused by the melting of ice lenses formed by frost action.

frost heave—the raising of a surface due to the accumulation of ice in the underlying soil or rock.

fundamental frequency—lowest frequency of periodic variation.

gage length, L (L)—distance over which the deformation measurement is made.

gage protector—*in grouting*, a device used to transfer grout pressure to a gage without the grout coming in actual contact with the gage.

gage saver—see **gage protector.**

gel—*in grouting*, the condition where a liquid grout begins to exhibit measurable shear strength.

gel time—*in grouting*, the measured time interval between the mixing of a grout system and the formation of a gel.

general shear failure—see **shear failure.**

glacial till (till)—material deposited by glaciation, usually composed of a wide range of particle sizes, which has not been subjected to the sorting action of water.

gradation (grain-size distribution) (texture)—the proportions by mass of a soil or fragmented rock distributed in specified particle-size ranges.

grain-size analysis (mechanical analysis) (particle-size analysis)—the process of determining grain-size distribution.

gravel—rounded or semirounded particles of rock that will pass a 3-in. (76.2-mm) and be retained on a No. 4 (4.75-μm) U.S. standard sieve.

gravitational water—see **free water.**

gravity grouting—grouting under no applied pressure other than the height of fluid in the hole.

groin—bank or shore-protection structure in the form of a barrier placed oblique to the primary motion of water, designed to control movement of bed load.

ground arch—the theoretical stable rock arch that develops some distance back from the surface of the opening and supports the opening. (ISRM)

ground water—that part of the subsurface water that is in the saturated zone.

Discussion—Loosely, all subsurface water as distinct from surface water.

ground-water barrier—soil, rock, or artificial material which has a relatively low permeability and which occurs below the land surface where it impedes the movement of ground water and consequently causes a pronounced difference in the potentiometric level on opposite sides of the barrier.

ground-water basin—a ground-water system that has defined boundaries and may include more than one aquifer of permeable materials, which are capable of furnishing a significant water supply.

Discussion—A basin is normally considered to include the surface area and the permeable materials beneath it. The surface-water divide need not coincide with ground-water divide.

ground-water discharge—the water released from the zone of saturation; also the volume of water released.

ground-water divide—a ridge in the water table or other potentiometric surface from which ground water moves away in both directions normal to the ridge line.

ground-water elevation—see **free water elevation.**

ground-water flow—the movement of water in the zone of saturation.

ground-water level—the level below which the rock and subsoil, to unknown depths, are saturated. (ISRM)

ground-water, perched—see **perched ground-water.**

ground-water recharge—the process of water addition to the saturated zone; also the volume of water added by this process.

ground-water surface—see **free water elevation.**

grout—*in soil and rock grouting*, a material injected into a soil or rock formation to change the physical characteristics of the formation.

groutability—the ability of a formation to accept grout.

groutability ratio of granular formations—the ratio of the 15 % size of the formation particles to be grouted to the 85 % size of grout particles (suspension-type grout). This ratio should be greater than 24 if the grout is to successfully penetrate the formation.

groutable rock bolts—rock bolts with hollow cores or with tubes adapted to the periphery of the bolts and extending to the bottom of the bolts to facilitate filling the holes surrounding the bolts with grout.

grouted-aggregate concrete—concrete that is formed by injecting grout into previously placed coarse aggregate. See also **preplaced aggregate concrete.**

grout cap—a "cap" that is formed by placing concrete along the top of a grout curtain. A grout cap is often used in weak foundation rock to secure grout nipples, control leakage, and to form an impermeable barrier at the top of a grout curtain.

grout gallery—an opening or passageway within a dam utilized for grouting or drainage operations, or both.

grout header—a pipe assembly attached to a ground hole, and to which the grout lines are attached for injecting grout. Grout injector is monitored and controlled by means of valves and a pressure gate mounted on the header; sometimes called grout manifold.

grout mix—the proportions or amounts of the various materials used in the grout, expressed by weight or volume. (The words "by volume" or "by weight" should be used to specify the mix.)

grout nipple—*in grouting*, a short length of pipe, installed at the collar of the grout hole, through which drilling is done and to which the grout header is attached for the purpose of injecting grout.

grout slope—the natural slope of grout injected into preplaced-aggregate or other porous mass.

grout system—formulation of different materials used to form a grout.

grout take—the measured quantity of grout injected into a unit volume of formation, or a unit length of grout hole.

hanging wall—the mass of rock above a discontinuity surface. (ISRM)

hardener—*in grouting*, in a two component epoxy or resin, the chemical component that causes the base component to cure.

hardness—resistance of a material to indentation or scratching. (ISRM)

hardpan—a hard impervious layer, composed chiefly of clay, cemented by relatively insoluble materials, that does not become plastic when mixed with water and definitely limits the downward movement of water and roots.

head—pressure at a point in a liquid, expressed in terms of the vertical distance of the point below the surface of the liquid. (ISRM)

heat of hydration—heat evolved by chemical reactions with water, such as that evolved during the setting and hardening of Portland cement.

heave—upward movement of soil caused by expansion or displacement resulting from phenomena such as: moisture absorption, removal of overburden, driving of piles, frost action, and loading of an adjacent area.

height of capillary rise—see **capillary rise**.

hemic peat—peat in which the original plant fibers are moderately decomposed (between 33 and 67 % fibers).

heterogeneity—having different properties at different points. (ISRM)

homogeneity—having different properties at different points. (ISRM)

homogeneity—having the same properties at all points. (ISRM)

homogeneous mass—a mass that exhibits essentially the same physical properties at every point throughout the mass.

honeycomb structure—see **soil structure**.

horizon (soil horizon)—one of the layers of the soil profile, distinguished principally by its texture, color, structure, and chemical content.

"A" horizon—the uppermost layer of a soil profile from which inorganic colloids and other soluble materials have been leached. Usually contains remnants of organic life.

"B" horizon—the layer of a soil profile in which material leached from the overlying "A" horizon is accumulated.

"C" horizon—undisturbed parent material from which the overlying soil profile has been developed.

humic peat—see **sapric peat**.

humification—a process by which organic matter decomposes.

Discussion—The degree of humification for peats is indicated by the state of the fibers. In slightly decomposed material, most of the volume consists of fibers. In moderately decomposed material, the fibers may be preserved but may break down with disturbance, such as rubbing between the fingers. In highly decomposed materials, fibers will be virtually absent; see **von Post humification scale**.

humus—a brown or black material formed by the partial decomposition of vegetable or animal matter; the organic portion of soil.

hydration—formation of a compound by the combining of water with some other substance.

hydraulic conductivity—see **coefficient of permeability**.

hydraulic fracturing—the fracturing of an underground strata by pumping water or grout under a pressure in excess of the tensile strength and confining pressure; also called hydrofracturing.

hydraulic gradient, i, s (D)—the loss of hydraulic head per unit distance of flow, dh/dL.

critical hydraulic gradient, i_c (D)—hydraulic gradient at which the intergranular pressure in a mass of cohesionless soil is reduced to zero by the upward flow of water.

hydrostatic head—the fluid pressure of formation water produced by the height of water above a given point.

hydrostatic pressure, u_o (FL^{-2})—a state of stress in which all the principal stresses are equal (and there is no shear stress), as in a liquid at rest; the product of the unit weight of the liquid and the different in elevation between the given point and the free water elevation.

excess hydrostatic pressure (*hydrostatic excess pressure*), $\bar{u}$, u (FL^{-2})—the pressure that exists in pore water in excess of the hydrostatic pressure.

hydrostatic pressure—a state of stress in which all the principal stresses are equal (and there is no shear stress). (ISRM)

hygroscopic capacity (hygroscopic coefficient), w_e (D)—ratio of: (*1*) the weight of water absorbed by a dry soil or rock in a saturated atmosphere at a given temperature, to (*2*) the weight of the oven-dried soil or rock.

hygroscopic water content, w_H (D)—the water content of an air-dried soil or rock.

hysteresis—incomplete recovery of strain during unloading cycle due to energy consumption. (ISRM)

impedance, acoustic—the product of the density and sonic velocity of a material. The extent of wave energy transmission and reflection at the boundary of two media is determined by their acoustic impedances. (ISRM)

inelastic deformation—the portion of deformation under stress that is not annulled by removal of stress. (ISRM)

inert—not participating in any fashion in chemical reactions.

influence value, I (D)—the value of the portion of a mathematical expression that contains combinations of the independent variables arranged in dimensionless form.

inhibitor—a material that stops or slows a chemical reaction from occurring.

initial consolidation (*initial compression*)—see **consolidation**.

initial set—a degree of stiffening of a grout mixture generally stated as an empirical value indicating the time in hours and minutes that is required for a mixture to stiffen sufficiently to resist the penetration of a weighted test needle.

injectability—see **groutability**.

inorganic silt—see **silt**.

in situ—applied to a rock or soil when occurring in the situation in which it is naturally formed or deposited.

intergranular pressure—see **stress**.

intermediate principal plane—see **principal plane**.

intermediate principal stress—see **stress**.

internal friction (shear resistance), (FL^{-2})—the portion of the shearing strength of a soil or rock indicated by the terms $p \tan \phi$ in Coulomb's equation $s = c + p \tan \phi$. It is usually considered to be due to the interlocking of the soil or rock grains and the resistance to sliding between the grains.

interstitial—occurring between the grains or in the pores in rock or soil.

intrinsic shear strength, S_o (FL^{-2})—the shear strength of a rock indicated by Coulomb's equation when $p \tan \phi$ (shear resistance or internal friction) vanishes. Corresponds to cohesion, *c*, in soil mechanics.

invert—on the cross section, the lowest point of the underground excavation or the lowest section of the lining. (ISRM)

isochrome—a curve showing the distribution of the excess hydrostatic pressure at a given time during a process of consolidation.

isotropic mass—a mass having the same property (or properties) in all directions.

isotropic material—a material whose properties do not vary with direction.

isotropy—having the same properties in all directions. (ISRM)

jackhammer—an air driven percussion drill that imparts a rotary hammering motion to the bit and has a passageway to the bit for the injection of compressed air for cleaning the hole of cuttings.

Discussion—These two characteristics distinguish it from the pavement breaker which is similar in size and general appearance.

jack-leg—a portable percussion drill of the jack-hammer type, used in underground work; has a single pneumatically adjustable leg for support.

jet grouting—technique utilizing a special drill bit with horizontal and vertical high speed water jets to excavate alluvial soils and produce hard impervious columns by pumping grout through the horizontal nozzles that jets and mixes with foundation material as the drill bit is withdrawn.

jetty—an elongated artificial obstruction projecting into a body of water form a bank or shore to control shoaling and scour by deflection of the force of water currents and waves.

joint—a break of geological origin in the continuity of a body of rock occurring either singly, or more frequently in a set or system, but not attended by a visible movement parallel to the surface of discontinuity. (ISRM)

joint diagram—a diagram constructed by accurately plotting the strike and dip of joints to illustrate the geometrical relationship of the joints within a specified area of geologic investigation. (ISRM)

joint pattern—a group of joints that form a characteristic geometrical relationship, and which can vary considerably from one location to another within the same geologic formation. (ISRM)

joint (fault) set—a group of more or less parallel joints. (ISRM)

joint (fault) system—a system consisting of two or more joint sets or any group of joints with a characteristic pattern, that is, radiating, concentric, etc. (ISRM)

jumbo—a specially built mobile carrier used to provide a work platform for one or more tunneling operations, such as drilling and loading blast holes, setting tunnel supports, installing rock bolts, grouting, etc.

kaolin—a variety of clay containing a high percentage of kaolinite.

kaolinite—a common clay mineral having the general formula $Al_2(Si_2O_5)(OH_4)$; the primary constituent of kaolin.

karst—a geologic setting where cavities are developed in massive limestone beds by solution of flowing water. Caves and even underground river channels are produced into which surface runoff drains and often results in the land above being dry and relatively barren. (ISRM)

kelly—a heavy-wall tube or pipe, usually square or hexagonal in cross section, which works inside the matching center hole in the rotary table of a drill rig to impart rotary motion to the drill string.

lagging, *n—in mining or tunneling,* short lengths of timber, sheet steel, or concrete slabs used to secure the roof and sides of an opening behind the main timber or steel supports. The process of installation is also called lagging or lacing.

laminar flow (streamline flow) (viscous flow)—flow in which the head loss is proportional to the first power of the velocity.

landslide—the perceptible downward sliding or movement of a mass of earth or rock, or a mixture of both. (ISRM)

landslide (slide)—the failure of a sloped bank of soil or rock in which the movement of the mass takes place along a surface of sliding.

leaching—the removal in solution of the more soluble materials by percolating or moving waters. (ISRM)

leaching—the removal of soluble soil material and colloids by percolating water.

lime—specifically, calcium oxide (CaO_2); also loosely, a general term for the various chemical and physical forms of quicklime, hydrated lime, and hydraulic hydrated lime.

ledge—see bedrock.

linear (normal) strain—the change in length per unit of length in a given direction. (ISRM)

line of creep (path of percolation)—the path that water follows along the surface of contact between the foundation soil and the base of a dam or other structure.

line of seepage (seepage line) (phreatic line)—the upper free water surface of the zone of seepage.

linear expansion, L_e (D)—the increase in one dimension of a soil mass, expressed as a percentage of that dimension at the shrinkage limit, when the water content is increased from the shrinkage limit to any given water content.

linear shrinkage, L_s (D)—decrease in one dimension of a soil mass, expressed as a percentage of the original dimension, when the water content is reduced from a given value to the shrinkage limit.

lineation—the parallel orientation of structural features that are lines rather than planes; some examples are parallel orientation of the long dimensions of minerals; long axes of pebbles; striae on slickensides; and cleavage-bedding plane intersections. (ISRM)

liquefaction—the process of transforming any soil from a solid state to a liquid state, usually as a result of increased pore pressure and reduced shearing resistance.

liquefaction potential—the capability of a soil to liquefy or develop cyclic mobility.

liquefaction (spontaneous liquefaction)—the sudden large decrease of the shearing resistance of a cohesionless soil. It is caused by a collapse of the structure by shock or other type of strain and is associated with a sudden but temporary increase of the prefluid pressure. It involves a temporary transformation of the material into a fluid mass.

liquid, limit, LL, L_w, w_L (D)—(*a*) the water content corresponding to the arbitrary limit between the liquid and plastic states of consistency of a soil.

(*b*) the water content at which a pat of soil, cut by a groove of standard dimensions, will flow together for a distance of ½ in. (12.7 mm) under the impact of 25 blows in a standard liquid limit apparatus.

liquidity index (water-plasticity ratio) (relative water content), B, R_w, I_L (D)—the ratio, expressed as a percentage, of: (*1*) the natural water content of a soil minus its plastic limit, to (*2*) its plasticity index.

liquid-volume measurement—*in grouting*, measurement of grout on the basis of the total volume of solid and liquid constituents.

lithology—the description of rocks, especially sedimentary clastics and especially in hand specimens and in outcrops, on the basis of such characteristics as color, structures, mineralogy, and particle size.

loam—a mixture of sand, silt, or clay, or a combination of any of these, with organic matter (see **humus**).

Discussion—It is sometimes called topsoil in contrast to the subsoils that contain little or no organic matter.

local shear failure—see **shear failure.**

loess—a uniform aeolian deposit of silty material having an open structure and relatively high cohesion due to cementation of clay or calcareous material at grain contacts.

Discussion—A characteristic of loess deposits is that they can stand with nearly vertical slopes.

logarithmic decrement—the natural logarithm of the ratio of any two successive amplitudes of like sign, in the decay of a single-frequency oscillation.

longitudinal rod wave—see **compression wave.**

longitudinal wave, v_l (LT^{-1})—wave in which direction of displacement at each point of medium is normal to wave front, with propagation velocity, calculated as follows:

$$v_l = \sqrt{(E/\rho)[(1-\nu)/(1+\nu)(1-2\nu)]} = \sqrt{(\lambda + 2\mu)/\rho}$$

where:

E = Young's modulus,
ρ = mass density,
λ and μ = Lamé's constants, and
ν = Poisson's ratio.

long wave (quer wave), W (LT^{-1})—dispersive surface wave with one horizontal component, generally normal to the direction of propagation, which decreases in propagation velocity with increase in frequency.

lubricity—*in grouting*, the physico-chemical characteristic of a grout material flow through a soil or rock that is the inverse of the inherent friction of that material to the soil or rock; comparable to "wetness."

lugeon—a measure of permeability defined by a pump-in test or pressure test, where one Lugeon unit is a water take of 1 L/min per metre of hole at a pressure of 10 bars.

major principal plane—see **principal plane.**

major principal stress—see **stress.**

manifold—see **grout header.**

marl—calcareous clay, usually containing from 35 to 65 % calcium carbonate ($CaCO_3$).

marsh—a wetland characterized by grassy surface mats which are frequently interspersed with open water or by a closed canopy of grasses, sedges, or other herbacious plants.

mass unit weight—see **unit weight.**

mathematical model—the representation of a physical system by mathematical expressions from which the behavior of the system can be deduced with known accuracy. (ISRM)

matrix—*in grouting*, a material in which particles are embedded, that is, the cement paste in which the fine aggregate particles of a grout are embedded.

maximum amplitude (L, LT^{-1}, LT^{-2})—deviation from mean or zero point.

maximum density (*maximum unit weight*)—see **unit weight.**

mechanical analysis—see **grain-size analysis.**

mesic peat—see **hemic peat.**

metering pump—a mechanical arrangement that permits pumping of the various components of a grout system in any desired proportions or in fixed proportions. (*Syn.* proportioning pump, variable proportion pump.)

microseism—seismic pulses of short duration and low amplitude, often occurring previous to failure of a material or structure. (ISRM)

minor principal plane—see **principal plane.**

minor principal stress—see **stress.**

mixed-in-place pile—a soil-cement pile, formed in place by forcing a grout mixture through a hollow shaft into the ground where it is mixed with the in-place soil with an auger-like head attached to the hollow shaft.

mixer—a machine employed for blending the constituents of grout, mortar, or other mixtures.

mixing cycle—the time taken for the loading, mixing, and unloading cycle.

mixing speed—the rotation rate of a mixer drum or of the paddles in an open-top, pan, or trough mixer, when mixing a batch; expressed in revolutions per minute.

modifier—*in grouting*, an additive used to change the normal chemical reaction or final physical properties of a grout system.

modulus of deformation—see **modulus of elasticity.**

modulus of elasticity (modulus of deformation), E, M (FL^{-2})—the ratio of stress to strain for a material under given loading conditions; numerically equal to the slope of the tangent or the secant of a stress-strain curve. The use of the term **modulus of elasticity** is recommended for materials that deform in accordance with Hooke's law; the term **modulus of deformation** for materials that deform otherwise.

modulus of subgrade reaction—see **coefficient of subgrade reaction.**

modulus of volume change—see **coefficient of volume compressibility.**

Mohr circle—a graphical representation of the stresses acting on the various planes at a given point.

Mohr circle of stress (strain)—a graphical representation of the components of stress (strain) acting across the various planes at a given point, drawn with reference to axes of normal stress (strain) and shear stress (strain). (ISRM)

Mohr envelope—the envelope of a sequence of Mohr circles representing stress conditions at failure for a given material. (ISRM)

Mohr envelope (rupture envelope) (rupture line)—the envelope of a series of Mohr circles representing stress conditions at failure for a given material.

Discussion—According to Mohr's rupture hypothesis, a rupture envelope is the locus of points the coordinates of which represent the combinations of normal and shearing stresses that will cause a given material to fail.

moisture content—see **water content.**

moisture-density curve—see **compaction curve.**

moisture-density test—see **compaction test.**

moisture equivalent:

centrifuge moisture equivalent, W_e, CME (D)—the water content of a soil after it has been saturated with water and then subjected for 1 h to a force equal to 1000 times that of gravity.

field moisture equivalent, FME—the minimum water content expressed as a percentage of the weight of the oven-dried soil, at which a drop of water placed on a smoothed surface of the soil will not immediately be absorbed by the soil but will spread out over the surface and give it a shiny appearance.

montmorillonite—a group of clay minerals characterized by a weakly bonded sheet-like internal molecular structure; consisting of extremely finely divided hydrous aluminum or magnesium silicates that swell on wetting, shrink on drying, and are subject to ion exchange.

muck—stone, dirt, debris, or useless material; or an organic soil of very soft consistency.

mud—a mixture of soil and water in a fluid or weakly solid state.

mudjacking—see **slab jacking.**

multibench blasting—the blasting of several benches (steps) in quarries and open pits, either simultaneously or with small delays. (ISRM)

multiple-row blasting—the drilling, charging, and firing of several rows of vertical holes along a quarry or opencast face. (ISRM)

muskeg—level, practically treeless areas supporting dense growth consisting primarily of grasses. The surface of the soil is covered with a layer of partially decayed grass and grass roots which is usually wet and soft when not frozen.

mylonite—a microscopic breccia with flow structure formed in fault zones. (ISRM)

natural frequency—the frequency at which a body or system vibrates when unconstrained by external forces. (ISRM)

natural frequency (displacement resonance), f_n—frequency for which phase angle is 90° between the direction of the excited force (or torque) vector and the direction of the excited excursion vector.

neat cement grout—a mixture of hydraulic cement and water without any added aggregate or filler materials.

Discussion—This may or may not contain admixture.

neutral stress—see **stress.**

newtonian fluid—a true fluid that tends to exhibit constant viscosity at all rates of shear.

node—point, line, or surface of standing wave system at which the amplitude is zero.

normal force—a force directed normal to the surface element across which it acts. (ISRM)

normal stress—see **stress.**

normally consolidated soil deposit—a soil deposit that has never been subjected to an effective pressure greater than the existing overburden pressure.

no-slump grout—grout with a slump of 1 in. (25 mm) or less according to the standard slump test (Test Method C 143).[2] See also **slump** and **slump test.**

open cut—an excavation through rock or soil made through a hill or other topographic feature to facilitate the passage of a highway, railroad, or waterway along an alignment that varies in topographic relief. An open cut can be comprised of single slopes or multiple slopes, or multiple slopes and horizontal benches, or both. (ISRM)

optimum moisture content (optimum water content), *OMC*, w_o (D)—the water content at which a soil can be compacted to a maximum dry unit weight by a given compactive effort.

organic clay—a clay with a high organic content.

organic silt—a silt with a high organic content.

organic soil—soil with a high organic content.

Discussion—In general, organic soils are very compressible and have poor load-sustaining properties.

organic terrain—see **peatland.**

oscillation—the variation, usually with time, of the magnitude of a quantity with respect to a specified reference when the magnitude is alternately greater and smaller than the reference.

outcrop—the exposure of the bedrock at the surface of the ground. (ISRM)

overbreak—the quantity of rock that is excavated or breaks out beyond the perimeter specified as the finished excavated tunnel outline. (ISRM)

overburden—the loose soil, sand, silt, or clay that overlies bedrock. In some usages it refers to all material overlying the point of interest (tunnel crown), that is, the total cover of soil and rock overlying an underground excavation. (ISRM)

overburden load—the load on a horizontal surface underground due to the column of material located vertically above it. (ISRM)

overconsolidated soil deposit—a soil deposit that has been subjected to an effective pressure greater than the present overburden pressure.

overconsolidation ratio, OCR—the ratio of preconsolidation vertical stress to the current effective overburden stress.

packer—*in grouting,* a device inserted into a hole in which grout or water is to be injected which acts to prevent return of the grout or water around the injection pipe; usually an expandable device actuated mechanically, hydraulically, or pneumatically.

---

[2] *Annual Book of ASTM Standards,* Vol 04.02.

paddle mixer—a mixer consisting essentially of a trough within which mixing paddles revolve about the horizontal axis, or a pan within which mixing blades revolve about the vertical axis.

pan mixer—a mixer comprised of a horizontal pan or drum in which mixing is accomplished by means of the rotating pan of fixed or rotating paddles, or both; rotation is about a vertical axis.

parent material—material from which a soil has been derived.

particle-size analysis—see **grain-size analysis**.

particle-size distribution—see **gradation, grain-size distribution**.

particulate grout—any grouting material characterized by undissolved (insoluble) particles in the mix. See also **chemical grout**.

passive earth pressure—see **earth pressure**.

passive state of plastic equilibrium—see **plastic equilibrium**.

path percolation (line of creep)—the path that water follows along the surface of contact between the foundation soil or rock and the base of a dam or other structure.

pavement pumping—ejection of soil and water mixtures from joints, cracks, and edges of rigid pavements, under the action of traffic.

peak shear strength—maximum shear strength along a failure surface. (ISRM)

peat—a naturally occurring highly organic substance derived primarily from plant materials.

Discussion—Peat is distinguished from other organic soil materials by its lower ash content (less than 25 % ash by dry weight) and from other phytogenic material of higher rank (that is, lignite coal) by its lower calorific value on a water saturated basis.

peatland—areas having peat-forming vegetation on which peak has accumulated or is accumulating.

penetrability—a grout property descriptive of its ability to fill a porous mass; primarily a function of lubricity and viscosity.

penetration—depth of hole cut in rock by a drill bit. (ISRM)

penetration grouting—filling joints or fractures in rock or pore spaces in soil with a grout without disturbing the formation; this grouting method does not modify the solid formation structure. See also **displacement grouting**.

penetration resistance (standard penetration resistance) (Proctor penetration resistance), p_R, N (FL^{-2} or Blows L^{-1})—(*a*) number of blows of a hammer of specified weight falling a given distance required to produce a given penetration into soil of a pile, casing, or sampling tube.

(*b*) unit load required to maintain constant rate of penetration into soil of a probe or instrument.

(*c*) unit load required to produce a specified penetration into soil at a specified rate of a probe or instrument. For a Proctor needle, the specified penetration is 2½ in. (63.5 mm) and the rate is ½ in. (12.7 mm)/s.

penetration resistance curve (Proctor penetration curve)—the curve showing the relationship between: (*1*) the penetration resistance, and (*2*) the water content.

percent compaction—the ratio, expressed as a percentage, of: (*1*) dry unit weight of a soil, to (*2*) maximum unit weight obtained in a laboratory compaction test.

percent consolidation—see **degree of consolidation**.

percent fines—amount, expressed as a percentage by weight, of a material in aggregate finer than a given sieve, usually the No. 200 (74 μm) sieve.

percent saturation (degree of saturation), S,S_r (D)—the ratio, expressed as a percentage, of: (*1*) the volume of water in a given soil or rock mass, to (*2*) the total volume of intergranular space (voids).

perched ground water—confined ground water separated from an underlying body of ground water by an unsaturated zone.

perched water table—a water table usually of limited area maintained above the normal free water elevation by the presence of an intervening relatively impervious confining stratum.

perched water table—groundwater separated from an underlying body of groundwater by unsaturated soil or rock. Usually located at a higher elevation than the groundwater table. (ISRM)

percolation—the movement of gravitational water through soil (see **seepage**).

percolation—movement, under hydrostatic pressure of water through the smaller interstices of rock or soil, excluding movement through large openings such as caves and solution channels. (ISRM)

percussion drilling—a drilling technique that uses solid or hollow rods for cutting and crushing the rock by repeated blows. (ISRM)

percussion drilling—a drilling process in which a hole is advanced by using a series of impacts to the drill steel and attached bit; the bit is normally rotated during drilling. See **rotary drilling**.

period—time interval occupied by one cycle.

permafrost—perennially frozen soil.

permanent strain—the strain remaining in a solid with respect to its initial condition after the application and removal of stress greater than the yield stress (commonly also called "residual" strain). (ISRM)

permeability—see **coefficient of permeability**.

permeability—the capacity of a rock to conduct liquid or gas. It is measured as the proportionality constant, k, between flow velocity, v, and hydraulic gradient, I; $v = k \cdot I$. (ISRM)

permeation grouting—filling joints or fractures in rock or pore spaces in soil with a grout, without disturbing the formation.

pH, pH (D)—an index of the acidity or alkalinity of a soil in terms of the logarithm of the reciprocal of the hydrogen ion concentration.

phase difference—difference between phase angles of two waves of same frequency.

phase of periodic quantity—fractional part of period through which independent variable has advanced, measured from an arbitrary origin.

phreatic line—the trace of the phreatic surface in any selected plane of reference.

phreatic line—see **line of seepage**.

phreatic surface—see **free water elevation**.

phreatic water—see **free water**.

piezometer—an instrument for measuring pressure head.

piezometric line (equipotential line)—line along which water will rise to the same elevation in piezometric tubes.

piezometric surface—the surface at which water will stand in a series of piezometers.

piezometric surface—an imaginary surface that everywhere coincides with the static level of the water in the aquifer. (ISRM)

pile—relatively slender structural element which is driven, or otherwise introduced, into the soil, usually for the purpose of providing vertical or lateral support.

pillar—in-situ rock between two or more underground openings: crown pillars; barrier pillars; rib pillars; sill pillars; chain pillars; etc. (ISRM)

pilot drift (pioneer tunnel)—a drift or tunnel first excavated as a smaller section than the dimensions of the main tunnel. A pilot drift or tunnel is usually used to investigate rock conditions in advance of the main tunnel, to permit installation of bracing before the principal mass of rock is removed, or to serve as a drainage tunnel. (ISRM)

piping—the progressive removal of soil particles from a mass by percolating water, leading to the development of channels.

pit—an excavation in the surface of the earth from which ore is obtained as in large open pit mining or as an excavation made for test purposes, that is, a testpit. (ISRM)

plane of weakness—surface or narrow zone with a (shear or tensile) strength lower than that of the surrounding material. (ISRM)

plane stress (strain)—a state of stress (strain) in a solid body in which all stress (strain) components normal to a certain plane are zero. (ISRM)

plane wave—wave in which fronts are parallel to plane normal to direction of propagation.

plastic deformation—see plastic flow.

plastic equilibrium—state of stress within a soil or rock mass or a portion thereof, which has been deformed to such an extent that its ultimate shearing resistance is mobilized.

active state of plastic equilibrium—plastic equilibrium obtained by an expansion of a mass.

passive state of plastic equilibrium—plastic equilibrium obtained by a compression of a mass.

plastic flow (plastic deformation)—the deformation of a plastic material beyond the point of recovery, accompanied by continuing deformation with no further increase in stress.

plasticity—the property of a soil or rock which allows it to be deformed beyond the point of recovery without cracking or appreciable volume change.

plasticity—property of a material to continue to deform indefinitely while sustaining a constant stress. (ISRM)

plasticity index, I_p, PI, I_w (D)—numerical difference between the liquid limit and the plastic limit.

plasticizer—*in grouting*, a material that increases the plasticity of a grout, cement paste, or mortar.

plastic limit, w_p, PL, P_w (D)—(*a*) the water content corresponding to an arbitrary limit between the plastic and the semisolid states of consistency of a soil. (*b*) water content at which a soil will just begin to crumble when rolled into a thread approximately ⅛ in. (3.2 mm) in diameter.

plastic soil—a soil that exhibits plasticity.

plastic state (plastic range)—the range of consistency within which a soil or rock exhibits plastic properties.

Poisson's ratio, (ν)—ratio between linear strain changes perpendicular to and in the direction of a given uniaxial stress change.

pore pressure (pore water pressure)—see **neutral** stress under stress.

pore water—water contained in the voids of the soil or rock.

porosity, n (D)—the ratio, usually expressed as a percentage, of: (*1*) the volume of voids of a given soil or rock mass, to (*2*) the total volume of the soil or rock mass.

porosity—the ratio of the aggregate volume of voids or interstices in a rock or soil to its total volume. (ISRM)

portal—the surface entrance to a tunnel. (ISRM)

positive displacement pump—a pump that will continue to build pressure until the power source is stalled if the pump outlet is blocked.

potential drop, Δh (L)—the difference in total head between two equipotential lines.

power spectral density—the limiting mean-square value (for example, of acceleration, velocity, displacement, stress, or other random variable) per unit bandwidth, that is the limit of the mean-square value in a given rectangular bandwidth divided by the bandwidth, as the bandwidth approaches zero.

pozzolan—a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties.

preconsolidation pressure (prestress), p_c (FL^{-2})—the greatest effective pressure to which a soil has been subjected.

preplaced aggregate concrete—concrete produced by placing coarse aggregate in a form and later injecting a portland cement-sand or resin grout to fill the interstices.

pressure, p (FL^{-2})—the load divided by the area over which it acts.

pressure bulb—the zone in a loaded soil or rock mass bounded by an arbitrarily selected isobar of stress.

pressure testing—a method of permeability testing with water or grout pumped downhole under pressure.

pressure-void ratio curve (compression curve)—a curve representing the relationship between effective pressure and void ratio of a soil as obtained from a consolidation test. The curve has a characteristic shape when plotted on semilog paper with pressure on the log scale. The various parts of the curve and extensions to the parts of the curve and extensions to the parts have been designated as recompression, compression, virgin compression, expansion, rebound, and other descriptive names by various authorities.

pressure washing—the cleaning of soil or rock surfaces accomplished by injection of water, air, or other liquids, under pressure.

primary consolidation (primary compression) (primary time effect)—see consolidation.

primary hole—*in grouting*, the first series of holes to be drilled and grouted, usually at the maximum allowable spacing.

primary lining—the lining first placed inside a tunnel or shaft, usually used to support the excavation. The primary lining may be of wood or steel sets with steel or wood lagging or rock bolts and shot-crete. (ISRM)

primary permeability—internal permeability of intack rock; intergranular permeability (not permeability due to fracturing).

primary porosity—the porosity that developed during the final stages of sedimentation or that was present within sedimentary particles at the time of deposition.

primary state of stress—the stress in a geological formation before it is disturbed by man-made works. (ISRM)

principal plane—each of three mutually perpendicular planes through a point in a soil mass on which the shearing stress is zero.

intermediate principal plane—the plane normal to the direction of the intermediate principal stress.

major principal plane—the plane normal to the direction of the major principal stress.

minor principal plane—the plane normal to the direction of the minor principal stress.

principal stress—see **stress.**

principal stress (strain)—the stress (strain) normal to one of three mutually perpendicular planes on which the shear stresses (strains) at a point in a body are zero. (ISRM)

Proctor compaction curve—see **compaction curve.**

Proctor penetration curve—see **penetration resistance curve.**

Proctor penetration resistance—see **penetration resistance.**

profile—see **soil profile.**

progressive failure—failure in which the ultimate shearing resistance is progressively mobilized along the failure surface.

progressive failure—formation and development of localized fractures which, after additional stress increase, eventually form a continuous rupture surface and thus lead to failure after steady deterioration of the rock. (ISRM)

proportioning pump—see **metering pump.**

proprietary—made and marketed by one having the exclusive right to manufacture and sell; privately owned and managed.

protective filter—see **filter.**

pumpability—*in grouting,* a measure of the properties of a particular grout mix to be pumped as controlled by the equipment being used, the formation being injected, and the engineering objective limitations.

pumping of pavement (pumping)—see **pavement pumping.**

pumping test—a field procedure used to determine in situ permeability or the ability of a formation to accept grout.

pure shear—a state of strain resulting from that stress condition most easily described by a Mohr circle centered at the origin. (ISRM)

quarry—an excavation in the surface of the earth from which stone is obtained for crushed rock or building stone. (ISRM)

Quer-wave (love wave), W—dispersive surface wave with one horizontal component, generally normal to the direction of propagation, which decreases in propagation velocity with increase in frequency.

quick condition (quicksand)—condition in which water is flowing upwards with sufficient velocity to reduce significantly the bearing capacity of the soil through a decrease in intergranular pressure.

quick test—see **unconsolidated undrained test.**

radius of influence of a well—distance from the center of the well to the closest point at which the piezometric surface is not lowered when pumping has produced the maximum steady rate of flow.

raise—upwardly constructed shaft; that is, an opening, like a shaft, made in the roof of one level to reach a level above. (ISRM)

range (of a deformation-measuring instrument)—the amount between the maximum and minimum quantity an instrument can measure without resetting. In some instances provision can be made for incremental extension of the range.

Rayleigh wave, v_R (LT^{-1})—dispersive surface wave in which element has retrograding elliptic orbit with one major vertical and one minor horizontal component both in plane of propagation velocity:

$$v_R = \alpha v_t \text{ with } 0.910 < \alpha < 0.995 \text{ for } 0.25 < \nu < 0.5$$

reactant—*in grouting,* a material that reacts chemically with the base component of grout system.

reactive aggregate—an aggregate containing siliceous material (usually in amorphous or crypto-crystalline state) which can react chemically with free alkali in the cement.

Discussion—The reaction can result in expansion of the hardened material, frequently to a damaging extent.

reflected (or refracted) wave—components of wave incident upon second medium and reflected into first medium (or refracted) into second medium.

reflection and refraction loss—that part of transmitted energy lost due to nonuniformity of mediums.

refusal—*in grouting,* when the rate of grout take is low, or zero, at a given pressure.

relative consistency, I_c, C_r (D)—ratio of: (*1*) the liquid limit minus the natural water content, to (*2*) the plasticity index.

relative density, D_d, I_D (D)—the ratio of (*1*) the difference between the void ratio of a cohesionless soil in the loosest state and any given void ratio, to (*2*) the difference between the void ratios in the loosest and in the densest states.

relative water content—see **liquidity index.**

remodeled soil—soil that has had its natural structure modified by manipulation.

remolding index, I_R (D)—the ratio of: (*1*) the modulus of deformation of a soil in the undisturbed state, to (*2*) the modulus of deformation of the soil in the remolded state.

remodeling sensitivity (sensitivity ratio), S_t (D)—the ratio of: (*1*) the unconfined compressive strength of an undisturbed specimen of soil, to (*2*) the unconfined compressive strength of a specimen of the same soil after remolding at unaltered water content.

residual soil—soil derived in place by weathering of the underlying material.

residual strain—the strain in a solid associated with a state of residual stress. (ISRM)

residual stress—stress remaining in a solid under zero external stress after some process that causes the dimensions of the various parts of the solid to be incompatible under zero stress, for example, (*1*) deformation under the action of external stress when some parts of the body suffer permanent strain; or (*2*) heating or cooling of a body in which the thermal expansion coefficient is not uniform throughout the body. (ISRM)

resin—*in grouting*, a material that usually constitutes the base of an organic grout system.

resin grout—a grout system composed of essentially resinous materials such as epoxys, polyesters, and urethanes.

Discussion—In Europe, this refers to any chemical grout system regardless of chemica. .rigin.

resolution (of a deformation-measuring instrument)—the ratio of the smallest divisional increment of the indicating scale to the sensitiv..y of the i. .trument. Interpolation within the increment may be possible, but is not recommended in specifying resolution.

resonance—the reinforced vibration of a body exposed to the vibration, at about the frequency, of another body.

resonant frequency—a frequency at which resonance exists.

response—the motion (or other output) in a device or system resulting from an excitation (stimulus) under specified conditions.

retard—bank-protection structure designed to reduce the riparian velocity and induce silting or accretion.

retardation—delay in deformation. (ISRM)

retarder—a material that slows the rate at which chemical reactions would otherwise occur.

reverse circulation—a drilling system in which the circulating medium flows down through the annulus and up through the drill rod, that is, in the reverse of the normal direction of flow.

revetment—bank protection by armor, that is, by facing of a bank or embankment with erosion-resistant material.

riprap stone—(generally less than 1 ton in weigh., specially selected and graded quarried stone placed to prevent erosion through wave action, tidal forces, or strong currents and thereby preserve the shape of a surface, slope, or underlying structure.

rise time (pulse rise time)—the interval of time required for the leading edge of a pulse to rise from some specified small fraction to some specified larger fraction of the maximum value.

rock—natural solid mineral matter occurring in large masses or fragments.

rock—any naturally formed aggregate of mineral matter occurring in large masses or fragments. (ISRM)

rock anchor—a steel rod or cable installed in a hole in rock; in principle the same as rock bolt, but generally used for rods longer than about four metres. (ISRM)

rock bolt—a steel rod placed in a hole drilled in rock used to tie the rock together. One end of the rod is firmly anchored in the hole by means of a mechanical device or grout, or both, and the threaded projecting end is equipped with a nut and plate that bears against the rock surface. The rod can be pretensioned. (ISRM)

rock burst—a sudden and violent expulsion of rock from its surroundings that occurs when a volume of rock is strained beyond the elastic limit and the accompanying failure is of such a nature that accumulated energy is released instantaneously.

rock burst—sudden explosive-like release of energy due to the failure of a brittle rock of high strength. (ISRM)

rock flour—see **silt**.

rock mass—rock as it occurs in situ, including its structural discontinuities. (ISRM)

rock mechanics—the application of the knowledge of the mechanical behavior of rock to engineering problems dealing with rock. Rock mechanics overlaps with structural geology, geophysics, and soil mechanics.

rock mechanics—theoretical and applied science of the mechanical behaviour of rock. (ISRM)

roof—top of excavation or underground opening, particularly applicable in bedded rocks where the top surface of the opening is flat rather than arched. (ISRM)

rotary drilling—a drilling process in which a hole is advanced by rotation of a drill bit under constant pressure without impact. See **percussion drilling**.

round—a set of holes drilled and charged in a tunnel or quarry that are fired instantaneously or with short-delay detonators. (ISRM)

running ground—*in tunneling*, a granular material that tends to flow or "run" into the excavation.

rupture—that stage in the development of a fracture where instability occurs. It is not recommended that the term rupture be used in rock mechanics as a synonym for fracture. (ISRM)

rupture envelope (*rupture line*)—see **Mohr envelope**.

sagging—usually occurs in sedimentary rock formations as a separation and downward bending of sedimentary beds in the roof of an underground opening. (ISRM)

sand—particles of rock that will pass the No. 4 (4.75-mm) sieve and be retained on the No. 200 (75-μm) U.S. standard sieve.

sand boil—the ejection of sand and water resulting from piping.

sand equivalent—a measure of the amount of silt or clay contamination in fine aggregate as determined by test (Test Method D 2419).[3]

sanded grout—grout in which sand is incorporated into the mixture.

sapric peat—peat in which the original plant fibers are highly decomposed (less than 33 % fibers).

saturated unit weight—see **unit weight**.

saturation curve—see **zero air voids curve**.

scattering loss—that part of transmitted energy lost due to roughness of reflecting surface.

schistosity—the variety of foliation that occurs in the coarser-grained metamorphic rocks and is generally the result of the parallel arrangement of platy and ellipsoidal mineral grains within the rock substance. (ISRM)

secant modulus—slope of the line connecting the origin and a given point on the stress-strain curve. (ISRM)

secondary consolidation (*secondary compression*) (*secondary time effect*)—see **consolidation**.

secondary hole—*in grouting*, the second series of holes to be drilled and grouted usually spaced midway between primary holes.

secondary lining—the second-placed, or permanent, structural lining of a tunnel, which may be of concrete, steel, or masonry. (ISRM)

[3] *Annual Book of ASTM Standards*, Vol 04.03.

secondary state of stress—the resulting state of stress in the rock around man-made excavations or structures. (ISRM)

sediment basin—a structure created by construction of a barrier or small dam-like structure across a waterway or by excavating a basin or a combination of both to trap or restrain sediment.

seep—a small area where water oozes from the soil or rock.

seepage—the infiltration or percolation of water through rock or soil to or from the surface. The term seepage is usually restricted to the very slow movement of ground water. (ISRM)

seepage (percolation)—the slow movement of gravitational water through the soil or rock.

seepage force—the frictional drag of water flowing through voids or interstices in rock, causing an increase in the intergranular pressure, that is, the hydraulic force per unit volume of rock or soil which results from the flow of water and which acts in the direction of flow. (ISRM)

seepage force, J (F)—the force transmitted to the soil or rock grains by seepage.

seepage line—see **line of seepage.**

seepage velocity, V_s, V_t (LT^{-1})—the rate of discharge of seepage water through a porous medium per unit area of void space perpendicular to the direction of flow.

segregation—*in grouting*, the differential concentration of the components of mixed grout, resulting in nonuniform proportions in the mass.

seismic support—mass (heavy) supported on springs (weak) so that mass remains almost at rest when free end of springs is subjected to sinusoidal motion at operating frequency.

seismic velocity—the velocity of seismic waves in geological formations. (ISRM)

seismometer—instrument to pick up linear (vertical, horizontal) or rotational displacement, velocity, or acceleration.

self-stressing grout—expansive-cement grout in which the expansion induces compressive stress in grout if the expansion movement is restrained.

sensitivity—the effect of remolding on the consistency of a cohesive soil.

sensitivity (of an instrument)—the differential quotient dQ_0/dQ_1, where Q_0 is the scale reading and Q_1 is the quantity to be measured.

sensitivity (of a transducer)—the differential quotient dQ_0/dQ_1, where Q_0 is the output and Q_1 is the input.

series grouting—similar to stage grouting, except each successively deeper zone is grouted by means of a newly drilled hole, eliminating the need for washing grout out before drilling the hole deeper.

set—*in grouting*, the condition reached by a cement paste, or grout, when it has lost plasticity to an arbitrary degree, usually measured in terms of resistance to penetration or deformation; initial set refers to first stiffening and final set refers to an attainment of significant rigidity.

setting shrinkage—*in grouting*, a reduction in volume of grout prior to the final set of cement caused by bleeding, by the decrease in volume due to the chemical combination of water with cement, and by syneresis.

set time—(*1*) the hardening time of portland cement; or (*2*) the gel time for a chemical grout.

shaft—generally a vertical or near vertical excavation driven downward from the surface as access to tunnels, chambers, or other underground workings. (ISRM)

shaking test—a test used to indicate the presence of significant amounts of rock flour, silt, or very fine sand in a fine-grained soil. It consists of shaking a pat of wet soil, having a consistency of thick paste, in the palm of the hand; observing the surface for a glossy or livery appearance; then squeezing the pat; and observing if a rapid apparent drying and subsequent cracking of the soil occurs.

shear failure (failure by rupture)—failure in which movement caused by shearing stresses in a soil or rock mass is of sufficient magnitude to destroy or seriously endanger a structure.

general shear failure—failure in which the ultimate strength of the soil or rock is mobilized along the entire potential surface of sliding before the structure supported by the soil or rock is impaired by excessive movement.

local shear failure—failure in which the ultimate shearing strength of the soil or rock is mobilized only locally along the potential surface of sliding at the time the structure supported by the soil or rock is impaired by excessive movement.

shear force—a force directed parallel to the surface element across which it acts. (ISRM)

shear plane—a plane along which failure of material occurs by shearing. (ISRM)

shear resistance—see **internal friction.**

shear strain—the change in shape, expressed by the relative change of the right angles at the corner of what was in the undeformed state an infinitesimally small rectangle or cube. (ISRM)

shear strength, s, T_f (FL^{-2})—the maximum resistance of a soil or rock to shearing stresses. See **peak shear strength.**

shear stress—stress directed parallel to the surface element across which it acts. (ISRM)

shear stress (*shearing stress*) (*tangential stress*)—see **stress.**

shear wave (rotational, equivoluminal)—wave in which medium changes shape without change of volume (shear-plane wave in isotropic medium is transverse wave).

shelf life—maximum time interval during which a material may be stored and remain in a usable condition; usually related to storage conditions.

shock pulse—a substantial disturbance characterized by a rise of acceleration from a constant value and decay of acceleration to the constant value in a short period of time.

shock wave—a wave of finite amplitude characterized by a shock front, a surface across which pressure, density, and internal energy rise almost discontinuously, and which travels with a speed greater than the normal speed of sound. (ISRM)

shotcrete—mortar or concrete conveyed through a hose and pneumatically projected at high velocity onto a surface. Can be applied by a "wet" or "dry" mix method. (ISRM)

shrinkage-compensating—*in grouting*, a characteristic of grout made using an expansive cement in which volume increase, if restrained, induces compressive stresses that are intended to offset the tendency of drying shrinkage to induce tensile stresses. See also **self-stressing grout.**

shrinkage index, SI (D)—the numerical difference between the plastic and shrinkage limits.

shrinkage limit, SL, w_s (D)—the maximum water content at which a reduction in water content will not cause a decrease in volume of the soil mass.

shrinkage ratio, R (D)—the ratio of: (*1*) a given volume change, expressed as a percentage of the dry volume, to (*2*) the corresponding change in water content above the shrinkage limit, expressed as a percentage of the weight of the oven-dried soil.

sieve analysis—determination of the proportions of particles lying within certain size ranges in a granular material by separation on sieves of different size openings.

silt (inorganic silt) (rock flour)—material passing the No. 200 (75-μm) U.S. standard sieve that is nonplastic or very slightly plastic and that exhibits little or no strength when air-dried.

silt size—that portion of the soil finer than 0.02 mm and coarser than 0.002 mm (0.05 mm and 0.005 mm in some cases).

simple shear—shear strain in which displacements all lie in one direction and are proportional to the normal distances of the displaced points from a given reference plane. The dilatation is zero. (ISRM)

single-grained structure—see **soil structure.**

size effect—influence of specimen size on its strength or other mechanical parameters. (ISRM)

skin friction, f (FL^{-2})—the frictional resistance developed between soil and an element of structure.

slabbing—the loosening and breaking away of relatively large flat pieces of rock from the excavated surface, either immediately after or some time after excavation. Often occurring as tensile breaks which can be recognized by the subconchoidal surfaces left on remaining rock surface. (ISRM)

slabjacking—*in grouting,* injection of grout under a concrete slab in order to raise it to a specified grade.

slaking—deterioration of rock on exposure to air or water.

slaking—the process of breaking up or sloughing when an indurated soil is immersed in water.

sleeved grout pipe—see **tube A manchette.**

sliding—relative displacement of two bodies along a surface, without loss of contact between the bodies. (ISRM)

slope—the excavated rock surface that is inclined to the vertical or horizontal, or both, as in an open-cut. (ISRM)

slow test—see **consolidated-drain test.**

slump—a measure of consistency of freshly mixed concrete or grout. See also **slump test.**

slump test—the procedure for measuring slump (Test Method C 143).[2]

slurry cutoff wall—a vertical barrier constructed by excavating a vertical slot under a bentonite slurry and backfilling it with materials of low permeability for the purpose of the containment of the lateral flow of water and other fluids.

slurry grout—a fluid mixture of solids such as cement, sand, or clays in water.

slurry trench—a trench that is kept filled with a bentonite slurry during the excavation process to stabilize the walls of the trench.

slush grouting—application of cement slurry to surface rock as a means of filling cracks and surface irregularities or to prevent slaking; it is also applied to riprap to form grouted riprap.

smooth (-wall) blasting—a method of accurate perimeter blasting that leaves the remaining rock practically undamaged. Narrowly spaced and lightly charged blastholes, sometimes alternating with empty dummy holes, located along the breakline and fired simultaneously as the last round of the excavation. (ISRM)

soil (earth)—sediments or other unconsolidated accumulations of solid particles produced by the physical and chemical disintegration of rocks, and which may or may not contain organic matter.

soil binder—see **binder.**

soil-forming factors—factors, such as parent material, climate, vegetation, topography, organisms, and time involved in the transformation of an original geologic deposit into a soil profile.

soil horizon—see **horizon.**

soil mechanics—the application of the laws and principles of mechanics and hydraulics to engineering problems dealing with soil as an engineering material.

soil physics—the organized body of knowledge concerned with the physical characteristics of soil and with the methods employed in their determinations.

soil profile (profile)—vertical section of a soil, showing the nature and sequence of the various layers, as developed by deposition or weathering, or both.

soil stabilization—chemical or mechanical treatment designed to increase or maintain the stability of a mass of soil or otherwise to improve its engineering properties.

soil structure—the arrangement and state of aggregation of soil particles in a soil mass.

flocculent structure—an arrangement composed of flocs of soil particles instead of individual soil particles.

honeycomb structure—an arrangement of soil particles having a comparatively loose, stable structure resembling a honeycomb.

single-grained structure—an arrangement composed of individual soil particles; characteristic structure of coarse-grained soils.

soil suspension—highly diffused mixture of soil and water.

soil texture—see **gradation.**

solution cavern—openings in rock masses formed by moving water carrying away soluble materials.

sounding well—*in grouting,* a vertical conduit in a mass of coarse aggregate for preplaced aggregate concrete which contains closely spaced openings to permit entrance of grout.

Discussion—The grout level is determined by means of a measuring line on a float within the sounding well.

spacing—the distance between adjacent blastholes in a direction parallel to the face. (ISRM)

spalling—(*1*) longitudinal splitting in uniaxial compression, or (*2*) breaking-off of plate-like pieces from a free rock surface. (ISRM)

specific gravity:

specific gravity of solids, G, G_s, S_s (D)—ratio of: (*1*) the weight in air of a given volume of solids at a stated temperature to (*2*) the weight in air of an equal volume of distilled water at a stated temperature.

apparent specific gravity, G_a, S_a (D)—ratio of: (*1*) the weight in air of a given volume of the impermeable portion of a permeable material (that is, the solid matter including its impermeable pores or voids) at a stated temperature to (*2*) the weight in air of an equal volume of distilled water at a stated temperature.

bulk specific gravity (*specific mass gravity*), G_m, S_m (D)—ratio of: (*1*) the weight in air of a given volume of a permeable material (including both permeable and impermeable voids normal to the material) at a stated temperature to (*2*) the weight in air of an equal volume of distilled water at a stated temperature.

specific surface (L^{-1})—the surface area per unit of volume of soil particles.

spherical wave—wave in which wave fronts are concentric spheres.

split spacing grouting—a grouting sequence in which initial (primary) grout holes are relatively widely spaced and subsequent grout holes are placed midway between previous grout holes to "split the spacing."; this process is continued until a specified hole spacing is achieved or a reduction in grout take to a specified value occurs, or both.

spring characteristics, c (FL^{-1})—ratio of increase in load to increase in deflection:

$$c = l/C$$

where:

C = compliance.

stability—the condition of a structure or a mass of material when it is able to support the applied stress for a long time without suffering any significant deformation or movement that is not reversed by the release of stress. (ISRM)

stability factor (stability number), N_s (D)—a pure number used in the analysis of the stability of a soil embankment, defined by the following equation:

$$N_s = H_c \gamma_e / c$$

where:

H_c = critical height of the sloped bank,
γ_e = effective unit of weight of the soil, and
c = cohesion of the soil

NOTE—Taylor's "stability number" is the reciprocal of Terzaghi's "stability factor."

stabilization—see **soil stabilization**.

stage—*in grouting*, the length of hole grouted at one time. See also **stage grouting**.

stage grouting—sequential grouting of a hole in separate steps or stages in lieu of grouting the entire length at once; holes may be grouted in ascending stages by using packers or in descending stages downward from the collar of the hole.

standard compaction—see **compaction test**.

standard penetration resistance—see **penetration resistance**.

standing wave—a wave produced by simultaneous transmission in opposite directions of two similar waves resulting in fixed points of zero amplitudes called nodes.

steady-state vibration—vibration in a system where the velocity of each particle is a continuing periodic quantity.

stemming—(*1*) the material (chippings, or sand and clay) used to fill a blasthole after the explosive charge has been inserted. Its purpose is to prevent the rapid escape of the explosion gases. (*2*) the act of pushing and tamping the material in the hole. (ISRM)

stick-slip—rapid fluctuations in shear force as one rock mass slides past another, characterized by a sudden slip between the rock masses, a period of no relative displacement between the two masses, a sudden slip, etc. The oscillations may be regular as in a direct shear test, or irregular as in a triaxial test.

sticky limit, T_w (D)—the lowest water content at which a soil will stick to a metal blade drawn across the surface of the soil mass.

stiffness—the ratio of change of force (or torque) to the corresponding change in translational (or rotational) deflection of an elastic element.

stiffness-force—displacement ratio. (ISRM)

stone—crushed or naturally angular particles of rock.

stop—*in grouting*, a packer setting at depth.

stop grouting—the grouting of a hole beginning at the lowest packer setting (stop) after the hole is drilled to total depth.

Discussion—Packers are placed at the top of the zone being grouted. Grouting proceeds from the bottom up. Also called upstage grouting.

strain, ϵ (D)—the change in length per unit of length in a given direction.

strain (linear or normal), ϵ (D)—the change in length per unit of length in a given direction.

strain ellipsoid—the representation of the strain in the form of an ellipsoid into which a sphere of unit radius deforms and whose axes are the principal axes of strain. (ISRM)

strain (stress) rate—rate of change of strain (stress) with time. (ISRM)

strain resolution (strain sensitivity), R_s (D)—the smallest subdivision of the indicating scale of a deformation-measuring device divided by the product of the sensitivity of the device and the gage length. The deformation resolution, R_d, divided by the gage length.

strain (stress) tensor—the second order tensor whose diagonal elements consist of the normal strain (stress) components with respect to a given set of coordinate axes and whose off-diagonal elements consist of the corresponding shear strain (stress) components. (ISRM)

streamline flow—see **laminar flow**.

strength—maximum stress which a material can resist without failing for any given type of loading. (ISRM)

stress, σ, p, f (FL^{-2})—the force per unit area acting within the soil mass.

effective stress (*effective pressure*) (*intergranular pressure*), $\bar{\sigma}$, f (FL^{-2})—the average normal force per unit area transmitted from grain to grain of a soil mass. It is the stress that is effective in mobilizing internal friction.

neutral stress (*pore pressure*) (*pore water pressure*), u, u_w (FL^{-2})—stress transmitted through the pore water (water filling the voids of the soil).

normal stress, σ, p (FL^{-2})—the stress component normal to a given plane.

principal stress, σ_1, σ_2, σ_3 (FL^{-2})—stresses acting normal to three mutually perpendicular planes intersecting at a point in a body, on which the shearing stress is zero.

major principal stress, σ_1 (FL^{-2})—the largest (with regard to sign) principal stress.

minor principal stress, σ_3 (FL^{-2})—the smallest (with regard to sign) principal stress.

intermediate principal stress, σ_2 (FL^{-2})—the principal stress whose value is neither the largest nor the smallest (with regard to sign) of the three.

shear stress (*shearing stress*) (*tangential stress*), τ, s FL^{-2})—the stress component tangential to a given plane.

total stress, σ, f (FL^{-2})—the total force per unit area acting within a mass of soil. It is the sum of the neutral and effective stresses.

stress ellipsoid—the representation of the state of stress in the form of an ellipsoid whose semi-axes are proportional to the magnitudes of the principal stresses and lie in the principal directions. The coordinates of a point P on this ellipse are proportional to the magnitudes of the respective components of the stress across the plane normal to the direction OP, where O is the center of the ellipsoid. (ISRM)

stress (strain) field—the ensemble of stress (strain) states defined at all points of an elastic solid. (ISRM)

stress relaxation—stress release due to creep. (ISRM)

strike—the direction or azimuth of a horizontal line in the plane of an inclined stratum, joint, fault, cleavage plane, or other planar feature within a rock mass. (ISRM)

structure—one of the larger features of a rock mass, like bedding, foliation, jointing, cleavage, or brecciation; also the sum total of such features as contrasted with texture. Also, in a broader sense, it refers to the structural features of an area such as anti-clines or synclines. (ISRM)

structure—see **soil structure**.

subbase—a layer used in a pavement system between the subgrade and base coarse, or between the subgrade and portland cement concrete pavement.

subgrade—the soil prepared and compacted to support a structure or a pavement system.

subgrade surface—the surface of the earth or rock prepared to support a structure or a pavement system.

submerged unit weight—see **unit weight**.

subsealing—*in grouting*, grouting under concrete slabs for the purpose of filling voids without raising the slabs.

subsidence—the downward displacement of the overburden (rock or soil, or both) lying above an underground excavation or adjoining a surface excavation. Also the sinking of a part of the earth's crust. (ISRM)

subsoil—(*a*) soil below a subgrade of fill. (*b*) that part of a soil profile occurring below the "A" horizon.

sulfate attack—*in grouting*, harmful or deleterious reactions between sulfates in soil or groundwater and the grout.

support—structure or structural feature built into an underground opening for maintaining its stability. (ISRM)

surface force—any force that acts across an internal or external surface element in a material body, not necessarily in a direction lying in the surface. (ISRM)

surface wave—a wave confined to a thin layer at the surface of a body. (ISRM)

suspension—a mixture of liquid and solid materials.

suspension agent—an additive that decreased the settlement rate of particles in liquid.

swamp—a forested or shrub covered wetland where standing or gently flowing water persists for long periods on the surface.

syneresis—*in grouting*, the exudation of liquid (generally water) from a set gel which is not stressed, due to the tightening of the grout material structure.

take—see **grout take**.

talus—rock fragments mixed with soil at the foot of a natural slope from which they have been separated.

tangential stress—see **stress**.

tangent modulus—slope of the tangent to the stress-strain curve at a given stress value (generally taken at a stress equal to half the compressive strength). (ISRM)

tensile strength (unconfined or uniaxial tensile strength), T_o (FL^{-2})—the load per unit area at which an unconfined cylindrical specimen will fail in a simple tension (pull) test.

tensile stress—normal stress tending to lengthen the body in the direction in which it acts. (ISRM)

tertiary hole—*in grouting*, the third series of holes to be drilled and grouted usually spaced midway between previously grouted primary and secondary holes.

texture—*of soil and rock*, geometrical aspects consisting of size, shape, arrangement, and crystallinity of the component particles and of the related characteristics of voids.

texture—the arrangement in space of the components of a rock body and of the boundaries between these components. (ISRM)

theoretical time curve—see **consolidation time curve**.

thermal spalling—the breaking of rock under stresses induced by extremely high temperature gradients. High-velocity jet flames are used for drilling blast holes with this effect. (ISRM)

thermo-osmosis—the process by which water is caused to flow in small openings of a soil mass due to differences in temperature within the mass.

thickness—the perpendicular distance between bounding surfaces such as bedding or foliation planes of a rock. (ISRM)

thixotropy—the property of a material that enables it to stiffen in a relatively short time on standing, but upon agitation or manipulation to change to a very soft consistency or to a fluid of high viscosity, the process being completely reversible.

throw—the projection of broken rock during blasting. (ISRM)

thrust—force applied to a drill in the direction of penetration. (ISRM)

tight—rock remaining within the minimum excavation lines after completion of a blasting record. (ISRM)

till—see **glacial till**.

time curve—see **consolidation time curve**.

time factor, T_v, T (D)—dimensionless factor, utilized in the theory of consolidation, containing the physical constants of a soil stratum influencing its time-rate of consolidation, expressed as follows:

$$T = k\,(1 + e)t/(a_v\gamma_w \cdot H^2) = (c_v \cdot t)/H^2$$

where:

k = coefficient of permeability (LT^{-1}),
e = void ratio (dimensionless),

t = elapsed time that the stratum has been consolidated (T),
a_v = coefficient of compressibility (L^2F^{-1}),
γ_w = unit weight of water (FL^{-3}),
H = thickness of stratum drained on one side only. If stratum is drained on both sides, its thickness equals $2H$ (L), and
c_v = coefficient of consolidation (L^2T^{-1}).

topsoil—surface soil, usually containing organic matter.

torsional shear test—a shear test in which a relatively thin test specimen of solid circular or annular cross-section, usually confined between rings, is subjected to an axial load and to shear in torsion. In-place torsion shear tests may be performed by pressing a dentated solid circular or annular plate against the soil and measuring its resistance to rotation under a given axial load.

total stress—see **stress**.

toughness index, I_T, T_w—the ratio of: (*1*) the plasticity index, to (*2*) the flow index.

traction, S_1, S_2, S_3 (FL^{-2})—applied stress.

transformed flow net—a flow net whose boundaries have been properly modified (transformed) so that a net consisting of curvilinear squares can be constructed to represent flow conditions in an anisotropic porous medium.

transported soil—soil transported from the place of its origin by wind, water, or ice.

transverse wave, v_t (LT^{-1})—wave in which direction of displacement of element of medium is parallel to wave front. The propagation velocity, v_t, is calculated as follows:

$$v_t = \sqrt{G/\rho} = \sqrt{\mu/\rho} = \sqrt{(E/\rho)[1/2(1 + \nu)]}$$

where:
G = shear modulus,
ρ = mass density,
ν = Poisson's ratio, and
E = Young's modulus.

transverse wave (shear wave)—a wave in which the displacement at each point of the medium is parallel to the wave front. (ISRM)

tremie—material placed under water through a tremie pipe in such a manner that it rests on the bottom without mixing with the water.

trench—usually a long, narrow, near vertical sided cut in rock or soil such as is made for utility lines. (ISRM)

triaxial compression—compression caused by the application of normal stresses in three perpendicular directions. (ISRM).

triaxial shear test (triaxial compression test)—a test in which a cylindrical specimen of soil or rock encased in an impervious membrane is subjected to a confining pressure and then loaded axially to failure.

triaxial state of stress—state of stress in which none of the three principal stresses is zero. (ISRM)

true solution—one in which the components are 100 % dissolved in the base solvent.

tube A manchette—*in grouting*, a grout pipe perforated with rings of small holes at intervals of about 12 in. (305 mm).

Discussion—Each ring of perforations is enclosed by a short rubber sleeve fitting tightly around the pipe so as to act as a one-way valve when used with an inner pipe containing two packer elements that isolate a stage for injection of grout.

tunnel—a man-made underground passage constructed without removing the overlying rock or soil. Generally nearly horizontal as opposed to a shaft, which is nearly vertical. (ISRM)

turbulent flow—that type of flow in which any water particle may move in any direction with respect to any other particle, and in which the head loss is approximately proportional to the second power of the velocity.

ultimate bearing capacity, q_o, q_{ult} (FL^{-2})—the average load per unit of area required to produce failure by rupture of a supporting soil or rock mass.

unconfined compressive strength—the load per unit area at which an unconfined prismatic or cylindrical specimen of material will fail in a simple compression test without lateral support.

unconfined compressive strength—see **compressive strength**.

unconsolidated-undrained test (quick test)—a soil test in which the water content of the test specimen remains practically unchanged during the application of the confining pressure and the additional axial (or shearing) force.

undamped natural frequency—*of a mechanical system*, the frequency of free vibration resulting from only elastic and inertial forces of the system.

underconsolidated soil deposit—a deposit that is not fully consolidated under the existing overburden pressure.

undisturbed sample—a soil sample that has been obtained by methods in which every precaution has been taken to minimize disturbance to the sample.

uniaxial (unconfined) compression—compression caused by the application of normal stress in a single direction. (ISRM)

uniaxial state of stress—state of stress in which two of the three principal stresses are zero. (ISRM)

unit weight, γ (FL^{-3})—weight per unit volume (with this, and all subsequent unit-weight definitions, the use of the term weight means force).

dry unit weight (*unit dry weight*), γ_d, γ_s (FL^{-3})—the weight of soil or rock solids per unit of total volume of soil or rock mass.

effective unit weight, γ_e (FL^{-3})—that unit weight of a soil or rock which, when multiplied by the height of the overlying column of soil or rock, yields the effective pressure due to the weight of the overburden.

maximum unit weight, γ_{max} (FL^{-3})—the dry unit weight defined by the peak of a compaction curve.

saturated unit weight, γ_G, γ_{sat} (FL^{-3})—the wet unit weight of a soil mass when saturated.

submerged unit weight (*buoyant unit weight*), γ_m, γ', γ_{sub} (FL^{-3})—the weight of the solids in air minus the weight of water displaced by the solids per unit of volume of soil or rock mass; the saturated unit weight minus the unit weight of water.

unit weight of water, γ_w (FL^{-3})—the weight per unit volume of water; nominally equal to 62.4 lb/ft^3 or 1 g/cm^3.

wet unit weight (*mass unit weight*), γ_m, γ_{wet} (FL^{-3})—the weight (solids plus water) per unit of total volume of soil or rock mass, irrespective of the degree of saturation.

zero air voids unit weight, γ_z, γ_s (FL^{-3})—the weight of solids per unit volume of a saturated soil or rock mass.

unloading modulus—slope of the tangent to the unloading stress-strain curve at a given stress value. (ISRM)

uplift—the upward water pressure on a structure.

	Symbol	Unit
unit symbol	u	FL^{-2}
total symbol	U	F or FL^{-1}

uplift—the hydrostatic force of water exerted on or underneath a structure, tending to cause a displacement of the structure. (ISRM)

uplift—*in grouting*, vertical displacement of a formation due to grout injection.

vane shear test—an in-place shear test in which a rod with thin radial vanes at the end is forced into the soil and the resistance to rotation of the rod is determined.

varved clay—alternating thin layers of silt (or fine sand) and clay formed by variations in sedimentation during the various seasons of the year, often exhibiting contrasting colors when partially dried.

vent hole—*in grouting*, a hole drilled to allow the escape of air and water and also used to monitor the flow of grout.

vent pipe—*in grouting*, a small-diameter pipe used to permit the escape of air, water, or diluted grout from a formation.

vibrated beam wall (injection beam wall)—barrier formed by driving an H-beam in an overlapping pattern of prints and filling the print of the beam with cement-bentonite slurry or other materials as it is withdrawn.

vibration—an oscillation wherein the quantity is a parameter that defines the motion of a mechanical system (see **oscillation**).

virgin compression curve—see **compression curve**.

viscoelasticity—property of materials that strain under stress partly elastically and partly viscously, that is, whose strain is partly dependent on time and magnitude of stress. (ISRM)

viscosity—the internal fluid resistance of a substance which makes it resist a tendency to flow.

viscous damping—the dissipation of energy that occurs when a particle in a vibrating system is resisted by a force that has a magnitude proportional to the magnitude of the velocity of the particle and direction opposite to the direction of the particle.

viscous flow—see **laminar flow**.

void—space in a soil or rock mass not occupied by solid mineral matter. This space may be occupied by air, water, or other gaseous or liquid material.

void ratio, e (D)—the ratio of: (*1*) the volume of void space, to (*2*) the volume of solid particles in a given soil mass.

critical void ratio, e_c (D)—the void ratio corresponding to the critical density.

volumetric shrinkage (volumetric change), V_s (D)—the decrease in volume, expressed as a percentage of the soil mass when dried, of a soil mass when the water content is reduced from a given percentage to the shrinkage limit.

von Post humification scale—a scale describing various stages of decomposition of peat ranging from H1, which is completely undecomposed, to H10, which is completely decomposed.

wall friction, f' (FL^{-2})—frictional resistance mobilized between a wall and the soil or rock in contact with the wall.

washing—*in grouting*, the physical act of cleaning the sides of a hole by circulating water, water and air, acid washes, or chemical substances through drill rods or tremie pipe in an open hole.

water-cement ratio—the ratio of the weight of water to the weights of Portland cement in a cement grout or concrete mix. See also **grout mix**.

water content, w (D)—the ratio of the mass of water contained in the pore spaces of soil or rock material, to the solid mass of particles in that material, expressed as a percentage.

water gain—see **bleeding**.

water-holding capacity (D)—the smallest value to which the water content of a soil or rock can be reduced by gravity drainage.

water-plasticity ratio (*relative water content*) (*liquidity index*)—see **liquidity index**.

water table—see **free water elevation**.

wave—disturbance propagated in medium in such a manner that at any point in medium the amplitude is a function of time, while at any instant the displacement at point is function of position of point.

wave front—moving surface in a medium at which a propagated disturbance first occurs.

wave front—(*1*) a continuous surface over which the phase of a wave that progresses in three dimensions is constant, or (*2*) a continuous line along which the phase of a surface wave is constant. (ISRM)

wave length—normal distance between two wave fronts with periodic characteristics in which amplitudes have phase difference of one complete cycle.

weathering—the process of disintegration and decomposition as a consequence of exposure to the atmosphere, to chemical action, and to the action of frost, water, and heat. (ISRM)

wetland—land which has the water table at, near, or above the land surface, or which is saturated for long enough periods to promote hydrophylic vegetation and various kinds of biological activity which are adapted to the wet environment.

wetting agent—a substance capable of lowering the surface tension of liquids, facilitating the wetting of solid surfaces, and facilitating the penetration of liquids into the capillaries.

wet unit weight—see **unit weight**.

working pressure—the pressure adjudged best for any particular set of conditions encountered during grouting.

Discussion—Factors influencing the determination are size of voids to be filled, depth of zone to be grouted, lithology of area to be grouted, grout viscosity, and resistance of the formation to fracture.

yield—*in grouting*, the volume of freshly mixed grout produced from a known quantity of ingredients.

yielding arch—type of support of arch shape, the joints of which deform plastically beyond a certain critical load, that is, continue to deform without increasing their resistance. (ISRM)

yield stress—the stress beyond which the induced deformation is not fully annulled after complete destressing. (ISRM)

Young's modulus—the ratio of the increase in stress on a test specimen to the resulting increase in strain under constant transverse stress limited to materials having a linear stress-strain relationship over the range of loading. Also called elastic modulus.

zero air voids curve (saturation curve)—the curve showing the zero air voids unit weight as a function of water content.

zero air voids density (zero air voids unit weight)—see unit weight.

APPENDIX

(Nonmandatory Information)

X1. ISRM SYMBOLS RELATING TO SOIL AND ROCK MECHANICS

NOTE—These symbols may not correlate with the symbols appearing in the text.

X1.1 Space

Ω, ω	solid angle
l	length
b	width
h	height or depth
r	radius
A	area
V	volume
t	time
v	velocity
ω	angular velocity
g	gravitational acceleration

X1.2 Periodic and Related Phenomena

T	periodic time
f	frequency
ω	angular frequency
λ	wave length

X1.3 Statics and Dynamics

m	mass
ρ	density (mass density)
G_m	mass specific gravity
G_s	specific gravity of solids
G_w	specific gravity of water
F	force
T	tangential force
W	weight
γ	unit weight
γ_d	dry unit weight
γ_w	unit weight of water
γ'	buoyant unit weight
γ_s	unit of solids
T	torque
I	moment of inertia
W	work
W	energy

X1.4 Applied Mechanics

e	void ratio
n	porosity
w	water content
S_r	degree of saturation
p	pressure
u	pore water pressure
σ	normal stress
σ_x, σ_y, σ_z	stress components in rectangular coordinates
σ_1, σ_2, σ_3	principal stresses
S_1, S_2, S_3	applied stresses (and reactions)
σ_h	horizontal stress
σ_v	vertical stress
τ	shear stress
τ_{xy}, τ_{yz}, τ_{zx}	shear stress components in rectangular coordinates
ϵ	strain
ϵ_x, ϵ_y, ϵ_z	strain components in rectangular coordinates
γ_{xy}, γ_{yz}, γ_{zx}	shear strain components in rectangular coordinates
θ	volume strain
E	Young's modulus; modulus of elasticity $E = \sigma/\epsilon$
ϵ_1, ϵ_2, ϵ_3	principal strains
G	shear modulus; modulus of rigidity $G = \tau/\gamma$
c	cohesion
ϕ_μ	angle of friction between solid bodies
ϕ	angle of shear resistance (angle of internal friction)
h	hydraulic head
i	hydraulic gradient
j	seepage force per unit volume or seepage pressure per unit length
k	coefficient of permeability
η	viscosity
η_{pl}	plasticity (viscosity of Bingham body)
t_{ret}	retardation time
t_{rel}	relaxation time
T_s	surface tension
q	quantity rate of flow; rate of discharge
Q	quantity of flow
FS	safety factor

X1.5 Heat

T	temperature
β	coefficient of volume expansion

X1.6 Electricity

I	electric current
Q	electric charge
C	capacitance
L	self-inductance
R	resistance
ρ	resistivity

REFERENCES

(1) Terzaghi, *Theoretical Soil Mechanics*, John Wiley & Sons, Inc., New York, NY (1943).

(2) Terzaghi and Peck, *Soil Mechanics in Engineering Practice*, John Wiley & Sons, Inc., New York, NY (1948).

(3) Taylor, D. W., *Fundamentals of Soil Mechanics*, John Wiley & Sons, Inc., New York, NY (1948).

(4) Krynine, D. P., *Soil Mechanics*, 2nd Edition, McGraw-Hill Book Co., Inc., New York, NY (1947).

(5) Plummer and Dore, *Soil Mechanics and Foundations*, Pitman Publishing Corp., New York, NY (1940).

(6) Tolman, C. F., *Ground Water*, McGraw-Hill Book Co., Inc., New York, NY (1937).

(7) Stewart Sharpe, C. F., *Land Slides and Related Phenomena*, Columbia University Press, New York, NY (1938).

(8) "Letter Symbols and Glossary for Hydraulics with Special Reference to Irrigation," Special Committee on Irrigation Hydraulics, *Manual of Engineering Practice*, Am. Soc. Civil Engrs., No. 11 (1935).

(9) "Soil Mechanics Nomenclature," Committee of the Soil Mechanics and Foundations Division on Glossary of Terms and

Definitions and on Soil Classification, *Manual of Engineering Practice*, Am. Soc. Civil Engrs., No. 22 (1941).
(10) "Pile Foundations and Pile Structures," Joint Committee on Bearing Value of Pile Foundations of the Waterways Division, Construction Division, and Soil Mechanics and Foundations Division, *Manual of Engineering Practice*, Am. Soc. Civil Engrs., No. 27 (1956).
(11) Webster's New International Dictionary of the English Language, unabridged, 2nd Edition, G. and C. Merriam Co., Springfield, MA (1941).
(12) Baver, L. D., *Soil Physics*, John Wiley & Sons, Inc., New York, NY (1940).
(13) Longwell, Knopf and Flint, "Physical Geology," *Textbook of Geology*, Part I, 2nd Edition, John Wiley & Sons, Inc., New York, NY (1939).
(14) Runner, D. G., *Geology for Civil Engineers*, Gillette Publishing Co., Chicago, IL (1939).
(15) Leggett, R. F., *Geology and Engineering*, McGraw-Hill Book Co., Inc., New York, NY (1939).
(16) Holmes, A., *The Nomenclature of Petrology*, Thomas Murby and Co., London, England (1920).
(17) Meinzer, O. E., "Outline of Ground Water Hydrology with Definitions," *U. S. Geological Survey Water Supply Paper 494* (1923).
(18) "Reports of the Committee on Sedimentation of the Division of Geology and Geography of the National Research Council," Washington, DC (1930–1938).
(19) Twenhofel, W. H., *A Treatise on Sedimentation*, 2nd Edition, Williams & Wilkins Co., Baltimore, MD (1932).
(20) Hogentogler, C. A., *Engineering Properties of Soils*, McGraw Hill Book Co., Inc., New York, NY (1937).
(21) *Special Procedures for Testing Soil and Rock for Engineering Purposes, ASTM STP 479*, ASTM, 1970.
(22) "Glossary of Terms and Definitions," Preliminary Report of Subcommittee G-3 on Nomenclature and Definitions of ASTM Committee D-18 on Soils for Engineering Purposes.
(23) Sowers and Sowers, *Introductory Soil Mechanics and Foundations*, The Macmillan Co., New York, NY (1951).
(24) Lambe, T. William, *Soil Testing for Engineers*, John Wiley & Sons, Inc., New York, NY (1951).
(25) Capper and Cassie, *The Mechanics of Engineering Soils*, McGraw-Hill Book Co., Inc., New York, NY (1949).
(26) Dunham, C. W., *Foundations of Structures*, McGraw-Hill Book Co., Inc., New York, NY (1950).
(27) Casagrande, A., "Notes on Soil Mechanics," Graduate School of Engineering, Harvard University (1938).
(28) Tschebotarioff, G. P., *Soil Mechanics, Foundations, and Earth Structures*, McGraw-Hill Book Co., Inc., New York, (1951).
(29) Rice, C. M., "Dictionary of Geological Terms," Edwards Bros., Inc., Ann Arbor, MI (1940).
(30) Creager, Justin and Hinds, *Engineering for Dams*, John Wiley & Sons, Inc., New York, NY (1945).
(31) Krumbein and Sloss, *Stratigraphy and Sedimentation*, W. H. Freeman and Co., San Francisco, CA (1951).
(32) Pettijohn, F. J. *Sedimentary Rocks*, Harper and Bros., New York, NY (1949).
(33) Reiche, Parry, *A Survey of Weathering Processes and Products*, University of New Mexico Press, Albuquerque, NM (1945).
(34) Garrels, R. M., *A Textbook of Geology*, Harper and Bros., New York, NY (1951).
(35) Ries and Watson, *Engineering Geology*, John Wiley & Sons, Inc., New York, NY (1936).
(36) Ross and Hendricks, *Minerals of the Montmorillonite Group*, U. S. Geological Survey Professional Paper 205-B (1945).
(37) Hartman, R. J., *Colloid Chemistry*, Houghton Mifflin Co., New York, NY (1947).
(38) "Frost Investigations," Corps of Engineers, Frost Effects Laboratory, Boston, MA, June 1951.
(39) "Standard Specifications for Highway Materials and Methods of Sampling and Testing," Parts I and II, adopted by the American Association of State Highway Officials (1950).
(40) Coates, D. G., "Rock Mechanics Principles," rev ed. Mines Br., Dept. Mines and Tech. Surv., Ottawa, Mines Br. Mon. 874 (1970)
(41) Gary, M., McAfee, R., Jr., and Wolf, C. L., (eds.), *Glossary of Geology*, American Geological Institute (1972).
(42) International Society for Rock Mechanics, Commission on Terminology, Symbols and Graphic Representation, Final Document on Terminology, English Version, 1972, and List of Symbols, 1970.
(43) Jaeger, J. C., and Cook, N. G. W., *Fundamentals of Rock Mechanics*, Methuen, London (1969).
(44) Nelson, A., and Nelson, K. D., *Dictionary of Applied Geology* (Mining and Civil Engineering), Philosophical Library, Inc., New York, NY (1967).
(45) Obert, L. A., and Duvall, W. I., *Rock Mechanics and the Design of Structures in Rock*, John Wiley & Sons, New York, NY (1967).
(46) SME Mining Engineering Handbook: Society Mining Engineers, Vol 2, New York, NY (1973).
(47) Thrush, R. P. (ed), et al., *A Dictionary of Mining, Mineral and Related Terms*, U. S. Bureau of Mines (1968).
(48) Lohman, W. W., and others, *Definitions of Selected Ground-Water Terms—Revisions and Conceptual Refinements*, U. S. Geological Survey Water-Supply Paper 1988, 21 pp., 1972.
(49) *Glossary of Soil Science Terms*, Madison, WI, Soil Science Society of America, 34 pp. (1975).
(50) *International Glossary of Hydrology*, Geneva, Switzerland, World Meterological Organization, WMO No. 385, 393 pp., (1974).

Designation: C 294 – 86 (Reapproved 1991)[ε1]

Standard Descriptive Nomenclature for Constituents of Natural Mineral Aggregates[1]

This standard is issued under the fixed designation C 294; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This standard has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

ε1 NOTE—Section 25, Keywords, was added editorially in April 1991.

1. Scope

1.1 This descriptive nomenclature provides brief descriptions of some of the more common, or more important, natural materials of which mineral aggregates are composed (Note 1). The descriptions provide a basis for understanding these terms as used to designate aggregate constituents. Many of the materials described frequently occur in particles that do not display all the characteristics given in the descriptions, and most of these rocks grade from varieties meeting one description to varieties meeting another with all intermediate stages being found.

NOTE 1—These descriptions characterize minerals and rocks as they occur in nature and do not include blast-furnace slag or lightweight aggregates that are prepared by the alteration of the structure and composition of natural material. Blast-furnace slag is defined in Definitions C 125. Information about lightweight aggregates is given in Specifications C 330, C 331, and C 332.

1.2 The accurate identification of rocks and minerals can, in many cases, be made only by a qualified geologist, mineralogist, or petrographer using the apparatus and procedures of these sciences. Reference to these descriptions may, however, serve to indicate or prevent gross errors in identification. Identification of the constituent materials in an aggregate may assist in recognizing its properties, but identification alone cannot provide a basis for predicting the behavior of aggregates in service. Mineral aggregates composed of any type or combination of types of rocks and minerals may perform well or poorly in service depending upon the exposure to which they are subjected, the physical and chemical properties of the matrix in which they are embedded, their physical condition at the time they are used, and other factors. Small amounts of minerals or rocks that may occur only as contaminants or accessories in the aggregate may decisively influence its quality.

2. Referenced Documents

2.1 *ASTM Standards*:

C 125 Terminology Relating to Concrete and Concrete Aggregates[2]
C 289 Test Method for Potential Reactivity of Aggregates (Chemical Method)[2]
C 330 Specification for Lightweight Aggregates for Structural Concrete[2]
C 331 Specification for Lightweight Aggregates for Concrete Masonry Units[2]
C 332 Specification for Lightweight Aggregates for Insulating Concrete[2]

3. Classes and Types

3.1 The materials found as constituents of natural mineral aggregates are rocks and minerals. Minerals are naturally occurring inorganic substances of more or less definite chemical composition and usually of a specific crystalline structure. Most rocks are composed of several minerals but some are composed of only one mineral. Certain examples of the rock quartzite are composed exclusively of the mineral quartz, and certain limestones are composed exclusively of the mineral calcite. Individual sand grains frequently are composed of particles of rock, but they may be composed of a single mineral, particularly in the finer sizes.

3.2 Rocks are classified according to origin into three major divisions: igneous, sedimentary, and metamorphic. These three major groups are subdivided into types according to mineral and chemical composition, texture, and internal structure. *Igneous rocks* form from molten rock matter either above or below the earth's surface. *Sedimentary rocks* form at the earth's surface by the accumulation and consolidation of the products of weathering and erosion of existing rocks. *Metamorphic rocks* form from pre-existing rocks by the action of heat, pressure, or shearing forces in the earth's crust. It is obvious that not only igneous but also sedimentary and metamorphic rocks may be weathered and eroded to form new sedimentary rocks. Similarly, metamorphic rocks may again be metamorphosed.

DESCRIPTIONS OF MINERALS

4. General

4.1 For the purpose of indicating significant relationships, the descriptions of minerals are presented in groups in the following sections.

[1] This descriptive nomenclature is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.02.06 on Petrography of Concrete and Concrete Aggregates.

Current edition approved Nov. 28, 1986. Published January 1987. Originally published as C 294 – 52. Last previous edition C 294 – 69 (1981)[ε1].

This standard has been extensively revised. The reader should compare this edition with the last previous edition for exact revisions.

[2] *Annual Book of ASTM Standards*, Vol 04.02.

5. Silica Minerals

5.1 **quartz**—a very common hard mineral composed of silica (SiO_2). It will scratch glass and is not scratched by a knife. When pure it is colorless with a glassy (vitreous) luster and a shell-like (conchoidal) fracture. It lacks a visible cleavage (the ability to break in definite directions along even planes) and, when present in massive rocks such as granite, it usually has no characteristic shape. It is resistant to weathering and is therefore an important constituent of many sand and gravel deposits and many sandstones. It is also abundant in many light-colored igneous and metamorphic rocks. Some strained or intensely fractured (granulated) quartz may be deleteriously reactive with alkalies in concrete.

5.2 **opal**—a hydrous form of silica ($SiO_2 \cdot nH_2O$) which occurs without characteristic external form or internal crystalline arrangement as determined by ordinary visible light methods. When X-ray diffraction methods are used, opal may show some evidences of internal crystalline arrangement. Opal has a variable water content, generally ranging from 3 to 9 %. The specific gravity and hardness are always less than those of quartz. The color is variable and the luster is resinous to glassy. It is usually found in sedimentary rocks, especially some cherts, and is the principal constituent of diatomite. It is also found as a secondary material filling cavities and fissures in igneous rocks and may occur as a coating on gravel and sand. The recognition of opal in aggregates is important because it reacts with the alkalies in portland-cement paste, or with the alkalies from other sources, such as aggregates containing zeolites, and ground water.

5.3 **chalcedony**—chalcedony has been considered both as a distinct mineral and a variety of quartz. It is frequently composed of a mixture of microscopic fibers of quartz with a large number of submicroscopic pores filled with water and air. The properties of chalcedony are intermediate between those of opal and quartz, from which it can sometimes be distinguished only by laboratory tests. It frequently occurs as a constituent of the rock chert and is reactive with the alkalies in portland-cement paste.

5.4 **tridymite and cristobalite**—crystalline forms of silica (SiO_2) sometimes found in volcanic rocks. They are metastable at ordinary temperatures and pressures. They are rare minerals in aggregates except in areas where volcanic rocks are abundant. A type of cristobalite is a common constituent of opal. Tridymite and cristobalite are reactive with the alkalies in portland-cement paste.

6. Feldspars

6.1 The minerals of the feldspar group are the most abundant rock-forming minerals in the crust of the earth. They are important constituents of all three major rock groups, igneous, sedimentary, and metamorphic. Since all feldspars have good cleavages in two directions, particles of feldspar usually show several smooth surfaces. Frequently, the smooth cleavage surfaces show fine parallel lines. All feldspars are slightly less hard than, and can be scratched by, quartz and will, when fresh, easily scratch a penny. The various members of the group are differentiated by chemical composition and crystallographic properties. The feldspars *orthoclase, sanidine*, and *microcline* are potassium aluminum silicates, and are frequently referred to as *potassium feldspars*. The *plagioclase* feldspars include those that are sodium aluminum silicates and calcium aluminum silicates, or both sodium and calcium aluminum silicates. This group, frequently referred to as the "soda-lime" group, includes a continuous series, of varying chemical composition and optical properties, from *albite*, the sodium aluminum feldspar, to *anorthite*, the calcium aluminum feldspar, with intermediate members of the series designated *oligoclase, andesine, labradorite*, and *bytownite*. Potassium feldspars and sodium-rich plagioclase feldspars occur typically in igneous rocks such as granites and rhyolites, whereas, plagioclase feldspars of higher calcium content are found in igneous rocks of lower silica content such as diorite, gabbro, andesite, and basalt.

7. Ferromagnesian Minerals

7.1 Many igneous and metamorphic rocks contain dark green to black minerals that are generally silicates of iron or magnesium, or of both. They include the minerals of the amphibole, pyroxene, and olivine groups. The most common amphibole mineral is hornblende; the most common pyroxene mineral is augite; and the most common olivine mineral is olivine. Dark mica, such as biotite and phlogopite, are also considered ferromagnesian minerals. The amphibole and pyroxene minerals are brown to green to black and generally occur as prismatic units. Olivine is usually olive green, glassy in appearance, and usually altered. Biotite has excellent cleavage and can be easily cleaved into thin flakes and plates. These minerals can be found as components of a variety of rocks, and in sands and gravels. Olivine is found only in dark igneous rocks where quartz is not present, and in sands and gravels close to the olivine source.

8. Micaceous Minerals

8.1 Micaceous minerals have perfect cleavage in one direction and can be easily split into thin flakes. The mica minerals of the muscovite group are colorless to light green; of the biotite group, dark brown to black or dark green; of the lepidolite group, white to pink and red or yellow; and of the chlorite group, shades of green. Another mica, phlogopite, is similar to biotite, commonly has a pearl-like luster and bronze color, and less commonly is brownish red, green, or yellow. The mica minerals are common and occur in igneous, sedimentary, and metamorphic rocks, and are common as minor to trace components in many sands and gravels. The muscovite, biotite, lepidolite, and phlogopite minerals cleave into flakes and plates that are elastic; the chlorite minerals, by comparison, form in elastic flakes and plates. Vermiculite (a mica-like mineral) forms by the alteration of other micas and is brown and has a bronze luster.

9. Clay Minerals

9.1 The term "clay" refers to natural material composed of particles in a specific size range, generally less than 2 μm (0.002 mm). Mineralogically, clay refers to a group of layered silicate minerals including the clay-micas (illites), the kaolin group, very finely divided chlorites, and the swelling clays—montmorillonites (smectites). Members of several groups, particularly micas, chlorites, and vermiculites, occur both in the clay-size range and in larger sizes. Some clays are made up of alternating layers of two or more clay groups. Random, regular, or both types of interlayering are known. If smectite is a significant constituent in such mixtures, then fairly large volume changes may occur with wetting and drying.

9.2 Clay minerals are hydrous aluminum, magnesium, and iron silicates that may contain calcium, magnesium, potassium, sodium and other exchangeable cations. They are formed by alteration and weathering of other silicates and volcanic glass. The clay minerals are major constituents of clays and shales. They are found disseminated in carbonate rocks as seams and pockets and in altered and weathered igneous and metamorphic rocks. Clays may also be found as matrix, void fillings, and cementing material in sandstones and other sedimentary rocks.

9.3 Most aggregate particles composed of, or containing large proportions of clay minerals are soft and, because of the large internal surface area of the constituents, they are porous. Some of these aggregates will disintegrate when wetted. Rocks in which the cementing matrix is principally clay, such as clay-bonded sandstones, and rocks in which montmorillonite is present as a continuous phase or matrix, such as some altered volcanics, may slake in water or may disintegrate in the concrete mixer. Rocks of this type are unsuitable for use as aggregates. Rocks having these properties less well developed will abrade considerably during mixing, releasing clay, and raising the water requirement of the concrete containing them. When such rocks are present in hardened concrete, the concrete will manifest greater volume change on wetting and drying than similar concrete containing non-swelling aggregate.

10. Zeolites

10.1 The zeolite minerals are a large group of hydrated aluminum silicates of the alkali and alkaline earth elements which are soft and usually white or light colored. They are formed as a secondary filling in cavities or fissures in igneous rocks, or within the rock itself as a product of hydrothermal alteration of original minerals, especially feldspars. Some zeolites, particularly *heulandite, natrolite,* and *laumontite,* reportedly produce deleterious effects in concrete, the first two having been reported to augment the alkali content in concrete by releasing alkalies through cation exchange and thus increasing alkali reactivity when certain siliceous aggregates are present. Laumontite and its partially dehydrated variety *leonhardite* are notable for their substantial volume change with wetting and drying. Both are found in rocks such as quartz diorites and some sandstones.

11. Carbonate Minerals

11.1 The most common carbonate mineral is *calcite* (calcium carbonate, $CaCO_3$). The mineral *dolomite* consists of calcium carbonate and magnesium carbonate ($CaCO_3 \cdot MgCO_3$ or $CaMg(CO_3)_2$) in equivalent molecular amounts, which are 54.27 and 45.73 by mass %, respectively. Both calcite and dolomite are relatively soft, the hardness of calcite being 3 and that of dolomite 3½ to 4 on the Mohs scale, and are readily scratched by a knife blade. They have rhombohedral cleavage, which results in their breaking into fragments with smooth parallelogram shaped sides. Calcite is soluble with vigorous effervescence in cold dilute hydrochloric acid; dolomite is soluble with slow effervescence in cold dilute hydrochloric acid and with vigorous effervescence if the acid or the sample is heated or if the sample is pulverized.

12. Sulfate Minerals

12.1 Carbonate rocks and shales may contain sulfates as impurities. The most abundant sulfate mineral is *gypsum* (hydrous calcium sulfate; $CaSO_4 \cdot 2H_2O$); *anhydrite* (anhydrous calcium sulfate, $CaSO_4$) is less common. Gypsum is usually white or colorless and characterized by a perfect cleavage along one plane and by its softness, representing hardness of 2 on the Mohs scale; it is readily scratched by the fingernail. Gypsum may form a whitish pulverulent or crystalline coating on sand and gravel. It is slightly soluble in water.

12.2 Anhydrite resembles dolomite in hand specimen but has three cleavages at right angles; it is less soluble in hydrochloric acid than dolomite, does not effervesce and is slightly soluble in water. Anhydrite is harder than gypsum. Gypsum and anhydrite occurring in aggregates offer risks of sulfate attack in concrete and mortar.

13. Iron Sulfide Minerals

13.1 The sulfides of iron, *pyrite, marcasite,* and *pyrrhotite* are frequently found in natural aggregates. Pyrite is found in igneous, sedimentary, and metamorphic rocks; marcasite is much less common and is found mainly in sedimentary rocks; pyrrhotite is less common but may be found in many types of igneous and metamorphic rocks. Pyrite is brass yellow, and pyrrhotite bronze brown, and both have a metallic luster. Marcasite is also metallic but lighter in color and finely divided iron sulfides are soot black. Pyrite is often found in cubic crystals. Marcasite readily oxidizes with the liberation of sulfuric acid and formation of iron oxides, hydroxides, and, to a much smaller extent, sulfates; pyrite and pyrrhotite do so less readily. Marcasite and certain forms of pyrite and pyrrhotite are reactive in mortar and concrete, producing a brown stain accompanied by a volume increase that has been reported as one source of popouts in concrete. Reactive forms of iron sulfides may be recognized by immersion in saturated lime water (calcium hydroxide solution); upon exposure to air the reactive varieties produce a brown coating within a few minutes.

14. Iron Oxide Minerals, Anhydrous and Hydrous

14.1 There are two common iron oxide minerals: (*1*) Black, magnetic: *magnetite* (Fe_3O_4), and (*2*) red or reddish when powdered: *hematite* (Fe_2O_3); and one common hydrous oxide mineral, brown or yellowish: *goethite* ($FeO(OH)$). Another common iron-bearing mineral is black, weakly magnetic, *ilmenite* ($FeTiO_3$). Magnetite and ilmenite are important accessory minerals in many dark igneous rocks and are common detrital minerals in sediments. Hematite is frequently found as an accessory mineral in reddish rocks. Limonite, the brown weathering product of iron-bearing minerals, is a field name for a variety of hydrous iron oxide minerals including goethite; it frequently contains adsorbed water, and various impurities such as colloidal or crystalline silica, clay minerals, and organic matter. The presence of substantial amounts of soft iron-oxide minerals in concrete aggregate can color concrete various shades of yellow or brown. Very minor amounts of iron minerals color many rocks, such as ferruginous sandstones, shales, clay-ironstones, and granites. Magnetite, ilmenite, and hematite ores are used as heavy aggregates.

DESCRIPTIONS OF IGNEOUS ROCKS

15. General

15.1 Igneous rocks are those formed by cooling from a molten rock mass (magma). They may be divided into two classes: (*1*) plutonic, or intrusive, that have cooled slowly within the earth; and (*2*) volcanic, or extrusive, that formed from quickly cooled lavas. Plutonic rocks have grain sizes greater than approximately 1 mm, and are classified as coarse- or medium-grained. Volcanic rocks have grain sizes less than approximately 1 mm, and are classified as fine-grained. Volcanic rocks frequently contain glass. Both plutonic and volcanic rocks may consist of porphyries, that are characterized by the presence of large mineral grains in a fine-grained or glassy groundmass. This is the result of sharp changes in rate of cooling or other physico-chemical conditions during solidification of the melt.

15.2 Igneous rocks are usually classified and named on the basis of their texture, internal structure, and their mineral composition which in turn depends to a large extent on their chemical composition. Rocks in the plutonic class generally have chemical equivalents in the volcanic class.

16. Plutonic Rocks

16.1 **granite**—granite is a medium- to coarse-grained, light-colored rock characterized by the presence of potassium feldspar with lesser amounts of plagioclase feldspars and quartz. The characteristic potassium feldspars are orthoclase or microcline, or both; the common plagioclase feldspars are albite and oligoclase. Feldspars are more abundant than quartz. Dark-colored mica (biotite) is usually present, and light-colored mica (muscovite) is frequently present. Other dark-colored ferromagnesian minerals, especially hornblende, may be present in amounts less than those of the light-colored constituents. *Quartz-monzonite* and *granodiorite* are rocks similar to granite, but they contain more plagioclase feldspar than potassium feldspar.

16.2 **syenite**—syenite is a medium- to coarse-grained, light-colored rock composed essentially of alkali feldspars, namely microcline, orthoclase, or albite. Quartz is generally absent. Dark ferromagnesian minerals such as hornblende, biotite, or pyroxene are usually present.

16.3 **diorite**—diorite is a medium- to coarse-grained rock composed essentially of plagioclase feldspar and one or more ferromagnesian minerals such as hornblende, biotite, or pyroxene. The plagioclase is intermediate in composition, usually of the variety andesine, and is more abundant than the ferromagnesian minerals. Diorite usually is darker in color than granite or syenite and lighter than gabbro. If quartz is present, the rock is called *quartz diorite*.

16.4 **gabbro**—gabbro is a medium- to coarse-grained, dark-colored rock consisting essentially of ferromagnesian minerals and plagioclase feldspar. The ferromagnesian minerals may be pyroxenes, amphiboles, or both. The plagioclase is one of the calcium-rich varieties, namely labradorite, bytownite, or anorthite. Ferromagnesian minerals are usually more abundant than feldspar. *Diabase* (in European usage *dolerite*) is a rock of similar composition to gabbro and basalt but is intermediate in mode of origin, usually occurring in smaller intrusions than gabbro, and having a medium to fine-grained texture. The terms "trap" or "trap rock" are collective terms for dark-colored, medium- to fine-grained igneous rocks especially diabase and basalt.

16.5 **peridotite**—peridotite is composed of olivine and pyroxene. Rocks composed almost entirely of pyroxene are known as *pyroxenites*, and those composed of olivine as *dunites*. Rocks of these types are relatively rare but their metamorphosed equivalent, serpentinite, is more common.

16.6 **pegmatite**—extremely coarse-grained varieties of igneous rocks are known as pegmatites. These are *usually* light-colored and are most frequently equivalent to granite or syenite in mineral composition.

17. Fine-Grained and Glassy Extrusive Igneous Rocks

17.1 **volcanic rock**—volcanic or extrusive rocks are the fine-grained equivalents of the coarse-and-medium-grained plutonic rocks described in Section 16. Equivalent types have similar chemical compositions and may contain the same minerals. Volcanic rocks commonly are so fine-grained that the individual mineral grains usually are not visible to the naked eye. Porphyritic textures are common, and the rocks may be partially or wholly glassy or non-crystalline. The glassy portion of a partially glassy rock usually has a higher silica content than the crystalline portion. Some volcanic or extrusive rocks may not be distinguishable in texture and structure from plutonic or intrusive rocks that originated at shallow depth.

17.2 **felsite**—light-colored, very fine-grained igneous rocks are collectively known as felsites. The felsite group includes *rhyolite, dacite, andesite*, and *trachyte*, which are the equivalents of granite, quartz diorite, diorite, and syenite, respectively. These rocks are usually light colored but they may be gray, green, dark red, or black. When they are dark they may incorrectly be classed as "trap" (see 16.4). When they are microcrystalline or contain natural glass, rhyolites, dacites, and andesites are reactive with the alkalies in portland-cement concrete.

17.3 **basalt**—fine-grained extrusive equivalent of gabbro and diabase. When basalt contains natural glass, the glass is generally lower in silica content than that of the lighter-colored extrusive rocks and hence is not deleteriously reactive with the alkalies in portland-cement paste; however, exceptions have been noted in the literature with respect to the alkali reactivity of basaltic glasses.

17.4 **volcanic glass**—igneous rocks composed wholly of glass are named on the basis of their texture and internal structure. A dense dark natural glass of high silica content is called *obsidian*, while lighter colored finely vesicular glassy froth filled with elongated, tubular bubbles is called *pumice*. Dark-colored coarsely vesicular types containing more or less spherical bubbles are called *scoria*. Pumices are usually silica-rich (corresponding to rhyolites or dacites), whereas scorias usually are more basic (corresponding to basalts). A high-silica glassy lava with an onion-like structure and a pearly luster, containing 2 to 5 % water, is called *perlite*. When heated quickly to the softening temperature, perlite puffs to become an artificial pumice. Glass with up to 10 % water and with a dull resinous luster is called *pitchstone*. Glassy rocks, particularly the more siliceous ones, are reactive with the alkalies in portland-cement paste.

DESCRIPTIONS OF SEDIMENTARY ROCKS

18. General

18.1 Sedimentary rocks are stratified rocks laid down for the most part under water, although wind and glacial action occasionally are important. Sediments may be composed of particles of preexisting rocks derived by mechanical agencies or they may be of chemical or organic origin. The sediments are usually indurated by cementation or compaction during geologic time, although the degree of consolidation may vary widely.

18.2 *Gravel, sand, silt*, and *clay* form the group of unconsolidated sediments. Although the distinction between these four members is made on the basis of their particle size, a general trend in the composition occurs. Gravel and, to a lesser degree, coarse sands usually consist of rock fragments; fine sands and silt consist predominantly of mineral grains; and clay exclusively of mineral grains, largely of the group of clay minerals. All types of rocks and minerals may be represented in unconsolidated sediments.

19. Conglomerates, Sandstones, and Quartzites

19.1 These rocks consist of particles of *sand* or *gravel*, or both, with or without interstitial and cementing material. If the particles include a considerable proportion of gravel, the rock is a *conglomerate*. If the particles are in the sand sizes, that is, less than 2 mm but more than 0.06 mm in major diameter, the rock is a *sandstone* or a *quartzite*. If the rock breaks around the sand grains, it is a sandstone; if the grains are largely quartz and the rock breaks through the grains, it is quartzite. Conglomerates, and sandstones are sedimentary rocks but quartzites may be sedimentary (*orthoquartzites*) or metamorphic (*metaquartzites*). The cementing or interstitial materials of sandstones may be quartz, opal, calcite, dolomite, clay, iron oxides, or other materials. These may influence the quality of a sandstone as concrete aggregate. If the nature of the cementing material is known, the rock name may include a reference to it, such as *opal-bonded sandstone* or *ferruginous conglomerate*.

19.2 **graywackes and subgraywackes**—gray to greenish gray sandstones containing angular quartz and feldspar grains, and sand-sized rock fragments in an abundant matrix resembling claystone, shale, argillite, or slate. Graywackes grade into subgraywackes, the most common sandstones of the geologic column.

19.3 **arkose**—coarse-grained sandstone derived from granite, containing conspicuous amounts of feldspar.

20. Claystones, Shales, Argillites, and Siltstones

20.1 These very fine-grained rocks are composed of, or derived by erosion of sedimentary *silts* and *clays*, or of any type of rock that contained clay. When relatively soft and massive, they are known as *claystones*, or *siltstones*, depending on the size of the majority of the particles of which they are composed. Siltstones consist predominantly of silt-sized particles (0.0625 to 0.002 mm in diameter) and are intermediate rocks between claystones and sandstones. When the claystones are harder and platy or fissile, they are known as *shales*. Claystones and shales may be gray, black, reddish, or green and may contain some carbonate minerals (calcareous shales). A massive, firmly indurated fine-grained argillaceous rock consisting of quartz, feldspar, and micaceous minerals is known as *argillite*. Argillites do not slake in water as some shales do. As an aid in distinguishing these fine-grained sediments from fine-grained, foliated metamorphic rocks such as slates and phyllites, it may be noted that the cleavage surfaces of shales are generally dull and earthy while those of slates are more lustrous. Phyllite has a glossier luster resembling a silky sheen.

20.2 Aggregates containing abundant shale are detrimental to concrete because they can produce high shrinkage, but not all shales are harmful. Some argillites are alkali-silica reactive.

20.3 Although aggregates which are volumetrically unstable in wetting and drying are not confined to any class of rock, they do share some common characteristics. If there is a matrix or continuous phase, it is usually physically weak and consists of material of high specific surface, frequently including clay. However, no general relation has been demonstrated between clay content or type of clay and large volume change upon wetting and drying. Volumetrically unstable aggregates do not have mineral grains of high modulus interlocked in a continuous rigid structure capable of resisting volume change.

20.4 Aggregates having high elastic modulus and low volume change from the wet to the dry condition contribute to the volume stability of concrete by restraining the volume change of the cement paste. In a relatively few cases, aggregates have been demonstrated to contribute to unsatisfactory performance of concrete because they have relatively large volume change from the wet to the dry condition combined with relatively low modulus of elasticity. On drying, such aggregates shrink away from the surrounding cement paste and consequently fail to restrain its volume change with change in moisture content.

21. Carbonate Rocks

21.1 *Limestones* are the most widespread of carbonate rocks. They range from pure limestones consisting of the mineral calcite to pure *dolomites* (dolostones) consisting of the mineral dolomite. Usually they contain both minerals in various proportions. If 50 to 90 % is the mineral dolomite, the rock is called *calcitic dolomite*. Magnesium limestone is sometimes applied to dolomitic limestones and calcitic dolomites but it is ambiguous and its use should be avoided. Most carbonate rocks contain some noncarbonate impurities such as quartz, chert, clay minerals, organic matter, gypsum, and sulfides. Carbonate rocks containing 10 to 50 % sand are *arenaceous* (or *sandy*) *limestones* (or *dolomites*); those containing 10 to 50 % clay are *argillaceous* (or *clayey* or *shaly*) *limestones* (or *dolomites*). *Marl* is a clayey limestone which is fine-grained and commonly soft. Chalk is fine-textured, very soft, porous, and somewhat friable limestone, composed chiefly of particles of microorganisms. Micrite is very fine-textured chemically precipitated carbonate or a mechanical ooze of carbonate particles, usually 0.001 to 0.003 mm in size. The term "limerock" is not recommended.

21.2 The reaction of the dolomite in certain carbonate rocks with alkalies in portland cement paste has been found to be associated with deleterious expansion of concrete containing such rocks as coarse aggregate. Carbonate rocks capable of such reaction possess a characteristic texture and composition. The characteristic microscopic texture is that in which relatively large crystals of dolomite (rhombs) are scattered in a finer-grained matrix of micritic calcite and clay. The characteristic composition is that in which the carbonate portion consists of substantial amounts of both dolomite and calcite, and the acid-insoluble residue contains a significant amount of clay. Except in certain areas, such rocks are of relatively infrequent occurrence and seldom make up a significant proportion of the material present in a deposit of rock being considered for use in making aggregate for concrete.

22. Chert

22.1 **chert**—the general term for a group of variously colored, very fine-grained (aphanitic), siliceous rocks composed of microcrystalline or cryptocrystalline quartz, chalcedony, or opal, either singly or in combinations of varying proportions. Identification of the form or forms of silica requires careful determination of optical properties, absolute specific gravity, loss on ignition, or a combination of these characteristics. Dense cherts are very tough, with a waxy to greasy luster, and are usually gray, brown, white, or red, and less frequently, green, black or blue. Porous varieties are usually lighter in color, frequently off-white, or stained yellowish, brownish, or reddish, firm to very weak, and grade to tripoli. Ferruginous, dense, red, and in some cases, dense, yellow, brown, or green chert is sometimes called *jasper*. Dense black or gray chert is sometimes called *flint*. A very dense, even textured, light gray to white chert, composed mostly of microcrystalline to cryptocrystalline quartz, is called *novaculite*. Chert is hard (scratches glass, but is not scratched by a knife blade) and has a conchoidal (shell-like) fracture in the dense varieties, and a more splintery fracture in the porous varieties. Chert occurs most frequently as nodules, lenses, or interstitial material, in limestone and dolomite formations, as extensively bedded deposits, and as components of sand and gravel. Most cherts have been found to be alkali-silica reactive to some degree when tested with high-alkali cement, or in the quick chemical test (Test Method C 289). However, the degree of the alkali-silica reactivity and whether a given chert will produce a deleterious degree of expansion in concrete are complex functions of several factors. Among them are: the mineralogic composition and internal structure of the chert; the amount of the chert as a proportion of the aggregates; the particle-size distribution; the alkali content of the cement; and the cement content of the concrete. In the absence of information to the contrary, all chert should be regarded as potentially alkali-silica reactive if combined with high-alkali cement. However, opaline cherts may produce deleterious expansion of mortar or concrete when present in very small proportions (less than 5 % by mass of the aggregate). Cherts that are porous may be susceptible to freezing and thawing deterioration in concrete and may cause popouts or cracking of the concrete surface above the chert particle.

DESCRIPTIONS OF METAMORPHIC ROCKS

23. General

23.1 Metamorphic rocks form from igneous, sedimentary, or pre-existing metamorphic rocks in response to changes in chemical and physical conditions occurring within the earth's crust after formation of the original rock. The changes may be textural, structural, or mineralogic and may be accompanied by changes in chemical composition. The rocks are dense and may be massive but are more frequently foliated (laminated or layered) and tend to break into platy particles. Rocks formed from argillaceous rocks by dynamic metamorphism usually split easily along one plane independent of original bedding; this feature is designated "platy cleavage." The mineral composition is very variable depending in part on the degree of metamorphism and in part on the composition of the original rock.

23.2 Most of the metamorphic rocks may derive either from igneous or sedimentary rocks but a few, such as marbles and slates, originate only from sediments.

23.3 Certain phyllites, slates, and metaquartzites containing low-temperature silica and silicate minerals or highly strained quartz may be deleteriously reactive when used with cements of high alkali contents.

24. Metamorphic Rocks

24.1 **marble**—a recrystallized medium- to coarse-grained carbonate rock composed of calcite or dolomite, or calcite and dolomite. The original impurities are present in the form of new minerals, such as micas, amphiboles, pyroxenes, and graphite.

24.2 **metaquartzite**—a granular rock consisting essentially of recrystallized quartz. Its strength and resistance to weathering derive from the interlocking of the quartz grains.

24.3 **slate**—a fine-grained metamorphic rock that is distinctly laminated and tends to split into thin parallel layers. The mineral composition usually cannot be determined with the unaided eye.

24.4 **phyllite**—a fine-grained thinly layered rock. Minerals, such as micas and chlorite, are noticeable and impart a silky sheen to the surface of schistosity. Phyllites are intermediate between slates and schists in grain size and mineral composition. They derive from argillaceous sedimentary rocks or fine-grained extrusive igneous rocks, such as felsites.

24.5 **schist**—a highly layered rock tending to split into nearly parallel planes (schistose) in which the grain is coarse enough to permit identification of the principal minerals. Schists are subdivided into varieties on the basis of the most prominent mineral present in addition to quartz or to quartz and feldspars; for instance, *mica schist*. *Greenschist* is a green schistose rock whose color is due to abundance of one or more of the green minerals, chlorite or amphibole, and is commonly derived from altered volcanic rock.

24.6 **amphibolite**—a medium- to coarse-grained dark-colored rock composed mainly of hornblende and plagioclase feldspar. Its schistosity, which is due to parallel alignment of hornblende grains, is commonly less obvious than in typical schists.

24.7 **hornfels**—equigranular, massive, and usually tough rock produced by complete recrystallization of sedimentary, igneous, or metamorphic rocks through thermal metamorphism sometimes with the addition of components of molten rock. Their mineral compositions vary widely.

24.8 **gneiss**—one of the most common metamorphic rocks, usually formed from igneous or sedimentary rocks by a higher degree of metamorphism than the schists. It is characteriz d by a layered or foliated structure resulting from approximately parallel lenses and bands of platy minerals, usually micas, or prisms, usually amphiboles, and of granular minerals, usually quartz and feldspars. All intermediate varieties between gneiss and schist, and between gneiss and granite are often found in the same areas in which well-defined gneisses occur.

24.9 **serpentinite**—a relatively soft, light to dark green to almost black rock formed usually from silica-poor igneous rocks, such as pyroxenites, peridotites, and dunites. It may contain some of the original pyroxene or olivine but is largely composed of softer hydrous ferromagnesian minerals of the serpentine group. Very soft talc-like material is often present in serpentinite.

25. Keywords

25.1 aggregates; carbonates; clays; concrete; feldspars; ferromagnesian minerals; igneous rocks; iron oxides; iron sulfides; metamorphic rocks; micas; minerals; nomenclature; rocks; sedimentary rocks; silica; sulfates; zeolites

RECOMMENDED PRACTICE FOR PETROGRAPHIC EXAMINATION OF ROCK CORES

1. Scope

1.1 This recommended practice outlines procedures for the petrographic examination of rock cores whose engineering properties can be determined by selected tests. The specific procedures employed in the petrographic examination of any sample will depend to a large extent on the purpose of the examination* and the nature of the sample. Complete petrographic examination may require use of such procedures as light microscopy, X-ray diffraction analysis, differential thermal analysis, infrared spectroscopy, or others; in some instances, such procedures are more rapid and more definitive than are microscopical methods. Petrographic examinations are made for the following purposes:

(a) To determine the physical and chemical properties of a material, by petrographic methods, that have a bearing on the quality of the material for its intended use.

(b) To describe and classify the constituents of the sample. (Note 1)

(c) To determine the relative amounts of the constituents of the sample, which is essential for proper evaluation of the sample, when the constituents differ significantly in properties that have a bearing on the quality of the material for its intended use.

NOTE 1--It is recommended that the rock and mineral names in "Descriptive Nomenclature of Constituents of Natural Mineral Aggregates" (ASTM Designation: C 294) be used insofar as they are appropriate in reports prepared according to this recommended practice.

---

*The practices described herein are applicable to the examination of rock cores from rock used as foundation or other similar purposes. However, if the cores are from rock proposed to be quarried for use as concrete aggregate or erosion control, reference should be made to ASTM C 295 or D 4992, respectively.

1.2 Detection of structural features and identification of the constituents of a sample are usually necessary steps toward recognition of the properties that may be expected to influence the behavior of the material in its intended use. However, the value of any petrographic examination will depend to a large extent on the representativeness of the samples examined, the completeness and accuracy of the information provided to the petrographer concerning the source and proposed use of the material, and the petrographer's ability to correlate these data with the findings of the examination.

1.3 This recommended practice does not attempt to outline the techniques of petrographic work since it is assumed that the method will be used by persons who are qualified by education and/or experience to employ such techniques for the recognition of the characteristic properties of rocks and minerals and to describe and classify the constituents of a sample. It is intended to outline the extent to which such techniques should be used, the selection of properties that should be looked for, and the manner in which such techniques may best be employed in the examination. These objectives will have been attained if engineers responsible for the application of the results of petrographic examinations have reasonable assurance that such results, wherever and whenever obtained, may confidently be compared.

2. Sampling and Examination Procedure

2.1 The purpose in specifying a sampling procedure is to ensure the selection of an adequate group of test specimens from each mechanically different rock type that forms an essential part of the core or cores that will be tested. The sampling procedure should also be guided by the project objectives and directed at obtaining the properties of the rock that eventually will comprise the roof, side walls, foundation, or other specific parts of the project structure. Samples for petrographic examination should be taken by or under the direct supervision of a geologist familiar with the requirements of the project. The exact location from which the sample was taken, the geology of the site, and other pertinent data should be submitted with the sample. The amount of material actually studied in the petrographic examination will be determined by the nature of the material to be examined. Areas to be studied should be sampled by means of cores drilled through the entire depth required for project investigation. Drilling of such cores should be in a direction that is essentially normal to the dominant structural feature of the rock. Massive material may be sampled by "NX"(2-1/8-in.-diam) (54-mm-diam) cores. Thinly bedded or complex material should be represented by cores not less than 4 in. (100 mm) in diameter, preferably 6 in. (150 mm). There should be an adequate number of cores to cover the limits of the rock mass under consideration.

2.2 The following is considered a preferable but not a mandatory procedure. A petrographer should inspect all of the rock core before any tests are made. Each core should be logged to show footage of core recovered, core loss, and location; location and spacing of fractures and parting planes; lithologic type or types; alternation of types; physical conditions and variations in conditions; toughness, hardness, coherence; obvious porosity; grain size, texture, variations in grain size and texture; type or types of breakage. If the surface of the core being examined is wetted, it is usually easier to recognize significant features and changes in lithology. Most of the information usually required can be obtained by careful visual examination, scratch and acid tests, and hitting the core with a hammer. A preliminary analysis of the test results may indicate that the results from one or another of the subdivisions are not significantly different and the groups may be combined. On the other hand, the analysis may disclose significant differences within a given group of specimens and a further subdivision may be required. Most rock is anisotropic and, if the core stock and sample procedure permit, a group of specimens should be obtained from the three mutually perpendicular directions. Usually these directions are oriented with respect to some petrographic property of the rocks such as bedding, schistosity, cleavage, or fabric. In bedded rock the greatest difference in properties occurs in specimens taken perpendicular and parallel to the bedding, and generally this type of rock is sampled only in these two directions. The petrographic examination may disclose mineral components that are soluble or that expand or soften in water, as for example, bentonites or other clays. The intent of this inspection is to provide a basis for the selection of samples for engineering tests. This basis will be rock types, amounts of rock types, differences within a rock type, etc. At this point, the petrographer, in conjunction with the project leader, should select the sections of core(s) that will be subjected to engineering tests. The detailed petrographic examination will usually be made on unused portions of some or all of the test pieces so that the petrographic data can be matched to the engineering data. This matching should mean that, in addition to petrographic characterization, the petrographic data should serve as a basis for understanding the physical test data within and between sample groups.

3. Apparatus and Supplies

3.1 The apparatus and supplies listed in the following subparagraphs (a) and (b) comprise a recommended selection which will permit the use of all of the procedures described in this recommended practice. All specific items have been used in connection with the performance of petrographic examinations by the procedures described herein; it is not, however, intended to imply that other items cannot be substituted to serve similar functions. Whenever possible the selection of particular apparatus and supplies should be left to the judgment of the petrographer who is to perform the work so that items obtained will be those with which he has the greatest experience and familiarity. The minimum equipment regarded as essential to the making of petrographic examinations are those items, or equivalent apparatus or supplies that will serve the same purpose, that are indicated by asterisks in the lists in subparagraphs (a) and (b).

(a) Apparatus and Supplies for Preparation of Specimens:

(1) Rock-Cutting Saw,* preferably with a 20-in. (508-mm) diameter blade or larger.

(2) Horizontal Grinding Wheel,* preferably 16 in. (400 mm) in diameter.

(3) Polishing Wheel, preferably 8 to 12 in. (200 to 300 mm) in diameter.

(4) Abrasives:* silicon carbide grit Nos. 100, 220, 320, 600, and 800; optical finishing alumina.

(5) Geologist's pick or hammer.

(6) Microscope Slides,* clear, noncorrosive, 25 by 45 mm in size.

(7) Canada Balsam,* neutral in xylene or other material to cement cover slips.

(8) Xylene.*

(9) Mounting Medium,* suitable for mounting rock slices for thin sections.

(10) Laboratory Oven.*

(11) Plate-Glass Squares,* about 12 in. (300 mm) on an edge for thin-section grinding.

(12) Micro Cover Glasses,* No. 1 noncorrosive, square, 12 to 12 mm, 25 mm, etc.

(13) Plattner mortar.

(b) Apparatus and Supplies for Examination of Specimens:

(1) Polarizing Microscope* with mechanical stage; low-, medium-, and high-power objectives, and objective centering devices; eyepieces of various powers; full- and quarter-wave compensators; quartz wedge.

(2) Microscope Lamps* (preferably including a sodium arc lamp).

(3) Stereoscopic Microscope* with objectives and oculars to give final magnifications from about 7X to about 140X.

(4) Magnet,* preferably Alnico, or an electromagnet.

(5) Needleholder and Points.*

(6) Dropping Bottles, 60-ml capacity.

(7) Forceps, smooth straight-pointed.

(8) Lens Paper.*

(9) Immersion Media,* $n = 1.410$ to $n = 1.785$ in steps of 0.005. (Note 2)

(10) Counter.

(11) Photomicrographic Camera and accessories. (Note 3)

(12) X-ray diffractometer.

(13) Differential thermal analysis system.

(14) Infrared absorption spectrometer.

NOTE 2--It is necessary that facilities be available to the petrographer to check the index of refraction of the immersion media. If accurate identification of materials is to be attempted, as for example the differentiation of quartz and chalcedony or the differentiation of basic from intermediate volcanic glass, the indices of refraction of the media need to be known with precision. Media will not be stable for very long periods of time and are subject to considerable variation due to temperature change. In laboratories not provided with close temperature control, it is often necessary to recalibrate immersion media several times during the course of a single day when accurate identifications are required. The equipment needed for checking immersion media consists of an Abbe Refractometer. The refractometer should be equipped with compensating prisms to read indices for sodium light from white light, or it should be used with a sodium arc lamp.

NOTE 3--It is believed that a laboratory that undertakes any considerable amount of petrographic work should be provided with facilities to make photomicrographic records of such features as cannot adequately be described in words. Photomicrographs can be taken using standard microscope lamps for illumination; however, it is recommended that whenever possible a zirconium arc lamp be provided for this purpose. For illustrations of typical apparatus, reference may be made to the paper by Mather and Mather.[1]

4. Report

4.1 First and foremost the report should be clear and useful to the engineer for whom it is intended. It should identify samples, give their source as appropriate, describe test procedures and equipment used as appropriate, describe the samples, and list the petrographic findings. Tabulations of data and photographs should be included as needed. Results that may bear on the engineering test data and the potential performance of the material should be clearly stated and their significance should be emphasized. It may also be appropriate to mention past performance records of the same or similar mate-

[1]This recommended practice is modified from the "Method of Petrographic Examination of Aggregates for Concrete," by Katharine Mather and Bryant Mather. Proceedings, American Society for Testing Materials, ASTM, Vol 50, 1950, pp 1288-1312.

rials if such information is available. In general, the report should be an objective statement. If any opinion is presented it should be clearly indicated to be an opinion. Finally, the petrographic report should make recommendations if and as appropriate.

5. References

5.1 ASTM Standards

C 294 (RTH 116) Descriptive Nomenclature for Constituents of Natural Mineral Aggregate

C 295 Guide for Petrographic Examination of Aggregates for Concrete

D 4992 Practice for Evaluation of Rock to be Used for Erosion Control

RTH 103-93

PREPARATION OF TEST SPECIMENS

1. Scope

1.1 In order to obtain valid results from tests on brittle materials, careful and precise specimen preparation is required. This method outlines preparatory procedures recommended for normal rock mechanics test progress.

2. Collection and Storage

2.1 Test material is normally collected from the field in the form of drilled cores. Field sampling procedures should be rational and systematic, and the material should be marked to indicate its original position and orientation relative to identifiable boundaries of the parent rock mass. Ideally, samples should be moistureproofed immediately after collection either by waxing, spraying, or packing in polyethylene bags or sheet. (Example: For moistureproofing by waxing, the following procedure can be used for core that will not fail apart in handling.

(a) Wrap core in a clear thin polyethylene such as GLAD WRAP or SARAN WRAP,

(b) Wrap in cheese cloth,

(c) Coat wrapped core with a lukewarm wax mixture to an approximate 1/4-in. (6.4-mm) thickness. The wax should consist of a 1 to 1 mixture of paraffin and microcrystalline wax, such as Sacony Vacuum Mobile Wax No. 2300 and 2305, Gulf Oil Corporation Petrowax A, and Humble Oil Company Microvan No. 1650.

Cores that could easily be broken by handling should be prepared using the soil sampling technique described on pages 4-20 and 4-21 of EM 1110-2-1907, 31 March 1972[9.1]). They should be transported as a fragile material and protected from excessive changes in humidity and temperature. The identification markings of all samples should be verified immediately upon their receipt at the laboratory, and an inventory of the samples received should be maintained. Samples should be examined and tested as soon as possible after receipt; however, it is often necessary to store samples for several days or even weeks to complete a large testing program. Every care must be taken to protect stored samples against damage. Core logs of samples should be available.

3. Avoidance of Contamination

3.1 The deformation and fracture properties of rock may be influenced by air, water, and other fluids in contact with their internal (crack and pore) surfaces. If these internal surfaces are contaminated by oils or other substances, their properties may be altered appreciably and give misleading test results. Of course, a cutting fluid is required with many types of specimen preparation equipment. Clean water is the preferred fluid. Even so, one must be cognizant of the effect of moisture on the test specimens. While it may be impossible to exactly duplicate the in situ conditions even if they were known, a concerted effort should be made to simulate the environment from which the samples came. Generally, there are three conditions to be considered:

(a) Hard, dense rock and low porosity will not normally be affected by moisture. This type of material is normally allowed to air-dry prior to testing to bring all samples to an equilibrium condition. Drying at temperatures above 120 °F (49 °C) is not recommended as excessive heat may cause an irreversible change in rock properties.

(b) Some shales and rocks containing clay will disintegrate if allowed to dry. Usually the disintegration of diamond drill cores can be prevented by wrapping the cores as they are drilled in a moistureproof material such as aluminum foil or chlorinated rubber, or sealing them in moistureproof containers.

(c) Mud shales and rock containing bentonites (e.g., tuff) may soften if the moisture content is too high. Most of the softer rocks can be cored or cut using compressed air to clear cuttings and to cool the bit or saw.

It is imperative to determine very early in the test program the moisture sensitivity of all types of material to be tested and to take steps to accommodate the requirements throughout the test life of the selected specimens.

4. Selection

4.1 Under the most favorable circumstances, a laboratory determination of the engineering properties of a small specimen gives an approximate guide to the behavior of an extensive, nonhomogeneous geological formation under the complex system of induced stresses. No other aspect of laboratory rock testing is as important as the selection of test specimens to best represent those features of a foundation which influence the analysis or design of a project. Closest teamwork of the laboratory personnel and the project engineer/geologist must be continued throughout the testing program since, as quantitative data become available, changes in the initial allocation of samples or the securing of additional samples may be necessary. Second in importance only to the selection of the most representative undisturbed material is the preparation and handling of the test specimens to preserve in every way possible the natural structure of the material. Indifferent handling of undisturbed rock can result in erroneous test data.

5. Coring

5.1 Virtually all laboratory coring is done with thin-wall diamond rotary bits, which may be detachable or integral to the core barrel. The usual size range for laboratory core drills is from 6-in.-diam (152.4-mm) down to 1-in. (25.4-mm) outside diameter. Typical sample diameters for uniaxial testing are 2.125 in. (54 mm). Drilling machines range from small quarry drills to modified machine shop drill presses. Almost any kind can be adapted for rock work by fitting a water swivel, but a heavy, rigid machine is desirable in order to assure consistent production of high quality core. The work block must be clamped tightly to a strong base or table so as to prevent any tilting, oscillation, or other shifting. To avoid unnecessary unclamping and rearrangement of the work block, it is desirable to have provision for traversing the drill head or the work block. Traversing devices must lock securely to eliminate any play between drill and work. The drill travel should be sufficient to permit continuous runs of at least 10 to 12 in. (254 to 304.8 mm), without need for stopping the machine. Optimum drilling speeds vary with bit size and rock type, and to some extent with condition of the bit and the characteristics of the machine. The general trend is that drill speed increases as drill diameter decreases; also, higher drill speeds are sometimes used on softer rocks. The broad range of drill speeds lies mainly between 200 and 2,000 rpm. No hard-and-fast rules can be given, but an experienced operator can easily choose a suitable speed by trial. Some core drills are hand-fed, but it is desirable to have some provision for automatic feed. The ideal feed arrangement is a constant-force hydraulic feed which can be set for each bit size and rock type, but such machines are quite rare. Constant-force feed can be improvised by means of a weight and pulley arrangement. On adapted metal-working drill presses, the automatic feed rate for a given drill size and rock type can be determined; however, since there is a danger of damaging the machine or the core barrel if too high a feed rate is used, an electrical overload breaker should be provided.

6. Sawing

6.1 For heavy sawing, a slabbing saw is adequate for most purposes. For exact sawing, a precision cutoff machine, with a diamond abrasive wheel about 10 in. (254 mm) in diameter, and a table with two-way screw traversing and provision for rotation are recommended. The speed of the wheel is usually fixed, but the feed rate of the wheel through the work can be controlled. Clean water, either direct from house supply or recirculated through a settling tank, is the standard cutting and cooling fluid. For crosscutting, core should be clamped in a vee-block slotted to permit passage of the wheel. By supporting the core on both sides of the cut, the problem of spalling and lip formation at the end of the cut is largely avoided. Saw cuts should be relatively smooth and perpendicular to the core axis in order to minimize the grinding or lapping needed to produce end conditions required for the various tests.

7. End Preparation

7.1 Due to the rather large degree of flatness required on bearing surfaces for many tests, end grinding or lapping is required. Conventional surface grinders provided the most practical means of preparing flat surfaces, especially on core samples with diameters greater than approximately 2 in. (50.8 mm). Procedures are essentially comparable to metal working. Quite often a special jig is constructed to hold one or more specimens in the grinding operation. The lathe can also be used for end-grinding cylindrical samples. A sample is held directly in the chuck, rotated at 200 to 300 rpm, and the grinding wheel, its axis inclined some 15 deg (0.26 radian) to the sample axis, is passed across end of the sample with rotating at 6,000 to 8,000 rpm. The "bite" ranges from about 0.003 in. (0.0762-mm) maximum to less than 0.001 in. (0.0254 mm) for finishing, and the grinding wheel is passed across the sample at about 0.5 in. (12.7 mm) per minute. For core diameters of 2-1/8 in. (54 mm) or less, a lap can be used for grinding flat end surfaces on specimens, although producing a sufficiently flat surface by this method is an art. To end-grind on the lap, a cylindrical specimen is placed in a steel-carrying tube which is machined to accept core with a clearance of about 0.002 in. (0.0508 mm). At the lower end of this tube is a steel collar which rests on the lapping wheel. The method requires use of grinding compounds and, hence, is not recommended where other methods are available.

8. Specimen Check

8.1 In general terms, test specimens should be straight, their diameter should be constant, and the ends should be flat, parallel, and normal to the long axis. Sample dimensions should be checked during machining with a micrometer or vernier caliper; final dimensions are normally measured with a micrometer and reported to the nearest 0.01 in. (0.254 mm). Tolerances are best checked on a comparator fitted with a dial micrometer reading to 0.0001 in. (0.00254 mm). There is a technique for revealing the roughness and planes qualitatively. Impressions are made by sandwiching a sheet of carbon paper and a sheet of white paper between the sample end and a smooth surface. The upper end of the sample is given a light blow with a rubber or plastic hammer, and an imprint is formed on the white paper. Areas where no impressions are made indicate dished or uneven surfaces. The importance of proper specimen preparation cannot be overemphasized. Specimens should not be tested which do not meet the dimensional tolerances specified in the respective test methods.

9. References

9.1 Department of the Army, Office, Chief of Engineers, "Soil Sampling," EM 1110-2-1907, Washington, D.C., 1972.

STATISTICAL CONSIDERATIONS

1. Scope

1.1 The purpose of this recommended practice is to outline some general statistical concepts which may be applied to small sample sizes typical of rock test data. It is not the intent to deal with accuracy or precision considerations, but rather to assess the results from the assumed point that the test has been conducted as specified in the respective test methods.

2. Variation and Sample Size

2.1 Most physical tests involve tabulation of a series of readings, with computation of an average said to be representative of the whole. The question arises as to how representative this average is as the measure of the characteristic under test. Three important factors introduce uncertainties in the result:

(a) Instrument and procedural errors.

(b) Variations in the sample being tested.

(c) Variations between the sample and the other samples that might have been drawn from the same source.

If a number of identical specimens were available for tests, or if the tests were nondestructive and could be repeated a number of times on the same specimens, determination of the procedural and instrument errors would be comparatively simple, because in such a test the sample variation would be zero. Periodic tests on this specimen or group of specimens could be used to check the performance of the test procedure and equipment. However, as most of the tests used in determining the mechanical properties of rock are destructive, the instrument and sample variations cannot be separated. Nevertheless, we may apply some elementary statistical concepts and still have confidence in the test results. Of course, the more test data available the more reliable the results. Due to the expense of testing occasionally the shortage of test specimens, rock test data almost always require treatment as groups of small samples. As a general rule at least 10 tests are recommended for any one condition of each individual test with an absolute minimum of 5.

3. Measures of Central Tendency and Deviation

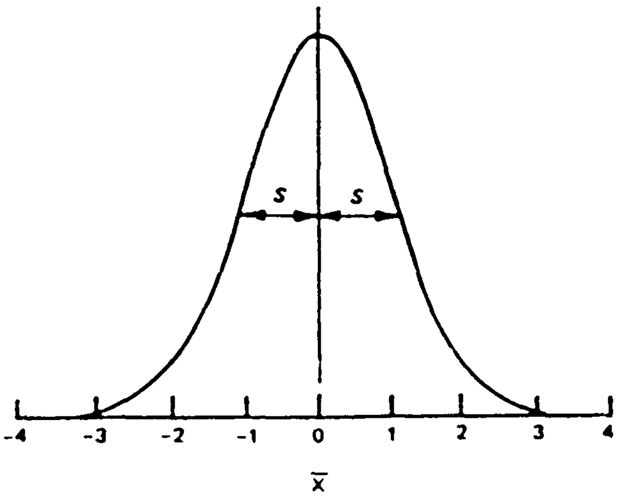
3.1 The most commonly used measure of the central tendency of a sample of n specimens is the arithmetic mean or average, $\bar{x}$. The standard deviation, s, is a measure of the sample variability. It is calculated as follows:

$$s = \sqrt{\frac{\sum (x_i - \bar{x})^2}{n-1}}$$

where x_i's are the individual observations. Most calculators have a built-in program that calculates standard deviation. Some have two programs, one that calculates s based on n-1 degrees of freedom and one that calculates s based on n degrees of freedom. The one that uses n-1 degrees of freedom is correct for most applications.

4. The Normal Distribution

4.1 A series of tests for any one property will, of course, have some variation in the individual determinations. The distribution of these bits of data quite often follows a pattern of normal distributions, i.e., a large portion of the data bits will be closely grouped to the mean on either side with progressively fewer bits distributed farther from the mean. The normal distribution is usually portrayed graphically as shown below.



Methods are available to check the normality of the distribution and to deal with those which are not normal (skewed).[7.1-7.3] Use of many statistical techniques requires the assumption that the data be normally distributed, but mild cases of skewness do not normally cause severe errors and can be ignored. A more important assumption is that the samples be taken at random from the population, as discussed in paragraph 5.

5. Sampling

5.1 It is extremely important that samples from a parent population be taken at random. Probably the most frequent cause of incorrect conclusions being drawn about a population from a sample is non-random sampling. Sometimes it is impossible to draw random samples because of physical or cost limitations on a project. When this happens, interpretations of results should be made with caution.

5.2 A study of sampling distributions of statistics for small samples (n < 30) is called small sampling theory. Statistical tables are available for use with the proper relationship to develop confidence in test data.[7.1] For small samples the confidence limits for a mean are given by:

$$\bar{x} \pm t_c \frac{s}{\sqrt{n - 1}}$$

$\bar{x}$, n , and s are determined as given in paragraph 3.1. t_c is from and depends on the level of confidence desired and the sample size. Values of t_c for 90, 95, and 99 percent confidence limits are given below for n from 3 to 12.

		t_c, %		
n	DF*	90	95	99
3	2	2.92	4.30	9.92
4	3	2.35	3.18	5.84
5	4	2.13	2.78	4.60
6	5	2.02	2.57	4.03
7	6	1.94	2.45	3.71
8	7	1.90	2.36	3.50
9	8	1.86	2.31	3.36
10	9	1.83	2.26	3.25
11	10	1.81	2.23	3.17
12	11	1.80	2.20	3.11

* Degrees of freedom

Using this calculation and given an estimate of the standard deviation of a population, one can estimate the number of samples needed to get a desired level of confidence on the mean. See paragraph 7.2 for an example.

6. Dealing with Outliers

6.1 An outlying observation or "outlier" is one that appears to deviate markedly from other members of the sample in which it occurs. Outliers may be merely unusual examples of the population extremes or they may actually be samples taken accidentally from another population. In the former case, the observations should not be deleted from the sample, whereas in the latter case they should be deleted. It is often difficult to know which applies. When some assignable cause is known, for example when one rock specimen is allowed to dry out prior to strength testing and all others are wet, then the dry specimen is reasonably considered to belong to a different population and should be discarded. In the absence of an assignable cause, outliers should not be discarded without first examining the observation in light of an objective procedure. This is particularly critical with small samples. For example, it is quite common when samples of 3 are taken that two observations fall quite close together and the third to be somewhat away. Without substantial experience or an objective method, intuition often misleads one into discarding the outlier. Many test methods provide guidance on handling outliers. ASTM E 178[7.5] provides a general technique for handling outliers.

7. Examples

7.1 For the purposes of illustrating a confidence limit calculation, assume there is a normal distribution of compressive strength data[7.4] for a particular rock type yielding the following individual specimen strengths, taken at random: 18,000; 18,700; 19,200; 19,600; 20,000; 20,100; 20,500; 20,800; 21,100; and 22,000 psi. Find the 95 percent confidence limits for the mean strength.

By computation:

$$n = 10$$

$$\bar{x} = 20{,}000 \text{ psi}$$

$$s = 1183$$

$$95\% \text{ confidence limits} = \bar{x} \pm t_{95}\ (s/\sqrt{n-1})\text{; thus}$$

$$\bar{x} \pm 2.26(1183/\sqrt{10-1}) =$$

$$20{,}000 \pm 2.26\ (420) = 20{,}000 \pm 891 \text{ psi}$$

Thus, we can be 95 percent confident that the true mean lies between 19,109 and 20,891.

7.2 As an example of the use of confidence limit calculation to estimate sample sizes, consider the population described in paragraph 7.1. Suppose the objective of a sample was to estimate the mean strength of that population with a confidence limit on that mean of ±1,000 psi. Then confidence limit could be calculated for a range of sample sizes, as follows:

Sample Size	95% Confidence Interval
4	1636 psi
5	1238 psi
6	1024 psi
7	891 psi

By inspection, a sample size of about 6 should be suitable to achieve the desired confidence limit.

8. References

8.1 Spiegel, M. R., Theory and Problems of Statistics, Schaun's Outline Series, McGraw-Hill Company, New York, 1961.

8.2 Volk, William, Applied Statistics for Engineers, McGraw-Hill Company, New York, 1958.

8.3 Obert, Leonard and Duvall, W. I., Rock Mechanics and the Design of Structures in Rock, John Wiley and Sons, Inc., New York, 1967.

8.4 "Engineering Geology; Special Issue, Uniaxial Testing in Rock Mechanics Laboratories," Elsevier Publishing Company, Amsterdam, Vol 4, No. 3, July 1970.

8.5 ASTM E 178. "Standard Practice for Dealing with Outlying Observations," Annual Book of ASTM Standards, Vol 14.02, ASTM, Philadelphia, PA.

METHOD FOR DETERMINATION OF REBOUND NUMBER OF ROCK

1. Scope

1.1 This method provides instructions for the determination of a rebound number of rock using a spring-driven steel hammer (Fig. 1).

1 Impact plunger
3 Housing compl.
4 Rider with guide rod
5 Scale (starting with serial No. 230 printed on window No. 19)
6 Pushbutton compl.
7 Hammer guide bar
8 Disk
9 Cap
10 Two-part ring
11 Rear cover
12 Compression spring
13 Pawl
14 Hammer mass
15 Retaining spring
16 Impact spring
17 Guide sleeve
18 Felt washer
19 Plexiglass window
20 Trip screw
21 Lock nut
22 Pin
23 Pawl spring

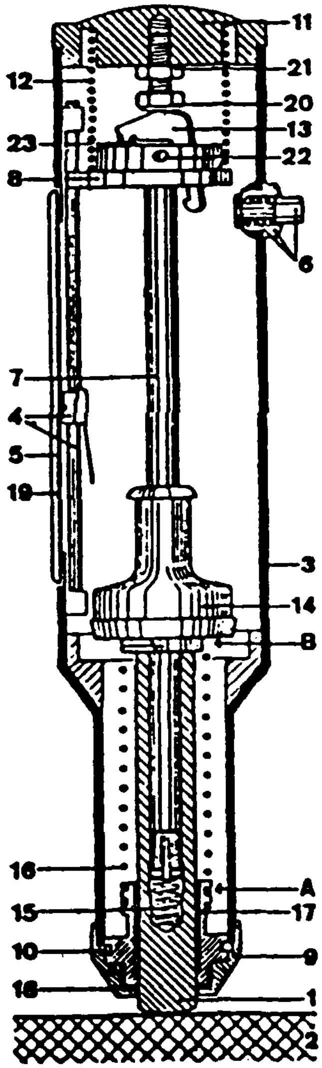


Fig. 1. Spring-driven steel hammer.

2. Significance

2.1 The rebound number determined by this method may be used to assess the uniformity of rock in situ, or on cored samples to indicate hardness characteristics of the rock.

3. Apparatus

3.1 Rebound Hammer - The rebound hammer consists of a spring-loaded steel hammer which when released strikes a steel plunger which is in contact with the test surface. The spring-loaded hammer must travel with a fixed and reproducible velocity. The rebound distance of the steel hammer from the steel plunger is measured by means of a linear scale attached to the frame of the instrument (Note 1).

NOTE 1—Several types and sizes of rebound hammers are commercially available. Hammers with an energy impact of 0.075 m-kg (0.542 ft-lb) have been found satisfactory for rock testing.

3.2 Abrasive Stone - An abrasive stone consisting of medium grain texture silicon carbide or equivalent material shall be provided.

4. Test Area

4.1 Selection of Test Surface - Surfaces to be tested shall be at least 2 in. (50 mm) thick and fixed within a stratum. Specimens should be rigidly supported. Some companies market a "rock cradle" for this purpose. Areas exhibiting scaling, rough texture, or high porosity should be avoided. Dry rocks give higher rebound numbers than wet.

4.2 Preparation of Test Surface - Heavily textured soft surfaces or surfaces with loose particles shall be ground smooth with the abrasive stone described in Section 3.2. Smooth surfaces shall be tested without grinding. The effects of drying and carbonation can be minimized by thoroughly wetting the surfaces for 24 hours prior to testing.

4.3 Factors Affecting Test Results - Other factors related to test circumstances may affect the results of the test:

(a) Rock at 32°F (0°C) or less may exhibit very high rebound values. Rock should be tested only after it has thawed.

(b) The temperature of the rebound hammer itself may affect the rebound number (Note 2).

NOTE 2—Rebound hammers at 0°F (-18°C) may produce rebound numbers reduced by as much as 2 or 3 units.

(c) For readings to be compared, the direction of impact--horizontal, downward, upward, etc.—must be the same.

(d) Different hammers of the same nominal design may give rebound numbers differing by 1 to 3 units, and therefore, to be compared, tests should be made with a single hammer. If more than one hammer is to be used, a sufficient number of tests must be made on typical rock surfaces to determine the magnitude of the differences to be expected. (Note 3)

NOTE 3—Rebound hammers require periodic servicing and verification annually for hammers in heavy use, biennially for hammers in less frequent use, and whenever there is reason to question their proper operation. Metal anvils are available for verification and are recommended. However, verification on an anvil will not guarantee that different hammers will yield the same results at other points on the rebound scale. Some users compare several hammers on surfaces encompassing the usual range of rebound values encountered in the field.

5. Test Procedure

5.1 The instrument shall be firmly held in a position which allows the plunger to strike perpendicular to the surface tested. The pressure on the plunger shall be gradually increased until the hammer impacts. After impact, the rebound number should be recorded to two significant figures. Ten readings shall be taken from each test area with no two impact tests being closer together than 1 in. (25.4 mm). Examine the impression made on the surface after impact and disregard the reading if the impact crushes or breaks through the surface.

6. Calculation

6.1 Readings differing from the average of 10 readings by more than 7 units are to be discarded and the average of the remaining readings determined. If more than 2 readings differ from the average by 7 units, the entire set of readings should be discarded.

7. Precision

7.1 The single-specimen-operator-machine-day precision is 2.5 units (1S) as defined in ASTM Recommended Practice E 177, "Use of the Terms Precision and Accuracy as Applied to Measurement of a Property of a Material."

8. Use and Interpretation of Rebound Hammer Results

8.1 Optimally, rebound numbers may be correlated with core testing information. There is a relationship between rebound number and strength and deformation and the relationship is normally provided by the rebound hammer manufacturer.

9. Report

9.1 The report should include the following information for each test area:

9.1.1 Rock identification.

9.1.2 Location of rock stratum.

9.1.3 Description and composition of rock if known.

9.1.4 Average rebound number for each test area or specimen.

9.1.5 Approximate angular direction of rebound hammer impact, with horizontal being considered 0, vertically upward being +90 deg (1.57 radians), and vertically downward being -90 deg (1.57 radians).

9.1.6 Hammer type and serial number.

RTH 106-93

METHOD FOR DETERMINATION OF THE WATER CONTENT OF A ROCK SAMPLE

1. Scope

1.1 This test method covers the determination of the percentage of evaporable water in the pores of a rock sample as a percentage of the ovendry sample mass.

2. Apparatus

(a) An oven capable of maintaining a temperature of 110 ± 5 °C for a period of at least 24 hr.

(b) A sample container of noncorrodible material, including an airtight lid.

(c) A desiccator to hold sample containers during cooling.

(d) A balance of adequate capacity, capable of weighing to an accuracy of 0.01 percent of the sample mass.

3. Procedure

(a) The container and lid is cleaned, dried, and its mass determined.

(b) A representative sample is selected, preferably comprising at least ten rock lumps each having a mass of at least 50 g to give a total sample mass of at least 500 g. For in situ water content determination, sampling, storage, and handling precautions should retain water content to within 1 percent of its in situ value.

(c) The sample is placed in the container, the lid replaced, and the mass of the sample plus container is determined.

(d) The lid is removed and the sample dried to constant mass. Constant mass is achieved when the mass loss is less than 0.1 percent of the sample mass in 4 hr of drying.

(e) The lid is replaced and the sample allowed to cool in the desiccator for 30 minutes. The mass of sample plus container is determined.

4. Calculation

$$\text{Water content } w = \frac{\text{pore water weight}}{\text{grain weight}} \cdot 100\% = \frac{Y-Z}{Z-X} \cdot 100\%$$

4.1 Calculate the water content of the rock sample as follows:

$$W = \frac{Y-Z}{Z-X}(100)$$

where

W - water content, percent

Y - original sample mass, g

Z - dried sample mass, g, and

X - sample container and lid mass, g

5. Reporting of Results

5.1 The water content should be reported to the nearest 0.1 percent stating whether this corresponds to in situ water content, in which case precautions taken to retain water during sampling and storage should be specified.

Designation: C 127 – 88

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa. 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate[1]

This standard is issued under the fixed designation C 127; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This test method has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

1. Scope

1.1 This test method covers the determination of specific gravity and absorption of coarse aggregate. The specific gravity may be expressed as bulk specific gravity, bulk specific gravity (SSD) (saturated-surface-dry), or apparent specific gravity. The bulk specific gravity (SSD) and absorption are based on aggregate after 24 h soaking in water. This test method is not intended to be used with lightweight aggregates.

1.2 The values stated in SI units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 29 Test Method for Unit Weight and Voids in Aggregate[2]

C 125 Definitions of Terms Relating to Concrete and Concrete Aggregates[2]

C 128 Test Method for Specific Gravity and Absorption of Fine Aggregate[2]

C 136 Method for Sieve Analysis of Fine and Coarse Aggregates[2]

C 566 Test Method for Total Moisture Content of Aggregate by Drying[3]

C 670 Practice for Preparing Precision Statements for Test Methods for Construction Materials[2]

C 702 Practice for Reducing Field Samples of Aggregate to Testing Size[3]

D 75 Practice for Sampling Aggregates[2]

D 448 Classification for Sizes of Aggregate for Road and Bridge Construction[2]

E 11 Specification for Wire-Cloth Sieves for Testing Purposes[4]

E 12 Definitions of Terms Relating to Density and Specific Gravity of Solids, Liquids, and Gases[5]

2.2 *AASHTO Standard:*

AASHTO No. T 85 Specific Gravity and Absorption of Coarse Aggregate[6]

3. Terminology

3.1 *Definitions:*

3.1.1 *absorption*—the increase in the weight of aggregate due to water in the pores of the material, but not including water adhering to the outside surface of the particles, expressed as a percentage of the dry weight. The aggregate is considered "dry" when it has been maintained at a temperature of 110 ± 5°C for sufficient time to remove all uncombined water.

3.1.2 *specific gravity*—the ratio of the mass (or weight in air) of a unit volume of a material to the mass of the same volume of water at stated temperatures. Values are dimensionless.

3.1.2.1 *apparent specific gravity*—the ratio of the weight in air of a unit volume of the impermeable portion of aggregate at a stated temperature to the weight in air of an equal volume of gas-free distilled water at a stated temperature.

3.1.2.2 *bulk specific gravity*—the ratio of the weight in air of a unit volume of aggregate (including the permeable and impermeable voids in the particles, but not including the voids between particles) at a stated temperature to the weight in air of an equal volume of gas-free distilled water at a stated temperature.

3.1.2.3 *bulk specific gravity (SSD)*—the ratio of the weight in air of a unit volume of aggregate, including the weight of water within the voids filled to the extent achieved by submerging in water for approximately 24 h (but not including the voids between particles) at a stated temperature, compared to the weight in air of an equal volume of gas-free distilled water at a stated temperature.

NOTE 1—The terminology for specific gravity is based on terms in Definitions E 12, and that for absorption is based on that term in Terminology C 125.

4. Summary of Test Method

4.1 A sample of aggregate is immersed in water for approximately 24 h to essentially fill the pores. It is then removed from the water, the water dried from the surface of

[1] This test method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.05 on Methods of Testing and Specifications for Physical Characteristics of Concrete Aggregates.

Current edition approved Oct. 31, 1988. Published December 1988. Originally published as C 127 – 36 T. Last previous edition C 127 – 84.

[2] *Annual Book of ASTM Standards*, Vols 04.02 and 04.03.

[3] *Annual Book of ASTM Standards*, Vol 04.02.

[4] *Annual Book of ASTM Standards*, Vol 14.02.

[5] *Annual Book of ASTM Standards*, Vols 04.02, 15.05.

[6] Available from American Association of State Highway and Transportation Officials, 444 North Capitol St. N.W., Suite 225, Washington, DC 20001.

the particles, and weighed. Subsequently the sample is weighed while submerged in water. Finally the sample is oven-dried and weighed a third time. Using the weights thus obtained and formulas in this test method, it is possible to calculate three types of specific gravity and absorption.

5. Significance and Use

5.1 Bulk specific gravity is the characteristic generally used for calculation of the volume occupied by the aggregate in various mixtures containing aggregate, including portland cement concrete, bituminous concrete, and other mixtures that are proportioned or analyzed on an absolute volume basis. Bulk specific gravity is also used in the computation of voids in aggregate in Test Method C 29. Bulk specific gravity (SSD) is used if the aggregate is wet, that is, if its absorption has been satisfied. Conversely, the bulk specific gravity (oven-dry) is used for computations when the aggregate is dry or assumed to be dry.

5.2 Apparent specific gravity pertains to the relative density of the solid material making up the constituent particles not including the pore space within the particles which is accessible to water.

5.3 Absorption values are used to calculate the change in the weight of an aggregate due to water absorbed in the pore spaces within the constituent particles, compared to the dry condition, when it is deemed that the aggregate has been in contact with water long enough to satisfy most of the absorption potential. The laboratory standard for absorption is that obtained after submerging dry aggregate for approximately 24 h in water. Aggregates mined from below the water table may have a higher absorption, when used, if not allowed to dry. Conversely, some aggregates when used may contain an amount of absorbed moisture less than the 24-h soaked condition. For an aggregate that has been in contact with water and that has free moisture on the particle surfaces, the percentage of free moisture can be determined by deducting the absorption from the total moisture content determined by Test Method C 566.

5.4 The general procedures described in this test method are suitable for determining the absorption of aggregates that have had conditioning other than the 24-h soak, such as boiling water or vacuum saturation. The values obtained for absorption by other test methods will be different than the values obtained by the prescribed 24-h soak, as will the bulk specific gravity (SSD).

5.5 The pores in lightweight aggregates may or may not become essentially filled with water after immersion for 24 h. In fact, many such aggregates can remain immersed in water for several days without satisfying most of the aggregates' absorption potential. Therefore, this test method is not intended for use with lightweight aggregate.

6. Apparatus

6.1 *Balance*—A weighing device that is sensitive, readable, and accurate to 0.05 % of the sample weight at any point within the range used for this test, or 0.5 g, whichever is greater. The balance shall be equipped with suitable apparatus for suspending the sample container in water from the center of the weighing platform or pan of the weighing device.

6.2 *Sample Container*—A wire basket of 3.35 mm (No. 6) or finer mesh, or a bucket of approximately equal breadth and height, with a capacity of 4 to 7 L for 37.5-mm (1½-in.) nominal maximum size aggregate or smaller, and a larger container as needed for testing larger maximum size aggregate. The container shall be constructed so as to prevent trapping air when the container is submerged.

6.3 *Water Tank*—A watertight tank into which the sample container may be placed while suspended below the balance.

6.4 *Sieves*—A 4.75-mm (No. 4) sieve or other sizes as needed (see 7.2, 7.3, and 7.4), conforming to Specification E 11.

7. Sampling

7.1 Sample the aggregate in accordance with Practice D 75.

7.2 Thoroughly mix the sample of aggregate and reduce it to the approximate quantity needed using the applicable procedures in Methods C 702. Reject all material passing a 4.75-mm (No. 4) sieve by dry sieving and thoroughly washing to remove dust or other coatings from the surface. If the coarse aggregate contains a substantial quantity of material finer than the 4.75-mm sieve (such as for Size No. 8 and 9 aggregates in Classification D 448), use the 2.36-mm (No. 8) sieve in place of the 4.75-mm sieve. Alternatively, separate the material finer than the 4.75-mm sieve and test the finer material according to Test Method C 128.

7.3 The minimum weight of test sample to be used is given below. In many instances it may be desirable to test a coarse aggregate in several separate size fractions; and if the sample contains more than 15 % retained on the 37.5-mm (1½-in.) sieve, test the material larger than 37.5 mm in one or more size fractions separately from the smaller size fractions. When an aggregate is tested in separate size fractions, the minimum weight of test sample for each fraction shall be the difference between the weights prescribed for the maximum and minimum sizes of the fraction.

Nominal Maximum Size, mm (in.)	Minimum Weight of Test Sample, kg (lb)
12.5 (½) or less	2 (4.4)
19.0 (¾)	3 (6.6)
25.0 (1)	4 (8.8)
37.5 (1½)	5 (11)
50 (2)	8 (18)
63 (2½)	12 (26)
75 (3)	18 (40)
90 (3½)	25 (55)
100 (4)	40 (88)
112 (4½)	50 (110)
125 (5)	75 (165)
150 (6)	125 (276)

7.4 If the sample is tested in two or more size fractions, determine the grading of the sample in accordance with Method C 136, including the sieves used for separating the size fractions for the determinations in this method. In calculating the percentage of material in each size fraction, ignore the quantity of material finer than the 4.75-mm (No. 4) sieve (or 2.36-mm (No. 8) sieve when that sieve is used in accordance with 7.2).

8. Procedure

8.1 Dry the test sample to constant weight at a temperature of 110 ± 5°C (230 ± 9°F), cool in air at room temperature for 1 to 3 h for test samples of 37.5-mm (1½-in.) nominal maximum size, or longer for larger sizes until the aggregate has cooled to a temperature that is comfortable to handle (approximately 50°C). Subsequently immerse the aggregate in water at room temperature for a period of 24 ± 4 h.

NOTE 2—When testing coarse aggregate of large nominal maximum size requiring large test samples, it may be more convenient to perform the test on two or more subsamples, and the values obtained combined for the computations described in Section 9.

8.2 Where the absorption and specific gravity values are to be used in proportioning concrete mixtures in which the aggregates will be in their naturally moist condition, the requirement for initial drying to constant weight may be eliminated, and, if the surfaces of the particles in the sample have been kept continuously wet until test, the 24-h soaking may also be eliminated.

NOTE 3—Values for absorption and bulk specific gravity (SSD) may be significantly higher for aggregate not oven dried before soaking than for the same aggregate treated in accordance with 8.1. This is especially true of particles larger than 75 mm (3 in.) since the water may not be able to penetrate the pores to the center of the particle in the prescribed soaking period.

8.3 Remove the test sample from the water and roll it in a large absorbent cloth until all visible films of water are removed. Wipe the larger particles individually. A moving stream of air may be used to assist in the drying operation. Take care to avoid evaporation of water from aggregate pores during the operation of surface-drying. Weigh the test sample in the saturated surface-dry condition. Record this and all subsequent weights to the nearest 0.5 g or 0.05 % of the sample weight, whichever is greater.

8.4 After weighing, immediately place the saturated-surface-dry test sample in the sample container and determine its weight in water at 23 ± 1.7°C (73.4 ± 3°F), having a density of 997 ± 2 kg/m^3. Take care to remove all entrapped air before weighing by shaking the container while immersed.

NOTE 4—The container should be immersed to a depth sufficient to cover it and the test sample during weighing. Wire suspending the container should be of the smallest practical size to minimize any possible effects of a variable immersed length.

8.5 Dry the test sample to constant weight at a temperature of 110 ± 5°C (230 ± 9°F), cool in air at room temperature 1 to 3 h, or until the aggregate has cooled to a temperature that is comfortable to handle (approximately 50°C), and weigh.

9. Calculations

9.1 *Specific Gravity:*

9.1.1 *Bulk Specific Gravity*—Calculate the bulk specific gravity, 23/23°C (73.4/73.4°F), as follows:

$$\text{Bulk sp gr} = A/(B - C)$$

where:

A = weight of oven-dry test sample in air, g,
B = weight of saturated-surface-dry test sample in air, g, and
C = weight of saturated test sample in water, g.

9.1.2 *Bulk Specific Gravity (Saturated-Surface-Dry)*—Calculate the bulk specific gravity, 23/23°C (73.4/73.4°F), on the basis of weight of saturated-surface-dry aggregate as follows:

$$\text{Bulk sp gr (saturated-surface-dry)} = B/(B - C)$$

9.1.3 *Apparent Specific Gravity*—Calculate the apparent specific gravity, 23/23°C (73.4/73.4°F), as follows:

$$\text{Apparent sp gr} = A/(A - C)$$

9.2 *Average Specific Gravity Values*—When the sample is tested in separate size fractions the average value for bulk specific gravity, bulk specific gravity (SSD), or apparent specific gravity can be computed as the weighted average of the values as computed in accordance with 9.1 using the following equation:

$$G = \frac{1}{\dfrac{P_1}{100\,G_1} + \dfrac{P_2}{100\,G_2} + \dots \dfrac{P_n}{100\,G_n}} \quad \text{(see Appendix X1)}$$

where:

G = average specific gravity. All forms of expression of specific gravity can be averaged in this manner.
$G_1, G_2 \dots G_n$ = appropriate specific gravity values for each size fraction depending on the type of specific gravity being averaged.
$P_1, P_2, \dots P_n$ = weight percentages of each size fraction present in the original sample.

NOTE 5—Some users of this test method may wish to express the results in terms of density. Density may be determined by multiplying the bulk specific gravity, bulk specific gravity (SSD), or apparent specific gravity by the weight of water (997.5 kg/m^3 or 0.9975 Mg/m^3 or 62.27 lb/ft^3 at 23°C). Some authorities recommend using the density of water at 4°C (1000 kg/m^3 or 1.000 Mg/m^3 or 62.43 lb/ft^3) as being sufficiently accurate. Results should be expressed to three significant figures. The density terminology corresponding to bulk specific gravity, bulk specific gravity (SSD), and apparent specific gravity has not been standardized

9.3 *Absorption*—Calculate the percentage of absorption, as follows:

$$\text{Absorption, \%} = [(B - A)/A] \times 100$$

9.4 *Average Absorption Value*—When the sample is tested in separate size fractions, the average absorption value is the average of the values as computed in 9.3, weighted in proportion to the weight percentages of the size fractions in the original sample as follows:

$$A = (P_1A_1/100) + (P_2A_2/100) + \dots (P_nA_n/100)$$

where:

A = average absorption, %,
$A_1, A_2 \dots A_n$ = absorption percentages for each size fraction, and
$P_1, P_2, \dots P_n$ = weight percentages of each size fraction present in the original sample.

10. Report

10.1 Report specific gravity results to the nearest 0.01, and indicate the type of specific gravity, whether bulk, bulk (saturated-surface-dry), or apparent.

10.2 Report the absorption result to the nearest 0.1 %.

10.3 If the specific gravity and absorption values were determined without first drying the aggregate, as permitted in 8.2, it shall be noted in the report.

11. Precision and Bias

11.1 The estimates of precision of this test method listed in Table 1 are based on results from the AASHTO Materials Reference Laboratory Reference Sample Program, with testing conducted by this test method and AASHTO Method T 85. The significant difference between the methods is that Test Method C 127 requires a saturation period of 24 ± 4 h, while Method T 85 requires a saturation period of 15 h minimum. This difference has been found to have an insignificant effect on the precision indices. The data are based on the analyses of more than 100 paired test results from 40 to 100 laboratories.

11.1 *Bias*—Since there is no accepted reference material for determining the bias for the procedure in this test method, no statement on bias is being made.

TABLE 1 Precision

	Standard Deviation (1S)[A]	Acceptable Range of Two Results (D2S)[A]
Single-Operator Precision:		
Bulk specific gravity (dry)	0.009	0.025
Bulk specific gravity (SSD)	0.007	0.020
Apparent specific gravity	0.007	0.020
Absorption[B], %	0.088	0.25
Multilaboratory Precision:		
Bulk specific gravity (dry)	0.013	0.038
Bulk specific gravity (SSD)	0.011	0.032
Apparent specific gravity	0.011	0.032
Absorption[B], %	0.145	0.41

[A] These numbers represent, respectively, the (1S) and (D2S) limits as described in Practice C 670. The precision estimates were obtained from the analysis of combined AASHTO Materials Reference Laboratory reference sample data from laboratories using 15 h minimum saturation times and other laboratories using 24 ± 4 h saturation times. Testing was performed on normal-weight aggregates, and started with aggregates in the oven-dry condition.

[B] Precision estimates are based on aggregates with absorptions of less than 2 %.

APPENDIXES

(Nonmandatory Information)

X1. DEVELOPMENT OF EQUATIONS

X1.1 The derivation of the equation is apparent from the following simplified cases using two solids. Solid 1 has a weight W_1 in grams and a volume V_1 in millilitres; its specific gravity (G_1) is therefore W_1/V_1. Solid 2 has a weight W_2 and volume V_2, and $G_2 = W_2/V_2$. If the two solids are considered together, the specific gravity of the combination is the total weight in grams divided by the total volume in millilitres:

$$G = (W_1 + W_2) / (V_1 + V_2)$$

Manipulation of this equation yields the following:

$$G = \frac{1}{\dfrac{V_1 + V_2}{W_1 + W_2}} = \frac{1}{\dfrac{V_1}{V_1 + W_2} + \dfrac{V_2}{W_1}}$$

$$G = \frac{1}{\dfrac{W_1}{W_1 + W_2}\left(\dfrac{V_1}{W_1}\right) + \dfrac{W_2}{W_1 + W_2}\left(\dfrac{V_2}{W_2}\right)}$$

However, the weight fractions of the two solids are:

$$W_1/(W_1 + W_2) = P_1/100 \text{ and } W_2/(W_1 + W_2) = P_2/100$$

and,

$$1/G_1 = V_1/W_1 \text{ and } 1/G_2 = V_2/W_2$$

Therefore,

$$G = 1/[(P_1/100)(1/G_1) + (P_2/100)(1/G_2)]$$

An example of the computation is given in Table X1.1.

TABLE X1.1 Example of Calculation of Average Values of Specific Gravity and Absorption for a Coarse Aggregate Tested in Separate Sizes

Size Fraction, mm (in.)	% in Original Sample	Sample Weight Used in Test, g	Bulk Specific Gravity (SSD)	Absorption, %
4.75 to 12.5 (No. 4 to ½)	44	2213.0	2.72	0.4
12.5 to 37.5 (½ to 1½)	35	5462.5	2.56	2.5
37.5 to 63 (1½ to 2½)	21	12593.0	2.54	3.0

Average Specific Gravity (SSD)

$$G_{SSD} = \frac{1}{\dfrac{0.44}{2.72} + \dfrac{0.35}{2.56} + \dfrac{0.21}{2.54}} = 2.62$$

Average Absorption

$$A = (0.44)(0.4) + (0.35)(2.5) + (0.21)(3.0) = 1.7\ \%$$

X2. INTERRELATIONSHIPS BETWEEN SPECIFIC GRAVITIES AND ABSORPTION AS DEFINED IN TEST METHODS C 127 AND C 128

X2.1 Let:

S_d = bulk specific gravity (dry basis),
S_s = bulk specific gravity (SSD basis),
S_a = apparent specific gravity, and
A = absorption in %.

X2.2 Then,

$$S_s = (1 + A/100)S_d \tag{1}$$

$$S_a = \frac{1}{\frac{1}{S_d} - \frac{A}{100}} = \frac{S_d}{1 - \frac{AS_d}{100}} \tag{2}$$

$$S_a = \frac{1}{\frac{1 + A/100}{S_s} - \frac{A}{100}} = \frac{S_s}{1 - \left[\frac{A}{100}(S_s - 1)\right]} \tag{2a}$$

$$A = \left(\frac{S_s}{S_d} - 1\right) 100 \tag{3}$$

$$A = \left(\frac{S_a - S_s}{S_a (S_s - 1)}\right) 100 \tag{4}$$

The American Society for Testing and Materials takes no position respecting the validity of any patent rights asserted in connection with any item mentioned in this standard. Users of this standard are expressly advised that determination of the validity of any such patent rights, and the risk of infringement of such rights, are entirely their own responsibility.

This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 1916 Race St., Philadelphia, PA 19103.

Designation: C 128 – 88

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa. 19103
Reprinted from the Annual Book of ASTM Standards. Copyright ASTM
If not listed in the current combined index, will appear in the next edition.

Standard Test Method for Specific Gravity and Absorption of Fine Aggregate[1]

This standard is issued under the fixed designation C 128; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This standard has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

1. Scope

1.1 This test method covers the determination of bulk and apparent specific gravity, 23/23°C (73.4/73.4°F), and absorption of fine aggregate.

1.2 This test method determines (after 24 h in water) the bulk specific gravity and the apparent specific gravity as defined in Definitions E 12, the bulk specific gravity on the basis of weight of saturated surface-dry aggregate, and the absorption as defined in Definitions C 125.

NOTE 1—The subcommittee is considering revising Test Methods C 127 and C 128 to use the term "density" instead of "specific gravity" for coarse and fine aggregate, respectively.

1.3 The values stated in SI units are to be regarded as the standard.

1.4 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 29/C 29M Test Method for Unit Weight and Voids in Aggregate[2]

C 70 Test Method for Surface Moisture in Fine Aggregate[2]

C 125 Terminology Relating to Concrete and Concrete Aggregates[2]

C 127 Test Method for Specific Gravity and Absorption of Coarse Aggregate[2]

C 188 Test Method for Density of Hydraulic Cement[3]

C 566 Test Method for Total Moisture Content of Aggregate by Drying[2]

C 670 Practice for Preparing Precision and Bias Statements for Test Methods for Construction Materials[2,3,4]

C 702 Practice for Reducing Field Samples of Aggregate to Testing Size[2]

D 75 Practice for Sampling Aggregates[2,4]

E 12 Definitions of Terms Relating to Density and Specific Gravity of Solids, Liquids, and Gases[2,5]

E 380 Metric Practice[6]

2.2 *AASHTO Standard:*

AASHTO No. T 84 Specific Gravity and Absorption of Fine Aggregates[7]

3. Significance and Use

3.1 Bulk specific gravity is the characteristic generally used for calculation of the volume occupied by the aggregate in various mixtures containing aggregate including portland cement concrete, bituminous concrete, and other mixtures that are proportioned or analyzed on an absolute volume basis. Bulk specific gravity is also used in the computation of voids in aggregate in Test Method C 29 and the determination of moisture in aggregate by displacement in water in Test Method C 70. Bulk specific gravity determined on the saturated surface-dry basis is used if the aggregate is wet, that is, if its absorption has been satisfied. Conversely, the bulk specific gravity determined on the oven-dry basis is used for computations when the aggregate is dry or assumed to be dry.

3.2 Apparent specific gravity pertains to the relative density of the solid material making up the constituent particles not including the pore space within the particles that is accessible to water. This value is not widely used in construction aggregate technology.

3.3 Absorption values are used to calculate the change in the weight of an aggregate due to water absorbed in the pore spaces within the constituent particles, compared to the dry condition, when it is deemed that the aggregate has been in contact with water long enough to satisfy most of the absorption potential. The laboratory standard for absorption is that obtained after submerging dry aggregate for approximately 24 h in water. Aggregates mined from below the water table may have a higher absorption when used, if not allowed to dry. Conversely, some aggregates when used may contain an amount of absorbed moisture less than the 24 h-soaked condition. For an aggregate that has been in contact with water and that has free moisture on the particle surfaces, the percentage of free moisture can be determined by deducting the absorption from the total moisture content determined by Test Method C 566 by drying.

---

[1] This test method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.05 on Methods of Testing and Specifications for Physical Characteristics of Concrete Aggregates.

Current edition approved Nov. 25, 1988. Published January 1989. Originally published as C 128 – 36. Last previous edition C 128 – 84.

[2] *Annual Book of ASTM Standards*, Vol 04.02.

[3] *Annual Book of ASTM Standards*, Vol 04.01.

[4] *Annual Book of ASTM Standards*, Vol 04.03.

[5] *Annual Book of ASTM Standards*, Vol 15.05.

[6] *Annual Book of ASTM Standards*, Vol 14.02. Excerpts in all volumes.

[7] Available from American Association of State Highway and Transportation Officials, 444 North Capitol St. N.W., Suite 225, Washington, DC 20001.

4. Apparatus

4.1 *Balance*—A balance or scale having a capacity of 1 kg or more, sensitive to 0.1 g or less, and accurate within 0.1 % of the test load at any point within the range of use for this test. Within any 100-g range of test load, a difference between readings shall be accurate within 0.1 g.

4.2 *Pycnometer*—A flask or other suitable container into which the fine aggregate test sample can be readily introduced and in which the volume content can be reproduced within ±0.1 cm^3. The volume of the container filled to mark shall be at least 50 % greater than the space required to accommodate the test sample. A volumetric flask of 500 cm^3 capacity or a fruit jar fitted with a pycnometer top is satisfactory for a 500-g test sample of most fine aggregates. A Le Chatelier flask as described in Test Method C 188 is satisfactory for an approximately 55-g test sample.

4.3 *Mold*—A metal mold in the form of a frustum of a cone with dimensions as follows: 40 ± 3 mm inside diameter at the top, 90 ± 3 mm inside diameter at the bottom, and 75 ± 3 mm in height, with the metal having a minimum thickness of 0.8 mm.

4.4 *Tamper*—A metal tamper weighing 340 ± 15 g and having a flat circular tamping face 25 ± 3 mm in diameter.

5. Sampling

5.1 Sampling shall be accomplished in general accordance with Practice D 75.

6. Preparation of Test Specimen

6.1 Obtain approximately 1 kg of the fine aggregate from the sample using the applicable procedures described in Methods C 702.

6.1.1 Dry it in a suitable pan or vessel to constant weight at a temperature of 110 ± 5°C (230 ± 9°F). Allow it to cool to comfortable handling temperature, cover with water, either by immersion or by the addition of at least 6 % moisture to the fine aggregate, and permit to stand for 24 ± 4 h.

6.1.2 As an alternative to 6.1.1, where the absorption and specific gravity values are to be used in proportioning concrete mixtures with aggregates used in their naturally moist condition, the requirement for initial drying to constant weight may be eliminated and, if the surfaces of the particles have been kept wet, the 24-h soaking may also be eliminated.

NOTE 2—Values for absorption and for specific gravity in the saturated surface-dry condition may be significantly higher for aggregate not oven dried before soaking than for the same aggregate treated in accordance with 6.1.1.

6.2 Decant excess water with care to avoid loss of fines, spread the sample on a flat nonabsorbent surface exposed to a gently moving current of warm air, and stir frequently to secure homogeneous drying. If desired, mechanical aids such as tumbling or stirring may be employed to assist in achieving the saturated surface-dry condition. Continue this operation until the test specimen approaches a free-flowing condition. Follow the procedure in 6.2.1 to determine whether or not surface moisture is present on the constituent fine aggregate particles. It is intended that the first trial of the cone test will be made with some surface water in the specimen. Continue drying with constant stirring and test at frequent intervals until the test indicates that the specimen has reached a surface-dry condition. If the first trial of the surface moisture test indicates that moisture is not present on the surface, it has been dried past the saturated surface-dry condition. In this case thoroughly mix a few millilitres of water with the fine aggregate and permit the specimen to stand in a covered container for 30 min. Then resume the process of drying and testing at frequent intervals for the onset of the surface-dry condition.

6.2.1 *Cone Test for Surface Moisture*—Hold the mold firmly on a smooth nonabsorbent surface with the large diameter down. Place a portion of the partially dried fine aggregate loosely in the mold by filling it to overflowing and heaping additional material above the top of the mold by holding it with the cupped fingers of the hand holding the mold. Lightly tamp the fine aggregate into the mold with 25 light drops of the tamper. Each drop should start about 5 mm (0.2 in.) above the top surface of the fine aggregate. Permit the tamper to fall freely under gravitational attraction on each drop. Adjust the starting height to the new surface elevation after each drop and distribute the drops over the surface. Remove loose sand from the base and lift the mold vertically. If surface moisture is still present, the fine aggregate will retain the molded shape. When the fine aggregate slumps slightly it indicates that it has reached a surface-dry condition. Some angular fine aggregate or material with a high proportion of fines may not slump in the cone test upon reaching a surface-dry condition. This may be the case if fines become airborne upon dropping a handful of the sand from the cone test 100 to 150 mm onto a surface. For these materials the saturated surface-dry condition should be considered as the point that one side of the fine aggregate slumps slightly upon removing the mold.

NOTE 3—The following criteria have also been used on materials that do not readily slump:

(1) *Provisional Cone Test*—Fill the cone mold as described in 6.2.1 except only use 10 drops of the tamper. Add more fine aggregate and use 10 drops of the tamper again. Then add material two more times using 3 and 2 drops of the tamper, respectively. Level off the material even with the top of the mold, remove loose material from the base; and lift the mold vertically.

(2) *Provisional Surface Test*—If airborne fines are noted when the fine aggregate is such that it will not slump when it is at a moisture condition, add more moisture to the sand, and at the onset of the surface-dry condition, with the hand lightly pat approximately 100 g of the material on a flat, dry, clean, dark or dull nonabsorbent surface such as a sheet of rubber, a worn oxidized, galvanized, or steel surface, or a black-painted metal surface. After 1 to 3 s remove the fine aggregate. If noticeable moisture shows on the test surface for more than 1 to 2 s then surface moisture is considered to be present on the fine aggregate.

(3) Colorimetric procedures described by Kandhal and Lee, Highway Research Record No. 307, p. 44.

(4) For reaching the saturated surface-dry condition on a single size material that slumps when wet, hard-finish paper towels can be used to surface dry the material until the point is just reached where the paper towel does not appear to be picking up moisture from the surfaces of the fine aggregate particles.

7. Procedure

7.1 Make and record all weight determinations to 0.1 g.

7.2 Partially fill the pycnometer with water. Immediately introduce into the pycnometer 500 ± 10 g of saturated surface-dry fine aggregate prepared as described in Section 6, and fill with additional water to approximately 90 % of capacity. Roll, invert, and agitate the pycnometer to eliminate all air bubbles. Adjust its temperature to 23 ± 1.7°C (73.4 ± 3°F), if necessary by immersion in circulating water, and bring the water level in the pycnometer to its calibrated capacity. Determine the total weight of the pycnometer, specimen, and water.

NOTE 4—It normally takes about 15 to 20 min to eliminate air bubbles.

7.2.1 *Alternative to Weighing in 7.2*—The quantity of added water necessary to fill the pycnometer at the required temperature may be determined volumetrically using a buret accurate to 0.15 mL. Compute the total weight of the pycnometer, specimen, and water as follows:

$$C = 0.9975\ V_a + S + W$$

where:
C = weight of pycnometer with specimen and water to calibration mark, g,
V_a = volume of water added to pycnometer, mL,
S = weight of the saturated surface-dry specimen, and
W = weight of the empty pycnometer, g.

7.2.2 *Alternative to the Procedure in 7.2*—Use a Le Chatelier flask initially filled with water to a point on the stem between the 0 and the 1-mL mark. Record this initial reading with the flask and contents within the temperature range of 23 ± 1.7°C (73.4 ± 3°F). Add 55 ± 5 g of fine aggregate in the saturated surface-dry condition (or other weight as necessary to result in raising the water level to some point on the upper series of gradation). After all fine aggregate has been introduced, place the stopper in the flask and roll the flask in an inclined position, or gently whirl it in a horizontal circle so as to dislodge all entrapped air, continuing until no further bubbles rise to the surface. Take a final reading with the flask and contents within 1°C (1.8°F) of the original temperature.

7.3 Remove the fine aggregate from the pycnometer, dry to constant weight at a temperature of 110 ± 5°C (230 ± 9°F), cool in air at room temperature for 1 ± ½ h, and weigh.

7.3.1 If the Le Chatelier flask method is used, a separate sample portion is needed for the determination of absorption. Weigh a separate 500 ± 10-g portion of the saturated surface-dry fine aggregate, dry to constant weight, and reweigh.

7.4 Determine the weight of the pycnometer filled to its calibration capacity with water at 23 ± 1.7°C (73.4 ± 3°F).

7.4.1 *Alternative to Weighing in 7.4*—The quantity of water necessary to fill the empty pycnometer at the required temperature may be determined volumetrically using a buret accurate to 0.15 mL. Calculate the weight of the pycnometer filled with water as follows:

$$B = 0.9975\ V + W$$

where:
B = weight of flask filled with water, g,
V = volume of flask, mL, and
W = weight of the flask empty, g.

8. Bulk Specific Gravity

8.1 Calculate the bulk specific gravity, 23/23°C (73.4/73.4°F), as defined in Definitions E 12, as follows:

$$\text{Bulk sp gr} = A/(B + S - C)$$

where:
A = weight of oven-dry specimen in air, g,
B = weight of pycnometer filled with water, g,
S = weight of the saturated surface-dry specimen, and
C = weight of pycnometer with specimen and water to calibration mark, g.

8.1.1 If the Le Chatelier flask method was used, calculate the bulk specific gravity, 23/23°C, as follows:

$$\text{Bulk sp gr} = \frac{S_1\,[1 - ((S - A)/A)]}{0.9975\,(R_2 - R_1)}$$

where:
S_1 = weight of saturated surface-dry specimen used in Le Chatelier flask, g,
R_1 = initial reading of water level in Le Chatelier flask, and
R_2 = final reading of water level in Le Chatelier flask.

9. Bulk Specific Gravity (Saturated Surface-Dry Basis)

9.1 Calculate the bulk specific gravity, 23/23°C (73.4/73.4°F), on the basis of weight of saturated surface-dry aggregate as follows:

$$\text{Bulk sp gr (saturated surface-dry basis)} = S/(B + S - C)$$

9.1.1 If the Le Chatelier flask method was used, calculate the bulk specific gravity, 23/23°C, on the basis of saturated surface-dry aggregate as follows:

$$\text{Bulk sp gr (saturated surface-dry basis)} = \frac{S_1}{0.9975\,(R_2 - R_1)}$$

10. Apparent Specific Gravity

10.1 Calculate the apparent specific gravity, 23/23°C (73.4/73.4°F), as defined in Definitions E 12, as follows:

$$\text{Apparent sp gr} = A/(B + A - C)$$

11. Absorption

11.1 Calculate the percentage of absorption, as defined in Definitions C 125, as follows:

$$\text{Absorption, \%} = [(S - A)/A] \times 100$$

12. Report

12.1 Report specific gravity results to the nearest 0.01 and absorption to the nearest 0.1 %. The Appendix gives mathematical interrelationships among the three types of specific gravities and absorption. These may be useful in checking the consistency of reported data or calculating a value that was not reported by using other reported data.

12.2 If the fine aggregate was tested in a naturally moist condition other than the oven dried and 24 h-soaked condition, report the source of the sample and the procedures used to prevent drying prior to testing.

13. Precision and Bias

13.1 *Precision*—The estimates of precision of this test method (listed in Table 1) are based on results from the AASHTO Materials Reference Laboratory Reference Sample Program, with testing conducted by this test method and AASHTO Method T 84. The significant difference between the methods is that Test Method C 128 requires a saturation period of 24 ± 4 h, and Method T 84 requires a saturation period of 15 to 19 h. This difference has been

TABLE 1 Precision

	Standard Deviation (1S)[A]	Acceptable Range of Two Results (D2S)[A]
Single-Operator Precision:		
Bulk specific gravity (dry)	0.011	0.032
Bulk specific gravity (SSD)	0.0095	0.027
Apparent specific gravity	0.0095	0.027
Absorption[B], %	0.11	0.31
Multilaboratory Precision:		
Bulk specific gravity (dry)	0.023	0.066
Bulk specific gravity (SSD)	0.020	0.056
Apparent specific gravity	0.020	0.056
Absorption[B], %	0.23	0.66

[A] These numbers represent, respectively, the (1S) and (D2S) limits as described in Practice C 670. The precision estimates were obtained from the analysis of combined AASHTO Materials Research Laboratory reference sample data from laboratories using 15 to 19 h saturation times and other laboratories using 24 ± 4 h saturation time. Testing was performed on normal weight aggregates, and started with aggregates in the oven-dry condition.

[B] Precision estimates are based on aggregates with absorptions of less than 1 % and may differ for manufactured fine aggregates and fine aggregates having absorption values greater than 1 %.

found to have an insignificant effect on the precision indices. The data are based on the analyses of more than 100 paired test results from 40 to 100 laboratories.

13.2 *Bias*—Since there is no accepted reference material suitable for determining the bias for this test method, no statement on bias is being made.

APPENDIX

(Nonmandatory Information)

X1. INTERRELATIONSHIPS BETWEEN SPECIFIC GRAVITIES AND ABSORPTION AS DEFINED IN TEST METHODS C 127 AND C 128

X1.1 Let:

S_d = bulk specific gravity (dry-basis),
S_s = bulk specific gravity (SSD-basis),
S_a = apparent specific gravity, and
A = absorption in %

Then:

(1) $$S_s = (1 + A/100)S_d$$

(2) $$S_a = \frac{1}{\frac{1}{S_d} - \frac{A}{100}} = \frac{S_d}{1 - \frac{AS_d}{100}}$$

(2*a*) $$\text{or } S_a = \frac{1}{\frac{1 + A/100}{S_s} - \frac{A}{100}} = \frac{S_s}{1 - \frac{A}{100}(S_s - 1)}$$

(3) $$A = \left(\frac{S_s}{S_d} - 1\right)100$$

(4) $$A = \left(\frac{S_a - S_s}{S_a(S_s - 1)}\right)100$$

RTH 108-93

METHOD OF DETERMINING DENSITY OF SOLIDS

1. Scope and Definition

1 This method covers procedures for determining the density of solids. The density of solids is the ratio of the mass in air of a given volume of crushed solids to the total volume of solids.

2. Apparatus

1 The apparatus shall consist of the following:

(a) Volumetric flask, 500-mL capacity.

(b) Vacuum pump or aspirator connected to vacuum line.

(c) Oven of the forced draft type, automatically controlled to maintain a uniform temperature of 110 ± 5 °C throughout the oven.

(d) Balance, sensitive and accurate to 0.01 g, capacity 500 g or more.

(e) Thermometer, range 0 to 50 °C, graduated in 0.1 °C.

(f) Evaporating dish.

(g) Water dish.

(h) Sieves, U.S. Standard 4.75-mm (No. 4) and 600-μm (No. 30) conforming to ASTM Designation E 11, "Specifications for Wire-Cloth Sieves for Testing Purposes."

(i) Sample splitter suitable for splitting material passing 4.75-mm (No. 4) and 600-μm (No. 30) sieves.

3. Calibration of Volumetric Flask

1 The volumetric flask shall be calibrated for the mass of the flask and water at various temperatures. The flask and water are calibrated by direct determination of mass at the range of temperatures likely to be encountered in the laboratory. The calibration procedure is as follows.

2 Fill the flask with de-aired, distilled, and demineralized water to slightly below the calibration mark and place in a water bath which is at a temperature between 30 and 35 °C. Allow the flask to remain in the bath until the water in the flask reaches the temperature of the water bath. This may take several hours. Remove the flask from the water bath and adjust the water

level in the flask so that the bottom of meniscus is even with the calibration mark on the neck of the flask. Thoroughly dry the outside of the flask and remove any water adhering to the inside of the neck above the graduation, then determine the mass of the flask and water to the nearest 0.01 g. Immediately after determination of mass, shake the flask gently and determine the temperature of the water to the nearest 0.1 °C by immersing a thermometer to the middepth of the flask. Repeat the procedure outlined above at approximately the same temperature, than make two more determinations, one at room temperature and the other at approximately 5 °C less than room temperature. Draw a calibration curve showing the relation between temperature and corresponding values of mass of the flask plus water. Prepare a calibration curve for each flask used for density determination and maintain the curves as a permanent record. A typical calibration curve is shown in Fig. 1.

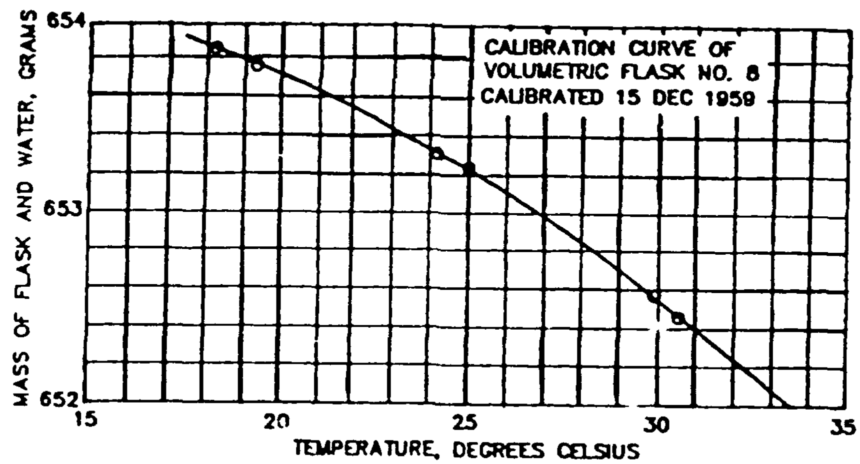


Fig. 1. Typical calibration curve of volumetric flask.

4. Sample

1 Crush the sample until it all passes a 4.75-mm (No. 4) sieve. With a sample splitter, separate out 120 to 150 g of representative crushed material. Pulverize this material to pass a 600-μm (No. 30) sieve. Oven-dry the crushed material to constant mass, determine the mass of the material to the nearest 0.01 g, and record the mass.

5. Procedure

1 After determination of mass, transfer the crushed material to a volumetric flask, taking care not to lose any material during this operation. To reduce possible error due to loss of material of known mass, the sample may have its mass determined after transfer to the flask. Fill the flask approximately half full with de-aired, distilled water. Shake the mixture well and allow it to stand overnight.

2 Then connect the flask to the vacuum line and apply a vacuum of approximately 99.99 Pa (750 mm of mercury) for approximately 4 to 6 h, agitating the flask at intervals during the evacuation process. Again, allow the flask to stand overnight. Finally, fill the flask with de-aired, distilled water to about 3/4 in. (19 mm) below the 500-mL graduation and again apply a vacuum to the flask until the suspension is de-aired, slowly and carefully remove the stopper from the flask, and observe the lowering of the water surface in the neck. If the water surface is lowered less than 1/8 in. (3.2 mm), the suspension can be considered sufficiently de-aired. Fill the flask until the bottom of the meniscus is coincident with the calibration line of the neck of the flask. Thoroughly dry the outside of the flask and remove the moisture on the inside of the neck by wiping with a paper towel. Determine the mass of the flask and contents to the nearest 0.01 g. Immediately after determination of mass, stir the suspension to assure uniform temperature, and determine the temperature of the suspension to the nearest 0.1 °C by immersing a thermometer to the middepth of the flask. Record the mass and temperature.

3 Compute the density of the solid, P_s, from the following formula:

$$P_s = \frac{W_s \quad Y_w}{W_s + W_{fW} - W_{fWs}}$$

where

W_s = the ovendry mass of the crushed rock sample, g

γ_w = density of water at test temperature, g/cc

W_{fw} = mass of flask plus water at test temperature, g (from calibration curve, Fig. 1)

W_{fws} = mass of flask plus water plus solids at test temperature, g

6. Report

1 The report shall include the following:

(a) The density of the solid.

(b) Ovendry mass of test sample.

(c) Water temperature during test.

RTH 109-93

METHOD OF DETERMINING EFFECTIVE (AS RECEIVED) AND DRY UNIT WEIGHTS AND TOTAL POROSITY OF ROCK CORES

1. Scope and Definition

1.1 This method covers the procedure for determining the effective unit weight, dry unit weight, and porosity of rock cores as defined in RTH 101. (This method covers determination of "total" porosity of a rock sample. Porosity calculated from the bulk volume and grain volume using the pulverization method is termed "total" since the pore volume obtained includes that of all closed pores. Other techniques give "effective" porosity since they measure the volume of interconnected pores only.)

2. Apparatus

2.1 The apparatus shall consist of the following:

(a) Balance having a capacity of 5 kg or more and sensitive and accurate to 0.5 g or 0.05 percent of the sample mass. Balances with capacities less than 5-kg shall be sensitive and accurate to 0.1 g.

(b) Wire basket of 4.75-mm (No. 4) mesh, diameter at least 50.8 mm (2 in.) greater than that of the core to be tested, walls at least one-half the height of the cylinder, and bail clearing the top the core by at least 25.4 mm (1 in.) at all points.

(c) Watertight container in which the wire basket may be suspended with a constant-level overflow spout at such a height that the wire basket, when suspended below the spout, will be at least 25.4 mm (1 in.) from the bottom of the container.

(d) Suspending apparatus suitable for suspending the wire basket in the container from the center of the balance platform or pan so that the basket will hang completely below the overflow spout and not be less than 25.4 mm (1 in.) from the bottom of the container.

(e) Thermometer, range 0 to 50 °C, graduated to 0.1 °C.

(f) Caliper or suitable measuring device capable of measuring lengths and diameters of test cores to the nearest 0.1 mm.

(g) Oven of the forced draft type, automatically controlled to maintain a uniform temperature 110 ± 5 °C throughout.

3. Sample

3.1 Select representative samples from the population and identify each sample. Individual sample mass should not exceed 5 kg.

4. Effective Unit Weight (As Received)

4.1 The test procedure for determining the effective unit weight of rock cores shall consist of the following steps:

(a) Determine the mass of the core (as received) to the nearest gram (0.1 g for 76.2-mm (3-in.) and smaller cores) (W_a) and the temperature in the working area near the core surface.

(b) Determine the bulk volume of the core in cubic centimetres by one of the following two methods:

(1) Determine the average length and diameter of the core from measurements of each of these dimensions at evenly spaced intervals covering the surface of the specimen. These measurements should be made to the nearest 0.1 mm. Calculate the volume using the formula $V = \frac{\pi}{4} d^2 L$, where V = volume, d = diameter of the core, and L = length of the core. (Note 1)

NOTE 1--If this method is used, the specimen should be sawed and machined to conform closely to the shape of a right cylinder or prism prior to determining its mass as in 4.1(a) above.

(2) Coat the surface of the core with wax or other suitable coating until it is watertight, making sure that the coating material does not measurably penetrate the pores of the core. Determine the mass of the specimen, after coating, to the nearest gram (or 0.1 g). The density of the coating material shall be determined. The volume of the coating on the core shall be determined by dividing the mass of coating by the density of the coating. Determine the volume of the coated core in cubic centimetres by liquid displacement. Subtract the volume of the coating material from the volume of the coated core to obtain the volume of the core (V) in cubic centimetres.

(c) Calculate the effective unit weight of the core from the following formula:

$$\gamma_e = \frac{W_a}{V}$$

where

γ_e = effective unit weight of the core, as received

W_a = mass of the core, in grams, as received

V = volume of the core, in cubic centrimetres

5. Dry Unit Weight

5.1 The test procedure for determining the dry unit weight of rock cores shall consist of the following steps:

(a) If a coating was utilized to waterproof the specimen as in 4.1(b)(2), remove it and, if applicable, brush to remove dust or elements of the coating. Then determine the mass of the core. (Note 2)

NOTE 2--If there is no mass loss in stripping or loss or gain in moisture, this mass should equal W_a.

(b) Crush the sample until it all passes a No. 4 sieve, taking care not to lose any material.

(c) Oven-dry the crushed material to constant mass W_b (constant mass is achieved when the mass loss is less than 0.1 percent of the sample mass during any 4-hour drying period), cool to room temperature, then record the mass and room temperature in the area of the test on the data sheet.

(d) Calculate the dry unit weight of the core from the following formula:

$$\gamma_d = \frac{W_b}{V}$$

where

γ_d = dry unit weight of the core

W_b = mass of the crushed, dried core, in grams

V = volume of the core, in cubic centrimetres

6. Porosity

6.1 The total porosity, n , may be determined from the dry unit weight and the gram unit weight of a sample. Determine the density of the solids, P_s , according to RTH 108.

Determine the total porosity by the expression:

$$n = \frac{(1 - W_b)}{P_s V} 100$$

where

n = total porosity, %

W_b = mass of crushed, dried core, g

V = volume of the core, cc

P_s = density of the solids, g/cc

(as determined by RTH - 108)

Designation: D 2845 – 90$^{\epsilon 1}$

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St. Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards. Copyright ASTM
If not listed in the current combined index, will appear in the next edition.

Standard Test Method for Laboratory Determination of Pulse Velocities and Ultrasonic Elastic Constants of Rock[1]

This standard is issued under the fixed designation D 2845; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

$^{\epsilon 1}$ NOTE—Section 10 was changed editorially in December 1.

1. Scope

1.1 This test method describes equipment and procedures for laboratory measurements of the pulse velocities of compression waves and shear waves in rock (1)[2] (Note 1) and the determination of ultrasonic elastic constants (Note 2) of an isotropic rock or one exhibiting slight anisotropy.

NOTE 1—The compression wave velocity as defined here is the dilatational wave velocity. It is the propagation velocity of a longitudinal wave in a medium which is effectively infinite in lateral extent. It should not be confused with the bar or rod velocity.

NOTE 2—The elastic constants determined by this test method are termed ultrasonic since the pulse frequencies used are above the audible range. The terms sonic and dynamic are sometimes applied to these constants but do not describe them precisely (2). It is possible that the ultrasonic elastic constants may differ from those determined by other dynamic methods.

1.2 This test method is valid for wave velocity measurements in both anisotropic and isotropic rocks although the velocities obtained in grossly anisotropic rocks may be influenced by such factors as direction, travel distance, and diameter of transducers.

1.3 The ultrasonic elastic constants are calculated from the measured wave velocities and the bulk density. The limiting degree of anisotropy for which calculations of elastic constants are allowed and procedures for determining the degree of anisotrophy are specified.

1.4 The values stated in U.S. customary units are to be regarded as the standard. The metric equivalents of U.S. customary units may be approximate.

2. Summary of Test Method

2.1 Details of essential procedures for the determination of the ultrasonic velocity, measured in terms of travel time and distance, of compression and shear waves in rock specimens include requirements of instrumentation, suggested types of transducers, methods of preparation, and effects of specimen geometry and grain size. Elastic constants may be calculated for isotropic or slightly anisotropic rocks, while anisotropy is reported in terms of the variation of wave velocity with direction in the rock.

3. Significance and Use

3.1 The primary advantages of ultrasonic testing are that it yields compression and shear wave velocities, and ultrasonic values for the elastic constants of intact homogeneous isotropic rock specimens (3). Elastic constants are not to be calculated for rocks having pronounced anisotropy by procedures described in this test method. The values of elastic constants often do not agree with those determined by static laboratory methods or the *in situ* methods. Measured wave velocities likewise may not agree with seismic velocities, but offer good approximations. The ultrasonic evaluation of rock properties is useful for preliminary prediction of static properties. The test method is useful for evaluating the effects of uniaxial stress and water saturation on pulse velocity. These properties are in turn useful in engineering design.

3.2 The test method as described herein is not adequate for measurement of stress-wave attenuation. Also, while pulse velocities can be employed to determine the elastic constants of materials having a high degree of anisotropy, these procedures are not treated herein.

4. Apparatus

4.1 *General*—The testing apparatus (Fig. 1) should have impedance matched electronic components and shielded leads to ensure efficient energy transfer. To prevent damage to the apparatus allowable voltage inputs should not be exceeded.

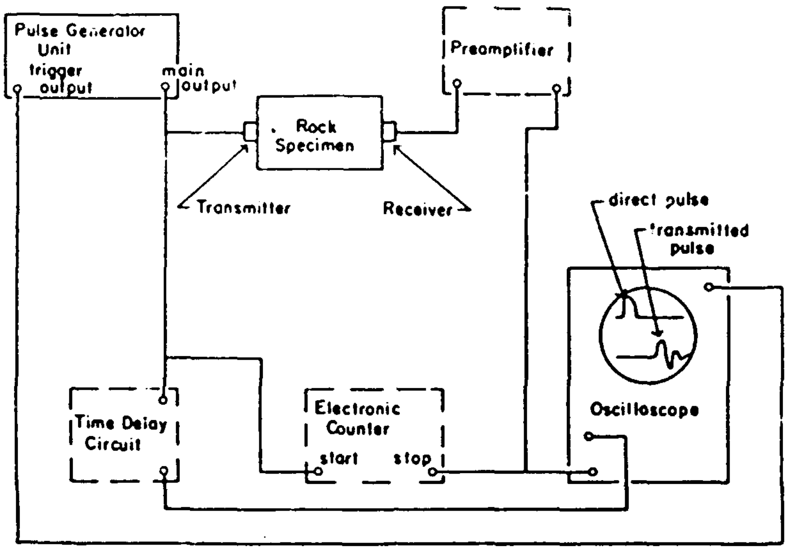
4.2 *Pulse Generator Unit*—This unit shall consist of an electronic pulse generator and external voltage or power amplifiers if needed. A voltage output in the form of either rectangular pulse or a gated sine wave is satisfactory. The generator shall have a voltage output with a maximum value after amplification of at least 50 V into a 50-Ω impedance load. A variable pulse width, with a range of 1 to 10 μs is desirable. The pulse repetition rate may be fixed at 60 repetitions per second or less although a range of 20 to 100 repetitions per second is recommended. The pulse generator shall also have a trigger-pulse output to trigger the oscilloscope. There shall be a variable delay of the main-pulse output with respect to the trigger-pulse output, with a minimum range of 0 to 20 μs.

4.3 *Transducers*—The transducers shall consist of a transmitter which converts electrical pulses into mechanical pulses and a receiver which converts mechanical pulses into electrical pulses. Environmental conditions such as ambient temperature, moisture, humidity, and impact should be

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved June 29, 1990. Published October 1990. Originally published as D 2845 – 69. Last previous edition D 2845 – 83.

[2] The boldface numbers in parentheses refer to the list of references at the end of this test method.



NOTE—Components shown by dashed lines are optional, depending on method of travel-time measurement and voltage sensitivity of oscilloscope.

FIG. 1 Schematic Diagram of Typical Apparatus

considered in selecting the transducer element. Piezoelectric elements are usually recommended, but magnetostrictive elements may be suitable. Thickness-expander piezoelectric elements generate and sense predominately compression-wave energy; thickness-shear piezoelectric elements are preferred for shear-wave measurements. Commonly-used piezoelectric materials include ceramics such as lead-zirconate-titanate for either compression or shear, and crystals such as a-c cut quartz for shear. To reduce scattering and poorly defined first arrivals at the receiver, the transmitter shall be designed to generate wavelengths at least three times the average grain size of the rock.

NOTE 3—Wavelength is the wave velocity in the rock specimen divided by the resonance frequency of the transducer. Commonly used frequencies range from 75 kHz to 3 MHz.

4.3.1 In laboratory testing, it may be convenient to use unhoused transducer elements. But if the output voltage of the receiver is low, the element should be housed in metal (grounded) to reduce stray electromagnetic pickup. If protection from mechanical damage is necessary, the transmitter as well as the receiver may be housed in metal. This also allows special backings for the transducer element to alter its sensitivity or reduce ringing (*4*). The basic features of a housed element are illustrated in Fig. 2. Energy transmission between the transducer element and test specimen can be improved by (*1*) machining or lapping the surfaces of the face plates to make them smooth, flat, and parallel, (*2*) making the face plate from a metal such as magnesium whose characteristic impedance is close to that of common rock types, (*3*) making the face plate as thin as practicable, and (*4*) coupling the transducer element to the face plate by a thin layer of an electrically conductive adhesive, an epoxy type being suggested.

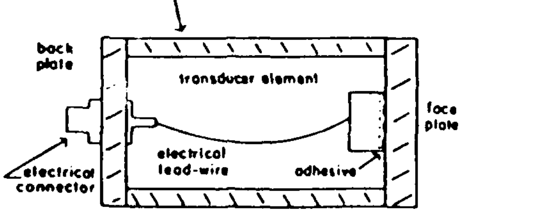


FIG. 2 Basic Features of a Housed Transmitter or Receiver

4.3.2 Pulse velocities may also be determined for specimens subjected to uniaxial states of stress. The transducer housings in this case will also serve as loading platens and should be designed with thick face plates to assure uniform loading over the ends of the specimen (*5*).

NOTE 4—The state of stress in many rock types has a marked effect on the wave velocities. Rocks *in situ* are usually in a stressed state and therefore tests under stress have practical significance.

4.4 *Preamplifier*—A voltage preamplifier is required if the voltage output of the receiving transducer is relatively low or if the display and timing units are relatively insensitive. To preserve fast rise times, the frequency response of the preamplifier shall drop no more that 2 dB over a frequency range from 5 kHz to 4 × the resonance frequency of the receiver. The internal noise and gain must also be considered in selecting a preamplifier. Oscilloscopes having a vertical-signal output can be used to amplify the signal for an electronic counter.

4.5 *Display and Timing Unit*—The voltage pulse applied to the transmitting transducer and the voltage output from the receiving transducer shall be displayed on a cathode-ray oscilloscope for visual observation of the waveforms. The oscilloscope shall have an essentially flat response between a frequency of 5 kHz and 4 × the resonance frequency of the transducers. It shall have dual beams or dual traces so that the two waveforms may be displayed simultaneously and their amplitudes separately controlled. The oscilloscope shall be triggered by a triggering pulse from the pulse generator. The timing unit shall be capable of measuring intervals between 2 μs and 5 ms to an accuracy of 1 part in 100. Two alternative classes of timing units are suggested, the respective positions of each being shown as dotted outlines in the block diagram in Fig. 1: (*1*) an electronic counter with provisions for time interval measurements, or (*2*) a time-delay circuit such as a continuously variable-delay generator, or a delayed-sweep feature on the oscilloscope. The travel-time measuring circuit shall be calibrated periodically with respect to its accuracy and linearity over the range of the instrument. The calibration shall be checked against signals transmitted by the National Institute of Standards and Technology radio station WWV, or against a crystal controlled time-mark or frequency generator which can be referenced back to the signals from WWV periodically. It is recommended that the calibration of the time measuring circuit be checked at least once a month and after any severe impact which the instrument may receive.

5. Test Specimens

5.1 *Preparation*—Exercise care in core drilling, handling, sawing, grinding, and lapping the test specimen to minimize the mechanical damage caused by stress and heat. It is recommended that liquids other than water be prevented from contacting the specimen, except when necessary as a coupling medium between specimen and transducer during the test. The surface area under each transducer shall be sufficiently plane that a feeler gage 0.001 in. (0.025 mm) thick will not pass under a straightedge placed on the surface. The two opposite surfaces on which the transducers will be placed shall be parallel to within 0.005 in./in. (0.1 mm/20 mm) of lateral dimension (Fig. 3). If the pulse velocity measurements are to be made along a diameter of a core, the above tolerance then refers to the parallelism of the lines of contact between the transducers and curved surface of the rock core. Moisture content of the test specimen can affect the measured pulse velocities (see 6.2). Pulse velocities may be determined on the velocity test specimen for rocks in the oven-dry state (0 % saturation), in a saturated condition (100 % saturation), or in any intermediate state. If the pulse velocities are to be determined with the rock in the same moisture condition as received or as exists underground, care must be exercised during the preparation procedure so that the moisture content does not change. In this case it is suggested that both the sample and test specimen be stored in moisture-proof bags or coated with wax and that dry surface-preparation procedures be employed. If results are desired for specimens in the oven-dried condition, the oven temperature shall not exceed 150°F (66°C). The specimen shall remain submerged in water up to the time of testing when results are desired for the saturated state.

5.2 *Limitation on Dimensions*—It is recommended that the ratio of the pulse-travel distance to the minimum lateral dimension not exceed 5. Reliable pulse velocities may not be measurable for high values of this ratio. The travel distance of the pulse through the rock shall be at least 10 × the average grain size so that an accurate average propagation velocity may be determined. The grain size of the rock sample, the natural resonance frequency of the transducers, and the minimum lateral dimension of the specimen are interrelated factors which affect test results. The wavelength corresponding to the dominant frequency of the pulse train in the rock is approximately related to the natural resonance frequency of the transducer and the pulse-propagation velocity, (compression or shear) as follows:

$$\Lambda \approx V/f, \tag{1}$$

where:

Λ = dominant wavelength of pulse train, in. (or m),
V = pulse propagation velocity (compression or shear), in./s (or m/s), and
f = natural resonance frequency of transducers, Hz.

The minimum lateral dimension of the test specimen shall be at least 5 × the wavelength of the compression wave so that the true dilational wave velocity is measured (Note 5), that is,

$$D \geq 5\Lambda, \tag{2}$$

where:

D = minimum lateral dimension of test specimen, in. (or m).

The wavelength shall be at least 3 × the average grain size (See 4.3) so that

$$\Lambda \geq 3d, \tag{3}$$

where:

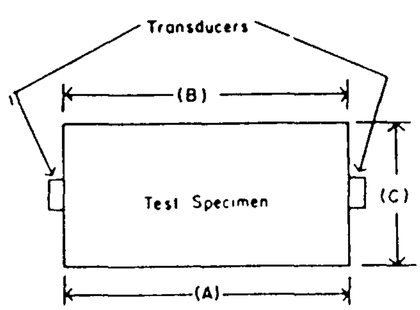
d = average grain size, in. (or m).

Equations 1, 2, and 3 can be combined to yield the relationship for compression waves as follows:

$$D \geq 5(V_p/f) \geq 15\,d, \tag{4}$$

where:

V_p = pulse propagation velocity (compression), in./s (or m/s).



NOTE—(A) must be within 0.1 mm of (B) for each 20 mm of width (C).

FIG. 3 Specification for Parallelism

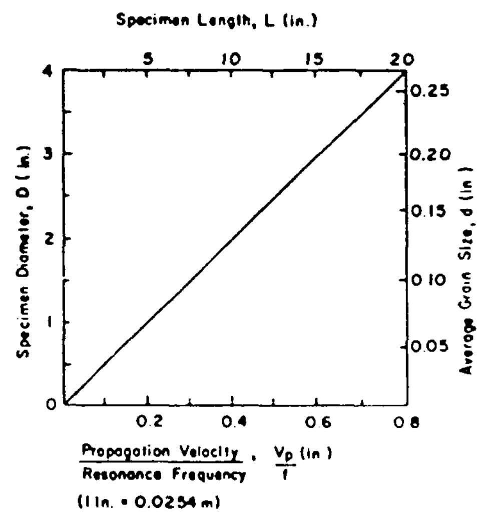


FIG. 4 Graph Showing Allowable Values of Specimen Diameter and Grain Size Versus the Ratio of Propagation Velocity to Resonance Frequency

Since V_p and d are inherent properties of the material, f and D shall be selected to satisfy Eq 4 (Fig. 4) for each test specimen. For any particular value of V_p/f the permissible values of specimen diameter D lie above the diagonal line in Fig. 4, while the permissible values of grain size d lie below the diagonal line. For a particular diameter, the permissible values for specimen length L lie to the left of the diagonal line.

NOTE 5—Silaeva and Shamina (6) found the limiting ratio of diameter to wavelength to be about 2 for metal rods. Data obtained by Cannady (3) on rock indicate the limiting ratio is at least 8 for a specimen length-to-diameter ratio of about 8.

6. Procedure

6.1 *Determination of Travel Distance and Density*—Mark off the positions of the transducers on the specimen so that the line connecting the centers of the transducer contact areas is not inclined more than 2° (approximately 0.1 in. in 3 in. (1 mm in 30 mm)) with a line perpendicular to either surface. Then measure the pulse-travel distance from center to center of the transducer contact area to within 0.1 %. The density of the test specimen is required in the calculation of the ultrasonic elastic constants (see 7.2). Determine the density of the test specimen from measurements of its mass and its volume calculated from the average external dimensions. Determine the mass and average dimensions within 0.1 %. Calculate the density as follows:

$$\rho = m/V$$

where:

ρ = density, lb sec^2/in.^{4} (or kg/m^3),
m = mass of test specimen, lb sec^2/386.4 in. (or kg), and
V = volume of test specimen, in^3 (or m^3).

6.2 *Moisture Condition*—The moisture condition of the sample shall be noted and reported as 8.1.3.

6.3 *Determination of Pulse-Travel Time:*

6.3.1 Increase the voltage output of the pulse generator, the gain of the amplifier, and the sensitivity of the oscilloscope and counter to an optimum level, giving a steeper pulse front to permit more accurate time measurements. The optimum level is just below that at which electromagnetic noise reaches an intolerable magnitude or triggers the counter at its lowest triggering sensitivity. The noise level shall not be greater than one tenth of the amplitude of the first peak of the signal from the receiver. Measure the travel time to a precision and accuracy of 1 part in 100 for compression waves and 1 part in 50 for shear waves by (*1*) using the delaying circuits in conjunction with the oscilloscope (see 6.1.1) or (*2*) setting the counter to its highest usable precision, (see 6.3.2).

6.3.1.1 The oscilloscope is used with the time-delay circuit to display both the direct pulse and the first arrival of the transmitted pulse, and to measure the travel time. Characteristically, the first arrival displayed on the oscilloscope consists of a curved transition from the horizontal zero-voltage trace followed by a steep, more or less linear, trace. Select the first break in a consistent manner for both the test measurement and the zero-time determination. Select it either at the beginning of the curved transition region or at the zero-voltage intercept of the straight line portion of the first arrival.

6.3.1.2 The counter is triggered to start by the direct pulse applied to the transmitter and is triggered to stop by the first arrival of the pulse reaching the receiver. Because a voltage change is needed to trigger the counter, it can not accurately detect the first break of a pulse. To make the most accurate time interval measurements possible, increase the counter's triggering sensitivity to an optimum without causing spurious triggering by extraneous electrical noise.

6.3.2 Determine the zero time of the circuit including both transducers and the travel-time measuring device and apply the correction to the measured travel times. This factor will remain constant for a given rock and stress level if the circuit characteristics do not change. Determine the zero time accordingly to detect any changes. Determine it by (*1*) placing the transducers in direct contact with each other and measuring the delay time directly, or (*2*) measuring the apparent travel time of some uniform material (such as steel) as a function of length, and then using the zero-length intercept of the line through the data points as the correction factor.

6.3.3 Since the first transmitted arrival is that of the compression wave, its detection is relatively easy. The shear-wave arrival, however, may be obscured by vibrations due to ringing of the transducers and reflections of the compression wave. The amplitude of the shear wave relative to the compression wave may be increased and its arrival time determined more accurately by means of thickness shear-transducer elements. This type of element generates some compressional energy, so that both waves may be detected. Energy transmission between the specimen and each transducer may be improved by using a thin layer of a coupling medium such as phenyl salicylate, high-vacuum grease, or resin, and by pressing the transducer against the specimen with a small seating force.

6.3.4 For specimens subjected to uniaxial stress fields, first arrivals of compression waves are usually well defined. However, the accurate determination of shear-wave first arrivals for specimens under stress is complicated by mode conversions at the interfaces on either side of the face plate and at the free boundary of the specimen (4). Shear-wave arrivals are therefore difficult to determine and experience is required for accurate readings.

6.4 *Ultrasonic Elastic Constants*—The rock must be isotropic or possess only a slight degree of anisotropy if the ultrasonic elastic constants are to be calculated (Section 7). In order to estimate the degree of anisotropy of the rock, measure the compression-wave velocity in three orthogonal directions, and in a fourth direction oriented at 45° from any one of the former three directions if required as a check. Make these measurements with the same geometry, that is, all between parallel flat surfaces or all across diameters. The equations in 7.2 for an isotropic medium shall not be applied if any of the three compression-wave velocities varies by more than 2 % from their average value. The error in E and G (see 7.2) due to both anisotropy and experimental error will then normally not exceed 6 %. The maximum possible error in μ, λ, and K depends markedly upon the relative values of V_p and V_s as well as upon testing errors and anisotropy. In common rock types the respective percent of errors for μ, λ, and K may be large as or even higher than 24, 36, and 6. For greater anisotropy, the possible percent of error in the elastic constants would be still greater.

7. Calculation

7.1 Calculate the propagation velocities of the compression and shear waves, V_p and V_s respectively, as follows:

$$V_p = L_p/T_p$$
$$V_s = L_s/T_s$$

where:

V = pulse-propagation velocity, in./s (or m/s),

L = pulse-travel distance, in. (or m),

T = effective pulse-travel time (measured time minus zero time correction), s,

and subscripts $_p$ and $_s$ denote the compression wave and shear wave, respectively.

7.2 If the degree of velocity anisotropy is 2 % or less, as specified in 6.4, calculate the ultrasonic elastic constants as follows:

$$E = [\rho V_s^2(3V_p^2 - 4V_s^2)]/(V_p^2 - V_s^2)$$

where:

E = Young's modulus of elasticity, psi (or Pa), and

ρ = density, lb/in.^{3} (or kg/m^3);

$$G = \rho V_s^2$$

where:

G = modulus of rigidity or shear modulus, psi (or Pa);

$$\mu = (V_p^2 - 2V_s^2)/[2(V_p^2 - V_s^2)]$$

where:

μ = Poisson's ratio;

$$\lambda = \rho(V_p^2 - 2V_s^2)$$

where:

λ = Lamé's constant, psi (or Pa); and

$$K = \rho(3V_p^2 - 4V_s^2)/3$$

where:

K = bulk modulus, psi (or Pa).

8. Report

8.1 The report shall include the following:

8.1.1 Identification of the test specimen including rock type and location,

8.1.2 Density of test specimen,

8.1.3 General indication of moisture condition of sample at time of test such as as-received, saturated, laboratory air dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation.

8.1.4 Degree of anisotropy expressed as the maximum percent deviation of compression-pulse velocity from the average velocity determined from measurements in three directions,

8.1.5 Stress level of specimens,

8.1.6 Calculated pulse velocities for compression and shear waves with direction of measurement,

8.1.7 Calculated ultrasonic elastic constants (if desired and if degree of anisotropy is not greater than specified limit),

8.1.8 Coupling medium between transducers and specimen, and

8.1.9 Other data such as physical properties, composition, petrography, etc., if determined.

9. Precision and Bias

9.1 *Precision*—Due to the nature of rock materials tested by this test method, it is either not feasible or too costly at this time to produce multiple specimens which have uniform physical properties. Any variation observed in the data is just as likely to be due to specimen variation as to operator or laboratory testing variation. Subcommittee D18.12 welcomes proposals that would allow for development of a valid precision statement.

9.2 *Bias*—There is no accepted reference value for this test method; therefore, bias cannot be determined.

10. Keywords

10.1 compression testing; isotrophy; ultrasonic testing; velocity-pulse

REFERENCES

(1) Simmons, Gene, "Ultrasonics in Geology," *Proceedings, Inst. Electrical and Electronic Engineers*, Vol 53, No. 10, 1965, pp. 1337-1345.

(2) Whitehurst, E. A., *Evaluation of Concrete Properties from Sonic Tests*, Am. Concrete Inst., Detroit, Mich., and the Iowa State Univ. Press, Ames, Iowa, 1966, pp. 1-2.

(3) Cannaday, F. X., "Modulus of Elasticity of a Rock Determined by Four Different Methods," *Report of Investigations U.S. Bureau of Mines 6533*, 1964.

(4) Thill, R. E., McWilliams, J. R., and Bur, T. R., "An Acoustical Bench for an Ultrasonic Pulse System," *Report of Investigations U.S. Bureau of Mines 7164*, 1968.

(5) Gregory, A. R., "Shear Wave Velocity Measurements of Sedimentary Rock Samples under Compression," *Rock Mechanics*, Pergamon Press, New York, N.Y., 1963, pp. 439-471.

(6) Silaeva, O. I., and Shamina, O. G., "The Distribution of Elastic Pulses in Cylindrical Specimens," *USSR Academy of Sciences (Izvestiya), Geophysics Series*, 1958, pp. 32-43, (English ed., Vol 1, No. 1, 1958, pp. 17-24).

Designation: D 2938 – 86 RTH 111-89

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa. 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Test Method for UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMENS[1]

This standard is issued under the fixed designation D 2938; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This test method covers the determination of the unconfined compressive strength of intact cylindrical rock specimens.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

D 4543 Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances[2]

E 4 Practices for Load Verification of Testing Machines[3]

E 122 Recommended Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[4]

3. Apparatus

3.1 *Loading Device*, to apply and measure axial load on the specimens, of sufficient capacity to apply load at a rate conforming to the requirements set forth in 5.3. It shall be verified at suitable time intervals in accordance with the procedures given in Practices E 4, and comply with the requirements prescribed therein.

3.2 *Bearing Surfaces*—The testing machine shall be equipped with two steel bearing blocks having a Rockwell hardness of not less than HRC 58 (Note 1). One of the blocks shall be spherically seated and the other a plain rigid block. The bearing faces shall not depart from a plane by more than 0.0005 in. (0.013 mm) when the blocks are new and shall be maintained within a permissible variation of 0.001 in. (0.025 mm). The diameter of the spherically seated bearing face shall be at least as large as that of the test specimen but shall not exceed twice the diameter of the test specimen. The center of the sphere for the spherically seated block shall coincide with the bearing face of the specimen. The movable portion of the bearing block shall be held closely in the spherical seat, but the design shall be such that the bearing face can be rotated and tilted through small angles in any direction.

NOTE 1—False platens, with plane bearing faces conforming to the requirements of this method may be used. These shall consist of disks about ½ to ¾ in. (12.7 to 19.05 mm) thick, oil-hardened to more than HRC 58, and surface ground. With abrasive rocks, these platens tend to roughen after a number of specimens have been tested, and hence need to be resurfaced from time to time.

4. Test Specimens

4.1 Prepare test specimens in accordance with Practice D 4543.

4.2 The moisture condition of the specimen

[1] This method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Oct. 31, 1986. Published December 1986. Originally published as D 2938 – 71. Last previous edition D 2938 – 79.

[2] *Annual Book of ASTM Standards*, Vol 4.08.

[3] *Annual Book of ASTM Standards*, Vols 03.01, 04.01, 07.01, and 08.03.

[4] *Annual Book of ASTM Standards*, Vol 14.02.

at time of test can have a significant effect upon the indicated strength of the rock. Good practice generally dictates that laboratory tests be made upon specimens representative of field conditions. Thus it follows that the field moisture condition of the specimen should be preserved until time of test. On the other hand, there may be reasons for testing specimens at other moisture contents including zero. In any case, tailor the moisture content of the test specimen to the problem at hand and report it in accordance with 7.1.7.

5. Procedure

5.1 Check the ability of the spherical seat to rotate freely in its socket before each test.

5.2 Wipe clean the bearing faces of the upper and lower bearing blocks and of the test specimen and place the test specimen on the lower bearing block. As the load is gradually brought to bear on the specimen, adjust the movable portion of the spherically seated block so that uniform seating is obtained.

5.3 Many rock types fail in a violent manner when loaded to failure in compression. A protective shield should be placed around the test specimen to prevent injury from flying rock fragments.

5.4 Apply the load continuously and without shock to produce an approximately constant rate of load or deformation such that failure will occur within 5 to 15 min of loading.

NOTE 2—Results of tests by several investigators have shown that strain rates within this range will provide strength values that are reasonably free of rapid loading effects and reproducible within acceptable tolerances.

6. Calculation

6.1 Calculate the unconfined compressive strength of the specimen by dividing the maximum load carried by the specimen during the test by the cross-sectional area calculated and express the result to the nearest 10 psi (68.9 kPa).

7. Report

7.1 The report should include the following:

7.1.1 Source of sample including project name and location, and, if known, storage environment. The location is frequently specified in terms of the borehole number and depth of specimen from collar of hole.

7.1.2 Physical description of sample including rock type; location and orientation of apparent weakness planes, bedding planes, and schisotosity; large inclusions or inhomogeneities, if any.

7.1.3 Date of sampling and testing.

7.1.4 Specimen diameter and length, conformance with dimensional requirements (see Note 4).

7.1.5 Rate of loading or deformation rate.

7.1.6 General indication of moisture condition of sample at time of test such as as-received, saturated, laboratory air dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation.

7.1.7 Unconfined compressive strength for each specimen as calculated average unconfined compressive strength of all specimens tested, standard deviation or coefficient of variation.

7.1.8 Type and location of failure. A sketch of the fractured sample is recommended.

7.1.9 Other available physical data.

NOTE 3—If only cores with a length-to-diameter ratio (L/D) of less than 2 are available, these may be used for the test and this fact noted in the report. However, a length-to-diameter ratio (L/D) as close to 2 as possible should be used. This apparent compressive strength may then be corrected in accordance with the following equation:

$$C' = C_a/(0.88 + (0.24b/h))$$

where:

C' = computed compressive strength of an equivalent L/D = 2 specimen,
C_a = measured compressive strength of the specimen tested,
b = test core diameter, and
h = test core height.

NOTE 4—The number of specimens tested may depend upon availability of specimens, but normally a minimum of ten is preferred. The number of specimens tested should be indicated. The statistical basis of relating the required number of specimens to the variability of measurements is given in Recommended Practice E 122.

8. Precision and Bias

8.1 The variability of rock and resultant in-

ability to determine a true reference value prevent development of a meaningful statement of bias. Data are being evaluated to determine the precision of this test method. In addition, the subcommittee is seeking pertinent data from users of the method.

Designation: D 2936 – 84 (Reapproved 1989)$^{\epsilon 2}$

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St. Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards. Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Test Method for Direct Tensile Strength of Intact Rock Core Specimens[1]

This standard is issued under the fixed designation D 2936; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

$^{\epsilon 1}$ NOTE—Section 9 was changed editorially in July 1989.
$^{\epsilon 2}$ NOTE—Section 10 was added editorially in December 1991.

1. Scope

1.1 This test method covers the determination of the direct tensile strength of intact cylindrical rock specimens.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

E 4 Practices for Load Verification of Testing Machines[2]

E 122 Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[3]

3. Significance and Use

3.1 Rock is much weaker in tension than in compression. Thus, in determining the failure condition for a rock structure, many investigators employ the tensile strength of the component rock as the failure strength for the structure. Direct tensile stressing of rock is the most basic test for determining the tensile strength of rock.

4. Apparatus

4.1 *Loading Device*, to apply and measure axial load on the specimen, of sufficient capacity to apply the load at a rate conforming to the requirements of 6.2. The device shall be verified at suitable time intervals in accordance with the procedures given in Practices E 4 and shall comply with the requirements prescribed therein.

4.2 *Caps*—Cylindrical metal caps that, when cemented to the specimen ends, provide a means through which the direct tensile load can be applied. The diameter of the metal caps shall not be less than that of the test specimen, nor shall it exceed the test specimen diameter by more than 0.0625 in. (1.6 mm). Caps shall have a thickness of at least 1¼ in. (32 mm). Caps shall be provided with a suitable linkage system for load transfer from the loading device to the test specimen. The linkage system shall be so designed that the load will be transmitted through the axis of the test specimen without the application of bending or torsional stresses. The length of the linkages at each end shall be at least two times the diameter of the metal end caps. One such system is shown in Fig. 1.

NOTE 1—Roller or link chain of suitable capacity has been found to perform quite well in this application. Because roller chain flexes in one plane only, the upper and lower segments should be positioned at right angles to each other to effectively reduce bending in the specimen. Ball-and-socket, cable, or similar arrangements have been found to be generally unsuitable as their tendency for bending and twisting makes the assembly unable to transmit a purely direct tensile stress to the test specimen.

5. Test Specimens

5.1 Test specimens shall be right circular cylinders within the following tolerances:

5.1.1 The sides of the specimen shall be generally smooth and free of abrupt irregularities with all the elements straight to within 0.020 in. (0.50 mm) over the full length of the specimen. The deviation from straightness of the elements shall be determined by either Method A or Method B as follows:

5.1.1.1 *Method A*—Roll the cylindrical specimen on a smooth flat surface and measure the height of the maximum gap between the specimen and the flat surface with a feeler gage. If the maximum gap exceeds 0.020 in. (0.50 mm), the specimen does not meet the required tolerance for straightness of the elements. The flat test surface on which the specimen is rolled shall not depart from a plane by more than 0.0005 in. (15 μm).

5.1.1.2 *Method B:*

5.1.1.2.1 Place the cylindrical surface of the specimen on a V-block that is laid flat on a surface. The smoothness of the surface shall not depart from a plane by more than 0.0005 in. (15 μm).

5.1.1.2.2 Place a dial indicator in contact with the top of the specimen as shown in Fig. 2, and observe the dial reading as the specimen is moved from one end of the V-block to the other along a straight line.

5.1.1.2.3 Record the maximum and minimum readings on the dial gage and calculate the difference, Δ_0. Repeat the same operations by rotating the specimen for every 90°, and obtain the differences, Δ_{90}, Δ_{180}, and Δ_{270}. The maximum value of these four differences shall be less than 0.020 in. (0.50 mm).

5.1.2 Cut the ends of the specimen parallel to each other,

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Sept. 28, 1984. Published November 1984. Originally published as D 2936 – 71. Last previous edition D 2936 – 71.

[2] *Annual Book of ASTM Standards*, Vols 03.01, 04.02, and 08.03.

[3] *Annual Book of ASTM Standards*, Vol 14.02.

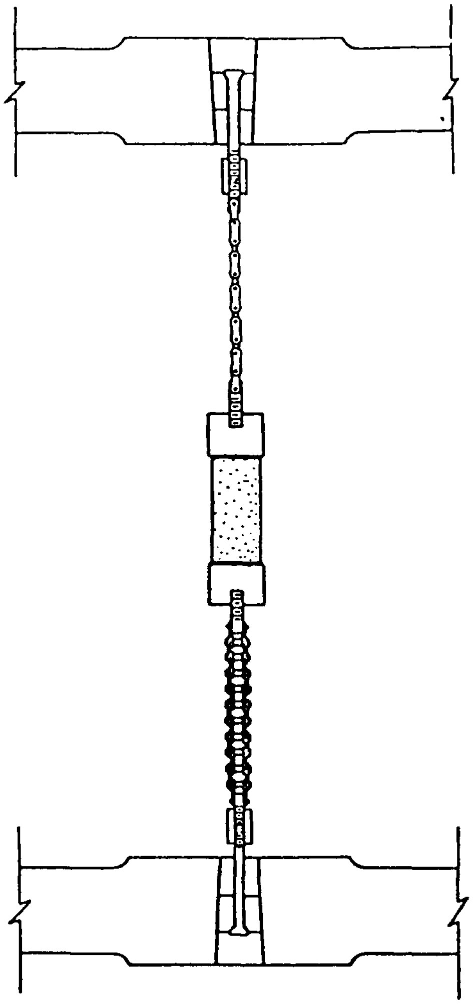


FIG. 1 Direct Tensile-Strength Test Assembly

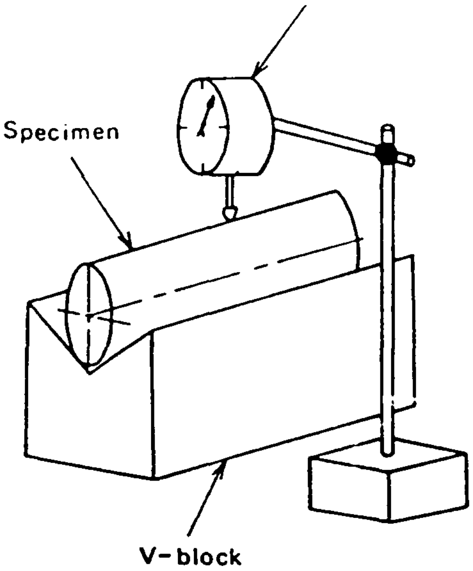


FIG. 2 Assembly for Determining the Straightness of the Cylindrical Surface

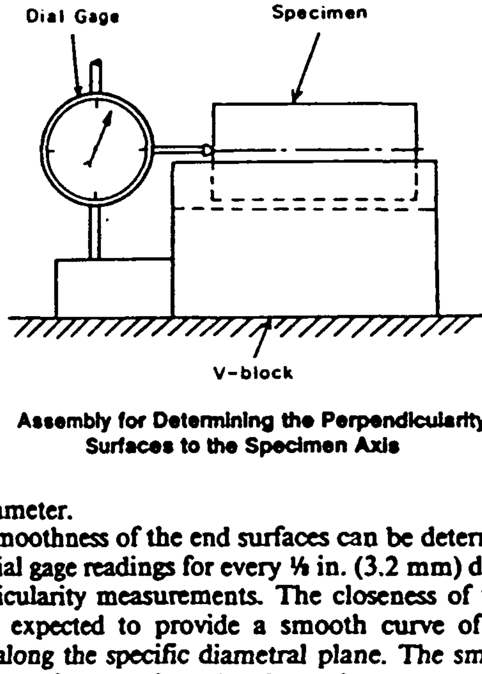


FIG. 3 Assembly for Determining the Perpendicularity of End Surfaces to the Specimen Axis

generally smooth (Note 2), and at right angles to the longitudinal axis. The ends shall not depart from perpendicularity to the axis of the specimen by more than 0.25°, approximately 0.01 in. (0.3 mm) in 2 in. (50.0 mm). The perpendicularity of the end surfaces to the longitudinal axis shall be determined by the similar setup as for the cylindrical surface (Fig. 3), except that the dial gage is mounted near the end of the V-block. Move the mounting pad horizontally so that the dial gage runs across the end surface of the specimen along a diametral direction. Take care to ensure that one end of the mounting pad maintains intimate contact with the end surface of the V-block during moving. Record the dial gage readings and calculate the difference between the maximum and the minimum values, Δ_1. Rotate the specimen 90° and repeat the same operations and calculate the difference, Δ_2. Turn the specimen around and repeat the same measurement procedures for the other end surface and obtain the difference values Δ'_1 and Δ'_2. The perpendicularity will be considered to have been met when:

$$\frac{\Delta_I}{D} \text{ and } \frac{\Delta'_I}{D} \leq 0.005$$

where:
I = 1 or 2, and
D = diameter.

The smoothness of the end surfaces can be determined by taking dial gage readings for every ⅛ in. (3.2 mm) during the perpendicularity measurements. The closeness of the readings are expected to provide a smooth curve of the end surface along the specific diametral plane. The smoothness requirement is met when the slope along any part of the curve is less than 0.25°.

NOTE 2—In this test method, the condition of the specimen ends with regard to the degree of flatness and smoothness is not as critical as it is, for example, in compression tests where good bearing is a prerequisite. In direct-tension tests it is more important that the ends be parallel to each other and perpendicular to the longitudinal axis of the specimen in order to facilitate the application of a direct tensile load. End surfaces, such as result from sawing with a diamond cut-off wheel, are entirely adequate. Grinding, lapping, or polishing beyond this point serves no useful purpose, and in fact, may adversely affect the adhesion of the cementing medium.

5.1.3 The specimen shall have a length-to-diameter ratio (L/D) of 2.0 to 2.5 (Note 3) and a diameter of not less than

NX wireline core size, approximately 1⅞ in. (48 mm) (Note 4).

NOTE 3—In direct-tension tests, the specimen should be free to select and fail on the weakest plane within its length. This degree of freedom becomes less as the specimen length diminishes. When cores of shorter than standard length must be tested, make suitable notation of this fact in the test report.

NOTE 4—It is desirable that the diameter of rock tension specimens be at least ten times greater than the diameter of the largest mineral grain. It is considered that the specified minimum specimen diameter of approximately 1⅞ in. (48 mm) will satisfy this criterion in the majority of cases. It may be necessary in some instances to test specimens that do not comply with this criterion. In this case, and particularly when cores of diameter smaller than the specified minimum must be tested because of the unavailability of larger size specimens (as is often the case in the mining industry) make suitable notation of these facts in the test report, and mention the grain size.

5.1.4 Determine the diameter of the test specimen to the nearest 0.01 in. (0.25 mm) by averaging two diameters measured at right angles to each other at about midlength of the specimen. Use this average diameter for calculating the cross-sectional area. Determine the length of the test specimen to the nearest 0.01 in. by averaging two height measurements along the diameter.

5.1.5 The moisture condition of the specimen at the time of test can have a significant effect upon the indicated strength of the rock. Good practice generally dictates that laboratory tests be made upon specimens representative of field conditions or conditions expected under the operating environment. However, consider also that there may be reasons for testing specimens at other moisture contents, or with none. In any case, relate the moisture content of the test specimen to the problem at hand and report the content in accordance with 8.1.6.

6. Procedure

6.1 Cement the metal caps to the test specimen to ensure alignment of the cap axes with the longitudinal axis of the specimen (Note 5). The thickness of the cement layer should not exceed 1/16 in. (1.6 mm) at each end. The cement layer must be of uniform thickness to ensure parallelism between the top surfaces of the metal caps attached to both ends of the specimens. This should be checked before the cement is hardened (Note 5) by measuring the length of the specimen and end-cap assembly at three locations 120° apart and near the edge. The maximum difference between these measurements should be less than 0.005 in. (0.13 mm) for each 1.0 in. (25.0 mm) of specimen diameter. After the cement has hardened sufficiently to exceed the tensile strength of the rock, place the test specimen in the testing machine, making certain that the load transfer system is properly aligned.

NOTE 5—In cementing the metal caps to the test specimens, use jigs and fixtures of suitable design to hold the caps and specimens in proper alignment until the cement has hardened. The chucking arrangement of a machine lathe or drill press is also suitable. The cement used should be one that sets at room temperature. Epoxy resin formulations of rather stiff consistency and similar to those used as a patching and filling compound in automobile body repair work have been found to be a suitable cementing medium.

6.2 Apply the tensile load continuously and without shock to failure. Apply the load or deformation at an approximately constant rate such that failure will occur in not less than 5 nor more than 15 min. Note and record the maximum load carried by the specimen during the test.

NOTE 6—In this test arrangement failure often occurs near one of the capped ends. Discard the results for those tests in which failure occurs either partly or wholly within the cementing medium.

7. Calculation

7.1 Calculate the tensile strength of the rock by dividing the maximum load carried by the specimen during test by the cross-sectional area computed in accordance with 5.3; express the result to the nearest 5 psi (35.0 kPa).

8. Report

8.1 The report shall include the following:

8.1.1 Source of sample including project name and location, and, if known, storage environment (the location is frequently specified in terms of the borehole number and depth of specimen from collar of hole),

8.1.2 Physical description of sample including: rock type, location and orientation of apparent planes, bedding planes, and schisotosity; and large inclusions or inhomogeneities, if any,

8.1.3 Date of sampling and testing,

8.1.4 Specimen length and diameter, also conformance with dimensional requirements as stated in Notes 3 and 4,

8.1.5 Rate of loading or deformation rate,

8.1.6 General indication of moisture condition of sample at time of test, such as as-received, saturated, laboratory air dry, or oven dry (It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation),

8.1.7 Direct tensile strength for each specimen as calculated, average direct tensile strength of all specimens, standard deviation or coefficient of variation,

8.1.8 Type and location of failure (A sketch of the fractured sample is recommended), and

8.1.9 Other available physical data.

NOTE 7—The number of specimens tested may depend upon the availability of specimens, but normally a minimum of ten is preferred. The number of specimens tested should be indicated. The statistical basis for relating the required number of specimens to the variability of measurements is given in Recommended Practice E 122.

9. Precision and Bias

9.1 *Precision*—Due to the nature of rock materials tested by this test method, it is, at this time, either not feasible or too costly to produce multiple specimens which have uniform physical properties. Therefore, since specimens which would yield the same test results cannot be tested, Subcommittee D18.12 cannot determine the variation between tests since any variation observed is just as likely to be due to specimen variation as to operator or laboratory testing variation. Subcommittee D18.12 welcomes proposals to resolve this problem that would allow for development of a valid precision statement.

9.2 *Bias*—There is no accepted reference value for this test method; therefore, bias cannot be determined.

10. Keywords

10.1 leading tests; rock; tension (tensile) properties/tests

Designation: D 3967 – 92 (Reapproved 1992)[ε1]

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St. Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards. Copyright ASTM
If not listed in the current combined index, will appear in the next edition.

Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens[1]

This standard is issued under the fixed designation D 3967; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

ε1 NOTE—Section 10 was corrected editorially November 1992.

1. Scope

1.1 This test method covers testing apparatus, specimen preparation, and testing procedures for determining the splitting tensile strength of rock by diametral line compression of a disk.

NOTE 1—The tensile strength of rock determined by tests other than the straight pull test is designated as the "indirect" tensile strength and, specifically, the value obtained in Section 8 of this test is termed the "splitting" tensile strength.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard does not purport to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Document

2.1 *ASTM Standard:*

E 4 Practices for Load Verification of Testing Machines[2]

3. Significance and Use

3.1 By definition the tensile strength is obtained by the direct uniaxial tensile test. But the tensile test is difficult and expensive for routine application. The splitting tensile test appears to offer a desirable alternative, because it is much simpler and inexpensive. Furthermore, engineers involved in rock mechanics design usually deal with complicated stress fields, including various combinations of compressive and tensile stress fields. Under such conditions, the tensile strength should be obtained with the presence of compressive stresses to be representative of the field conditions. The splitting tensile strength test is one of the simplest tests in which such stress fields occur. Since it is widely used in practice, a uniform test method is needed for data to be comparable. A uniform test is also needed to insure positively that the disk specimens break diametrally due to tensile pulling along the loading diameter.

4. Apparatus

4.1 *Loading Device*, to apply and measure axial load on the specimen, of sufficient capacity to apply the load at a rate conforming to the requirements in 7.3. It shall be verified at suitable time intervals in accordance with Practices E 4 and shall comply with the requirements prescribed therein.

4.2 *Bearing Surfaces*—The testing machine shall be equipped with two steel bearing blocks having a Rockwell hardness of not less than 58 HRC (Note 2). One of the blocks shall be spherically seated and the other a plain rigid block. The bearing faces shall not depart from a plane by more than 0.0005 in. (0.0127 mm) when the blocks are new and shall be maintained within a permissible variation of 0.001 in. (0.025 mm). The diameter of the spherically seated bearing face shall be at least as large as that of the test specimen but shall not exceed twice the diameter of the test specimen. The movable portion of the bearing block shall be held closely in the spherical seat, but the design shall be such that the bearing face can be rotated and tilted through small angles in any direction.

NOTE 2—False platens, with plane bearing faces conforming to the requirements of this standard, may be used. These shall consist of disks about ½ to ¾ in. (12.7 to 19.05 mm) thick, oil hardened to more than 58 HRC, and surface ground. With abrasive rocks these platens tend to roughen after a number of specimens have been tested, and hence need to be resurfaced from time to time.

4.2.1 During testing the specimen can be placed in direct contact with the machine bearing plates (or false platens, if used) (Fig. 1). Otherwise, curved supplementary bearing plates or bearing strips should be placed between the specimen and the machine bearing plates to reduce high stress concentration.

4.2.2 Curved supplementary bearing plates with the same specifications as described in 4.2 may be used to reduce the contact stresses. The radius of curvature of the supplementary bearing plates shall be so designed that their arc of contact with the specimen will in no case exceed 15° or that the width of contact is less than $D/6$, where D is the diameter of the specimen.

NOTE 3—Since the equation used in 8.1 for splitting tensile strength is derived based on a line load, the applied load shall be confined to a very narrow strip if the splitting tensile strength test is to be valid. But a line load creates extremely high contact stresses which cause premature cracking. A wider contact strip can reduce the problem significantly. Investigations show that an arc of contact smaller than 15° causes no more than 2 % of error in principal tensile stress while reducing the incidence of premature cracking greatly.

4.3 *Bearing Strips* (0.01 D thick cardboard cushion, where D is the specimen diameter; or up to 0.25 in. thick plywood cushion are recommended to place between the machine bearing surfaces (or supplementary bearing plates; if used)

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved May 15, 1992. Published July 1992. Originally published as D 3967 – 81. Last previous edition D 3967 – 86[ε1].

[2] *Annual Book of ASTM Standards*, Vols 03.01, 04.02, and 08.03.

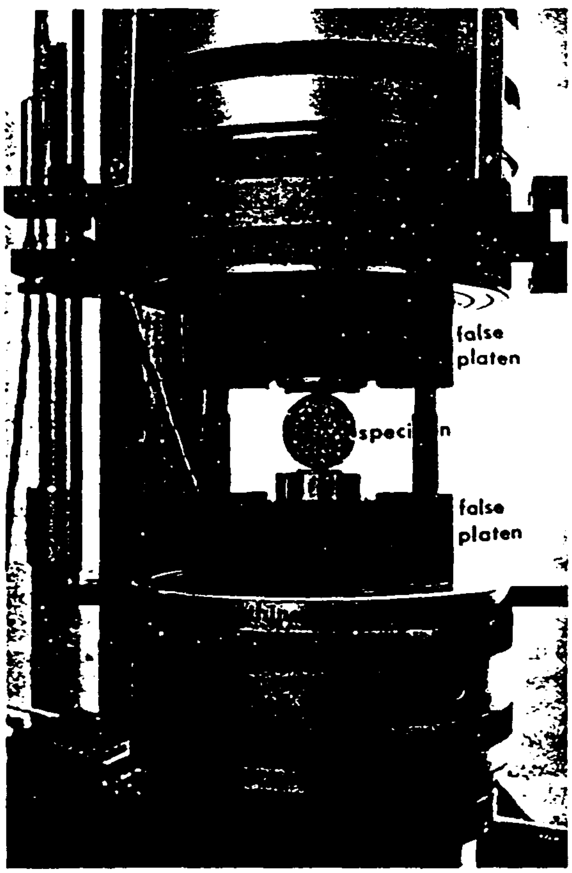


FIG. 1 One of the Proposed Testing Setup for Splitting Tensile Strength

and the specimen to reduce high stress concentration.

NOTE 4—Experiences have indicated that test results using the curved supplementary bearing plates and bearing strips, as specified in 4.2.2 and 4.3, respectively, do not significantly differ from each other, but there may be some consistent difference from the results of tests in which direct contact between the specimen and the machine platen is used.

5. Sampling

5.1 The specimen shall be selected from the core to represent a true average of the type of rock under consideration. This can be achieved by visual observations of mineral constituents, grain sizes and shape, partings, and defects such as pores and fissures.

6. Test Specimens

6.1 *Dimensions*—The test specimen shall be a circular disk with a thickness-to-diameter ratio (L/D) between 0.5 and 0.75. The diameter of the specimen shall be at least 10 times greater than the largest mineral grain constituent. A diameter of 1-$^{15}/_{16}$ in. (wireline core) will generally satisfy this criterion.

NOTE 5—When cores smaller than the specified minimum must be tested because of the unavailability of material, notation of the fact shall be made in the test report.

NOTE 6—If the specimen shows apparent anisotropic features such as bedding or schistosity, care shall be exercised in preparing the specimen so that the orientation of the loading diameter relative to anisotropic features can be determined precisely. Notation of this orientation shall be made in the test report.

6.2 *Number of specimens*—At least ten specimens shall be tested to obtain a meaningful average value. If the reproducibility of the test results is good (coefficient of variation less than 5 %), a smaller number of specimens is acceptable.

6.3 The circumferential surface of the specimen shall be smooth and straight to 0.020 in. (0.50 mm).

6.4 Cut the ends of the specimen parallel to each other and at right angles to the longitudinal axis. The ends of the specimen shall not deviate from perpendicular to the core axis by more than 0.5°. This requirement can be generally met by cutting the specimen with a precision diamond saw.

6.5 Determine the diameter of the specimen to the nearest 0.01 in. (0.25 mm) by taking the average of at least three measurements, one of which shall be along the loading diameter.

6.6 Determine the thickness of the specimen to the nearest 0.01 in. (0.25 mm) by taking the average of at least three measurements, one of which shall be at the center of the disk.

6.7 The moisture conditions of the specimen at the time of test can have a significant effect upon the indicated strength of the rock. The field moisture condition for the specimen shall be preserved until the time of test. On the other hand, there may be reasons for testing specimens at other moisture contents, including zero, and preconditioning of specimen when moisture control is needed. In any case, tailor the moisture content of the test specimen to the problem at hand and report it in accordance with 9.1.6.

7. Procedure

7.1 *Marking*—The desired vertical orientation of the specimen shall be indicated by marking a diametral line on each end of the specimen. These lines shall be used in centering the specimen in the testing machine to ensure proper orientation, and they are also used as the reference lines for thickness and diameter measurements.

NOTE 7—If the specimen is anisotropic, take care to ensure that the marked lines in each specimen refer to the same orientation.

7.2 *Positioning*—Position the test specimen to ensure that the diametral plane of the two lines marked on the ends of the specimen lines up with the center of thrust of the spherically seated bearing surface to within 0.05 in. (0.013 mm).

NOTE 8—A good line loading can often be attained by rotating the specimen about its axis until there is no light visible between the specimen and the loading platens. Back lighting helps in making this observation.

7.3 *Loading*—Apply a continuously increasing compressive load to produce an approximately constant rate of loading or deformation such that failure will occur within 1 to 10 min of loading, which should fall between 500 and 3000 psi/min of loading rate, depending on the rock type.

NOTE 9—Results of tests by several investigators indicate that rates of loading at this range are reasonably free from rapid loading effects.

8. Calculation

8.1 The splitting tensile strength of the specimen shall be calculated as follows:

$$\sigma_t = 2P/LD$$

and the result shall be expressed to the appropriate number of significant figures (usually 3), where:

σ_t = splitting tensile strength, psi or Pa,
P = maximum applied load indicated by the testing machine, lbf (or N),
L = thickness of the specimen, in. (or m), and
D = diameter of the specimen, in. (or m).

9. Report

9.1 The report shall include as much of the following as possible:

9.1.1 Sources of the specimen including project name and location, and if known, storage environment. The location is frequently specified in terms of the borehole number and depth of specimen from collar of hole.

9.1.2 Physical description of the specimen including rock type; location and orientation of apparent weakness planes, bedding planes, and schistosity; large inclusions or inhomogeneities, if any.

9.1.3 Dates of sampling and testing.

9.1.4 Specimen diameter and length, conformance with dimensional requirements, direction of loading if anisotropy exists. Type of contact between the specimen and the loading platens.

9.1.5 Rate of loading or deformation rate.

9.1.6 General indication of moisture condition of the specimen at time of test such as as-received, saturated, laboratory air dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation.

9.1.7 Splitting tensile strength of each specimen as calculated, average splitting tensile strength of all specimens, standard deviation or coefficient of variation.

9.1.8 Type and location of failure. A sketch of the fractured specimen is recommended.

10. Precision and Bias

10.1 Data are being evaluated via an interlaboratory test program for rock properties to determine the precision of this test method. There is no accepted reference value of rock for this test method, therefore, bias cannot be determined.

11. Keywords

11.1 indirect tensile strength; rock; splitting tensile strength

RTH 114-93

PROPOSED METHOD OF TEST FOR GAS PERMEABILITY OF ROCK CORE SAMPLES

1. Scope

1.1 This method describes procedures for determining gas permeabilities of rock core samples. Test procedures are described for both large-diameter cores (NX size--5.4 cm and larger) and small-diameter cores which include plugs taken from larger diameter cores. Samples are normally right cylinders although cube samples may be used with a suitably modified sample holder.

1.2 The permeability measurement will be made according to standard procedures with dry air as the permeant. The value obtained with this fluid may then be corrected to a corresponding value for a nonreacting liquid using a standard table of Klinkenberg corrections.[1] The use of such corrections should be specifically noted in reporting results. Determination of permeability of rock specimens may be made with other gases or with a liquid as the permeant. However, measurements of liquid permeability are difficult to standardize because of potential interaction between rock constituents and the liquid.

1.3 Permeability measurements made parallel to the bedding planes of sedimentary rocks shall be reported as horizontal permeability while those measured perpendicular to the bedding shall be reported as vertical permeability. Structural features other than bedding may be used to describe the direction of flow during measurement. These shall be specifically noted in reporting results.

1.4 Samples of hard, consolidated rock may be cut to shape and tested without artificial support. Friable, soft, shaly, or otherwise weak rock may require additional support to resist deformation or alteration during testing. Deformation or alteration will affect test results. Such samples shall be supported by mounting in a suitable potting material (i.e. cement slurry) exercising care not to alter the surfaces through which fluid flow will occur.

---

[1]"Recommended Practice for Determining Permeability of Porous Media," 3rd ed., American Petroleum Institute RP 27 (1952).

Testing procedures are the same for both unsupported and artificially supported samples.

1.5 The conventional direction of permeability measurement for small cylindrical samples is parallel to the axis of the cylinder. Large-diameter core samples are conventionally measured by one of two methods. In one method, referred to as the "linear permeability measurement," gas flow is either across the core perpendicular to a plane formed by a diameter of the core and the vertical axis or parallel to the core axis as with the small-diameter samples. When flow is across the sample, screens are used over diametrically opposite quadrants of the core circumference to uniformly distribute the gas flow. The second method is referred as a "radial permeability measurement" in which gas flows from the outside surface of the core radially through the core to a small-diameter hole drilled concentric with the axis of the core sample.

1.6 The permeability test is applicable to a wide range of rock types with a correspondingly wide range of permeabilities. With proper selection of test equipment components, permeabilities as low as 0.01 millidarcys and as high as 10 darcys can be measured accurately.

2. Summary of Method

2.1 The method for small samples consists of (a) placing the prepared sample in a Hassler- or Fancher-type core holder, Figs. 1 and 2, (b) applying the air pressure required to seal the sleeve around the core for the Hassler-type holder or loading the rubber compression ring for the Fancher holder, and (c) initiating dry gas flow through the sample. Because of the sensitivity of permeability to minor changes in lithology, no prescribed number of samples can be recommended to define the permeability of a given rock stratum. Reproducibility of approximately ±2 percent for samples of 0.1 millidarcy or greater permeability should be obtained for a given sample.

2.2 The method used to measure permeability for large-diameter cylindrical samples with vertical gas flow is the same as that described in 2.1.

2.3 The method for large-diameter samples with horizontal flow consists of (a) positioning appropriate-size screens diametrically opposite each other, (b) attaching the screens to the sample by light rubber bands, (c) placing the

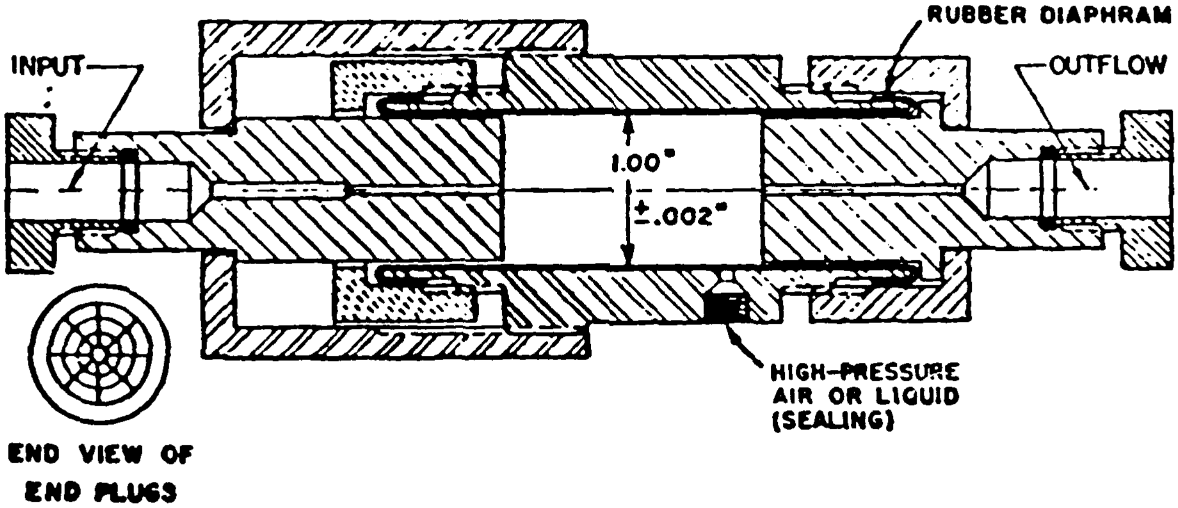


Fig. 1. Hassler-type permeability cell.

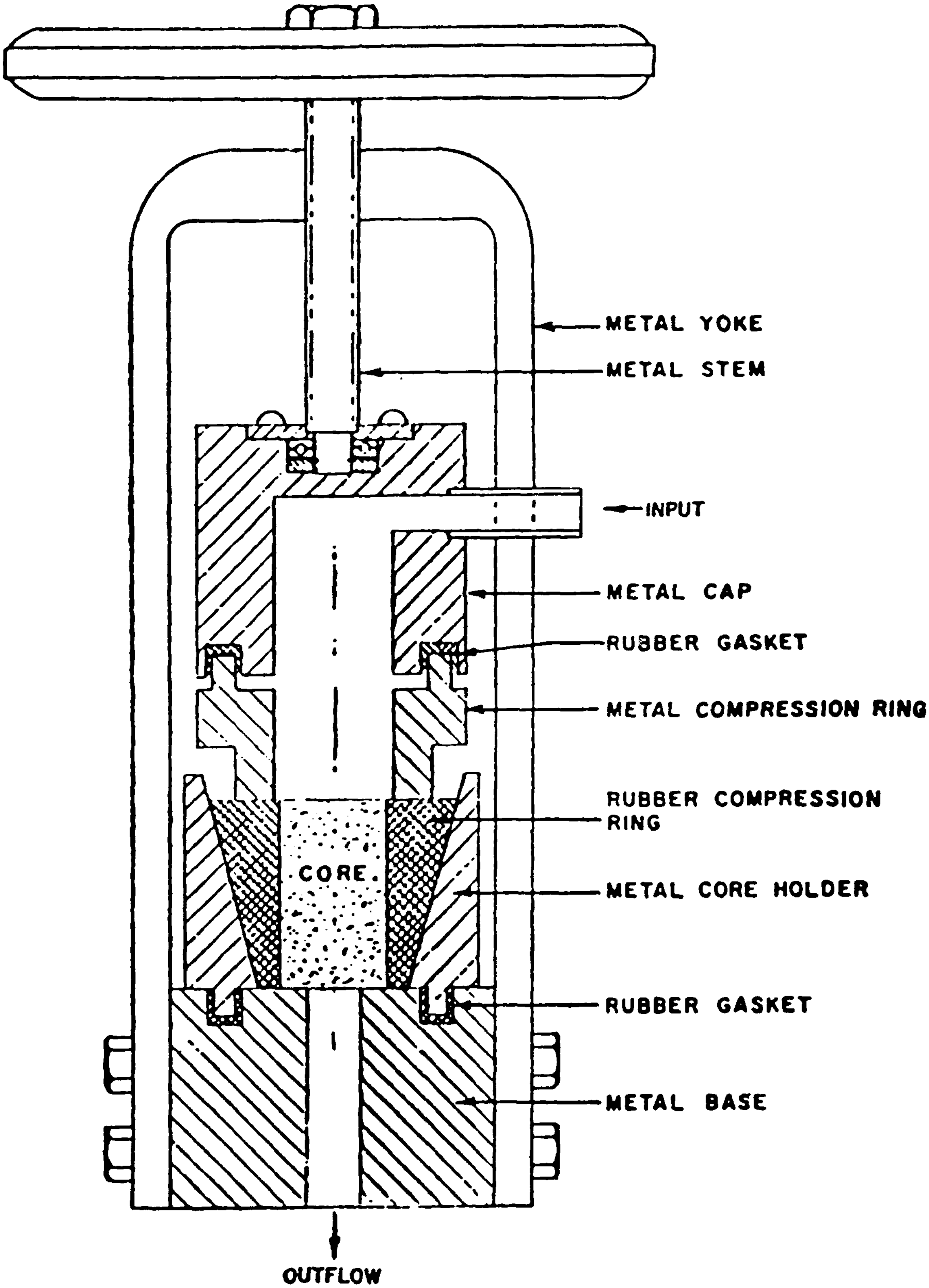


Fig. 2. Fancher-type core holder.

prepared sample in a Hassler-type or compression (ram) core holder, Figs. 3 and 4, (d) compressing the rubber gaskets which seal the ends of the samples in the Hassler-type holder (the ends of the samples used in the compression holder are presealed with plastic which is overlapped by the compression halves), (e) applying the air pressure (Hassler-type) or hydraulic force (compression) necessary to seal the sides of the sample except the area covered by the screens, and (f) initiating dry gas flow through the sample. Permeability is normally measured in two directions across the core: one is in the direction of apparent maximum permeability and the other is perpendicular to the first.

2.4 The method for large-diameter specimens with radial flow consists of (a) positioning the prepared sample in the radial flow core holder, Fig. 5, (b) raising the core against the closed lid by means of a piston, and (c) initiating dry gas flow through the sample.

3. Apparatus

3.1 Components - The apparatus used in dry air permeability testing consists of the following major components:

(a) A source of dry air

(b) Pressure regulator

(c) Inlet-pressure measuring device

(d) Core holder

(e) Outlet-pressure measuring device

(f) A dry air flow-rate metering device

3.2 Source of Air - The source of air for permeability measurements can be either the normal laboratory air supply or cylinders. Provisions should be made to filter particulate matter, absorb oil vapor, and remove water vapor. These devices should be periodically checked to insure proper operation.

3.3 Pressure Regulator - A suitable pressure regulator should be provided for the source of dry air. This regulator should apply air at a constant pressure and should be capable of doing so over a range of pressures between 1 and 80 cm of mercury (Note 1) which will produce the desired flow rate

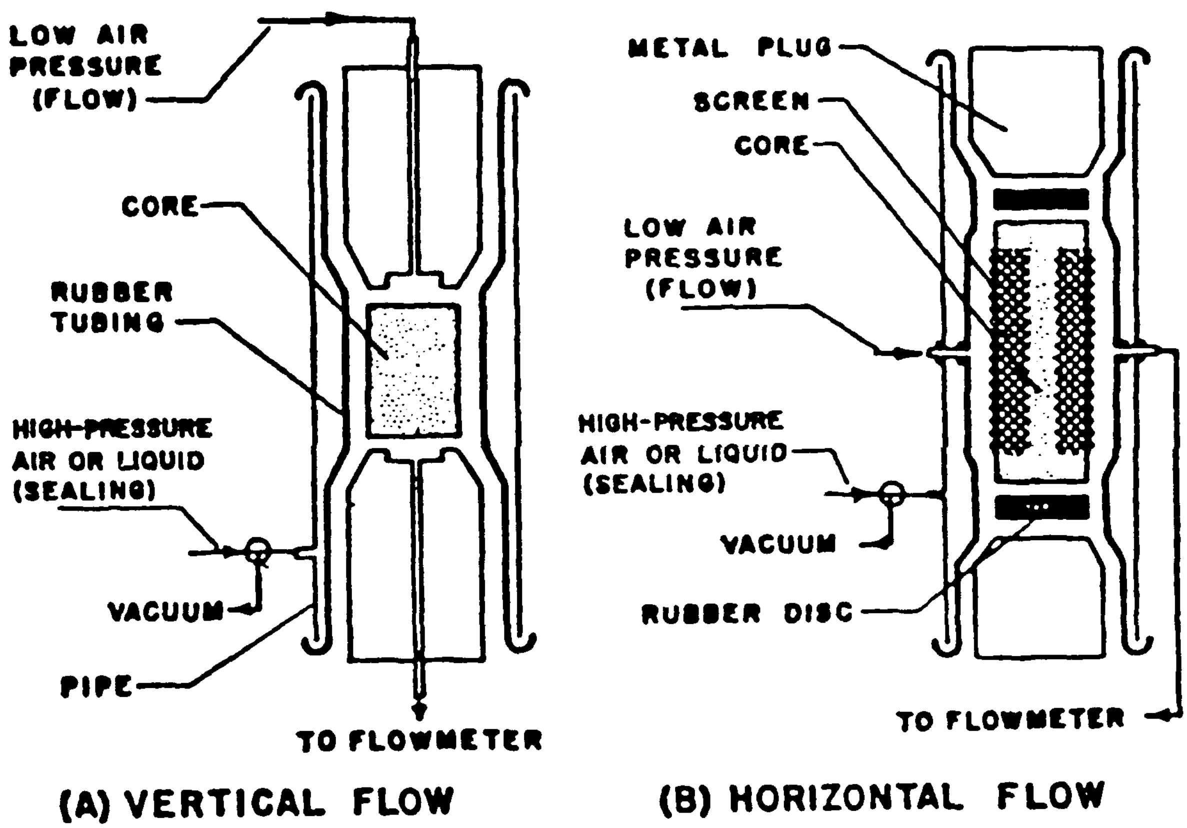


Fig. 3. Hassler-type permeameter.

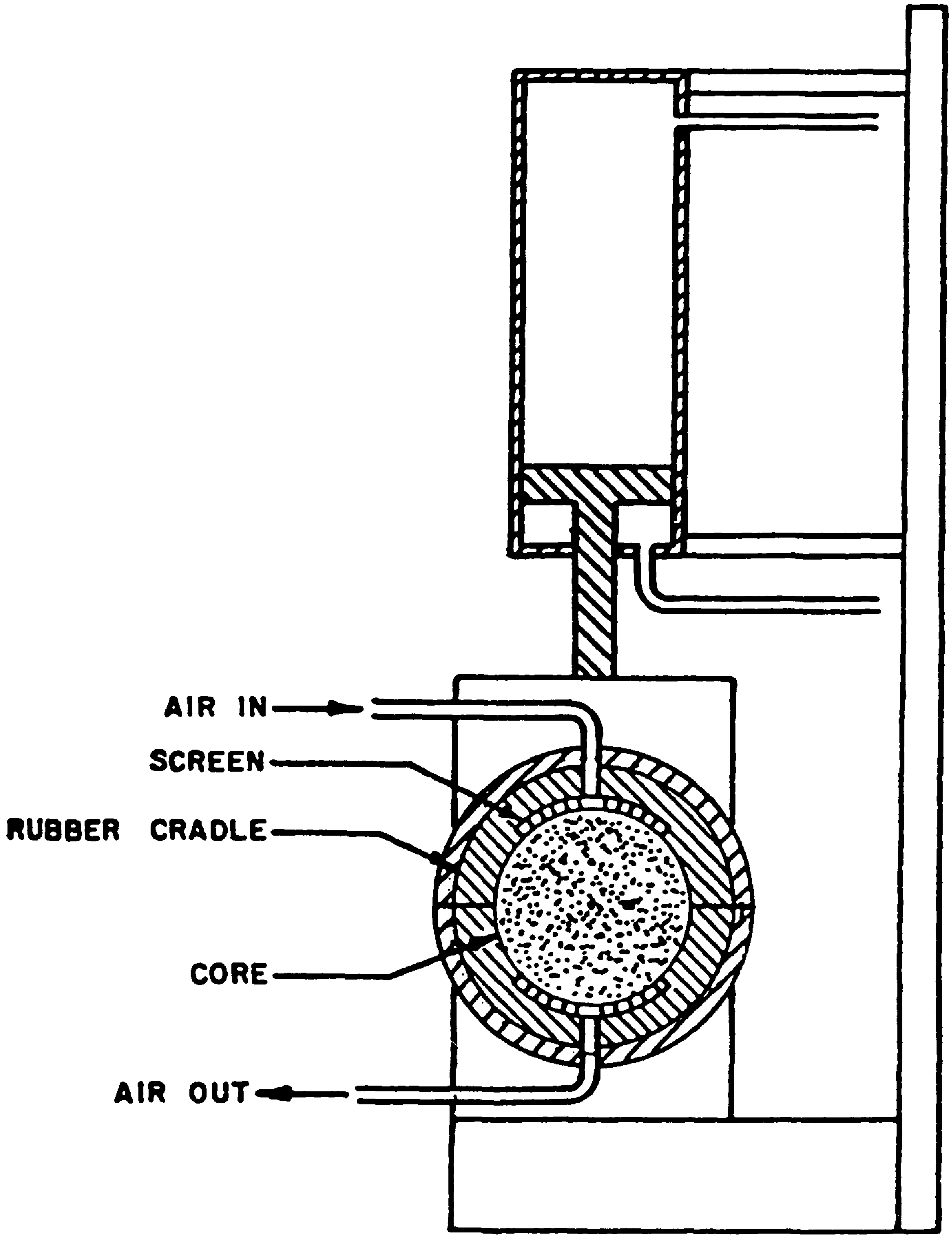


Fig. 4. Compression (RAM) permeameter.

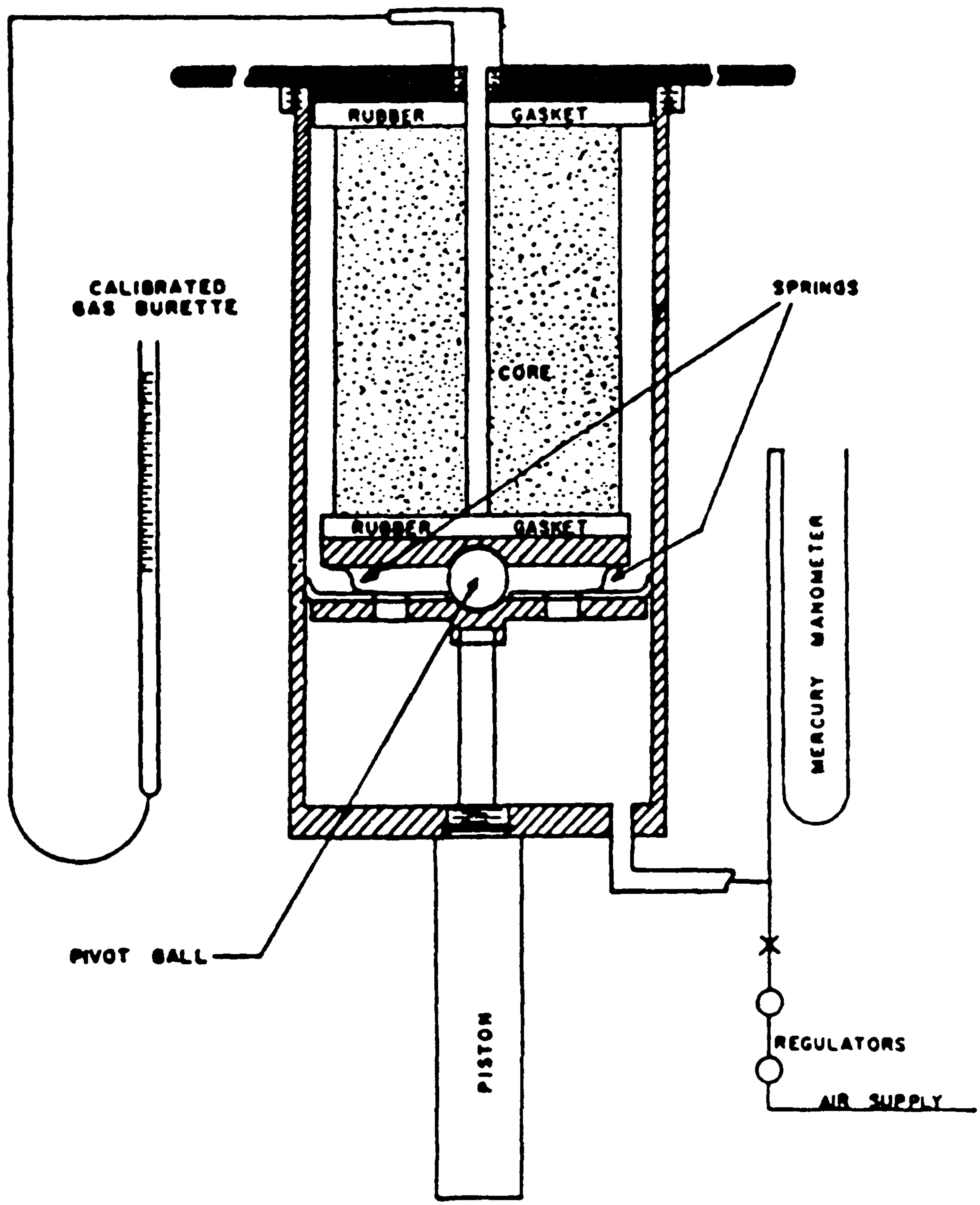


Fig. 5. Full-diameter radial permeameter.

(Note 1) of approximately 1 cu cm per second. Regulators of the pneumatic type are suitable for this purpose.

NOTE 1--The pressures required to produce the desired flow rate (1 cu cm per second) depend on the permeability and dimensions of the rock sample. Since the dimensions and permeability range of samples are not specified, the range of pressures over which pressure control is required cannot be specified. Equipment in common use operates over the pressure range of 1 to 80 cm of mercury. This pressure range is capable of producing laminar flow. This is the flow region required for permeability measurements. It is usually observed at flow rates up to 1 cu cm per second.

3.4 Pressure Measuring Devices - Inlet- and outlet-pressure measuring devices are manometers. These are water-, oil-, or mercury-filled and of a convenient length, usually 80 cm or less, with selection of length and fluid depending upon pressures to be measured. Manometers may be used in parallel to obtain the necessary accuracy over a range of pressures. Manometers are either open to the atmosphere or connected across the core. Connected across the core, they measure "differential" pressure. Where the pressure is in excess of 80 cm of mercury, a bourbon-type gage may be employed. This type of gage is normally only used in measuring extremely low permeabilities where high inlet pressures are required.

3.5 Core Holders

3.5.1 Three types of core measurements are commonly made:

(a) Axial flow in both small- and large-diameter cores

(b) Diametric flow in large-diameter cores

(c) Radial flow in large-diameter cores

3.5.2 Except for radial flow, more than one type of core holder may be used for permeability determinations. Irrespective of direction of gas flow or type of core holder used, the core holder must be such that when pressure is applied to one end of the system, all flow is through the sample. Care must be taken that no fluid bypasses the sample either through an imperfect seal between the core holder and sample or between the sample and the supporting material if the sample is mounted. Holders which accommodate several cores for simultaneous or sequential testing using matched pairs of inlet-outlet valves must be designed so that no fluid can leak between samples.

3.5.3 Selection of the type of core measurement is based on both the size of the sample available and the desired representativeness of the permeability value. For uniform, homogeneous rock small-diameter cores taken parallel and perpendicular to the bedding should provide representative permeability values. Measuring permeability of large-diameter cores is recommended for rock which is nonhomogeneous, vugular, fractured, or laminated. The radial method provides more representative data than the horizontal flow method since the entire diameter surface area of the core rather than a concentrated section of the diameter surface area of the core is involved in flow.

3.6 Screens to Distribute Gas Flow - When flow is across the core parallel to a diameter, screens should be of a wire diameter and mesh size such that a uniform gas distribution is obtained Care should be exercised in positioning the screens prior to placing the sample in the holder to ensure that they maintain their proper position while the core is being seated.

3.7 Air Flow-Rate Metering Device- Three types of dry air flow-rate metering devices may be used:

(a) Calibrated orifices (a capillary tube which is calibrated for the conditions of testing, so that the pressure drop across the orifice is small compared to the core)

(b) Soap bubble in a calibrated burette

(c) Water-displacement meters

The calibrated orifice is the most commonly used type of flow metering device. Timing the movement of a soap bubble in a burette is also frequently used. Differential pressures across the core are adjusted to minimize turbulence in the flow of gas through the sample. Flow rates of 1 cu cm per second or less are used.

3.8 Sample Preparation Equipment - Sample preparation equipment shall include the following:

3.8.1 Diamond coring equipment for taking small-diameter cylinders from larger samples.

3.8.2 Diamond saw for trimming ends of samples. Ends should be sufficiently flat and parallel for leakage seating of rubber end seals.

3.8.3 Drill press and diamond drill for drilling axial holes for radial flow measurement samples. The holes should be concentric with the axis of the cylindrical sample. Care should be exercised in drilling to prevent cracking of the sample.

3.8.4 Sample cleaning to remove original fluids from the cores, external coatings such as drilling muds, and, in the case of highly saline original fluid, deposited salts. Where the original fluids are hydrocarbons, cleaning may be accomplished by solvent extraction, gas-driven solvent extraction, distillation-extraction, or other suitable method.[2] Drilling muds may be removed from the surface with water washing. If the interstitial water is very saline, several thorough freshwater washing should remove deposited salts.

3.8.5 Drying oven that can be maintained at 110 ± 5 °C.

3.8.6 Micrometer or vernier caliper for measuring length and diameter of test specimen. Micrometer or caliper should be direct reading to 1/50 mm.

3.8.7 Equipment as required for mounting samples of weak rock in optical pitch or suitable potting plastic.

3.8.8 Miscellaneous equipment such as timing devices, magnifying lenses, etc., used in preparing the sample and measuring pressures and gas flow rates.

4. Calibration

4.1 The permeameter should be calibrated regularly by means of capillary tubes of various known permeabilities or with standard plugs.

4.2 Orifices used to measure gas flow rates should be calibrated by allowing air to flow from the orifice to a burette containing a soap bubble.

4.3 Micrometers or vernier calipers used to measure sample dimensions should be checked against length standards.

5. Sample Preparation

5.1 There are no standard sample sizes. Small-diameter samples are commonly 1.9 to 3.8 cm in diameter. Large-diameter cores are arbitrarily 5.4 cm

---

[2]Darcy, H., The Public Fountain of the Village of Dijon, Paris: Victor Dalmont, 1856 (French text).

in diameter (equivalent to NX core) and larger. Cores as large as 15.2 cm in diameter are commonly tested. Core holders are designed for a specific size or sizes of cores. The core holders are designed for a specific size or sizes of cores. The core holder may be modified to accommodate a smaller sized sample by placing the core in a rubber sleeve whose inner diameter is that of the core and whose outer diameter is that for which the permeameter was designed. Thus a 1.9-cm-diam sample can be tested in a permeameter designed for 2.54-cm samples. Sample length is also not standardized. Sample lengths vary from 2.54 cm for small-diameter samples to 60.96 cm for large-diameter cores. Core holding devices are designed to accept different length samples. Where a sleeve is used to adapt a core, differences in sleeve and sample length may be compensated for by means of spacer rings placed on top of the sample.

5.2 Ratio of sample length to diameter also is not standardized. A ratio of 1:1 is recommended as a minimum for small-diameter samples. For large-diameter samples, a minimum L/D ratio of 1:2 is recommended.

5.3 The diameter of the test specimen is measured to the nearest 0.1 mm by averaging two diameters measured at right angles to each other at about midlength of the specimen. This average diameter is used for calculating the cross-sectional area. The length of the test specimen is determined to the nearest 0.1 mm by averaging three length measurements taken at third points around the circumference.

5.4 If the sample requires artificial support, the mounting material and method of mounting should be such that penetration into the sample is minimized, the sample does not extend beyond the surface of the mounting material, and mounting material does not cover any surface perpendicular to the direction of flow. Specific details of the mounting procedures, as well as mounting materials, are given in reference 1.

5.5 In cleaning and mounting the sample prior to testing, care should be used to prevent any alteration of the minerals comprising the rock sample which may produce changes in permeability. Samples containing clays or other hydratable minerals are especially susceptible.

5.6 Before measuring permeability any sample which is suspected of being cracked should be tested by coating with a liquid while air is passing through the sample. A row of bubbles on the downstream surface will serve to indicate the presence of a crack parallel to the direction of flow. Such samples should either be discarded as nonrepresentative or, if retained, note should be made of the crack in reporting permeability values.

6. Procedure

6.1 The prepared sample is mounted in the specified core holder as follows:

6.1.1 Hassler-type Holder - Retract rubber diaphragm or sleeve, Fig. 1, by applying a vacuum to the space between the diaphragm and the body of the core holder. (Note 2)

NOTE 2--A vacuum source of 5 cm of mercury is sufficient to retract the diaphragm.

Remove one end of the core holder and insert the cylindrical sample. Reinsert the end of the core holder, applying sufficient force to seat the sample. Remove the vacuum from the space between the body and the diaphragm, thus allowing the diaphragm to constrict around the sample. (Note 3)

NOTE 3--Fine-grained, smoothly formed samples can be effectively sealed at 690 N/m^2 pressure. Coarse-grained samples will require from 1035 to 1370 N/m^2 sealing pressure.

The pressure between the diaphragm and core holder body is maintained during the permeability measurement.

6.1.2 Fancher-type Holder - Select a rubber stopper drilled to the diameter and length of the sample. Carefully push the sample plug into the tapered stopper base until it is flush with the small end surface. Remove all loose sand grains. Place the stopper containing the test sample inside the tapered material core holder, Fig. 2. Turn the ram hand wheel to move the upper ram plug down to seal tightly against the top of the holder. Compress the tapered rubber stopper with the ram to effect a seal around the sample perimeter.

6.1.3 Compression (Ram) Holder - Place the sample in the lower half of the rubber cradle, Fig. 4, making certain that the screen is centered over the air outlet and the ends of the sample are overlapped by the rubber cradle. Lower the upper compression half by applying air or hydraulic pressure above the piston. After seating the two halves, apply sufficient pressure to compress the rubber cradle with the ram to effect the seal around the sample perimeter. (See Note 3).

6.1.4 Radial Flow Holder - Place the core on a (2.54-cm) solid rubber gasket which is attached to the lower floating plate, Fig. 5. Raise the core by means of the piston against the closed lid with the center hole of the core matching that of the upper gaskets. After the core contacts the upper gasket, increase the piston pressure slightly to adjust the lower floating plate if the core ends are not parallel. Sufficient piston pressure is then applied to effect a seal of the upper and lower core surfaces. (Note 4)

NOTE 4--With gas flowing through the core at a constant rate, the piston pressure is increased. Decreased flow rate with increasing piston pressure indicates a leak at the rubber gasket. Repeat the test until no change in the flow rate is noted.

6.2 Connect the source of dry gas and inlet pressure measuring devices to the inlet fitting on the core holder. Connect the outlet pressure measuring devices and flow rate metering device to the outlet fitting on the core holder.

6.3 Initiate dry gas flow through the sample. Control the flow rate to minimize turbulence. Measure the flow rate through the sample intermittently for a period of 3 to 10 minutes until the flow becomes constant. Record the constant flow rate and pressures.

7. Calculations

7.1 The calculation of permeability is based on the empirical expression of Darcy known as Darcy's law.[2] The coefficient, k, of proportionality is the permeability.

7.2 The unit of the permeability coefficient, k, is the darcy. For convenience the subunit millidarcy may be used where 1 millidarcy equals 0.001 darcy. A porous medium has a permeability of one darcy when a single-phase fluid of one centipoise viscosity that completely fills the voids of the medium will flow through it under "conditions of viscous flow" at a rate of 1 cu cm per second per square centimetre of cross-sectional area under a pressure gradient of one atmosphere per centimetre. "Conditions of viscous flow" mean that the rate of flow is sufficiently low to be directly proportional to the pressure gradient. The permeability coefficient so defined has the units of length squared or area.

7.3 Vertical Flow Parallel to Core Axis - Darcy's law in differential form for linear flow is

$$q = \frac{k}{\mu}\frac{dp}{dL}$$

where

q - macroscopic velocity of flow, in centimetres per second

k - permeability coefficient, in darcys

μ - viscosity of the fluid that is flowing, in centipoises

dp/dL - pressure gradient in the direction of flow, in atmospheres per centimetres

Relating the velocity of flow to the volume rate of flow through the cross-sectional area and performing the indicated integration produces the following working equation for permeability coefficient in millidarcys:

$$k = (2000\ Q_o O_o\ L\ \mu)/\left(P_i{}^2 - P_o{}^2\right)$$

where

Q_o = rate of flow of outlet air, in cubic centimetres per second

P_o = outlet pressure, in atmospheres (absolute)

P_i = inlet pressure, in atmospheres (absolute)

L = length of sample, in centimetres

A = cross-sectional area perpendicular to direction of flow, in square centimetres

Several methods of simplification exist for calculating the permeability coefficient. Two such methods are given below:

7.3.1 Method 1 - For this method inlet pressure and pressure drop across the rate measuring orifice during the test are selected so that the outlet pressure is essentially one atmosphere. The working equation then reduces to

$$k = Q\ CL/A$$

where

$C = (2000\ \mu)/(P_i^2 - 1)$

μ = viscosity of air under the conditions used to calibrate the orifice

C then is a constant for each fixed inlet pressure since the fact that the same air flows through both the core and the orifice means that any change in air viscosity resulting from temperature changes or water vapor will have no effect on the relative pressure readings.

7.3.2 Method 2 - For this method calibration charts or tables of permeance (Note 5) versus outlet pressure for given inlet pressures and orifices are prepared based on the following equation

$$k_c = \frac{L_c}{A_c}\left(\frac{k_{or}}{L_{or}/A_{or}} \cdot \frac{\Delta P_{or} Q_c}{\Delta P_c Q_{or}}\right)$$

where

k_c, k_{or} = permeability of the core and the equivalent permeability of the orifice, respectively, in millidarcys

L_c, L_{or} = length of core and orifice, respectively, in centimetres

A_c, A_{or} = cross-sectional area of core and orifice, respectively, in square centimetres

ΔP_c, ΔP_{or} = pressure drop across the core and orifice, respectively, in atmospheres

Q_c, Q_{or} = flow rate through the core and orifice, respectively, in cubic centimetres per second

NOTE 5--Permeance or apparent permeability is the proper term for flow capacity. The term as used is analogous to the term conductance for the flow of current through an electrolyte solution. Permeance and permeability, therefore, are related in the same way as conductance and conductivity.

This working equation can be simplified to

$$k_c = \frac{L_c}{A_c} \frac{Q_c}{\Delta P_c} \cdot L$$

where

L - orifice constant

L may be determined directly by use of a known permeability plug. When tables or nomographs are used to calculate the permeability coefficient from measured outlet pressure for given inlet pressures and orifices, the working equation reduces to

$$k_c = \frac{L_c}{A_c} k_c^{\theta}$$

where

k_c^{θ} - permeance of the core

The permeance as calculated when multiplied by the L/A ratio gives the core permeability coefficient, k_c.

7.4 <u>Horizontal Flow Parallel to Core Diameter</u> - The same differential form of Darcy's law and working equation as used for vertical flow are applied. The working equation is modified by a factor for shape to the following form:

$$k = (Q_m \mu / L \Delta P)(1000)(G)$$

where

k = permeability, in millidarcys

Q_m = volume rate of air flow at mean core pressure, in cubic centimetres per second

μ - viscosity of flowing fluid, in centipoises

L - length of sample, in centimetres

ΔP - pressure drop across the core, in atmospheres

G - shape factor,[3] Fig. 6

7.5 Radial Flow - The radial permeability is calculated directly from the integrated form of Darcy's law for radial flow. The equation in terms of permeability coefficient is

$$k = (\mu Q_a)(\ln d_e/d_w)(P_o)/(\pi h)\left(P_i^2 - P_o^2\right) \cdot 1000$$

where

k - permeability, in millidarcys

μ - viscosity of flowing fluid at test temperature, in centipoises

Q_a - measured flow rate at test temperature and pressure - P_o, in cubic centimetres per second

ln - logarithm to the base e

d_e - outside diameter of sample, in centimetres

d_w - inside diameter of inner hole, in centimetres

P_o - outlet pressure, in atmospheres (absolute)

h - height of sample, in centimetres

P_i - inlet pressure, in atmospheres (absolute)

As with vertical flow, if the outlet pressure is atmospheric and the orifices are calibrated over the range of inlet pressures, the working equation simplifies to

$$k = \mu Q_m (\ln d_e/d_w)/2\pi h\Delta \cdot 1000$$

---

[3]Collins, R. E., "Determination of the Transverse Permeabilities of Large Core Samples from Petroleum Reservoirs," Journal of Applied Physics 23, 681-84 (1952).

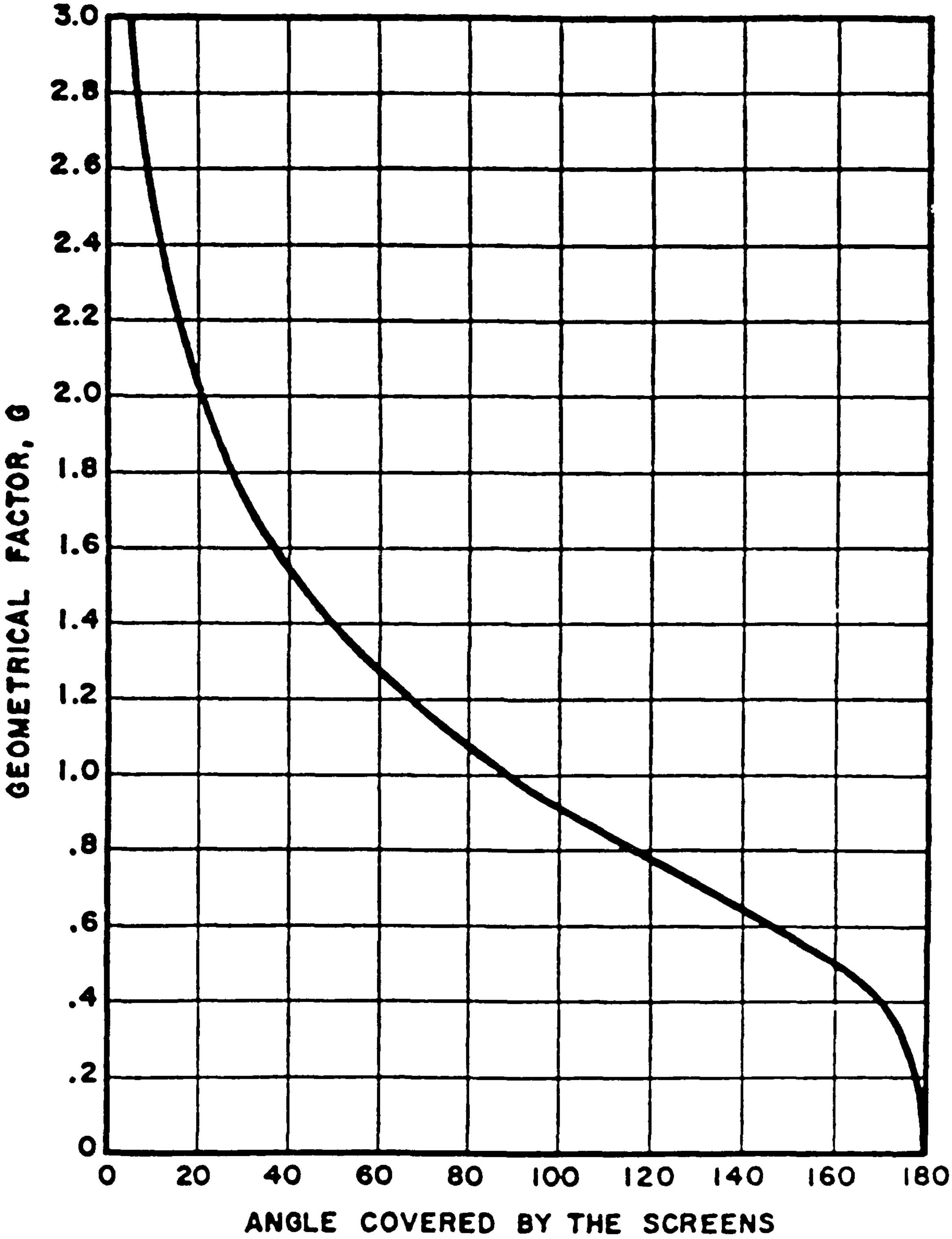


Fig. 6. Theoretical curve relating the geometric factor and the angular segment of the core covered by the screens (Collins, 1952).

where

Q_m = volume rate of air flow at mean core pressure, in cubic centimetres per second

ΔP - pressure drop across sample, in atmospheres (absolute)

8. Report

8.1 The report shall include the following:

8.1.1 A lithologic description of the rock tested.

8.1.2 Source of sample including depth and orientation, dates of sampling and testing, and storage environment.

8.1.3 Methods used for sample cleaning.

8.1.4 Methods used for sample support, capping, or other preparation such as sawing, grinding, or drilling.

8.1.5 Specimen length and diameter.

8.1.6 Type of core holder used.

8.1.7 Pressures used to seal core surfaces in core holder.

8.1.8 Flowing fluid used and direction flow.

8.1.9 Method used for calculating permeability coefficient.

8.1.10 Results of other physical tests, citing the method of determination for each.

8.1.11 Permeability corrections used.

8.2.12 Description of air source.

8.1.13 Calculation of permeability, with values defined for all variables.

Designation: C 535 - 89

RTH 115-93

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa. 19103
Reprinted from the Annual Book of ASTM Standards. Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Test Method for Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine[1]

This standard is issued under the fixed designation C 535; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This method has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

1. Scope

1.1 This test method covers testing sizes of coarse aggregate larger than ¾ in. (19 mm) for resistance to degradation using the Los Angeles testing machine.

NOTE 1—A procedure for testing coarse aggregate smaller than 1½ in. (37.5 mm) is covered in Method C 131.

1.2 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 131 Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine[2]

C 136 Method for Sieve Analysis of Fine and Coarse Aggregates[2]

C 670 Practice for Preparing Precision Statements for Test Methods for Construction Materials[2]

C 702 Practice for Reducing Field Samples of Aggregate to Testing Size[3]

D 75 Practice for Sampling Aggregates[2]

E 11 Specification for Wire-Cloth Sieves for Testing Purposes[4]

3. Summary of Method

3.1 The Los Angeles test is a measure of degradation of mineral aggregates of standard gradings resulting from a combination of actions including abrasion or attrition, impact, and grinding in a rotating steel drum containing a specified number of steel spheres, the number depending upon the grading of the test sample. As the drum rotates, a shelf plate picks up the sample and the steel spheres, carrying them around until they are dropped to the opposite side of the drum, creating an impact-crushing effect. The contents then roll within the drum with an abrading and grinding action until the shelf plate impacts and the cycle is repeated. After the prescribed number of revolutions, the contents are removed from the drum and the aggregate portion is sieved to measure the degradation as percent loss.

4. Significance and Use

4.1 The Los Angeles test has been widely used as an indicator of the relative quality or competence of various sources of aggregate having similar mineral compositions. The results do not automatically permit valid comparisons to be made between sources distinctly different in origin, composition, or structure. Specification limits based on this test should be assigned with extreme care in consideration of available aggregate types and their performance history in specific end uses.

5. Apparatus

5.1 *Los Angeles Machine* conforming to the requirements of Test Method C 131.

5.1.1 The machine shall be so driven and so counterbalanced as to maintain a substantially uniform peripheral speed (Note 2). If an angle is used as the shelf, the direction of rotation shall be such that the charge is caught on the outside surface of the angle.

NOTE 2—Backlash or slip in the driving mechanism is very likely to furnish test results that are not duplicated by other Los Angeles machines producing constant peripheral speed.

5.2 *Sieves,* conforming to Specification E 11.

5.3 *Balance*—A balance or scale accurate within 0.1 % of test load over the range required for this test

5.4 *Charge*—The charge shall consist of 12 steel spheres averaging approximately 1²⁷⁄₃₂ in. (46.8 mm) in diameter, each weighing between 390 and 445 g, and having a total weight of 5000 ± 25 g.

NOTE 3—Steel ball bearings 1¹³⁄₁₆ in. (46.038 mm) and 1⅞ in. (47.625 mm) in diameter, weighing approximately 400 and 440 g each, respectively, are readily available. Steel spheres 1²⁷⁄₃₂ in. (46.8 mm) in diameter weighing approximately 420 g may also be obtainable. The charge may consist of a mixture of these sizes.

6. Sampling

6.1 The field sample shall be obtained in accordance with Practice D 75 and reduced to test portion in accordance with Practice C 702.

[1] This test method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.05 on Methods of Testing and Specifications for Physical Characteristics of Concrete Aggregates.

Current edition approved April 28, 1989. Published June 1989. Originally published as C 535 - 64 T. Last previous edition C 535 - 81 (1987).

[2] *Annual Book of ASTM Standards,* Vols 04.02 and 04.03.

[3] *Annual Book of ASTM Standards,* Vol 04.02.

[4] *Annual Book of ASTM Standards,* Vol 14.02.

TABLE 1 Gradings of Test Samples

Sieve Size, mm (in.) (Square Openings)		Weights of Indicated Sizes, g		
		Grading[A]		
Passing	Retained on	1	2	3
75 (3)	63 (2½)	2 500 ± 50	...	...
63 (2½)	50 (2)	2 500 ± 50	...	...
50 (2)	37.5 (1½)	5 000 ± 50	5 000 ± 50	...
37.5 (1½)	25.0 (1)	...	5 000 ± 25	5 000 ± 25
25.0 (1)	19.0 (¾)	...	...	5 000 ± 25
	Total	10 000 ± 100	10 000 ± 75	10 000 ± 50

[A] Gradings 1, 2, and 3 correspond, respectively, in their size distribution to Gradings, E, F, and G in the superseded ASTM Method C 131 – 55, Test for Abrasion of Coarse Aggregate by Use of the Los Angeles Machine, which appears in the 1961 *Book of ASTM Standards*, Part 4.

7. Test Sample

7.1 The test sample shall be washed and oven-dried at 221 to 230°F (105 to 110°C) to substantially constant weight (Note 4), separated into individual size fractions, and recombined to the grading of Table 1 most nearly corresponding to the range of sizes in the aggregate as furnished for the work. The weight of the sample prior to test shall be recorded to the nearest 1 g.

NOTE 4—If the aggregate is essentially free of adherent coatings and dust, the requirement for washing before and after test may be waived. Elimination of washing after test will seldom reduce the measured loss by more than about 0.2 % of the original sample weight.

8. Procedure

8.1 Place the test sample and charge in the Los Angeles testing machine and rotate the machine at 30 to 33 r/min for 1000 revolutions. After the prescribed number of revolutions, discharge the material from the machine and make a preliminary separation of the sample on a sieve coarser than the 1.70-mm (No. 12). The finer portion shall then be sieved on a 1.70-mm sieve in a manner conforming to Method C 136. The material coarser than the 1.70-mm sieve shall be washed (Note 4), oven-dried at 221 to 230°F (105 to 110°C) to substantially constant weight, and weighed to the nearest 5 g (Note 5).

NOTE 5—Valuable information concerning the uniformity of the sample under test may be obtained by determining the loss after 200 revolutions. This loss should be determined without washing the material coarser than the 1.70-mm (No. 12) sieve. The ratio of the loss after 200 revolutions to the loss after 1000 revolutions should not greatly exceed 0.20 for material of uniform hardness. When this determination is made, take care to avoid losing any part of the sample; return the entire sample, including the dust of fracture, to the testing machine for the final 800 revolutions required to complete the test.

9. Calculation

9.1 Express the loss (difference between the original weight and the final weight of the test sample) as a percentage of the original weight of the test sample. Report this value as the percent loss.

NOTE 6—The percent loss determined by this method has no known consistent relationship to the percent loss for the same material when tested by Test Method C 131.

10. Precision

10.1 *Precision*—The precision of this test method has not been determined. It is expected to be comparable to that of Test Method C 131.

10.2 *Bias*—No statement is being made about the bias of this Test Method since there is no accepted reference material suitable for determining the bias of this procedure.

APPENDIX

(Nonmandatory Information)

X1. MAINTENANCE OF SHELF

X1.1 The shelf of the Los Angeles machine is subject to severe surface wear and impact. With use, the working surface of the shelf is peened by the balls and tends to develop a ridge of metal parallel to and about 1¼ in. (32 mm) from the junction of the shelf and the inner surface of the cylinder. If the shelf is made from a section of rolled angle, not only may this ridge develop but the shelf itself may be bent longitudinally or transversely from its proper position.

X1.2 The shelf should be inspected periodically to determine that it is not bent either lengthwise or from its normal radial position with respect to the cylinder. If either condition is found, the shelf should be repaired or replaced before further tests are made. The influence on the test result of the ridge developed by peening of the working face of the shelf is not known. However, for uniform test conditions, it is recommended that the ridge be ground off if its height exceeds 0.1 in. (2 mm).

PART I. LABORATORY TEST METHODS

B. Engineering Design Tests

Designation: D 3148 – 86[ε1] RTH 201-89

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Test Method for Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression[1]

This standard is issued under the fixed designation D 3148; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

ε1 NOTE—Figure 2 was corrected editorially in December 1987.

INTRODUCTION

The deformation and strength properties of rock cores measured in the laboratory usually do not accurately reflect large scale *in situ* properties because the latter are strongly influenced by joints, faults, inhomogeneities, weakness planes, and other factors. Therefore, laboratory values for intact specimens must be employed with proper judgment in engineering applications.

1. Scope

1.1 This test method covers the determination of elastic moduli of intact rock core specimens in uniaxial compression. Procedure A specifies the apparatus, instrumentation, and procedures for determining the axial stress - strain curve and Young's modulus, *E*, of cylindrical rock specimens loaded in uniaxial compression. Method B specifies the additional apparatus, instrumentation, and procedures which are necessary also to determine the lateral stress - strain curve and Poisson's ratio, ν.

NOTE 1—Some applications require the value of Young's modulus, but not Poisson's ratio. Thus, the decision to use Procedure A or Procedure A and B shall be determined by the engineer in charge of the project.

NOTE 2—This test method does not include the procedures necessary to obtain a stress - strain curve beyond the ultimate strength.

1.2 Test methods are not normally specified for rock moduli in tension because its low tensile strength does not permit sufficient data points to be obtained to be significant. However, the basic principles given here may be applied to tension testing.

1.3 The relation between the three elastic constants,

$$G = E/2(1 + \nu)$$

where:

G = modulus of rigidity,
E = Young's modulus, and
ν = Poisson's ratio.

and most elastic design equations are based on the assumption of isotropy. The engineering applicability of these equations is therefore decreased if the rock is anisotropic. When possible, it is desirable to conduct tests in the plane of foliation, bedding, etc., and at right angles to it to determine the degree of anisotropy. It is noted that equations developed for isotropic materials may give only approximate calculated results if the difference in elastic moduli in any two directions is greater than 10 % for a given stress level.

NOTE 3—Elastic moduli measured by sonic methods may often be employed as preliminary measures of anisotropy.

1.4 The values stated in inch-pound units are to be regarded as the standard.

1.5 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

D 2938 Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens[2]
D 4543 Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances[2]
E 4 Practices for Load Verification of Testing Machines[3]
E 122 Recommended Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[4]

3. Apparatus

3.1 *Loading Device* (Procedures A and B), for applying and measuring axial load to the specimen and of sufficient capacity to apply load at a rate in accordance with the requirements prescribed in 5.4. It shall be verified at suitable time intervals in accordance with the procedures given in Practices E 4, and comply with the requirements prescribed therein.

NOTE 4—The loading apparatus employed for this test is the same as that required for Test Method D 2938.

3.2 *Bearing Surfaces* (Procedures A and B)—The testing machine shall be equipped with two steel bearing blocks having a Rockwell hardness of not less than 58 HRC. One of the blocks shall be spherically seated and the other a plain

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Oct. 31, 1986. Published December 1986. Originally published as D 3148 – 72. Last previous edition D 3148 – 80.

[2] *Annual Book of ASTM Standards*, Vol 04.08.

[3] *Annual Book of ASTM Standards*, Vols 03.01, 04.02, 07.01, and 08.03.

[4] *Annual Book of ASTM Standards*, Vol 14.02.

rigid block. The bearing faces shall not depart from a plane by more than 0.0006 in. (15 µm) when the blocks are new and shall be maintained within a permissible variation of 0.001 in. (25 µm). The diameter of the spherically seated bearing face shall be at least as large as that of the test specimen but shall not exceed twice the diameter of the test specimen. The center of the sphere for the spherically seated block shall coincide approximately with the center of the bearing face of the specimen. The movable portion of the bearing block shall fit closely in the spherical seat, but the design shall be such that the bearing face can be rotated and tilted through small angles in any direction. Accomplish seating by rotating the movable bearing block while the specimen is held in contact with the fixed block.

NOTE 5—False platens with plane bearing faces conforming to the requirements of this method may be used. These shall consist of disks about 0.6 to 0.8 in. (15 to 20 mm) thick, oil-hardened preferably through the disks to more than 58 HRC and surface ground. With abrasive rocks these platens tend to roughen after a number of specimens have been tested, and hence need to be resurfaced from time to time.

3.3 *Axial Strain Determination* (Procedure A)—The axial deformations or strains may be determined from data obtained by electrical resistance strain gages, compressometers, optical devices, or other suitable means. The design of the measuring device shall be such that the average of at least two axial strain measurements can be determined for each increment of load. Measuring positions shall be equally spaced around the circumference of the specimen close to midheight. The gage length over which the axial strains are determined shall be at least 10 grain diameters in magnitude. The axial strains shall be determined with an accuracy of 2 % of the reading and a precision of 0.2 % of full-scale.

NOTE 6—Accuracy should be within 2 % of value of readings above 250 µm/m strain and within 5 µm/m strain for readings lower than 250 µm/m strain.

3.4 *Lateral Strain Determination* (Procedure B)—The lateral deformations or strains may be measured by any of the methods mentioned in 3.3. Either circumferential or diametric deformations (or strains) may be measured. At least two lateral deformation sensors shall be used. These shall be equally spaced around the circumference of the specimen close to midheight. The average deformation (or strain) from the two sensors shall be recorded at each load increment. The use of a single transducer that wraps all the way around the specimen to measure the total change in circumference is also permitted. The gage length and the accuracy and precision of the lateral strain measurement system shall be the same as those specified in 3.3 for the axial direction.

4. Test Specimens

4.1 Test specimens shall be prepared in accordance with Practice D 4543.

4.2 The moisture condition of the sample shall be noted and reported in 8.1.9.

NOTE 7—The moisture condition of the specimen at time of test can have a significant effect upon the strength of the rock and, hence, upon the shape of the deformation curves. Good practice generally dictates that laboratory tests be made upon specimens representative of field conditions. Thus, it follows that the field moisture condition of the specimen should be preserved until time of test. On the other hand, there may be reasons for testing specimens at other moisture contents from saturation to dry. In any case the moisture content of the test specimen should be tailored to the problem at hand. Excess moisture will affect the adhesion of strain gages, if used, and the accuracy of their performance.

5. Procedure (Procedures A and B)

5.1 Check the ability of the spherical seat to rotate freely in its socket before each test.

5.2 Wipe clean the bearing faces of the upper and lower bearing blocks and of the test specimen and place the test specimen with the strain-measuring device attached on the lower bearing block. Carefully align the axis of the specimen with the center of thrust of the spherically seated block. Make electrical connections or adjustments to the strain- or deformation-measuring device. As the load is gradually brought to bear on the specimen, adjust the movable portion of the spherically seated block so that uniform seating is obtained.

5.3 Many rock types fail in a violent manner when loaded to failure in compression. A protective shield should be placed around the test specimen to prevent injury from flying rock fragments.

5.4 Apply the load continuously and without shock to produce an approximately constant rate of load or deformation such that failure would occur within 5 to 15 min from initiation of loading, if carried to failure (Note 8). Record the load and the axial strain or deformation frequently at evenly spaced load intervals during Procedure A of Practice D 4543. Take at least ten readings over the load range to define the axial stress-strain curve. Also record the circumferential or diametric strains (or deformations) at the same increments of load for Procedure B of Practice D 4543. Continuous recording of data with strip chart or X-Y recorders is permitted as long as the precision and accuracy of the recording system meets the requirements in 3.3.

NOTE 8—Results of tests by several investigators have shown that strain rates within this range will provide strength values that are reasonably free from rapid loading effects and reproducible within acceptable tolerances.

6. Calculation (Procedure A)

6.1 The axial strain, ϵ_a, may be recorded directly from strain-indicating equipment, or may be calculated from deformation readings depending upon type of apparatus or instrumentation employed. Calculate the axial strain, ϵ_a, as follows:

$$\epsilon_a = \Delta l / l$$

where:

l = original undeformed axial gage length, in. (mm), and
Δl = change in measured axial length (negative for a decrease in length) in. (mm).

6.2 Calculate the compressive stress in the test specimen from the compressive load on the specimen and the initial computed cross-sectional area as follows:

$$\sigma = -P/A$$

where:

σ = stress, psi (MPa),
P = load, lbf (N), and
A = area, in.^{2} (mm^2).

NOTE 9—Tensile stresses and strains are used as being positive herein. A consistent application of a compression-positive sign convention may be employed if desired.

6.3 Plot the stress versus strain curve for the axial direction

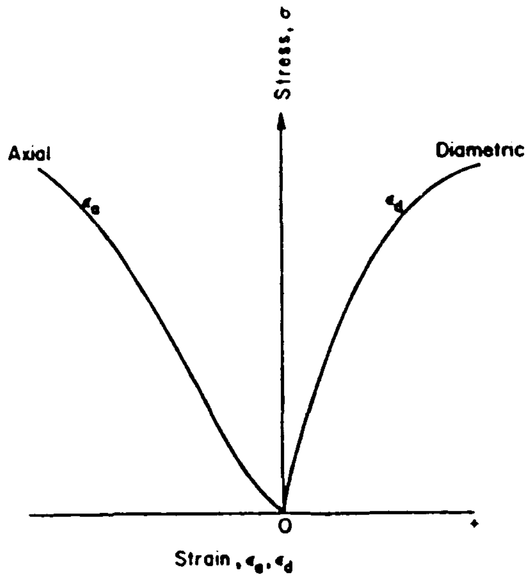


FIG. 1 Format for Graphical Presentation of Data

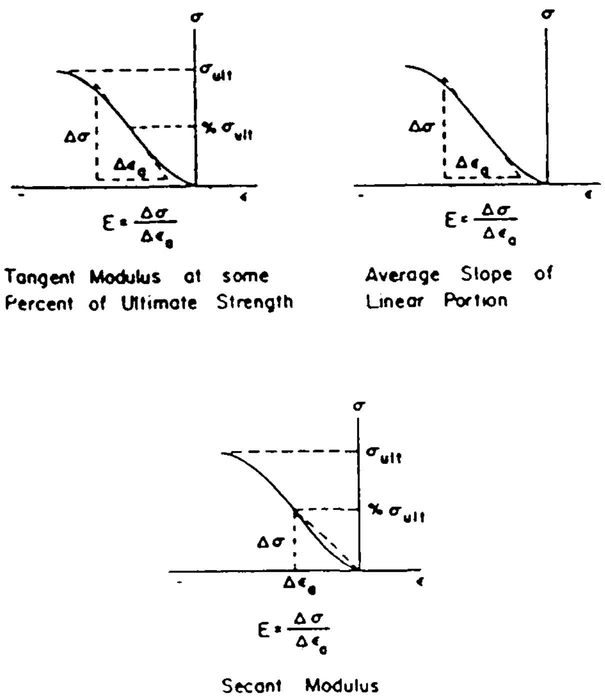


FIG. 2 Methods for Calculating Young's Modulus, *E*

(Fig. 1). The complete curve gives the best description of the deformation behavior of rocks having nonlinear stress-strain relationships at low and high stress levels.

6.4 The axial Young's modulus, E, may be calculated using any one of several methods employed in engineering practice. The most common methods, described in Fig. 2, are as follows:

6.4.1 Tangent modulus at a stress level which is some fixed percentage of the maximum strength.

6.4.2 Average slope of the more-or-less straight line portion of the stress-strain curve.

6.4.3 Secant modulus, usually from zero stress to some fixed percentage of maximum strength.

7. Calculation (Procedure B)

7.1 The circumferential or diametric strain, ϵ_d, may be recorded directly from strain-indicating equipment, or may be calculated from deformation readings depending upon the type of apparatus or instrumentation employed.

7.1.1 Calculate the diametric strain, ϵ_d, from the following equation:

$$\epsilon_d = \Delta d / d$$

where:

d = original undeformed diameter, in. (mm), and
Δd = change in diameter (positive for an increase in diameter), in. (mm).

NOTE 10—It should be noted that the circumferentially applied electrical resistance strain gages reflect diametric strain, the value necessary in computing Poisson's ratio, ν. Since $C = \pi d$, and $\Delta C = \pi \Delta d$, the circumferential strain, ϵ_c, is related to the diametric strain, ϵ_d, through the relation:

$$\epsilon_c = \Delta C/C = \pi \Delta d / \pi d = \Delta d / d$$

so that $\epsilon_c = \epsilon_d$ where C and d are the specimen circumference and diameter, respectively.

7.2 Plot the stresses calculated in 6.2 versus the corresponding diametric strains determined in 7.1.

7.3 The value of Poisson's ratio, ν, is greatly affected by nonlinearities at low stress levels in the axial and lateral stress-strain curves. It is suggested that Poisson's ratio be calculated from the equation:

$$\nu = -\text{slope of axial curve/slope of lateral curve}$$
$$= -E/\text{slope of lateral curve}$$

where the slope of the lateral curve is determined in the same manner as was done in 6.4 for Young's modulus, E.

NOTE 11—The denominator in the equation in 7.3 will have a negative value if the sign convention is applied properly. The negative sign in the equation thereby assures a positive value for ν.

8. Report

8.1 *Procedure A*—The report shall include the following:

8.1.1 Source of sample including project name and location. Often the location is specified in terms of the drill hole number and depth of specimen from collar of hole.

8.1.2 Date test is performed.

8.1.3 Specimen diameter and height, conformance with dimensional requirements.

8.1.4 Rate of loading or deformation rate.

8.1.5 Values of applied load, stress and axial strain as tabulated results or as recorded on a chart.

8.1.6 Plot of stress versus axial strain as shown in Fig. 1, if data are tabulated in 8.1.5. If data are recorded directly on a chart, the load and deformation axes may be scaled to give stress and strain without replotting the curve.

8.1.7 Young's modulus, E, method of determination as given in Fig. 2, and at what stress level or levels determined.

8.1.8 Physical description of sample including rock type such as sandstone, limestone, granite, etc.; location and orientation of apparent weakness planes, bedding planes, and schistosity; and large inclusions or inhomogeneities, if any.

8.1.9 General indication of moisture condition of sample

at time of test such as as-received, saturated, laboratory air-dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation.

8.2 *Procedure B*—Procedure B of the test method must always accompany Procedure A in order to determine Poisson's ratio. The report for Procedure B, which is in addition to that for Procedure A, shall include the following:

8.2.1 Values of applied load, stress, and diametric strain as tabulated results or as recorded on a chart.

8.2.2 Plot of stress versus diametric strain as shown in Fig. 1 if data is tabulated in 8.2.1. If data is recorded directly on a chart, the load and deformation axes may be scaled to give stress and strain without replotting the curve.

8.2.3 Poisson's ratio, ν, method of determination in 7.3, and at what stress level or levels determined.

9. Precision and Bias

9.1 The variability of rock and resultant inability to determine a true reference value prevent development of a meaningful statement of bias. Data are being evaluated to determine the precision of this test method. In addition, the subcommittee is seeking pertinent data from users of the method.

Designation: D 2664 - 86 RTH 202-89

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa. 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Test Method for TRIAXIAL COMPRESSIVE STRENGTH OF UNDRAINED ROCK CORE SPECIMENS WITHOUT PORE PRESSURE MEASUREMENTS[1]

This standard is issued under the fixed designation D 2664; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This test method covers the determination of the strength of cylindrical rock core specimens in an undrained state under triaxial compression loading. The test provides data useful in determining the strength and elastic properties of rock, namely: shear strengths at various lateral pressures, angle of internal friction, (angle of shearing resistance), cohesion intercept, and Young's modulus. It should be observed that this method makes no provision for pore pressure measurements. Thus the strength values determined are in terms of total stress, that is, not corrected for pore pressures.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

D 4543 Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances[2]

E 4 Practices for Load Verification of Testing Machines[3]

E 122 Recommended Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[4]

3. Significance and Use

3.1 Rock is known to behave as a function of the confining pressure. The triaxial compression test is commonly used to simulate the stress conditions under which most underground rock masses exist.

4. Apparatus

4.1 *Loading Device*—A suitable device for applying and measuring axial load to the specimen. It shall be of sufficient capacity to apply load at a rate conforming to the requirements specified in 7.2. It shall be verified at suitable time intervals in accordance with the procedures given in Practices E 4 and comply with the requirements prescribed in the method.

4.2 *Pressure-Maintaining Device*—A hydraulic pump, pressure intensifier, or other system of sufficient capacity to maintain constant the desired lateral pressure, σ_3.

NOTE 1—A pressure intensifier as described by Leonard Obert in U.S. Bureau of Mines Report of Investigations No. 6332, "An Inexpensive Triaxial Apparatus for Testing Mine Rock," has been found to fulfill the above requirements.

4.3 *Triaxial Compression Chamber*[5]—An apparatus in which the test specimen may be en-

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Oct. 31, 1986. Published December 1986. Originally published as D 2664 - 67. Last previous edition D 2664 - 80.

[2] *Annual Book of ASTM Standards*, Vol 04.08.

[3] *Annual Book of ASTM Standards*, Vols 03.01, 04.02, 07.01 and 08.03.

[4] *Annual Book of ASTM Standards*, Vol 14.02.

[5] Assembly and detail drawings of an apparatus that meets these requirements and which is designed to accommodate 2⅛-in. (53.975-mm) diameter specimens and operate at a lateral fluid pressure of 10 000 psi (689 MPa) are available from Headquarters. Request Adjunct No. 12-426640-00.

closed in an impermeable flexible membrane; placed between two hundred platens, one of which shall be spherically seated; subjected to a constant lateral fluid pressure; and then loaded axially to failure. The platens shall be made of tool steel hardened to a minimum of Rockwell 58 HRC, the bearing faces of which shall not depart from plane surfaces by more than 0.0005 in. (0.0127 mm) when the platens are new and which shall be maintained within a permissible variation of 0.001 in. (0.025 mm). In addition to the platens and membrane, the apparatus shall consist of a high-pressure cylinder with overflow valve, a base, suitable entry ports for filling the cylinder with hydraulic fluid and applying the lateral pressure, and hoses, gages, and valves as needed.

4.4 *Deformation and Strain-Measuring Devices*—High-grade dial micrometers or other measuring devices graduated to read in 0.0001-in. (0.0025-mm) units, and accurate within 0.0001 in. (0.0025 mm) in any 0.0010-in. (0.025-mm) range, and within 0.0002 in. (0.005 mm) in any 0.0100-in. (0.25-mm) range shall be provided for measuring axial deformation due to loading. These may consist of micrometer screws, dial micrometers, or linear variable differential transformers securely attached to the high pressure cylinder.

4.4.1 Electrical resistance strain gages applied directly to the rock specimen in the axial direction may also be used. In addition, the use of circumferentially applied strain gages will permit the observation of data necessary in the calculation of Poisson's ratio. In this case two axial (vertical) gages should be mounted on opposite sides of the specimen at mid-height and two circumferential (horizontal) gages similarly located around the circumference, but in the direction perpendicular to the axial gages.

4.5 *Flexible Membrane*—A flexible membrane of suitable material to exclude the confining fluid from the specimen, and that shall not significantly extrude into abrupt surface pores. It should be sufficiently long to extend well onto the platens and when slightly stretched be of the same diameter as the rock specimen.

NOTE 2—Neoprene rubber tubing of 1/16-in. (1.588-mm) wall thickness and of 40 to 60 Durometer hardness, Shore Type A or various sizes of bicycle inner tubing, have been found generally suitable for this purpose.

5. Sampling

5.1 The specimen shall be selected from the cores to represent a true average of the type of rock under consideration. This can be achieved by visual observations of mineral constituents, grain sizes and shape, partings and defects such as pores and fissures.

6. Test Specimens

6.1 *Preparation*—The test specimens shall be prepared in accordance with Practice D 4543.

6.2 Moisture condition of the specimen at the time of test can have a significant effect upon the indicated strength of the rock. Good practice generally dictates that laboratory tests be made upon specimens representative of field conditions. Thus it follows that the field moisture condition of the specimen should be preserved until the time of test. On the other hand, there may be reasons for testing specimens at other moisture contents, including zero. In any case the moisture content of the test specimen should be tailored to the problem at hand and reported in accordance with 9.1.6.

7. Procedure

7.1 Place the lower platen on the base. Wipe clean the bearing faces of the upper and lower platens and of the test specimen, and place the test specimen on the lower platen. Place the upper platen on the specimen and align properly. Fit the flexible membrane over the specimen and platen and install rubber or neoprene O-rings to seal the specimen from the confining fluid. Place the cylinder over the specimen, ensuring proper seal with the base, and connect the hydraulic pressure lines. Position the deformation measuring device and fill the chamber with hydraulic fluid. Apply a slight axial load, approximately 25 lbf (110 N), to the triaxial compression chamber by means of the loading device in order to properly seat the bearing parts of the apparatus. Take an initial reading on the deformation device. Slowly raise the lateral fluid pressure to the predetermined test level and at the same time apply sufficient axial load to prevent the deformation measuring device from deviating from the initial reading. When the predetermined test level of fluid pressure is reached note and record the axial load registered by the loading device. Consider this load to be the zero or starting load for the test.

7.2 Apply the axial load continuously and without shock until the load becomes constant, or reduces, or a predetermined amount of strain is achieved. Apply the load in such a manner as to produce a strain rate as constant as feasible throughout the test. Do not permit the strain rate at any given time to deviate by more than 10 % from that selected. The strain rate selected should be that which will produce failure of a similar test specimen in unconfined compression, in a test time of between 2 and 15 min. The selected strain rate for a given rock type shall be adhered to for all tests in a given series of investigation (Note 3). Maintain constant the predetermined confining pressure throughout the test and observe and record readings of deformation as required.

NOTE 3—Results of tests by other investigators have shown that strain rates within this range will provide strength values that are reasonably free from rapid loading effects and reproducible within acceptable tolerances.

7.3 To make sure that no testing fluid has penetrated into the specimen, the specimen membrane shall be carefully checked for fissures or punctures at the completion of each triaxial test. If in question, weigh the specimen before and after the test.

8. Calculations

8.1 Make the following calculations and graphical plots:

8.1.1 Construct a stress difference versus axial strain curve (Note 5). Stress difference is defined as the maximum principal axial stress, σ_1, minus the lateral pressure, σ_3. Indicate the value of the lateral pressure, σ_3, on the curve.

NOTE 4—If the specimen diameter is not the same as the piston diameter through the chamber, a correction must be applied to the measured load to account for differences in area between the specimen and the loading piston where it passes through the seals into the chamber.

NOTE 5—If the total deformation is recorded during the test, suitable calibration for apparatus deformation must be made. This may be accomplished by inserting into the apparatus a steel cylinder having known elastic properties and observing differences in deformation between the assembly and steel cylinder throughout the loading range. The apparatus deformation is then subtracted from the total deformation at each increment of load in order to arrive at specimen deformation from which the axial strain of the specimen is computed.

8.1.2 Construct the Mohr stress circles on an arithmetic plot with shear stresses as ordinates and normal stresses as abscissas. Make at least three triaxial compression tests, each at a different confining pressure, on the same material to define the envelope to the Mohr stress circles.

NOTE 6—Because of the heterogeneous nature of rock and the scatter in results often encountered, it is considered good practice to make at least three tests of essentially identical specimens at each confining pressure or single tests at nine different confining pressures covering the range investigated. Individual stress circles shall be plotted and considered in drawing the envelope.

8.1.3 Draw a "best-fit", smooth curve (the Mohr envelope) approximately tangent to the Mohr circles as in Fig. 1. The figure shall also include a brief note indicating whether a pronounced failure plane was or was not developed during the test and the inclination of this plane with reference to the plane of major principal stress.

NOTE 7—If the envelope is a straight line, the angle the line makes with the horizontal shall be reported as the angle of interval friction, ϕ (or the slope of the line as tan ϕ depending upon preference) and the intercept of this line at the vertical axis reported as the cohesion intercept, C. If the envelope is not a straight line, values of ϕ (or tan ϕ) should be determined by constructing a tangent to the Mohr circle for each confining stress at the point of contact with the envelope and the corresponding cohesion intercept noted.

9. Report

9.1 The report shall include as much of the following as possible:

9.1.1 Sources of the specimen including project name and location, and if known, storage environment. The location is frequently specified in terms of the borehole number and depth of specimen from collar of hole.

9.1.2 Physical description of the specimen including rock type; location and orientation of apparent weakness planes, bedding planes, and schistosity; large inclusions or inhomogeneities, if any.

9.1.3 Dates of sampling and testing.

9.1.4 Specimen diameter and length, conformance with dimensional requirements.

9.1.5 Rate of loading or deformation or strain rate.

9.1.6 General indication of moisture condition of the specimen at time of test such as: as-received, saturated, laboratory air-dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible

and reported as either water content or degree of saturation.

9.1.7 Type and location of failure. A sketch of the fractured specimen is recommended.

NOTE 8—If it is a ductile failure and $\sigma_1 - \sigma_3$ is still increasing when the test is terminated, the maximum strain at which $\sigma_1 - \sigma_3$ is obtained shall be clearly stated.

10. Precision and Bias

10.1 The variability of rock and resultant inability to determine a true reference value prevent development of a meaningful statement of bias. Data are being evaluated to determine the precision of this test method. In addition, the subcommittee is seeking pertinent data from users of the method.

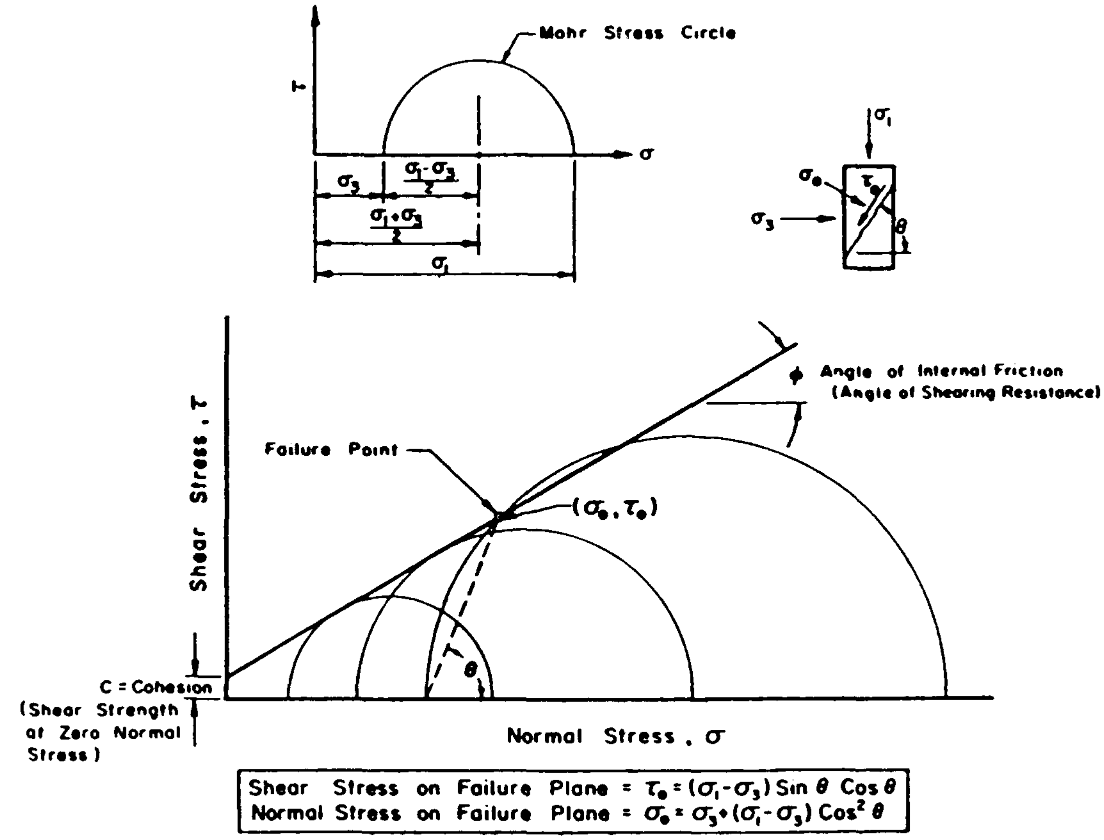


FIG. 1 Typical Mohr Stress Circles

RTH 203-80

METHOD OF TEST FOR DIRECT SHEAR STRENGTH OF ROCK CORE SPECIMENS

1. Scope

1.1 This method describes apparatus and procedures for determining the shear strength of a rock material in direct shear. The test can be made on rock core specimens from 2 to 6 in. (5 to 15 cm) in diameter. The test can be made on intact specimens to determine intact shear strength, on intact specimens with recognizable thin weak planes to determine the shearing resistance along these planes, on presawn shear surfaces to determine lower bound residual shear strengths, and on rock core to concrete bond specimens to determine the shearing resistance between the bond. The principle of the rock core direct shearing is illustrated schematically in Fig. 1.

1.2 A minimum of three test specimens of any rock type are subjected to different but constant normal stresses during the shearing process. For each type of intact rock, cohesion and an angle of internal friction are determined. For each type of rock with sawn failure surfaces, a lower bound residual angle of internal friction is determined.

1.3 The test is not suited to the development of exact stress-strain relationships within the test specimen because of the nonuniform distribution of shearing stresses and displacements. Care should be taken so that the testing conditions represent those being investigated. The results of these tests are used where field design requirements dictate unconsolidated, undrained parameters.

2. Apparatus

2.1 Test Specimen Saw - For cores of 3 to 6 in. (7.5 to 15 cm) in diameter, use a rock saw with 20-in.- (50-cm-) diam safety abrasive blade fitted for dry and for wet cutting. Alternatively for wet cutting, a diamond blade may be used. For cores 2 to 2-1/2 in. (5 to 6.25 cm) in diameter, a rock saw with 12-in.- (60-cm-) diam blade should be used.

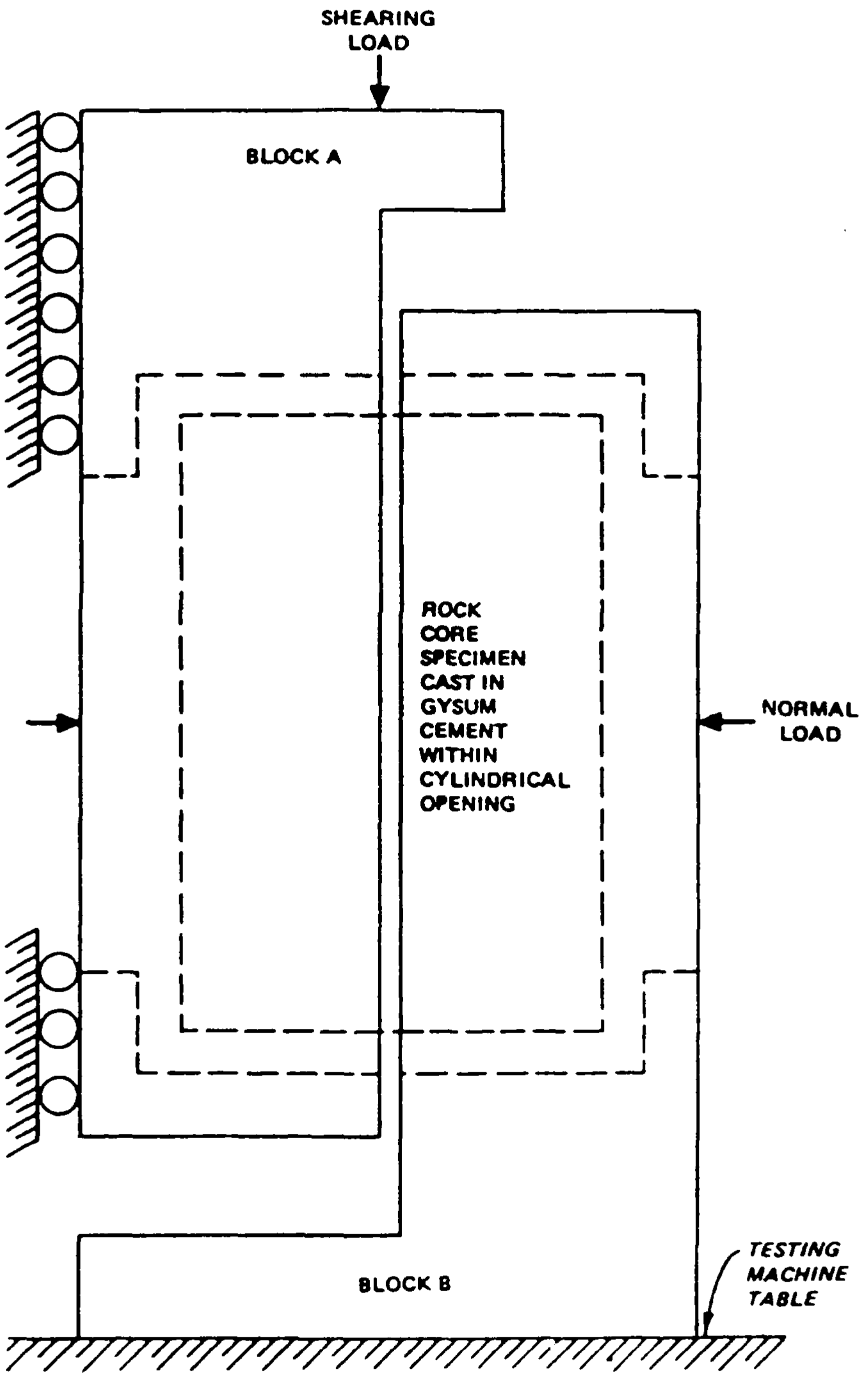


Fig 1. Schematic showing direct shear of rock core.

2.2 Shear Device - The shear device shall consist of a pair of shear boxes constructed so as to provide a means of applying a normal stress to the face of the specimen while applying a force to shear the specimen along a predetermined plane parallel to the vertical axis of the specimen. The device shall securely hold the specimen in such a way that torque cannot be applied to the specimen. The shear boxes that hold the specimen shall be sufficiently rigid to prevent their distortion during shearing. The various parts of the shear device shall be made of material not subject to corrosion by substances within the rock or moisture within the rock.

2.2.1 Shear boxes suitable for testing specimens from 3 to 6 in. (7.5 to 15 cm) in diameter should each have a recess 6-5/8 in. (16.6 cm) in diameter and 2-1/4 in. (5.67 cm) deep. Smaller shear boxes for 2- to 2-1/2-in.- (5- to 6.25-cm-) diam specimens should each have a recess 2-7/8 in. (7.36 cm) in diameter and 2 in. (5 cm) in depth. The shear boxes should be designed for a shear travel greater than 10 percent of the specimen shear plane length.

2.2.2 In both cases the two shear boxes assembled with the specimen shall be placed within a framework constructed so as to hold the boxes in proper position during testing. The framework shall include a pair of hardened stainless steel plates machined to accommodate roller bearings or ball bearings for minimizing friction of the moving shear box as indicated in Fig. 1. The roller plate device should ensure that resistance of the equipment to shear displacement is less than 1 percent of the maximum shear force applied in the test.

2.2.3 The shear device framework shall include capability of providing a submerging tank for tests in which maintaining specimen saturation is important to duplicate field conditions.

2.3 Loading Devices

2.3.1 Normal Force - The normal force device shall be capable of applying the specified force quickly without exceeding it and capable of maintaining it with an accuracy of ±2 percent for the duration of the

test. The device shall have a travel greater than the amount of dilation or compression to be expected.

2.3.2 Shear Force - The device for applying the shear force shall distribute the load uniformly along one-half face of the specimen with the resultant applied shear force acting in the plane of shearing. The required capabilities will depend upon whether a controlled-displacement test or a controlled-stress test is used. Controlled-displacement equipment shall be capable of shearing the specimen at a uniform rate of displacement with less than ±15 percent deviation and shall permit adjustment of the rate of displacement over a relatively wide range. Controlled-stress equipment shall be capable of applying the shear force in increments to the specimen in the same manner and to the same degree of accuracy as that described in 2.3.1.

2.4 Displacement Indicators - Equipment for measuring shear and normal displacements may consist of mechanical devices, such as dial gages or electric transducers. Displacement indicators shall have a sensitivity of at least 0.001 in. (0.025 mm). The shear displacement measuring system shall have a travel greater than 10 percent of the specimen shear plane length. Normal displacement systems shall have the capability of measuring both dilation and compression of the specimen. Resetting of gages during the test should if possible be avoided. If electric transducers or an automatic recording system is used, a recent calibration shall be included in the report.

2.5 Casting Compound - High-strength gypsum cement (such as hydrostone) or a capping compound (such as leadite) should be used to hold the test specimen in the recesses of the test device.

2.6 Spacer Plate - The spacer plate separating the shear boxes for development of the shear zone shall be 1/16 in. (1.6 mm) thick and constructed of a noncorrosive material.

3. Test Specimen

3.1 Intact Specimens

3.1.1 Test specimens shall be prepared by sawing rock cores into 3- to 4-in. (7.5- to 10-cm) lengths (Note 1). The diameter of each specimen shall be measured to the nearest 0.01 in. (0.025 mm) at several different positions along the length of the specimen axis. The average diameter shall be used to compute the cross-sectional area of the specimen. The volume of the specimen shall be determined by the volumetric or displacement method presented in EM 1110-2-1906, "Laboratory Soils Testing." The initial weight of the specimen shall be determined to the nearest 0.1 g for subsequent use in determining initial moisture content and density.

NOTE 1—Soft rock such as clay shales may be as short as 2-1/2 in. (6.25 cm) if material is scarce. Although helpful in the setup, ends of the test specimens need not be smooth, flat, nor square with the axis of the core. Generally, harder rocks are best cut in the wet; softer rocks are best cut in the dry, depending on fissility and reaction to pressure of cutting water.

3.1.2 A block of the shear box shall be set on a flat surface with the shear surface up. The inside of the recess shall be lightly coated with lubricant. A grout of the gypsum cement (hydrostone) and water shall be placed in the recess to approximately the one-third or midpoint. After approximately three minutes of setting, the specimen shall be set or pushed into the grout until the approximate midpoint (desired shear plane) of the specimen is opposite the top recess (Note 2). Excess grout shall be screeded off at the shear plane (Note 3).

NOTE 2—To prepare intact test specimens for testing along recognizable thin weak planes, orient the specimen so that the plane of weakness is parallel with the 1/16-in. (1.6-mm) shear gap provided by the spacer plate.

NOTE 3—Gypsum cement grout has only a few minutes pot life; hence a fresh mix will have to be prepared for each block. An alternative to gypsum cement grout for holding the test specimen in the recesses is capping compound, such as leadite. The procedure for preparing specimens with a capping compound is essentially the same as for gypsum cement. Capping compound has a shorter pot life after pouring than gypsum cement and must be heated to proper temperature and handled quickly and with great care. An overnight curing period is generally required. Capping compound is stronger in compression and shear than gypsum cement and is preferred for hard rock testing. Because capping compound must be placed hot, it should not be used to secure specimens subject to structural damage with loss of natural moisture or for tests in which it is desirable to maintain natural moisture.

3.1.3 A 1/16-in.- (1.6-mm-) thick spacer plate having a hole equal to the diameter of the specimen and split on a diameter from the front to the back of the block shall be coated with lubricant and placed on the block around the specimen. The spacer separates the two blocks of the shear box to prevent friction between the blocks during shearing. The recess of the remaining shear box block shall be lightly coated with lubricant and the block placed over the now protruding half of the specimen. The two blocks shall be aligned and temporarily clamped together with C clamps. The recess between the top block and specimen shall then be filled with the gypsum cement grout using appropriate tools to rod the grout thoroughly around the specimen. For soft rock such as clay shale, a 2-hour curing is usually sufficient before loading. For hard rocks, the grout must be allowed to cure overnight.

3.1.4 At the end of the curing period, the two halves of the spacer shall be pulled out and the C clamps removed. The specimens secured in the shear boxes are then ready for further assembly and shear testing.

3.2 <u>Presawn Shear Surfaces</u> - Test specimens shall be prepared the same as presented in paragraphs 3.1.1 to 3.1.4, except that the specimen shall be sawn in half near the center length before grouting the

specimen in the shear box blocks. The presawn shear surface shall be smooth and oriented in the shear box so as to be centered within the 1/16-in. (1.6-mm) shear gap provided by the spacer plate.

3.3 Concrete to Rock Core Bond

3.3.1 Test specimens shall be prepared by sawing rock cores into 1.5- to 2-in. (3.75- to 5-cm) length. The sawn specimen shall be tightly encased in the bottom of a 3- to 4-in.- (7.5- to 10-cm-) high mold (Note 4) with the smooth sawn surface (shear plane) facing upward and perpendicular to the axis of the mold. The remaining portion of the mold shall be filled with concrete, which is then consolidated and cured according to the procedures presented in CRD-C 10-73 (Note 5). The concrete mix design shall be compatible in consistency and strength with the anticipated field design mix and have a maximum aggregate no larger than 1/6 of the specimen diameter.

NOTE 4—Molds shall be made of steel, cast iron, or other nonabsorbent material, nonreactive with concrete containing portland or other hydraulic cements. Mold diameters shall conform to the dimensions of the rock core test specimen. Molds shall hold their dimensions and shape and be watertight under conditions adverse to use.

NOTE 5—"Handbook for Concrete and Cement," U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, published in quarterly supplements.

3.3.2 Procedures for measuring specimen weight, diameter, and volume are the same as presented in paragraph 3.1.1. Procedures for securing the test specimen in the shear box are the same as presented in paragraphs 3.1.2 to 3.1.4.

4. Procedure

4.1 Following the removal of the spacer plates and C clamps, transfer final assembly operations to the test shear and normal load area. Final assembly of the testing apparatus, to include orientation of the resultant normal and shear loads, will depend on the equipment utilized

in the testing. In general, the resultant of the normal load shall react through the axial center of the specimen, and the shear load shall react through the radial center of the specimen so as to pass through the shear plane. Position or activate, or both, the displacement indicators for measuring shear deformation and changes in specimen thickness.

4.2 Apply the selected normal force (normal stress " cross-section area) to the specimen as rapidly as practical (Note 6). Record and allow any initial elastic compression of the specimen to reach equilibrium. For those tests where applicable, as soon as possible after applying the initial normal force, fill the water reservoir to at least submerge the shear plane.

NOTE 6—The normal force used for each of the three or more specimens will depend upon the input information required for field analysis and/or design.

4.3 Shear the specimen.

4.3.1 After any elastic compression has reached equilibrium, apply the shearing force and shear the specimen. In a controlled-displacement test, the rate of displacement shall be less than 0.004 in./min (0.1 mm/min) until peak strength is reached. Approximately 10 sets of readings should be taken before reaching peak strength. If it is desired to determine the ultimate strength, the normal load shall be relieved and the specimen recentered. The normal load is then reapplied and the specimen sheared again. The rate of shear displacement to determine the ultimate strength shall be no greater than 0.01 in./min (0.25 mm/min). Readings should be taken at increments of from 0.02- to 0.2-in. (0.5- to 5-mm) shear displacement as required to adequately define the force-displacement curves.

4.3.2 In a controlled-stress test the rate of stress application should not exceed 5 psi/min (34.47 kPa/min) for soft rock (such as clay shale) and up to 100 psi (689.4 kPa) for the very hardest rock.

Concurrent time, shear load, and deformation readings shall be taken at convenient intervals (a minimum of 10 readings before reaching peak strength). After reaching peak strength, the ultimate strength may be determined as presented in paragraph 4.3.1.

5. Calculations

5.1 Calculate the following:

5.1.1 Initial cross-sectional area.

5.1.2 Initial water content.

5.1.3 Initial wet and dry unit weights.

5.1.4 Shear stress data.

5.1.5 Initial and final degrees of saturation, if desirable.

6. Report

6.1 The report shall include the following:

6.1.1 Description of type of shear device used in the test.

6.1.2 Identification and description of the sample.

6.1.3 Description of the shear surface.

6.1.4 Initial water content.

6.1.5 Initial wet and dry unit weights.

6.1.6 Initial and final degrees of saturation, if desirable.

6.1.7 All basic test data including normal stress, shear displacement, corresponding shear resistance values, and specimen thickness changes.

6.1.8 For each test specimen, a plot of shear and specimen thickness change versus shear displacement and a plot of composite maximum and ultimate shear stress versus normal stress.

6.1.9 Departures from the procedure outlined, such as special loading sequences or special wetting requirements.

STANDARD METHOD OF TEST FOR MULTISTAGE TRIAXIAL STRENGTH OF UNDRAINED ROCK CORE SPECIMENS WITHOUT PORE PRESSURE MEASUREMENTS

1. Scope

1.1 This method covers the determination of the strength of cylindrical rock specimens in an undrained state under multistage triaxial loading. The test provides data useful in delineating the strength of joints, seams, bedding planes, etc. This method makes no provision for pore pressure measurements. Thus, the strength values determined are in forms of total stress, i.e. not corrected for pore pressures.

2. Apparatus

2.1 The apparatus is identical with that used in RTH 202, "Triaxial Compressive Strength of Undrained Rock Core Specimens Without Pore Pressure Measurements."

3. Test Specimens

3.1 In the case of intact specimens that develop a well-defined shear failure plane, it is possible to continue testing beyond the first failure; that is, the confining pressure can be raised to a higher level and another peak stress recorded. This may be done immediately following the completion of a conventional triaxial test as conducted according to RTH 202.

3.2 Multistage testing can also be used with cores that are intact initially. The key factor in making these studies is the use of specimens with the failure plane preestablished to cause failure along an inherent weakness, such as seams, open joints, bedding planes, faults, schistosity bands, or laminations. These planes of weakness, when tested, should be oriented at 45 to 65 deg (0.79 to 1.14 radians) from the horizontal, which will normally produce a failure in the preoriented zone. When including these specific geologic features in an NX size core, the specimens can be drilled from 6-in. (15-cm) or larger cores by suitable orientation in the drilling apparatus. Care must be taken when coring these specimens to prevent breakage. However, if the core is

broken along a weakness plane, it may still be tested as an open joint. Broken cores can be taped together with plastic tape, only sufficiently to maintain the matching contact between the broken parts. The test specimens are then prepared in the manner described in Section 3 of RTH 202.

4. Procedure

4.1 The test procedure described in Section 4 of RTH 202 shall be used for the first stage of test. Subsequent stages should be achieved in like manner by applying progressively higher levels of lateral fluid pressure. This may be done as many times as desired, provided the total strain does not cause excessive misalignment of the steel platens and an eccentric loading. This procedure is referred to as multistage testing. Fig. 1 illustrates this loading sequence.

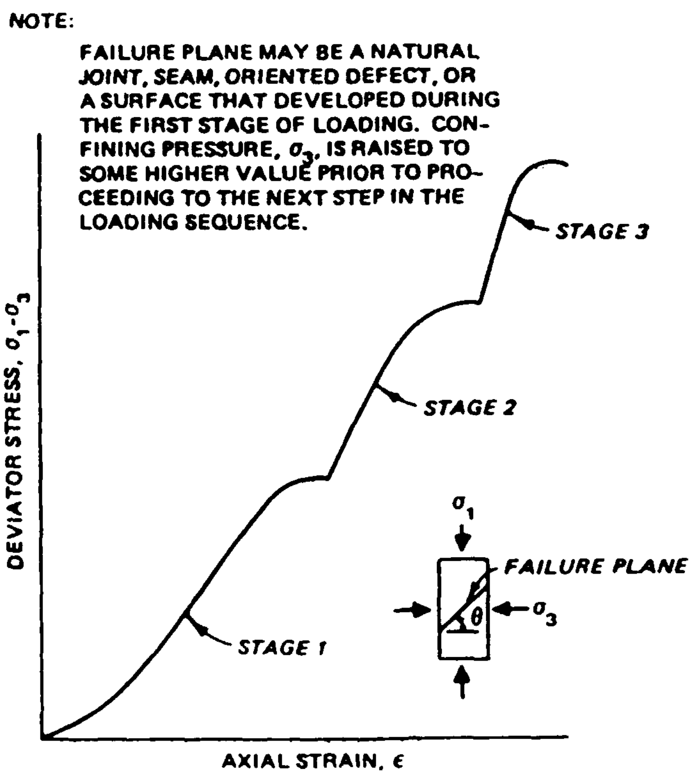


Fig. 1. Stress-strain curve for multistage triaxial test.

5. Calculations

5.1 The shear strength on the joint can be determined graphically by constructing a Mohr circle as shown in Fig. 2. Proof of this construction may be found in most soil mechanics texts.[1] For a multistage test on a given specimen, several Mohr's circles can be drawn and the same angle used to plot the stresses on the failure plane. A strength envelope for this condition, Fig. 3, which is the average of the results determined for each Mohr's circle, is considered to be the joint friction angle.

5.2 There are variations of this plotting technique that may also be employed. The multistage test described above produces a joint friction angle from a single specimen. For a strength envelope derived from tests of several intact specimens, plotting failure plane stresses would yield a higher limiting strength criterion than that of open joints. To report this type of data properly, all orientation data must be carefully and fully stated to ensure proper interpretation of results.

5.3 Various orientations of seams may also be tested to determine that which is most critical. Direct tension and unconfined compression tests may be included to completely define the strength envelope as shown in Fig. 4.

6. Report

6.1 In addition to the plots discussed in Section 5, the report should include the following:

6.1.1 Lithologic description of the rock, including the type of joint, seam, etc., tested.

6.1.2 Source of sample including depth and orientation, dates of sampling and testing, and storage environment.

6.1.3 Specimen diameter and height.

---

[1] Taylor, D., "Fundamentals of Soil Mechanics," John Wiley and Sons, Inc., p 317, or Spangler, M., "Soil Engineering," International Textbook Co., p 277.

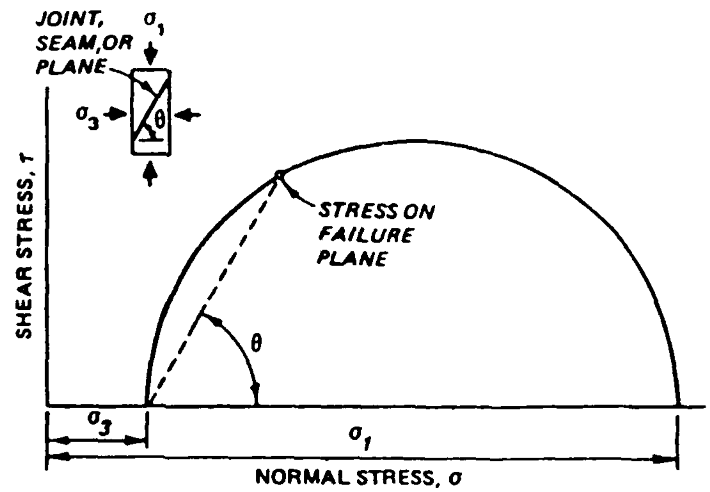


Fig. 2. Mohr circle showing method of construction for locating stresses in failure plane.

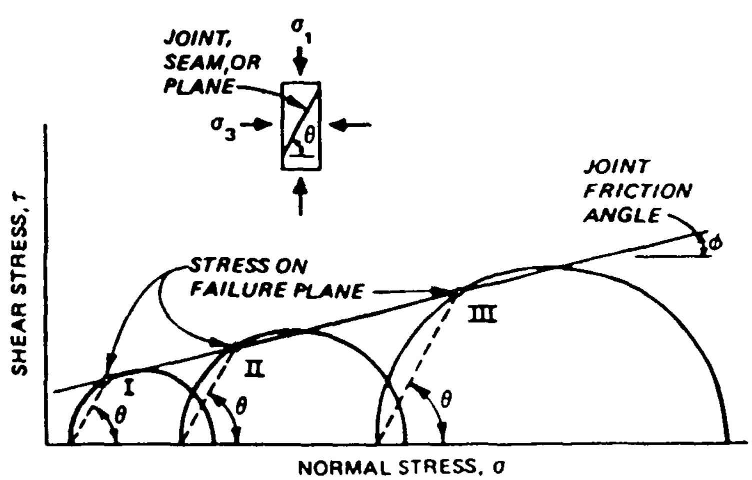


Fig. 3. Mohr diagram for locating stresses on failure plane in a multistage test.

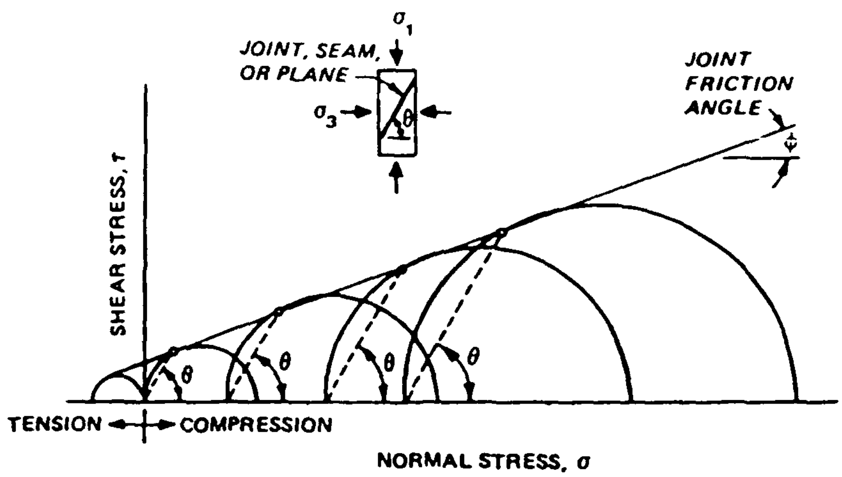


Fig. 4. Mohr diagram for locating stresses on failure plane, including direct tension and unconfined compression test data.

6.1.4 Moisture content and degree of saturation at time of test.

6.1.5 Other physical data, such as specific gravity, absorption, porosity, and permeability, citing the method of determination for each.

Designation: D 4405 – 84 (Reapproved 1989)[ε2]

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St. Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards. Copyright ASTM
If not listed in the current combined index, will appear in the next edition.

Standard Test Method for Creep of Cylindrical Soft Rock Core Specimens in Uniaxial Compressions[1]

This standard is issued under the fixed designation D 4405; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

[ε1] NOTE—Section 9 was changed editorially in July 1989.
[ε2] NOTE—Section 10 was added editorially in December 1991.

1. Scope

1.1 This test method covers the determination of the creep behavior of intact cylindrical soft rock core specimens subjected to uniaxial compression up to a temperature of 572°F (300°C). Creep is the time-dependent strain or deformation under sustained axial stress. Soft rocks include such materials as salt and potash, which often exhibit very large strain at failure.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

E 4 Practices for Load Verification of Testing Machines[2]

E 122 Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[3]

3. Significance and Use

3.1 There are many underground structures that are created for permanent use. These structures are subjected to an approximately constant load. Creep tests provide quantitative parameters for stability analysis of these structures.

4. Apparatus

4.1 *Loading Device*—The loading device shall be capable of applying and maintaining the required load on the specimen, regardless of any changes in the dimensions of the specimen. The loading device consists of a reaction frame and a load generating component which may be a hydraulic actuator, spring, dead weight, or controlled screw-driven loading mechanism. Means shall be provided for measuring the load to within 2 % of the applied load. The applied load shall be maintained within ±2 % of the required test load for the duration of the testing. The stability of the loading device and the reaction frame as a function of time shall be evaluated prior to testing, and periodically during testing. To maintain the necessary stability of the loading device, the room temperature shall be maintained to within ±2°F (±1°C).

NOTE 1—By definition, creep is time-dependent deformation under constant axial stress. The loading device is specified to maintain constant axial load. The engineering stress (load divided by original cross-sectional area) will therefore be constant, while the true stress (load divided by actual cross-sectional area at the time) will decrease somewhat during the test as the cross-sectional area increases. Standard practice in creep testing is to maintain constant load (constant engineering stress) because of the experimental difficulties in controlling the load to maintain a constant true stress.

4.2 *Bearing Surfaces*—The testing machine shall be equipped with two steel bearing blocks having a hardness of not less than 58 HRC. One of the blocks shall be spherically seated and the other a plain rigid block. The bearing faces shall not depart from a plane by more than 0.0005 in. (0.013 mm) when the blocks are new and shall be maintained within a permissible variation of 0.001 in. (0.025 mm). The diameter of the spherically seated bearing face shall be at least as large as that of the test specimen but shall not exceed twice the diameter of the test specimen. The center of the sphere in the spherically seated block shall coincide with that of the bearing face of the specimen. The spherically seated block shall be properly lubricated to assure free movement. The movable portion of the bearing block shall be held closely in the spherical seat, but the design shall be such that the bearing face can be rotated and tilted through small angles in any direction.

4.2.1 *Room Temperature Bearing Blocks*—In a room temperature test, the specimen may be placed directly between the upper and lower machine platens, or false platens may be used between the specimen and the machine platens. False platens (steel spacer disks) shall possess the same material and flatness specifications as the machine platens. The diameter of the false platens shall be at least as great as the specimen, but not exceeding the specimen diameter by more than 0.050 in. (1.3 mm).

4.2.2 *Elevated Temperature Bearing Blocks*—The steel false platens shall be thermally insulated from the machine platens by insulating spacers in the load column or by cooling coils. These insulating spacers or cooling coils are necessary if the temperature enclosure directly surrounds the specimen, but not if the entire room is maintained at the elevated temperature.

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Sept. 28, 1984. Published November 1984.

[2] *Annual Book of ASTM Standards*, Vols 04.02, 07.01, and 08.03.

[3] *Annual Book of ASTM Standards*, Vol 14.02.

4.3 *Elevated Temperature Enclosure*—The elevated temperature tests can be conducted either in a temperature controlled room or in a small chamber directly surrounding the test specimen. Temperature shall be measured at three locations, with one sensor near the top, one at midheight, and one near the bottom of the specimen. The average specimen temperature based on the midheight sensor shall be maintained to within ±2 % of the required test temperature, measured in degrees Celcius. The maximum temperature difference between the midheight sensor and either end sensor shall not exceed 5 % of the test temperature in degrees Celcius.

4.4 *Temperature Measuring Devices*—The type of instrument chosen to monitor temperature depends primarily on the test apparatus and the maximum test temperature. Special limits-of-error thermocouples or platinum resistance thermometers (RTDs) are recommended. The temperature transducer shall be accurate to at least ±1.0°F (±0.5°C), with a resolution of 0.20°F (0.1°C).

4.5 *Strain/Deformation Measuring Devices:*

4.5.1 *Room Temperature:*

4.5.1.1 *Axial Strain Determination*—For these materials, the axial deformation or strain may not normally be determined by electrical strain gages, and some other forms of strain/deformation measuring devices must be utilized. Such devices include compressometers, optical devices, and the like. The design of the measuring device shall be such that the average of at least two axial strain measurements can be recorded either continuously or for each increment of time. Measuring positions shall be equally spaced around the circumference of the specimen close to midheight. The gage length over which the axial strains are determined shall be at least ten grain diameters in magnitude. The axial strains shall be determined with an accuracy of 2 % of the reading and a precision 0.2 % of full-scale over the duration of the test. Furthermore, for readings below 250 μin./in. (μm/m) strain, accuracy shall be within 5 μin./in. (μm/m) strain.

Note 2—The use of strain gage adhesives requiring cure temperatures above 150°F (65°C) is not permitted unless it is known that microfractures do not develop at the cure temperature.

4.5.1.2 *Lateral Strain Determination*—The lateral deformations or strains may be measured by any of the methods mentioned in 4.5.1.1. Either circumferential or diametrical deformations (or strains) may be measured. At least two lateral deformation sensors shall be used. These shall be equally spaced around the circumference of the specimen close to midheight. The average deformation (or strain) from the two sensors shall be recorded either continuously or for each increment of time. The use of a single transducer that wraps completely around the specimen to measure the total change in circumference may also be used. The gage length and the accuracy and precision of the lateral strain measurement system shall be the same as those specified in 4.5.1.1.

4.5.2 *Elevated Temperature*—Deformation measurements are normally made externally from the heating enclosure using suitable extensometers.

Note 3—An example of such a measurement arrangement is shown in Fig. 1.

5. Test Specimens

5.1 Test specimens shall be right circular cylinders within the tolerances specified herein.

5.1.1 The specimen shall have a length-to-diameter ratio (L/D) of 2.0 to 2.5 and a diameter of not less than NX wireline core size, approximately 1⅞ in. (48 mm).

Note 4—It is desirable that the diameter of rock compression specimens be at least ten times the diameter of the largest mineral grain.

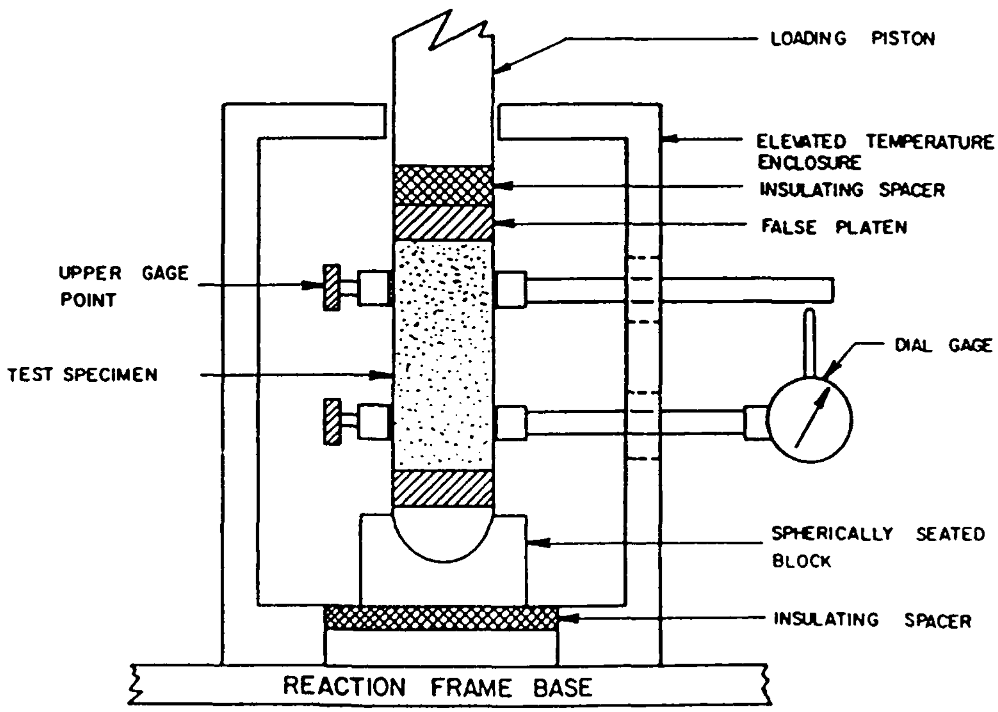


FIG. 1 Example of Technique for Monitoring Axial Specimen Deformation External to Elevated Temperature Enclosure

In soft rock, such as salt and potash, the average grain size is normally larger than it is for most hard rock. Research has shown that creep behavior of salt is highly dependent upon the specimen dimensions. Therefore, in order to obtain meaningful results, sufficiently large specimens should be used.

5.1.2 The sides of the specimen shall be generally smooth and free of abrupt irregularities with all the elements straight to within 0.020 in. (0.50 mm) over the full length of the specimen. The deviation from straightness of the elements shall be determined by either Method A or Method B.

5.1.2.1 *Method A*—Roll the cylindrical specimen on a smooth flat surface and measure the height of the maximum gap between the specimen and the flat surface with a feeler gage. If the maximum gap exceeds 0.020 in. (0.50 mm), the specimen does not meet the required tolerance for straightness of the elements. The flat test surface on which the specimen is rolled shall not depart from a plane by more than 0.0005 in. (13 µm).

5.1.2.2 *Method B:*

5.1.2.2.1 Place the cylindrical surface of the specimen on a V-block that is laid flat on a surface. The smoothness of the surface shall not depart from a plane by more than 0.0005 in. (13 µm).

5.1.2.2.2 Place a dial indicator in contact with the top of the specimen, as shown in Fig. 2, and observe the dial reading as the specimen is moved from one end of the V-block to the other along a straight line.

5.1.2.2.3 Record the maximum and minimum readings on the dial gage and calculate the difference, Δ_0. Repeat the same operations by rotating the specimen for every 90°, and obtain the differences, Δ_{90}, Δ_{180}, and Δ_{270}. The same maximum value of these four differences shall be less than 0.020 in. (0.50 mm).

5.1.3 The ends of the specimen shall be cut parallel to each other and at right angles to the longitudinal axis. They shall be surface ground or lapped flat to 0.001 in. (25 µm). Water shall not be used in any of the sample preparation procedures. The flatness tolerance shall be checked by a setup similar to that for the cylindrical surface (Fig. 3) except that the dial gage is mounted near the end of the V-block. Move the mounting pad horizontally so that the dial gage runs across the end surface of the specimen along a diametral direction. Take care to make sure that one end of the mounting pad maintains intimate contact with the end surface of the V-block during moving. Record the dial gage readings every ⅛ in. (3 mm) across the diameter.

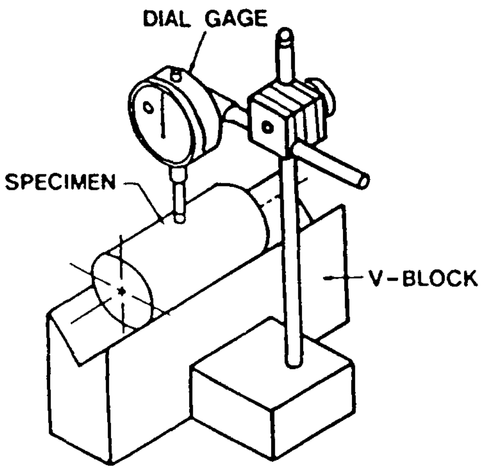


FIG. 2 Assembly for Determining the Straightness of the Cylindrical Surface

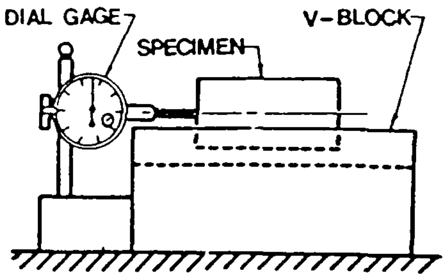


FIG. 3 Assembly for Determining the Flatness and Perpendicularity of End Surfaces to the Specimen Axis

5.1.3.1 Plot the readings and draw a smooth curve through the points to represent the surface profile along the specified diametral plane. The flatness tolerance is met when the smooth curve so determined does not depart from a visual best-fit straight line by more than 0.001 in. (25 µm).

5.1.3.2 Rotate the specimen 90° about its longitudinal axis and repeat the same operations and tolerance check for the new diametral plane. Turn the specimen end for end and repeat the same measurement procedures and tolerance checks for the other end surface.

5.1.4 The ends of the specimen shall not depart from perpendicularity to the axis of the specimen by more than 0.25°, which is a slope of approximately one part in 200. The tolerance shall be checked using the measurements taken in 5.1.3. Calculate the difference between the maximum and minimum readings on the dial gage along diameter 1. This difference is denoted as Δ_1. Calculate the corresponding difference for diameter 2, which is 90° from diameter 1. This difference for diameter 2 is Δ_2. Calculate the corresponding differences for the other end of the specimen, Δ'_1 and Δ'_2. The perpendicularity tolerance will be considered to have been met when:

$$\frac{\Delta_i}{D} \text{ and } \frac{\Delta'_i}{D} \leq 0.005$$

where:
i = 1 or 2, and
D = diameter.

5.1.5 The use of capping materials or end surface treatments other than the grinding and lapping specified herein is not permitted.

5.2 The diameter of the test specimen shall be determined to the nearest 0.01 in. (0.25 mm) by averaging two diameters measured at right angles to each other at about midheight of the specimen. This average diameter shall be used for calculating the cross-sectional area. The height of the test specimen shall be determined to the nearest 0.01 in. (0.25 mm) at the centers of the end faces.

5.3 The specimen should be prepared in a room with a relative humidity of less than 40 %. Prior to and following preparation, specimens shall be stored in an airtight container. During the creep test, the specimen shall be coated or jacketed with an impervious material.

6. Procedure

6.1 *Loading:*

6.1.1 *Room Temperature:*

6.1.1.1 Check the ability of the spherical seat to rotate freely in its socket before each test.

6.1.1.2 Wipe clean the bearing faces of the upper and lower bearing blocks, false platens, if used, and the ends of the test specimen. Place the test specimen with the attached strain-measuring device on the lower bearing block. Carefully align the axis of the specimen with the center of thrust of the spherically seated block. Make electrical connections or adjustments to the strain or deformation-measuring device.

6.1.1.3 Slowly load the specimen to a preload value of 25 psi (170 kPa). During this process, adjust the movable portion of the spherically seated block so that uniform seating is obtained. Zero the strain/deformation-measuring devices. Then rapidly raise the load, without shock to the required test load, within 20 s. Thereafter, the test load shall be held constant during the duration of the test.

6.1.2 *Elevated Temperature:*

6.1.2.1 Wipe clean the bearing faces of the upper and lower false platens and the ends of the test specimen. Jacket the specimen as described in 5.3. The jacket shall be such that after installation, it encloses the specimen tightly and extends over the false platens.

6.1.2.2 Place the specimen on the lower machine platen. Carefully align the axis of the specimen with the center of thrust of the lower platen of the test machine. Slowly load the specimen to a preload value of 25 psi (170 kPa). During this process, adjust the movable portion of the spherically seated block so that uniform seating is obtained. If the test temperature is below 350°F (175°C) and electrical resistance strain gages are used, make the necessary connections. Zero the strain/deformation-measuring devices.

6.1.2.3 Place the heating enclosure in position. Raise the temperature at a rate not exceeding 3.6°F (2°C)/min until it reaches the required temperature (Note 5). The test specimen shall be considered to have attained thermal equilibrium when the deformation transducer output is stable for at least three readings taken at equal intervals over a period of no less than 30 min. Stability is defined as a constant reading showing only the effects of normal instrument and heater unit fluctuations.

NOTE 5—It has been observed that for some rock types microfracturing will occur for heating rates above 1.8°F (1°C)/min. The operator is cautioned to select a heating rate such that microfracturing due to thermal gradients does not occur.

6.1.2.4 Rapidly raise the load without shock to the required test load within 20 s. Thereafter, the test load shall be held constant during the duration of the test.

6.2 Record the strain/deformation immediately after the required test load has been applied. Thereafter, record the strain/deformation at suitable time intervals. During the transient creep period (Fig. 4), strain/deformation readings shall be taken every few minutes to few hours until the strain/deformation rate becomes constant. Readings shall be taken at least twice daily during the steady-state phase of creep (Fig. 4) until the test is terminated. If the test extends into the tertiary creep period (Fig. 4), frequency of readings shall be increased appropriately.

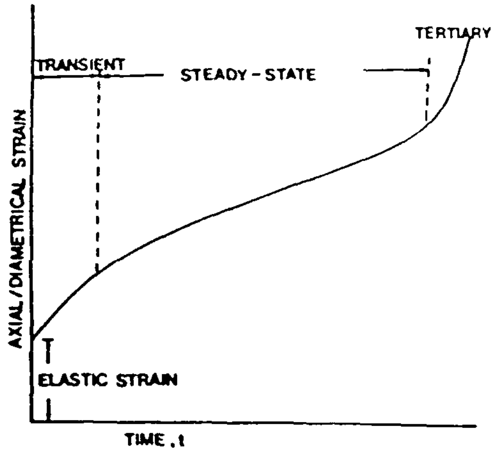


FIG. 4 Typical Creep Curve

6.3 Record the load and specimen temperature either continuously or each time the strain/deformation is read.

7. Calculation

7.1 The axial strain ϵ_a, circumferential strain, ϵ_c and diametrical strain, ϵ_d, may be obtained directly from strain-indicating equipment, or may be calculated from deformation readings, depending on the type of apparatus or instrumentation employed.

7.1.1 Calculate the axial strain, ϵ_a, as follows:

$$\epsilon_a = \Delta L/L$$

where:

L = original undeformed axial gage length, in. (mm), and
ΔL = change in measured axial length (negative for decrease in length) in. (mm).

7.1.2 Calculate the diametrical strain, ϵ_d, as follows:

$$\epsilon_d = \Delta D/D$$

where:

D = original undeformed diameter, in. (mm), and
ΔD = change in diameter (positive for an increase in diameter) in. (mm).

NOTE 6—Tensile stresses and strains are used as being positive herein. A consistent application of a compression-positive sign convention may be employed if desired.

7.2 Calculate the compressive stress in the test specimen from the compressive load on the specimen and the initial computed cross-sectional area as follows:

$$\sigma = -P/A$$

where:

σ = stress, psi (MPa),
P = load, lbf (N), and
A = area, in.^{2} (mm^2).

7.3 Plot the strain versus time curve as shown in Fig. 4. Frequently it may be necessary to use a logarithmic scale for the "time" if a long test period is involved.

8. Report

8.1 The report shall include the following:

8.1.1 Source of sample, including project name and location (often the location is specified in terms of the drill hole number and depth of specimen from the collar of the hole),

8.1.2 Lithologic description of the rock and load direction with respect to lithology,

8.1.3 Moisture condition of specimen before test,

8.1.4 Specimen diameter and height (conformance with dimensional requirements, see Note 4),

8.1.5 Stress level at which test was performed,

8.1.6 Temperature at which test was performed,

8.1.7 Tabulation of strain and time data,

8.1.8 Plot of the strain versus time curve (Fig. 4),

8.1.9 A description of physical appearance of specimen after test, and

8.1.10 If the actual equipment or procedure has varied from the requirements contained in this test method, each variation and the reasons for it shall be discussed.

9. Precision and Bias

9.1 *Precision*—Due to the nature of rock materials tested by this test method, it is, at this time, either not feasible or too costly to produce multiple specimens which have uniform physical properties. Therefore, since specimens which would yield the same test results cannot be tested, Subcommittee D18.12 cannot determine the variation between tests since any variation observed is just as likely to be due to specimen variation as to operator or laboratory testing variation. Subcommittee D18.12 welcomes proposals to resolve this problem that would allow for development of a valid precision statement.

9.2 *Bias*—There is no accepted reference value for this test method; therefore, bias cannot be determined.

10. Keywords

10.1 compression testing; creep; deformation; loading tests; rock

METHOD OF TEST FOR THERMAL DIFFUSIVITY OF ROCK

1. Scope

1.1 This method of test outlines a procedure for determining the thermal diffusivity of rock. The thermal diffusivity is equal to the thermal conductivity divided by the heat capacity per unit volume and may be used as an index of the facility with which the material will undergo temperature change.

2. Apparatus

2.1 The apparatus shall consist of:

2.1.1 Bath - A heating bath in which specimens can be raised to uniform high temperature (212°F (100°C)).

2.1.2 Diffusion Chamber - A diffusion chamber containing running cold water.

2.1.3 Temperature-Indicating or Recording Instrument - Consisting of iron-constantan thermocouples, Type K potentiometer, ice bath, standard cell, galvanometer, switch, and storage battery; or thermocouples and suitable recording potentiometer.

2.1.4 Timer - Timer capable of indicating minutes and seconds.

3. Procedure

3.1 Preparation of Specimen - The test specimen shall be a 6- by 12-in. (152- by 305-mm) core (for other shapes and sizes, see Section 5). A thermocouple shall be inserted in an axially drilled hole 3/8 in. (9.5 mm) in diameter and subsequently grouted.

3.2 Heating - Each specimen shall be heated to the same temperature by continuous immersion in boiling water until the temperature of the center is 212°F (100°C). The specimen shall then be transferred to a bath of running cold water and suspended in the bath so that the entire surface of the specimen is in contact with the water. The temperature of the cold water shall be determined by means of another thermocouple.

3.3 Cooling - The cooling history of the specimen shall be obtained from readings of the temperature of the interior of the specimen at 1-minute intervals from the time the temperature difference between the center and the water is 120°F (67°C) until the temperature difference between the center and water is 8°F (4°C). The data shall be recorded. Two such cooling histories shall be obtained for each test specimen, and the calculated diffusivities shall check within 40.002 ft^2/h ($0.0052 \cdot 10^{-5}$ m^2/s).

4. Calculations

4.1 The temperature difference in degrees F shall be plotted against the time in minutes on a semilogarithmic scale. The best possible straight line shall then be drawn through the points so obtained. A typical graph is shown in Fig. 1. The time elapsed between the temperature differences of 80°F (44°C) and 20°F (11°C) shall be read from the graph, and this value inserted in the equation below, from which the thermal diffusivity, α , shall be calculated as follows:

$$\alpha = 0.812278/(t_1 - t_2)$$

where α = thermal diffusivity, ft^2/hr (Note 1)

$(t_1 - t_2)$ = elapsed time between temperature differences of 80°F (44°C) and 20°F (11°C), minutes

0.812278 = numerical factor applicable to 6- by 12-in. (152- by 305-mm) cylinder

NOTE 1--The SI equivalent of ft^2/hr is m^2/s; $ft^2/hr \cdot 2.580640$ E - 05 = m^2/s.

5. Specimens of Other Sizes and Shapes

5.1 The method given above is directly applicable to a 6- by 12-in. (152- by 305-mm) cylinder. Specimens of other sizes and shapes may be treated in the manner described below.

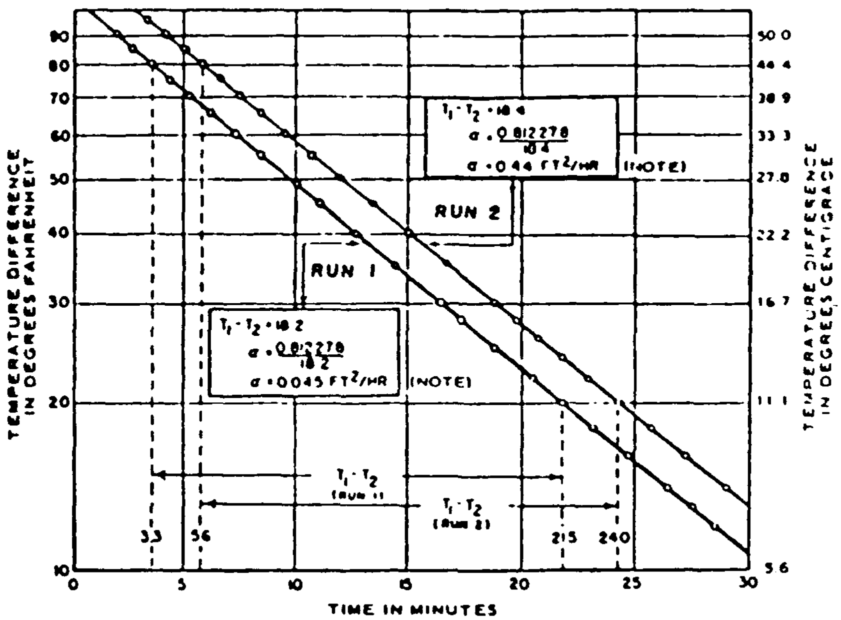


Fig. 1. Calculation of thermal diffusivity.

5.2 The thermal diffusivity of a specimen of regular shape is, to a first approximation,

$$\alpha = M/(t_2 - t_1)$$

where α = thermal diffusivity, ft^2/hr

M = a factor depending on the size and shape of the specimen

t_1, t_2 = times at which the center of the specimen reaches any specified temperature differences, minutes

5.3 For a prism,

$$M = \frac{60 \ln(T_1/T_2)}{\pi^2\left(\frac{1}{a^2} + \frac{1}{b^2} + \frac{1}{c^2}\right)}$$

where $\ln(T_1/T_2)$ = natural logarithm of the temperature difference ratio

T_1, T_2 = temperature differences at times t_1 and t_2, deg F

a, b, c = dimensions of prism, feet

5.4 For a cylinder,

$$M = \frac{60 \ln(T_1/T_2)}{\left(\frac{5.783}{r^2} + \frac{\pi^2}{l^2}\right)}$$

where $\ln(T_1/T_2)$ = natural logarithm, as above

r = radius of cylinder, feet

l = length of cylinder, feet

5.5 For specimens whose minimum dimension is more than 3 in. (76 mm), this approximate calculation will yield the required accuracy. For smaller specimens or when more precise determinations are desired, reference may be made to "Heat Conduction," by L. R. and A. C. Ingersoll, and O. J. Zebel, McGraw-Hill Book Company, Inc., 1948, pp. 183-185 and appended tables. Charts which may be used are also found in Williamson and Adams, Phys. Rev. XIV, p. 99 (1919) and "Heat Transmission," W. H. McAdams, McGraw-Hill Book Company, Inc., 1942, pp. 27-44.

PART II. IN SITU TEST METHODS

A. Rock Mass Monitoring

USE OF INCLINOMETERS FOR ROCK MASS MONITORING

1. Scope

1.1 This method describes the use of inclinometers for rock mass monitoring, lists some available instruments, outlines operating techniques and maintenance requirements, and presents data reduction methods.

2. Apparatus

2.1 An inclinometer is a device for measuring the deviation from the vertical of a flexible casing installed in a borehole. Deviations can be converted to displacements by trigonometric functions. Successive measurements enable the determination of the depth, magnitude, and rate of lateral movement. Fig. 1 shows a typical inclinometer installation.[6.1]

2.2 Many types of inclinometers are commercially available (Tables 1 and 2); however, the most commonly used is the probe type. This type consists of a control box and a probe which is lowered into the casing on a cable. In some probes a cantilevered pendulum with resistance strain gages, vibrating wire, or inductive transducers is used to measure cantilever deflection.[6.2] Other probes use the Wheatstone bridge principle (Slope Inclinometer Model 200 B), the servo accelerometer principle (Slope Indicator Digitilt), or a differential transformer (Dames and Moore, EDR). The probe generally requires a special flexible casing as indicated in Table 2. The electrical output from the probe is measured at the control box and converted to visual display, punched tape, or graphic form.

3. Procedure

3.1 Installation - Inclinometer casing should be installed in a near-vertical hole that intersects the zone of suspected movement.* The hole should extend beyond the zone of expected movement and at least

* Measurements in nonvertical holes can be made with some inclinometers; however, before planning such holes manufacturers' specifications should be checked to determine the limitations of the particular instrument being used.

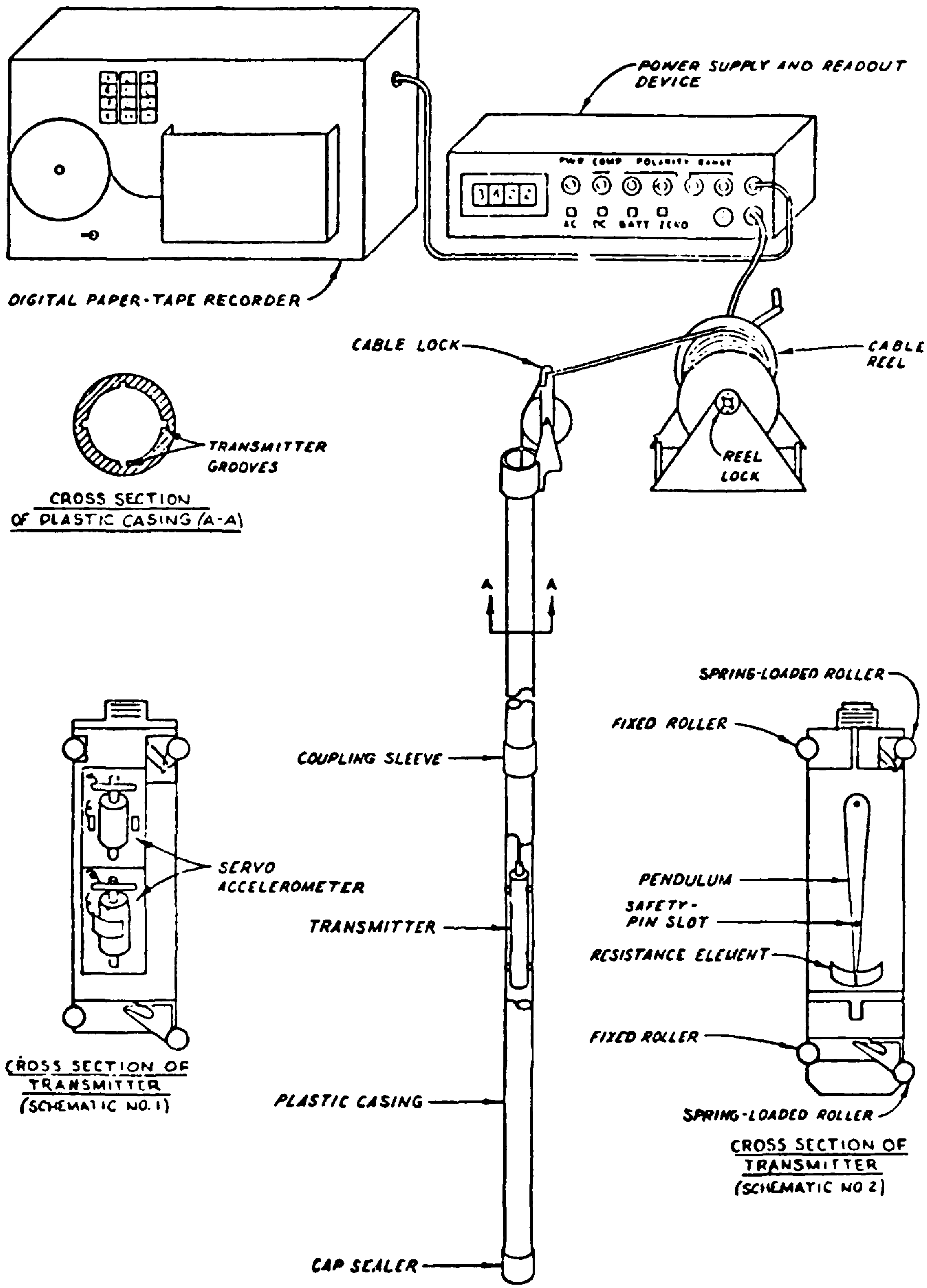


Fig. 1. Typical inclinometer installation (Leach, 1976).

Table 1. Fixed-Position Inclinometers (Franklin and Denton, 1973).

Type	Trade name	Drillhole diameter (mm)	Maximum No. and type of anchors	Range mm/m	Range mins	Sensitivity mm/m	Sensitivity secs	Manufacturer
Anchored chain of rods with transducers at pivots	Lateral deformation indicator/chain deflectometer	116–146 (cased)	Not determined	±10	35′	0·1–0·01	20–2	Eastman Interfels
Pivoted rod and proximity transducer	Multiple position deflectometer	75–100	Not determined	±12	40′	0·03	6	Terrametrics
Flexible steel strip with strain gauges in parallel to monitor	Strip gauge	75 or larger as required	Continuous	60 mm radius subject to metal thickness	5400	6·0	1200	Savage (not ye marketed)
Tiltmeter incorporating pendulum and vibrating wire measurement for mounting on retaining walls etc. or rods in drill hole	MDS 81 MDS 81B MDS 82B		As required	±3 ±6 ±12	10′ 20′ 40′	0·01–0·002 0·025–0·004 0·050–0·007	2–0·3 5–0·7 10–1·4	Maihak
Flexible breakable strip with resistors in series to detect depth of movement horizon	Shear strips	76 or larger	60 m strip lengths in series	Shear detection movement only		2–50 mm		Terrametrics

Table 2. Probe Inclinometers (EM 1110-2-1908, 1975, and Franklin and Denton, 1973).

Type	Trade Name	Approximate Casing Size mm	Casing Type	Range mm/m	Range deg	Sensitivity mm/m	Sensitivity sec	Manufacturer
Strain-gaged pendulum	CRL Inclinometer	45 x 45	Square allumi-num duct	±88 from vertical	±5	0.075	15	Cementation Research
	Inclinometer	50	Aluminum tubing with keyways	360 from vertical	±20	0.2	36	Soil Instruments
	Borehole clinometer	76 x 76	Square steel tube	±175 from vertical	±10	0.1	20	Structural Behavior Eng. Lab.
	C-350 slope meter	45 x 45	Square steel tube	±577 from vertical	±30	0.075	15	Soiltest
Pendulum with rheostat	Series 200-B slope indi-cator	81	Aluminum tubing	±467 ±87 from vertical	±25 ±5	1.0	180	Slope Indicator
2 electrolevels at 90 deg, servomotor and compass	Slope reader	51	Plastic	±175 from vertical	±10	0.1	20	Eastman
Servo accelerometers	Digitilt	30/70/81	Aluminum/ plastic tube	±577 infinite	±30 ±90	0.1	18	Slope Indicator
Pendulum with vibrating wire, 2 direction, compass or keyway	MDS 83	50 or larger	Aluminum or plastic, keyways optional	±290	±15	0.05	10	Maihak
Pendulum with vibrating wire	68-062 inclinometer	50	Aluminum alloy	±792	±45	0.15	30	ELE/Geonor
Pendulum with differential trans-former, automatic recorder	Earth deformation recorder (EDR)	89	Plastic with grooves			0.3% for angles up to 4 deg; 0.15% for angles up to 8 deg		Dames & Moore
Pendulum with vibrating wire	MPF clinometer				15	1		Telemac

15 ft (4.5 m) into soil or rock in which no movement is anticipated. Allowance should be made for loss of the bottom 5 ft (1.5 m) of the hole where sediment accumulation may occur. Casing should be held in place with a sand backfill or a weak cement grout. Casings over 50 ft (15 m) deep should be checked for twist using equipment described in paragraph 4.6 since some of the casings may be received with a built-in twist which would cause considerable errors in observations.[6.3]

3.1.1 Inclinometer casings are commonly installed in either 5- or 10-ft (1.5- or 3.0-m) lengths and are available in either plastic or aluminum. Plastic casing joints are glued. Aluminum casings are coupled with aluminum couplings and riveted, Fig. 2. Care should be taken to ensure that all joints are sealed since leakage can introduce fines into grooves and cause errors in readings. Joints can be sealed with caulking and taped. Greater installation details can be obtained from manufacturers' literature[6.4, 6.5, 6.6, 6.7] or from other sources.[6.3, 6.8]

3.2 Observations - Initial observations should be made after allowing sufficient setting time for the grout around the casing or time for the backfill to settle where sand or gravel is used. Since all displacements are computed based on the position of the casing when installed, the initial position should be verified with at least three separate sets of observations. These observations should be checked closely to see that they agree within the accuracy of the inclinometer being used. Observations should be repeated until satisfactory agreement is obtained. When initial observations are made, the top of the casing should be located with respect to a point outside the zone of expected movement by conventional surveying means and its elevation determined.

3.2.1 The frequency of observations depends upon several factors, the most important of which is the rate of movement. It is necessary to read inclinometers frequently just after installation and, based on

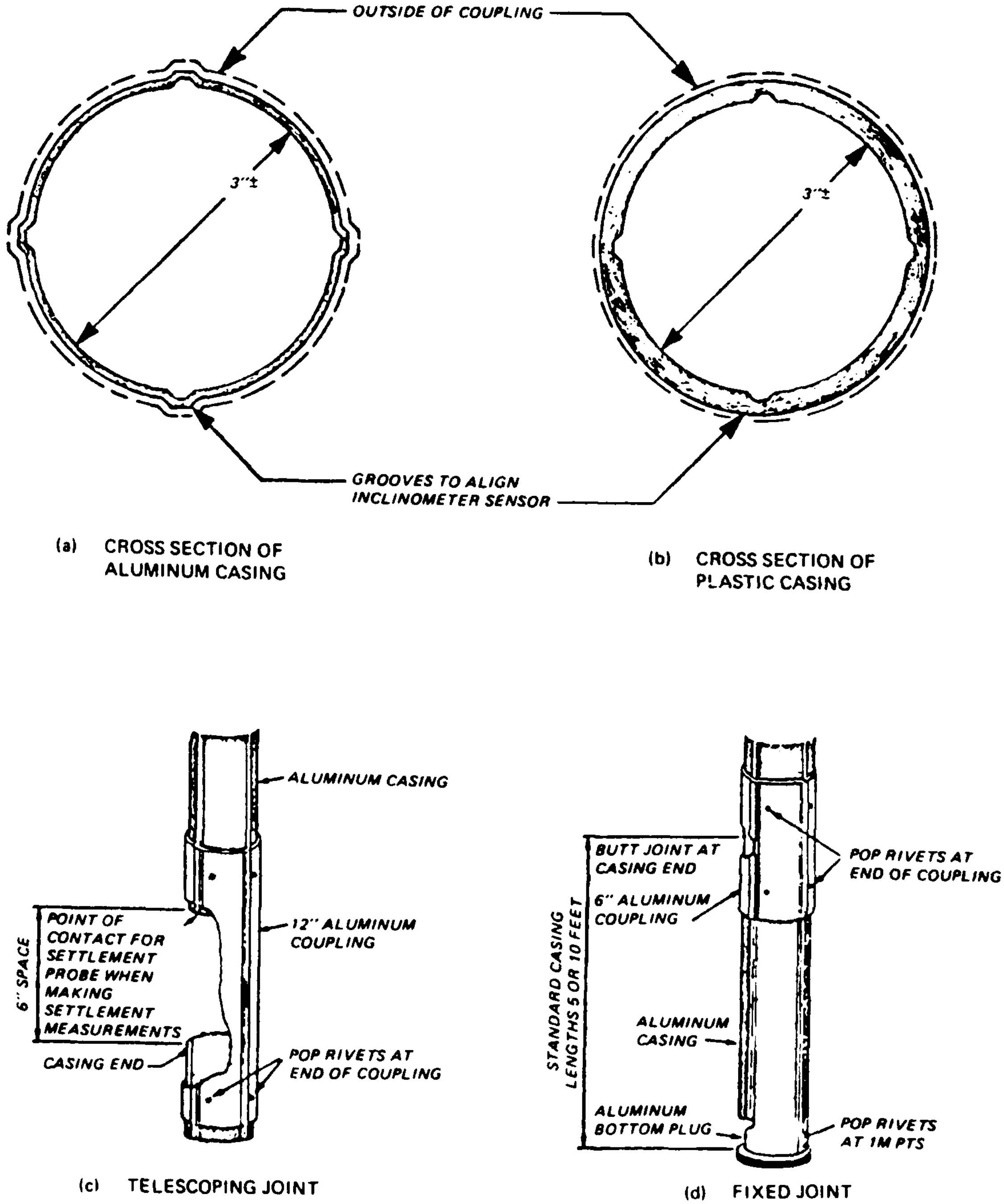


Fig. 2. Inclinometer casing details (Slope Indicator Co.).

these results, to adjust the interval of observations. Observations should coincide with observations of other instrumentation such as extensometers, piezometers, settlement devices, movement surveys, etc.

3.2.2 The procedure for obtaining readings with various inclinometers may vary slightly and manufacturers' literature should be consulted for the current procedure for a particular instrument. However, the general procedure consists of lowering the inclinometer to the bottom of the borehole and beginning the readings. The inclinometer is raised a specified interval,* readings are made, and the procedure repeated until the top of the hole is reached. The inclinometer is removed from the casing, inserted again with the guide wheels in a different groove, lowered to the bottom of the casing, and readings are again made to the top of the hole. This procedure is repeated until a set of readings is obtained for all four grooves. A field check is made by comparing the value of the sum of each set of readings (opposite grooves) and the mean of all sets of readings for the length of the casing. When variations greater than specified by the manufacturer are found, the inclinometer is relocated at that depth and an additional reading is taken. Care should be taken to ensure that readings are obtained at the same depths each time observations are made.

3.3 Maintenance - Maintenance that can be performed in the field on inclinometers is very limited. On probes using O ring connections between the probe and the cable, the O ring should be checked and replaced as necessary. Electrical connections should be kept clean and dry. On probes using batteries, the battery should be checked and charged when necessary. Manufacturers' literature should be consulted for other maintenance operations and precautions to be exercised in operation of inclinometers.

4. Data Reduction

4.1 General - The numerical values of the readings (R) obtained from observations with most inclinometers are equal to plus or minus an

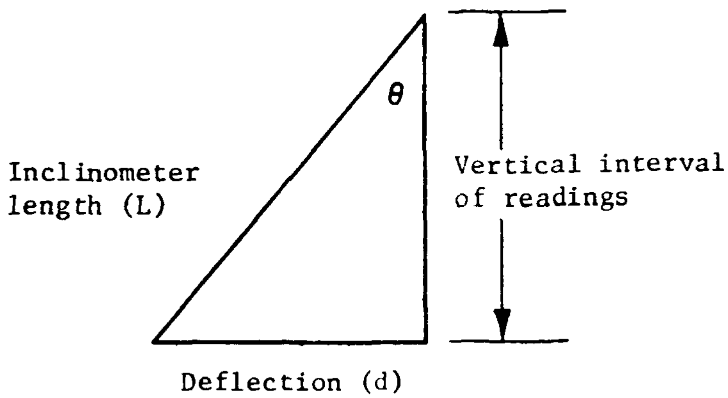
* Greatest accuracy is obtained when the interval of observations equals the wheel spacing of the probe.

instrument constant (K) times the sine of the inclination angle (θ). Expressed mathematically, this is:

$$R = \pm K \sin \theta \qquad (1)$$

where the plus or minus sign indicates the direction of movement--plus away from the groove in which the measuring wheel is located and minus toward the groove.

4.1.1 To compute the deflection of the casing from the vertical at any measurement point, the right triangle depicted below is solved:



where L is the distance between measuring wheels. This results in the following expression:

$$d = L \sin \theta \qquad (2)$$

The algebraic difference in readings ($R_1 - R_3$) in opposite grooves (180 deg apart) can be used to minimize errors contributed by casing and instrument irregularities.

* The formula is true for the Hall Inclinometer and the Digitilt. For the Model 200-B, the reading is equal to a constant times the tangent of the inclination angle; however, for the range of operation (±12 deg) the tangent is approximately equal to the sine. Manufacturers' literature should be consulted for applicability of this discussion to a particular instrument.

$$\text{Difference} = (R_1 - R_3) = \pm\, 2K \sin\theta \qquad (3)$$

Solving for $\sin\theta$

$$\sin\theta = \frac{\text{Difference}}{2K}$$

and substituting into Equation 2 we have

$$d = \frac{L}{2K} \cdot \text{Difference} \qquad (4)$$

4.1.2 Because the prime interest is not the magnitude of the deflection (d) but the change in deflection or magnitude of movement since the initial readings, the initial deflection of the casing must be subtracted from the deflection at some later time.

$$d = d_t - d_i = \frac{L}{2K}\,(\text{Difference}_t - \text{Difference}_i) \qquad (5)$$

It is also desirable to know the deflected shape of the casing with reference to a fixed point or length. This fixed length is normally considered to be the bottom of the casing so that the formula now becomes:

$$D = \sum_{m=o}^{n} (d_t - d_i) = \frac{1}{2K} \sum_{m=o}^{n} (\text{Difference}_t - \text{Difference}_i)$$

$$D = \frac{1}{2K} \sum_{m=o}^{n} \text{Change} \qquad (6)$$

where m = o is at the bottom of the casing or first measuring point and m = n is at the top of the casing or last measuring point. It is obvious from the above that the initial deflection of the casing need not be computed. Only the Difference is needed.*

---

* As stated previously, several observations should be taken initially. The Difference used above is an average of these observations.

4.2 Hand Calculations - Hand calculations of the deflections in a borehole can be made using the above formula (Equation 6) and the data sheets in Figs. 3 and 4. However, the calculations would require checking of many additions, subtractions, and multiplications for each vertical plane in which deflection measurements were made. Where several boreholes are observed, this would be a long and tedious operation and therefore computer reduction of data is usually performed.

4.3 Computer Data Reduction - Equation 6 is readily adaptable to computer reduction either from data recorded by hand or with automatic data recording devices. Computer programs are available that reduce the data, tabulate, and plot the results. Documentation of two such programs is contained in reference 6.1.

4.4 Twist Corrections - In casings over 50 ft (15 m) in length, accumulated twist can cause significant errors in the assumed direction of movement. Casing should therefore be checked for twist using commercially available equipment. If the twist is found to be significant, readings can be corrected using computer programs currently available.[6.1]

5. Reporting Results

5.1 Results of inclinometer measurements are usually reported in two ways: in tabulations of deflections with depth (Fig. 4) and in plots (Fig. 5) showing movement versus depth in relation to the structure, tunnel, or embankment near which movement is being monitored.

6. References

6.1 Leach, Roy E., "Evaluation of Some Inclinometers, Related Instruments, and Data Reduction Techniques," Miscellaneous Paper S-76-12, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi, 1976.

6.2 Franklin, J. A. and Denton, P. E., "The Monitoring of Rock Slopes," The Quarterly Journal of Engineering Geology, Vol 6, No. 3-4, 1973.

FIELD SHEET INCLINOMETER WELL NO. 1N 3

Project X - Mining of Tunnel 2
After Shove 216, Face + 3

SHEET 1 OF 1

DATE 9/15/72 0900 0915 INSTR 1Z0021 READ WH/FS CALC WH CHKD HM

DEPTH	DIFF	AZ = SPRING N		DIFF	CHANGE	DIFF	AZ = SPRING S		DIFF	CHANGE
		AN	AS				BN	BS		
74		+137	-126	+263	+11		+121	-141	+262	-20
72		+136	-126	+262	+10		+142	-150	+292	-8
70		+021	-019	+040	+2		+103	-108	+211	-5
68		-045	+038	-103	+13		+132	-145	+277	-13
66		-070	+083	-153	+13		+136	-154	+290	-18
64		+068	-058	+126	+10		+380	-395	+775	-15
62		+104	-090	+194	+14		+405	-423	+828	-18
60		+104	-081	+185	+23		+373	-406	+779	-33
58		+060	-048	+108	+12		+264	-278	+542	-14
56		+053	-039	+092	+14		+245	-265	+510	-20
54		+031	-026	+057	+05		+262	-267	+529	-5
52		+066	-054	+120	+11		+186	-206	+392	-20
50		+004	+011	-007	+15		+191	-211	+402	-20
48		-010	+028	-038	+18		+315	-339	+654	-24
46		+173	-161	+334	+12		+334	-353	+687	-19
44		+145	-134	+279	+11		+376	-370	+746	+6
42		+118	-107	+227	+11		+353	-388	+741	-35
40		+092	-080	+172	+12		+326	-339	+665	-13
38		+080	-073	+153	+7		+321	-364	+685	-43
36		+038	-027	+065	+11		+293	-307	+600	-14
34		-078	+092	-170	+14		+371	-384	+755	-13
32		+083	-062	+145	+21		+318	-325	+643	-7
30		+179	-165	+344	+14		+191	-212	+403	-21
28		+192	-178	+370	+14		+154	-173	+327	-19
26		+245	-231	+476	+14		+000	-022	+022	-22
24		+256	-242	+498	+14		-079	+068	-147	-11
22		+185	-172	+357	+13		-132	+101	-233	-31
20		+114	-107	+221	+7		-044	+037	-081	-7
18		+043	-031	+074	+12		-032	+015	-047	-17
16		+009	-000	+009	+9		-014	-003	-011	-17
14		+141	-128	+269	+13		-001	-021	+020	-22
12		+124	-110	+234	+14		+009	-028	+037	-19
10		+009	+013	-004	+22		-026	+001	-027	-25
8		-076	+087	-163	+11		+021	-039	+060	-18
6		-046	+060	-106	+14		+025	-044	+069	-19
4		+038	-030	+068	+8		+040	-047	+087	-7
2		+057	-044	+101	+13		+011	-028	+039	-17
					Σ180°					Σ180°

Fig. 3. Field data sheet for borehole inclinometer measurements (Cording, 1975).

PROJECT X - MINING of TUNNEL 2

JOB NO 121 SHT 5 of 5
INCLINOMETER WELL NO 1N3
SLOT BEARING B Axis: East(-) West(+)

DEPTH	INITIAL	DATE 9/15/72 0900 INST 180021 R: .0006 C:				DATE 9/15/72 [illegible] INST 180021 R: .0006 C:				DATE INST R: C:				DATE INST R: C:				DATE INST R: C:				DATE INST R: C:			
	8/21/71	DIFF	CHANGE	ΣCHANGE	DEFL	DIFF	CHANGE	ΣCHANGE	DEFL	DIFF	CHANGE	ΣCHANGE	DEFL	DIFF	CHANGE	ΣCHANGE	DEFL	DIFF	CHANGE	ΣCHANGE	DEFL	DIFF	CHANGE	ΣCHANGE	DEFL
2'	+128	+039	-89	-650	-.29"	+050	-78	-711	-.43"																
4	+136	+087	-69	-561	-.34	+090	-66	-633	-.38																
6	+139	+069	-60	-492	-.30	+070	-59	-567	-.34																
8	+117	+060	-57	-432	-.26	+057	-60	-508	-.30																
10	+088	-027	-60	-375	-.22	-022	-65	-448	-.27																
12	+083	+037	-46	-315	-.19	+033	-50	-383	-.23																
14	+066	+020	-46	-269	-.16	+014	-52	-333	-.20																
16	+026	-011	-37	-223	-.13	-015	-41	-281	-.17																
18	-020	-047	-27	-186	-.11	-049	-29	-240	-.14																
20	-068	-081	-13	-159	-.10	-083	-15	-211	-.13																
22	-211	-233	-22	-146	-.09	-235	-24	-196	-.12																
24	-131	-147	-16	-136	-.08	-149	-18	-172	-.10																
26	+035	+022	-13	-110	-.07	+028	-7	-154	-.09																
28	+324	+327	+3	-97	-.06	+333	+9	-147	-.09																
30	+413	+403	-10	-100	-.06	+405	-8	-156	-.09																
32	+682	+643	-39	-90	-.05	+637	-45	-148	-.09																
34	+800	+755	-45	-51	-.03	+753	-47	-103	-.06																
36	+640	+600	-40	-6	0	+600	-40	-56	-.03																
38	+730	+685	-45	+34	+.02	+685	-45	-16	-.01																
40	+700	+665	-35	+79	+.05	+661	-39	+29	+.02																
42	+770	+741	-29	+114	+.07	+739	-31	+68	+.04																
44	+740	+746	+6	+143	+.09	+750	+10	+99	+.06																
46	+677	+687	+10	+137	+.08	+685	+8	+89	+.05																
48	+641	+654	+13	+127	+.08	+649	+8	+81	+.05																
50	+383	+402	+19	+114	+.07	+405	+22	+73	+.04																
52	+375	+392	+17	+95	+.06	+395	+20	+51	+.03																
54	+515	+529	+14	+78	+.05	+529	+14	+31	+.02																
56	+494	+510	+16	+64	+.04	+504	+10	+17	+.01																
58	+525	+542	+17	+48	+.03	+538	+13	+7	0																
60	+770	+779	+9	+31	+.02	+777	+7	-6	0																
62	+824	+828	+4	+22	+.01	+822	-2	-13	-.01																
64	+780	+775	-5	+18	+.01	+769	-11	-11	-.01																
66	+291	+290	-1	+23	+.01	+284	-7	0	0																
68	+275	+277	+2	+24	+.01	+273	-2	+7	0																
70	+207	+211	+4	+22	+.01	+205	-2	+9	+.01																
72	+282	+292	+10	+18	+.01	+288	+6	+11	+.01																
74	+254	+262	+8	+8	0	+259	+5	+5	0																
	[illegible]	+13183		-663		13224		-211																	

Fig. 4. Summary sheet for borehole inclinometer calculations (Cording, 1975).

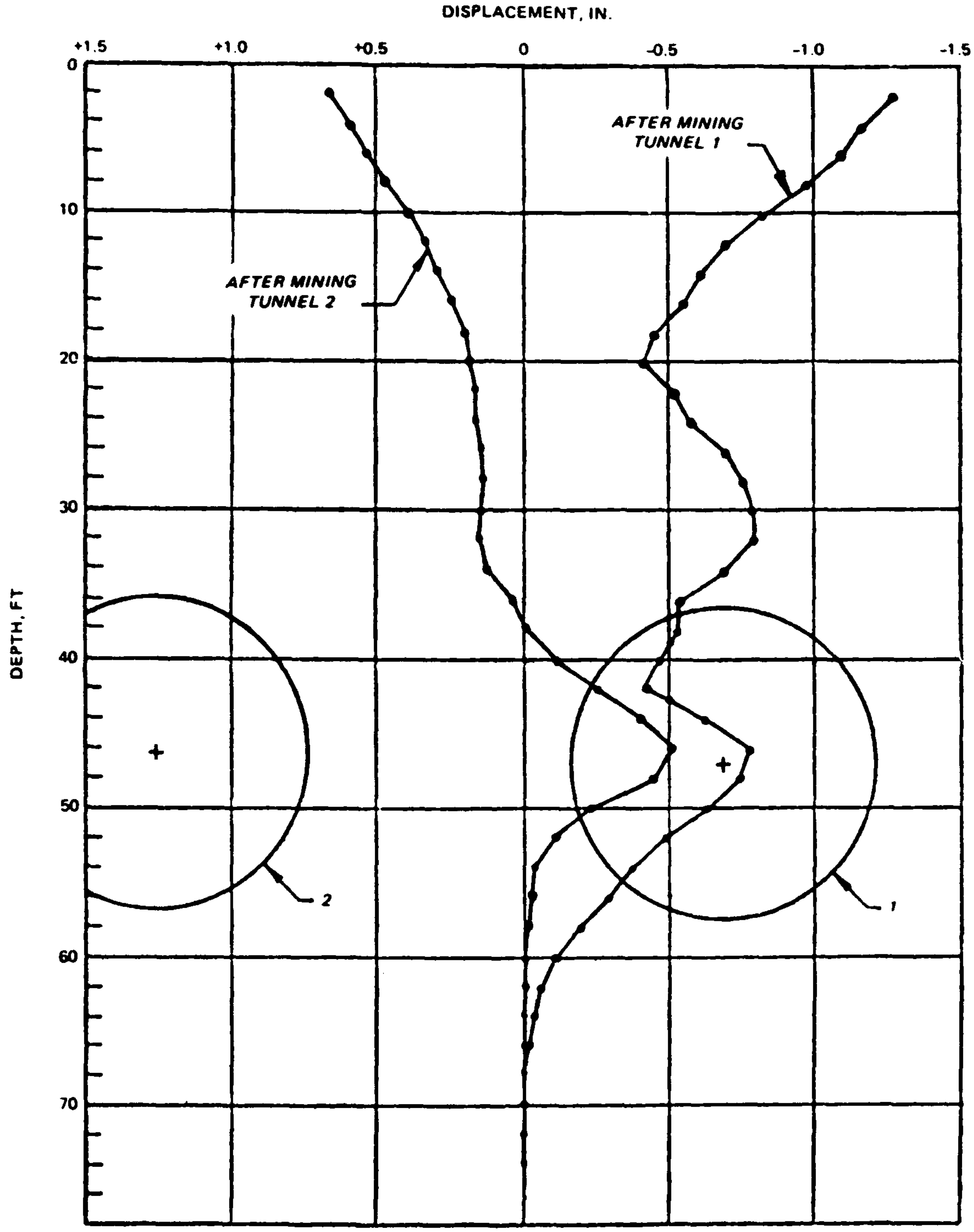


Fig. 5. Typical lateral movement profile from borehole inclinometer (Cording, 1975).

6.3 Department of the Army, Office, Chief of Engineers, "Instrumentation of Earth and Rock-Fill Dams, Part 2, Earth-Movement and Pressure Measuring Devices," Engineering Manual EM 1110-2-1908, Washington, D. C., 1975.

6.4 "Instruction Manual, Series 200-B Instrument," Seattle, Washington.

6.5 ______________________, "Instruction, Digitilt with Tally Tape Perforator Model 50303," Seattle, Washington.

6.6 Geo-Testing, Inc., "Instruction Manual, Hall Inclo-Meter System," San Rafael, California.

6.7 Soiltest, Inc., "Operating Instructions, Model C-350 Slope Meter," Evanston, Illinois.

6.8 Cording, E. J., et al., "Methods for Geotechnical Observations and Instrumentation of Tunneling," Vol I and II, NSF Research Grant GI-33644X, Department of Civil Engineering, University of Illinois, Urbana, Illinois, 1975.

SUGGESTED METHODS FOR MONITORING ROCK MOVEMENTS USING TILTMETERS

(International Society for Rock Mechanics)

1. (a) A tiltmeter consists of a housing containing a gravity operated sensor which detects static or dynamic angular movements at a point. Normally a portable tiltmeter is temporarily located on a reference plate (Fig. 1) cemented or bolted to intact rock at the ground surface or in a tunnel or adit[1]. Periodic measurement of the surface tilt of each plate enables determination of the magnitude and rate of angular deformation.

(b) Nonportable tiltmeters are also available which may be used for static or dynamic angular measurement and can provide continuous monitoring[2]. These sensors are enclosed in a waterproof housing and cemented or bolted and grouted directly to the rock surface. Local access for reading may be utilized or remote reading facilities may be installed.

(c) The tiltmeter, unlike the probe or fixed-in-place inclinometers, only measures the tilt at a discrete, normally accessible point. It does not operate at depth along a borehole although in certain situations it can be permanently buried.

(d) The reference plate must be anchored on a surface which properly reflects movements of the rock mass under investigation. The weathered initial few meters at the surface can often be avoided by locating the reference plate on a pipe or concrete pillar founded on intact rock 1 to 2 meters below the surface. The monument must be free from contact with the upper 1 to 2 meters of rock. Failure to protect the reference plate from the effects of surficial temperature and moisture variations may result in erroneous readings.

Fig. 1. Portable tiltmeter, reference plate and readout.

Apparatus

2. Surface preparation and drilling equipment including:

(a) Hand tools or bush hammer or jack hammer for surface cleanup, leveling and preparation of a fresh dust-free surface.

(b) A rock drill to give drillholes 15 to 40 mm in diameter, up to about 300 mm deep for fixing bolts.

(c) A rock drill to give a shallow drillhole of the required diameter for a steel anchor tube 70 to 150 mm in diameter, up to 3 m long. Core drilling should be used, but if no core is available, a borehole periscope may be used to inspect the hole.

(d) A rock drill to excavate a large diameter shallow hole for an isolated concrete pillar. Blasting should be avoided.

3. Surface reference plates and installation equipment:

(a) The surface reference plate on which the portable tiltmeter sensor is periodically located should be made of dimensionally stable metal, ceramic or rock. The reference surface should be corrosion-free, easy to wipe clean and not easily damaged. The surface of the reference plate should incorporate a precise positioning system compatible with the portable sensor and which locates the sensor in two mutually perpendicular directions. The positioning system must permit the sensor to be reversed through 180 deg to enable zero errors in the portable sensor to be eliminated. Inaccurate replacement may be the greatest source of error with a sensitive tiltmeter.

(b) Epoxy or polyester resin cement, Portland cement or similar grouts for fixing the reference plate directly to prepared rock surface or steel plate.

(c) Anchor bolts where additional anchorage is needed.

(d) A steel tube 70 to 150 mm in diameter, up to 3 m long to be grouted into a shallow hole and isolated from the weathered initial 1 to 2 meters of rock at the surface. The tube should have an integral reference plate welded in place.

(e) Materials for constructing a concrete pillar or monument founded below the rock surface and isolated from the weathered initial 1 to 2 meters of rock at the surface.

4. Tiltmeter sensor and readout (for example, Fig. 1) including:

(a) If a portable tiltmeter sensor is to be used this should comprise a housing containing a sensing device[3], and with a reference surface incorporating a precise positioning system compatible with the fixed reference plates. The reference surface on the portable sensor should be corrosion-free, easy to wipe clean and not easily damaged. The electrical sensing device is connected by a cable to a compatible portable readout box[4].

(b) If a nonportable tiltmeter is to be used, this should comprise an electrically operated sensor[3] enclosed in a waterproof corrosion resistant housing designed for direct and permanent fixing to the rock surface. The electrical sensing device is connected by a permanently installed cable to a compatible monitoring and readout system. The tiltmeter housing should preferably incorporate a reference surface suitable for a portable tiltmeter which is used periodically to check the permanently installed tiltmeter for zero drift or other malfunction.

(c) The measuring range, sensitivity and accuracy of the tiltmeter with its readout system should be specified according to the requirements of the project. The range and resolution of tiltmeters varies considerably; ±30 deg and 10 seconds, ±0.7 deg and 2 seconds are typical specifications.

(d) The equipment should be designed to ensure that the specified accuracy is maintained irrespective of normal mechanical handling, water pressures, and corrosive environments encountered in use.

5. Calibration equipment including:

(a) A calibrating device to enable initial checking of fixed tiltmeters or routine on-site checking of the portable sensor and readout unit. The device should allow the sensor to be set in its normal operating position and should be adjustable from horizontal to the maximum operating angle of the sensor, with at least one intermediate setting either side of the horizontal. The calibrator should ideally have an independent angle measuring accuracy better than the resolution of the portable tiltmeter sensor[5].

Procedure

6. Preparatory investigations:

(a) The site and project characteristics should be considered in detail in order to specify the performance requirements of equipment to be used.

(b) Tiltmeter locations should be selected on the basis of a study of the geotechnical features of the site, taking into consideration the directions and magnitudes of anticipated ground movements and the nature of other instrumentation to be installed.

(c) The amount and depth of weathering of the rock surface should be investigated so that the tiltmeter is located on or anchored in intact rock. Failure to eliminate the influence of localized surficial movements or inhomogeneities may entirely invalidate the tiltmeter measurements especially if the true subsurface movements are small. If extensive weathering exists, subsurface location in a tunnel or adit is desirable.

7. Installation:

(a) If suitable fresh, hard, unweathered, sound rock exists at the surface, minimal cleanup is required to form a dust-free, level surface.

(b) Holes should be drilled for fixing bolts when the grout to rock adhesion is unreliable. Alternatively, a shallow hole should be drilled or excavated to install a tubular steel anchor or to construct a concrete pillar, isolated from the weathered near-surface rock and founded on fresh rock.

(c) The underside of the reference plate or of the nonportable tiltmeter, thoroughly cleaned and abraded, is oriented to correspond with the required direction of measurement and is then cemented or grouted in place on the rock, anchor or pillar and leveled. The azimuth should be recorded to an accuracy of ±3 deg.

(d) A protective cap or cover should be installed over the exposed reference plate or tiltmeter to prevent damage.

8. Readings:

(a) The nonportable tiltmeter should be calibrated prior to installation.

(b) The portable tiltmeter should be checked on site both before and after each day's readings. Instrument errors should be promptly investigated and corrected and a diary of calibrations and adjustments should be kept. Unnecessary adjustment must be avoided.

(c) Several sets of initial readings should be taken immediately after the cement or grout has set. These readings are averaged to provide a baseline for all subsequent observations. Thereafter, readings should be taken at intervals specified by the project engineer on the basis of site requirements. A set of readings with the portable tiltmeter should comprise, as a minimum, steps 9(d) and 9(e) below.

(d) The reference plate and portable tiltmeter are wiped with a clean dry cloth and inspected for dirt or damage. The tiltmeter is accurately located on the reference plate and a reading taken. The tiltmeter is removed, the contact surfaces rewiped and replaced. This procedure is repeated three or four times until consistent readings are obtained. The tiltmeter is then rotated through 180 deg and the readings repeated, rewiping the contact surfaces each time.

(e) The procedure 9(d) is repeated with the tiltmeter located at 90 deg to the initial position.

(f) A permanently installed tiltmeter may be read manually or automatically at suitable time intervals. It should, where feasible, be periodically checked by a portable tiltmeter as in 9(d) and 9(e).

Calculations and Data Processing

9. (a) Unless otherwise specified all data should be processed within 24 hours of readings being taken[6].

(b) The field data are scrutinized and obvious errors marked on the field data sheet. If corrections are made, these should be clearly noted.

(c) Pairs of opposite face readings obtained with the portable tiltmeter are averaged to correct for face error. The direction of angular rotation must be carefully checked and recorded.

(d) Single face readings obtained with the permanently installed tiltmeter may be corrected for zero error, based on intermittent portable tiltmeter readings on both faces.

(e) Corrected readings are compared with initial readings at the same location to determine the incremental change in angle or displacement.

(f) Graphs of angular change or displacement versus time are plotted for each reference plate location (for example, Fig. 2).

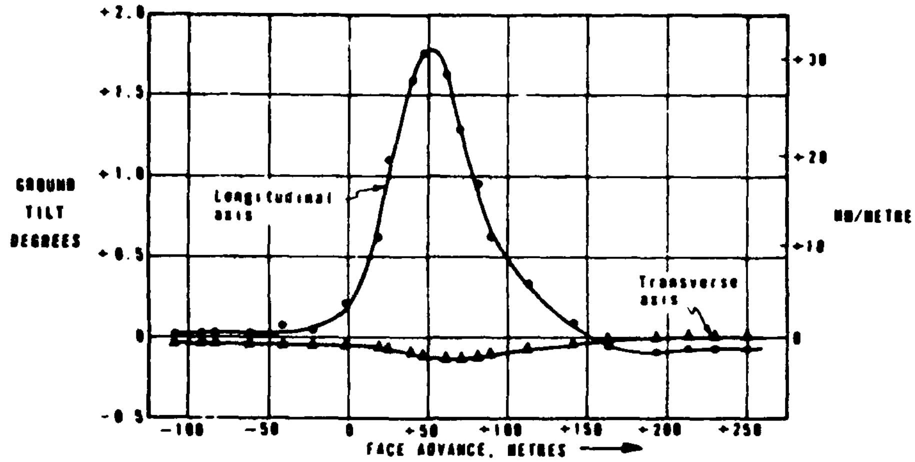


Fig. 2. Tilt at ground surface due to longwall mining face advance at depth.

Reporting of Results

10. Results should, unless otherwise specified, be presented in two forms of report: an Installation Report giving basic data on the instrumentation system at the time of installation; followed by Monitoring Reports presenting periodically the results of routine observations. The Monitoring Reports will generally be required at frequent intervals to minimize delay between the detection of adverse behavior and the implementation of any remedial measures that may be necessary.

11. The Installation Report should include the following:

(a) A description and diagrams of the monitoring equipment used including detailed performance specifications and manufacturers' literature.

(b) A station location plan with details of the reference plates, their surveyed positions and elevations.

(c) Details of methods used for tiltmeter installation, calibration, and monitoring; reference may be made to this ISRM Suggested Method stating only departures from the recommended procedures.

(d) For each station, a diagram showing the geotechnical characteristics of the ground and the position of the reference plate. The azimuth of reference plate guides should be reported clearly stating conventions adopted for the sign of movement and angle directions.

(e) For each station, a tabulated list of initial tiltmeter readings.

12. The Monitoring Reports should include the following:

(a) A set of field monitoring result tabulations; the set to cover all observations since the preceding report.

(b) Graphs of angular change or displacement versus time, sufficient to show clearly the magnitudes, rates, and directions of all significant movements.

(c) A brief commentary drawing attention to significant movements and to all instrument malfunctions occurring since the preceding report.

Notes

[1]Tiltmeters may also be installed on structures founded on rock, for example on concrete dams or turbine foundations.

[2]In addition, there exist very sensitive and narrow range tiltmeters designed to resolve angles as small as 1×10^{-8} radians, used mainly for detecting earthtides and other geodetic or seismic events.

[3]The sensing device may, for example, include an electrolytic spirit level, a pendulum actuated vibrating wire or closed loop servo-accelerometer or alternatively a precise spirit level with a manual mechanical-optical micrometer system, which is not suitable for automatic recording.

[4]The readout box may for example include a direct reading voltmeter, a manual null-balance bridge circuit, a digital voltmeter, or an automatic null balance bridge with digital display and an integrating circuit to sum incremental displacements. Units are available that allow recording on magnetic or paper tape.

[5]It is recognized that this may be difficult to achieve if a narrow range tiltmeter with a resolution of one or two seconds of arc is used, and some compromise will be necessary.

[6]Data processing may be manual or with the aid of a computer; however, the various stages of computation must in either case be fully supervised and checked for reading or transcribing errors and to ensure that the significance of any anomalous behavior is fully appreciated. For example, it is often difficult to distinguish between anomalies due to ground behavior and those due to instrument malfunction.

Designation: D 4403 – 84 RTH 303-89

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa. 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition.

Standard Practice for EXTENSOMETERS USED IN ROCK[1]

This standard is issued under the fixed designation D 4403; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This practice covers the description, application, selection, installation, data collecting, and data reduction of the various types of extensometers used in the field of rock mechanics.

1.2 Limitations of each type of extensometer system are covered in Section 3.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Significance and Use

2.1 Extensometers are widely used in the field of engineering and include most devices used to measure displacements, separation, settlements, convergence, and the like.

2.2 For tunnel instrumentation, extensometers are generally used to measure roof and sidewall movements and to locate the tension arch zone surrounding the tunnel opening.

2.3 Extensometers are also used extensively as safety monitoring devices in tunnels, in underground cavities, on potentially unstable slopes, and in monitoring the performance of rock support systems.

2.4 An extensometer should be selected on the basis of its intended use, the preciseness of the measurement required, the anticipated range of deformation, and the details accompanying installation. No single instrument is suitable for all applications.

3. Apparatus

3.1 *General*—Experience and engineering judgment are required to match the proper type of extensometer systems to the nature of investigation for a given project.

3.1.1 In applications for construction in rock, precise measurements will usually allow the identification of significant, possibly dangerous, trends in rock movement; however, precise measurement is much less important than the overall pattern of movement. Where measurements are used to determine rock properties (such as in plate-jack tests), accurate measurements involving a high degree of precision are required. For in-situ rock testing, instrument sensitivity better than 0.0012 in. (0.02 mm) is necessary for proper interpretation.

3.1.2 Most field measurements related to construction in rock do not require the precision of in-situ testing. Precision in the range of 0.001 to 0.01 in. (0.025 to 0.25 mm) is typically required and is readily obtainable by several instruments

3.1.3 As the physical size of an underground structure or slope increases, the need for highly precise measurements diminishes. A precision of 0.01 to 0.04 in. (0.25 to 1.0 mm) is often sufficient. This range of precision is applicable to underground construction in soil or weak rock. In most hard rock applications, however, an instrument sensitivity on the order of 0.001 in. (0.025 mm) is preferred.

3.1.4 The least precision is required for very large excavations, such as open pit mines and large moving landslides. In such cases, the deformations are large before failure and, thus, relatively coarse precision is required, on the order

[1] This practice is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Aug. 31, 1984. Published November 1984.

of 1 % of the range where the range may be 3 ft. (1 m) or more.

3.1.5 For long-term monitoring, displacements are typically smaller than those that occur during construction. Therefore, greater precision may be required for the long-term measurements.

3.2 *Extensometers:*

3.2.1 *Rod Extensometers*—A large variety of rod extensometers are manufactured. They range from simple single-point units to complicated multipoint systems with electrical readout. The single-point extensometer is generally used to detect support system failures. The rod can also serve as a safety warning device in hazardous areas. Generally, the rod extensometer is read with a depth-measuring instrument such as a dial gage or depth micrometer, however, various electrical transducers such as LVDTs (linear variable differential transformers), linear potentiometers, and microswitches have been used where remote or continuous readings are required (as shown in Fig. 1). Another type of readout recently developed is a noncontact removable sonic probe digital readout system which is interchangeable with the depth micrometer type. Multipoint rod extensometers have up to eight measuring points. Reduced rod diameters are required for multipoint instruments and have been used effectively to depths of at least 150 ft (45 m). The rod acts as a rigid member and must react in both tension and compression. When used in deep applications, friction caused by drill hole misalignment and rod interference can cause erroneous readings.

3.2.2 *Bar Extensometers*—Bar extensometers are generally used to measure diametric changes in tunnels. Most bar extensometers consist of spring-loaded, telescopic tubes that have fixed adjustment points to cover a range of several feet. The fixed points are generally spaced at 1 to 4-in. (25 to 100-mm) increments. A dial gage is used to measure the displacements between the anchor points in the rock (as shown in Fig. 2). If the device is not constructed from invar steel, ambient temperature should be recorded and the necessary corrections applied to the results. Bar extensometers are primarily used for safety monitoring devices in mines and tunnels.

3.2.3 *Tape Extensometers*—Such devices are designed to be used in much the same manner as bar extensometers, however, tape extensometers allow the user to measure much greater distances, such as found in large tunnels or powerhouse openings. Tape extensometers consist of a steel tape (preferably invar steel), a tensioning device to maintain constant tension, and a readout head. Lengths of tape may be pulled out from the tape spool according to the need. The readout may be a dial gage or a vernier, and the tensioning mechanism may be a spring-loading device or a dead-weight (as shown in Figs. 3 and 4). The tape and readout head are fastened, or stretched in tension, between the points to be measured. Accuracies of 0.010 to 0.002 in. (0.25 to 0.05 mm) can be expected, depending on the length of the tape and the ability to tension the tape to the same value on subsequent readings, and provided that temperature corrections are made when necessary.

3.2.4 *Joint Meters*—Normally, joint meters consist of an extensometer fixed across the exposed surface of a joint (as demonstrated in Fig. 5), and are used to measure displacements along or across joints. The joint movements to be measured may be the opening or closing of the joint or slippage along the joint. Rod-type extensometers are generally used as joint meters with both ends fixed across the joint. Preset limit switches are often mounted on the joint meter to serve as a warning device in problem areas such as slopes and foundations.

3.2.5 *Wire Extensometers*—Such devices utilize a thin stainless steel wire to connect the reference point and the measuring point of the instrument (as shown in Fig. 6). This allows a greater number of measuring points to be placed in a single drill hole. The wire or wires are tensioned by springs or weights. The wire is extended over a roller shiv and connected to a hanging weight. Wire extensometers tensioned by springs have the advantage of variable spring tension caused by anchor movements. This error must be accounted for when reducing the data. Wire-tensioned extensometers have been used to measure large displacements at drill hole depths up to approximately 500 ft (150 m). The instruments used for deep measurements generally require much heavier wire and greater spring tensions. Although wire extensometers are often used in open drill holes for short-term measurements, in areas of poor ground or unstable holes it is necessary to run a protective sleeve or tube over the measuring wires between the anchors.

3.3 *Anchor Systems:*

3.3.1 *Groutable Anchors*—These were one of the first anchoring systems used to secure wire extensometer measuring points in the drill hole. Groutable anchors are also used for rod type extensometers. Initially PVC (poly(vinyl chloride)) pipes clamped between the anchor points were employed to isolate the measuring wires from the grout column (as shown in Fig. 7), however, this arrangement was unreliable at depths greater than 25 ft (7.5 m) because the hydrostatic head pressure of the grout column often collapsed the PVC tubing. To counteract this condition, oil-filled PVC tubes were tried. The use of oil enabled this method to be used to depths of over 50 ft (15 m). As an alternative to this system, liquid-tight flexible steel conduit is used to replace the PVC pipe. This alternative system seems to work well and can be used in most applications. Resin anchors fall in this category and are very successful.

3.3.2 *Wedge-Type Anchors*—These consist of a mechanical anchor that has been widely used for short-term anchoring applications in hard rock. Fig. 8 shows the two basic types of wedge anchors: (*1*) the self-locking spring-loaded anchor, and (*2*) the mechanical-locking anchor. Self-locking anchors, when used in areas subject to shock load vibrations caused by blasting or other construction disturbances, may tend to slip in the drill holes or become more deeply-seated, causing the center wedge to move. Another disadvantage of the wedge anchor is that no protection is offered, if using wires, to the measuring wires in the drill hole against damage that might be caused by water or loose rock.

3.3.3 *Hydraulic Anchors*—These anchors have proven to be successful in most types of rock and soil conditions. Fig. 9 shows the two basic types of hydraulic anchors manufactured for use with extensometer systems: (*1*) the uncoiling Bourdon tube anchor, and (*2*) the hydraulic piston of grappling hook anchor, which is limited to soft rock and soils. Both anchors have the disadvantage of being rather costly. The Bourdon tube anchor works well in most rock and soil conditions and the complete anchor system can be fabricated before installing it in the drill hole. There have been other specialized anchor systems developed, however, these systems have proven to be too costly and unsuccessful for most applications.

3.4 *Extensometer Transducers*—These extensometers convert displacements occurring in insitu materials between two anchored points to mechanical movements that can be measured with conventional measuring devices such as dial gages, LVDTs, strain gages, and the like.

3.4.1 *Depth-Measuring Instruments*—A dial gage, or a depth micrometer are the simplest and most commonly used mechanical measuring instruments. Used in conjunction with extensometers, they provide the cheapest and surest methods of making accurate measurements. When using the dial gage or depth micrometer, the operator is required to take readings at the instrument head, however, local readings may not be practical or possible due to the instrument location or area conditions.

3.4.2 *Electrical Transducers*—For remote or continuous readings, electrical transducers are used rather than dial gages. LVDTs are often used because of their accuracy, small size, and availability. LVDTs require electrical readout equipment consisting of an a-c regulated voltage source and an accurate voltmeter, such as a digital voltmeter or bridge circuit. The use of linear potentiometers or strain gages is often desirable because of the simplicity of the circuitry involved. The disadvantage of using linear potentiometers is their inherently poor linearity and resolution.

3.4.3 When very accurate measurements are dictated by certain excavations, for example, the determination of the tension arch zone around a tunnel opening, extensometers which can be calibrated in the field after installation shall be used. In all cases, the accuracy of extensometers, either determined through calibration or estimation, should be given in addition to the sensitivity of the transducers. The strain-gaged cantilever extensometer (shown in Fig. 10) has been used successfully for many years. The strain-gaged cantilever operates on the principles of the linear strain produced across a given area of a spring material when flexed. This type of extensometer readout is normally used when rock movements of 0.5 in. (12.5 mm) or less are expected. Strain gages produce a linear change in resistance of 1 to 3% of their initial resistance, over their total measurement range. Because of this small change in resistance, it is absolutely necessary to provide extremely good electrical connections and cable insulation when using this type of transducer. Standard strain-gage readout equipment can be used with this type of extensometer, however, care must be taken to protect this equipment from the hostile environments found in most field applications. Vibrating wire and sonic readouts are also reliable and are becoming more common than strain-gage readouts. Provision should always be made for mechanical readout capability.

4. Procedure

4.1 *Preparatory Investigations:*

4.1.1 Select the location, orientation, length, and number of anchors for each extensometer on the basis of a thorough review of both the construction and geotechnical features of the project. Among the items to be considered are: direction and magnitude of anticipated rock movements, location and nature of other instruments to be installed, and the procedures and timing of construction activities before, during, and after installation of the instrument. If the instrument is installed where rock bolts are used for support, the deepest extensometer anchor shall be located beyond the end of the rock bolt. The length of the extensometer shall depend upon the anticipated depth of rock influenced by excavation, expressed for example in terms of tunnel diameter or slope height. As a general rule, the deepest anchor (reference point for all subsequent anchors) shall be placed at least 2½ tunnel diameters beyond the perimeter of the tunnel.

4.1.2 Displacement measurements are most valuable when extensometers are installed at, or before, the beginning of excavation, and when measurements have been taken regularly throughout the entire excavation period at several locations so that a complete history of movements is recorded. Documentation of the geologic conditions and construction events in the vicinity of the measurements is essential to the proper interpretation of the field data.

4.2 *Drilling:*

4.2.1 The size of borehole required for extensometers depends on the type, character, and number of anchors. The borehole size shall conform to the recommendations of the extensometer manufacturer.

4.2.2 The method of drilling used depends upon the nature of the rock, the available equipment, the cost of each method, and the need for supplemental geologic data. Percussion drilling equipment of the type used for blast holes is usually available and is the least costly. Coring methods, like those used for subsurface exploration, are usually more expensive but provide important information on the presence and nature of rock discontinuities. On large projects, coring or close observation of the percussion hole is usually justified to better define the geology. In addition, coring affords the opportunity to position extensometers accurately in the vicinity of major discontinuties.

4.2.3 Immediately prior to drilling, verify the location and orientation of the drill hole.

4.2.4 For percussion-drilled holes, maintain visual inspection of the drilling operation from start to completion of the hole. At all times, the operation shall be under the direct supervision of an individual familiar with drilling and knowledgeable in the peculiarities and intended use of the extensometer. For later use in summarizing the installation, keep notes on drilling rates, use of casing, soft zones, hole caving, plugging of drilling equipment, and any other drilling difficulties.

4.2.5 For cored holes, similar inspection and observation as that for percussion-drilled holes shall be recorded, giving particular attention to drilling techniques that may affect the quality of the rock core obtained. The core shall be logged, including rock lithology, joint orientation, joint roughness, and degree of weathering. For both percussion-drilled and cored holes, note the location of water bearing seams or joints and water flows.

4.2.6 Immediately prior to installing the anchor assembly, thoroughly clean the completed borehole by washing with a pressure water hose. Holes in which instruments are not installed for a lengthy time after drilling (a day or more) shall be carefully cleaned immediately prior to installing the anchor. If hole caving or other blocking in zones of poor rock is suspected, verify the openness of the hole by inserting a pipe or wooden dowel the full length of the hole. In very poor ground conditions, special procedures involving grouting and temporary casing may be required to keep the borehole open sufficiently long to allow installation.

4.3 *Installation:*

4.3.1 Installation of the anchor assembly and connection of the displacement sensor to the anchor assembly shall be performed by a suitably qualified instrumentation specialist. This specialist may be the manufacturer's representative or an individual who, through previous experience and training, is qualified to perform the task.

4.3.2 Whenever possible, adjust the position of the anchors to maximize the information obtained by the extensometer. For instance, it is desirable to have one anchor to each side of a shear zone or filled joint. If not determined from rock cores, discontinuties can be located by borehole television or borehole periscope surveys (Fig. 11 illustrates a typical extensometer installation in a tunnel).

4.3.3 For grouted anchor assemblies, allow sufficient time for setting and hardening of the grout before installation of the extensometer sensor unit. During this time, keep notes on any blasting or other construction activities in the vicinity of the instrument. The strength and compressibility of the grout should somewhat match the surrounding soil or rock.

4.3.4 Install a protective cover if the instrument does not have an inherently rugged protective cover or is not recessed within a borehole. The protective cover must provide full protection from damage due to workmen, equipment, and fly rock from blasting. For installations in blast-damaged areas, it is preferable to initially install the instrument with a mechanical sensor only. After the risks of damage have been reduced, an electrical, remotely read sensor can be installed. If an instrument is an electrical, remotely read type, suitably protect the electrical cable (such as by armored cable or steel pipe) to prevent damage during the intended period of use. Instruments installed at the ground surface shall be installed below the depth of frost penetration. Manholes shall be watertight in cold climates to prevent icing.

4.3.5 Verify zero readings of each extensometer at least two times prior to the start of construction, or at the time of installation or resetting if construction is in progress. Instruments installed several weeks or months in advance of construction should be monitored to detect equipment malfunctions and reading variations due to temperature or operation. Two calibrations are required per extensometer, one following installation and one following the conclusion of the test series. Additional calibrations are required only where: (*1*) a transducer is replaced or rewired, (*2*) the power supply is changed, (*3*) the instrument head is damaged, or (*4*) a transducer is moved or reset to change its zero point.

4.3.6 Completion of installation of the sensing unit requires a thorough check for electrical or mechanical malfunctions. For future reference, keep notes of any measurements, in-situ calibrations, or settings performed during this final checkout.

4.4 *Readings:*

4.4.1 Readings shall be taken by a person familiar with the equipment and trained to recognize critical measurements and their relevance to the particular project.

4.4.2 The mechanical or electrical device used to read an extensometer shall be checked on site both before and after each day's use. For instance, verify zero settings on dial gages, and compare readouts for resistance or vibrating-wire gages to standards.

4.4.3 Electrical readout equipment shall undergo full-range calibration by the manufacturer, or an appropriately qualified commercial calibration service, on a routine basis. This calibration shall take place before and after times of critical measurement, or periodically for long-term measurements.

4.4.4 For those instruments having such a feature, a periodic in-situ calibration shall be performed to determine changes in the behavior of the instrument. The calibration may be done at times of critical measurements or during regular maintenance.

4.4.5 After reading the extensometer, record the data in a field notebook or data sheet that contains a record of previous readings. When a reading is taken, check it immediately with the previous reading to determine if any significant displacements have taken place since the last reading, or to determine if the reading is in error. If a reading is in question (that is, unanticipated displacements are indicated), the observer shall take additional readings. The observer shall also check to see if the extensometer is dirty or has been damaged, or if any construction events have taken place that would explain the change in the readings. For all readings, record construction conditions and temperature.

4.4.6 Several observations will aid the interpretation of displacement measured by borehole extensometers and shall be noted in a "remarks" column on the data sheet or field book. Examples of these observations are as follows:

4.4.6.1 Opening of joints or movement of rock blocks.

4.4.6.2 Mapping of joints, shear zones, and other geologic features that could be related to movement. Observations of overbreak and rock loosening along the joints and shear zones will aid in evaluating the significance of these features.

4.4.6.3 Crack surveys in shotcrete. The width, length, and relative movement of the crack shall be measured with time, and the thickness of the shotcrete in the vicinity of the crack determined.

4.4.6.4 In tunnels, evidence of distress or displacement of steel ribs and timber blocking.

4.4.6.5 Evidence of distress or loosening of rock bolts.

4.4.6.6 The increase of waterflow in the drainage system of dams that can reflect the opening of joints in the upstream part of a rock foundation. This is also helpful in tunnels to indicate loosening and opening of rock mass.

5. Calculation

5.1 Unless otherwise specified, process all data as soon as possible, but within 24 h of the reading.

5.2 Again scrutinize the field data in the office and clearly mark obvious errors in the field book. Supposedly erroneous readings shall be replaced by additional readings and shall not be discarded or obliterated from the field records.

5.3 If not entered on a special data sheet at the time of the reading, the field data shall be transferred to a computation and data summary sheet, such as shown in Fig. 12.

5.4 The method of calculating displacements from the field data depends on the particular instrument. The procedure recommended by the manufacturer shall be followed unless an alternative method is proven acceptable. Thermal displacement corrections to extensometer output are made by interpolating the change in measuring rod or wire temperature between the depths where thermocouples are able to directly measure temperature change, integrating the interpolation function (a cubic spine) over each length and multiplying this quantity by the thermal expansion coefficient of the wire or rod.

5.5 A plot of displacement versus time is the best means of summarizing current data and should be kept up to date. Interpretation of the measurements is facilitated by considering not only displacement, but the rate of displacement and the rate of change of displacement with time. Rate of displacement is equal to the slope of the displacement curve.

5.6 Periodically, prepare displacement-depth plots, as illustrated for a tunnel in Figs. 11 and 13. The deepest anchor (No. 6) has been assumed a fixed point of reference for all anchors. The rock movements can be correlated with the position of supports.

6. Report

6.1 *General*—Present results, unless otherwise specified, in two forms: (*1*) an installation report giving basic data on the instrumentation system at the time of installation, and (*2*) a monitoring report that presents periodically the results of routine observations. The monitoring reports will generally be required at frequent intervals to minimize delay between the detection of adverse behavior and the implementation of any remedial measures that may be necessary.

6.2 Installation reports shall include the following:

6.2.1 A description, with diagrams, of all components of the extensometer (anchor assembly, displacement-sensing unit, readout equipment), including detailed performance specifications.

6.2.2 Type and details of drilling equipment used.

6.2.3 Log of drilling—For cored holes, a summary log including the log of drilling and a log of the core. Also include summaries of borehole television or periscope investigations when undertaken.

6.2.4 Details and methods of installation, calibration, and monitoring; reference may be made to this practice, stating only departures from the recommended procedures.

6.2.5 A borehole location diagram that relates the specific instrument to the entire project and other instrumentation. This diagram shall include (*1*) the station or coordinates and elevation of the head of instrument, (*2*) depth, orientation, and diameter of borehole, (*3*) distances between anchors and the reference head, and (*4*) the relative position of the instrument to present and future structures and other construction.

6.2.6 A plan and section of the installation that illustrates present and anticipated construction and geology.

6.3 Monitoring reports shall include the following:

6.3.1 A set of tabulated field monitoring results (containing information in the manner shown in Fig. 12), including all observations since the preceding report.

6.3.2 Updated diagrams of displacement of all individual sensing points with respect to time.

6.3.3 For selected instruments and locations, a diagram of displacement versus depth for various times. The reading times shall be correlated to the construction activity and shall emphasize the development or progressive nature of displacements that might be taking place.

6.3.4 A brief summary of the most significant displacements and all instrument malfunctions since the preceding report.

7. Precision and Bias

7.1 The precision and bias of any extensometer system are limited to the type of extensometer, transducer, and data acquisition system. The situation dictates the degree of precision and bias required.

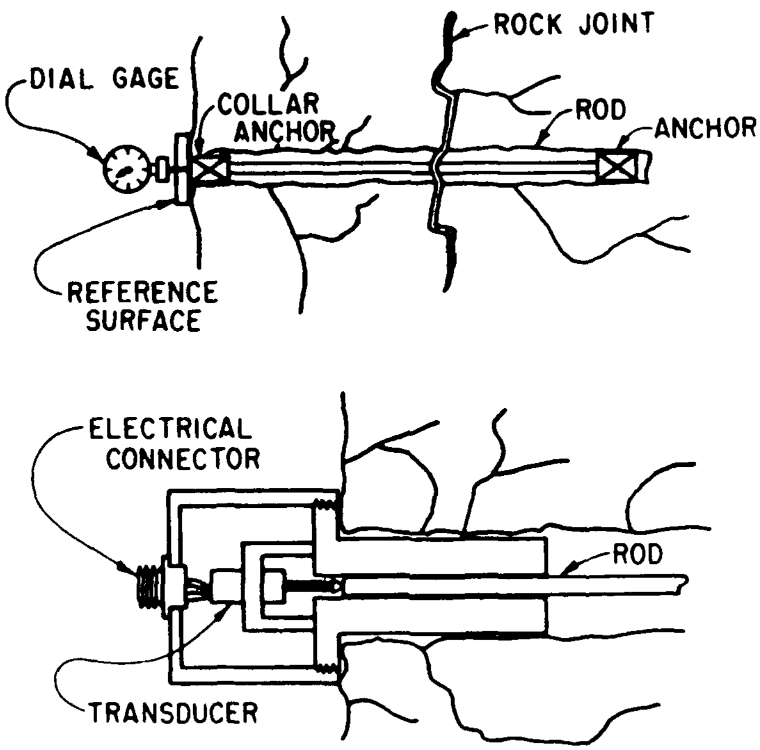


FIG. 1 Rod Extensometer

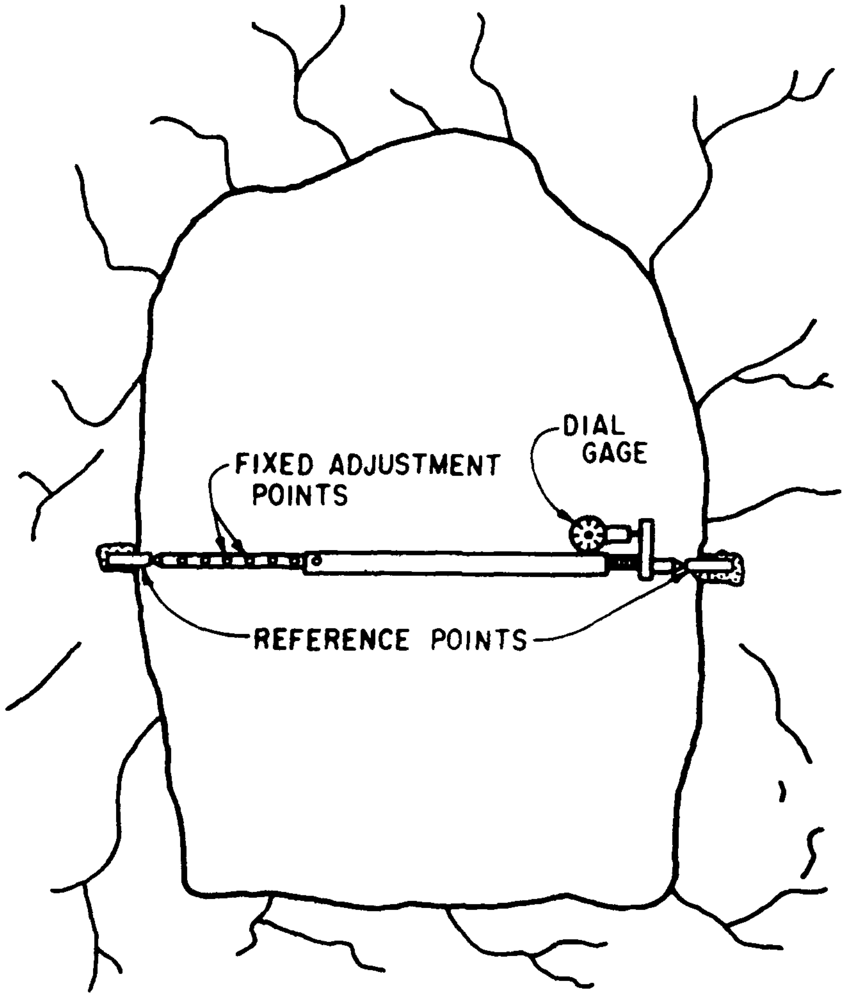


FIG. 2 Bar Extensometer

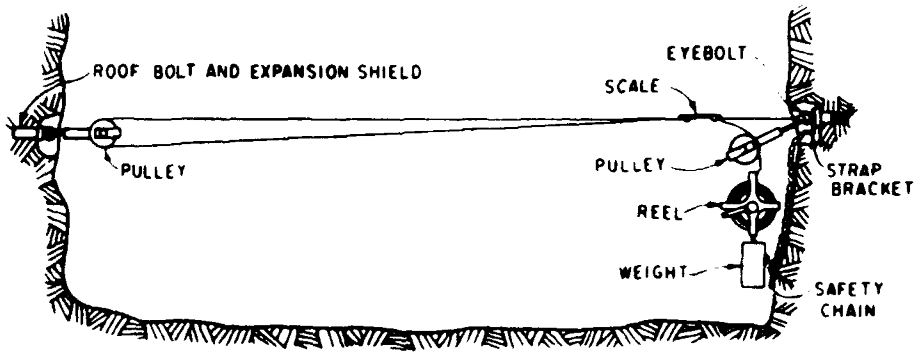


FIG. 3 Tape Extensometer with Vernier Readout and Deadweight

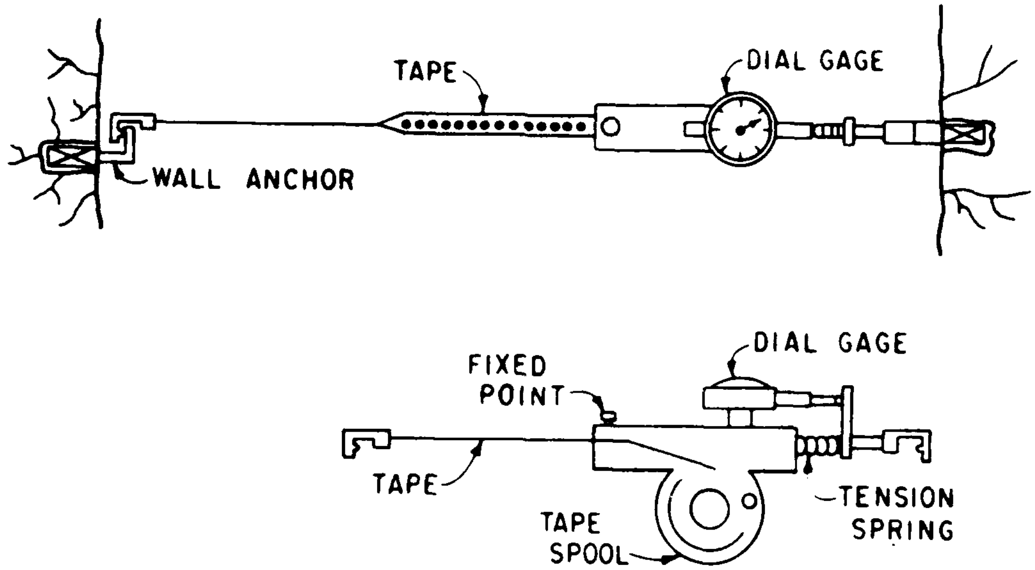


FIG. 4 Tape Extensometer with Dial Gage and Tension Spring

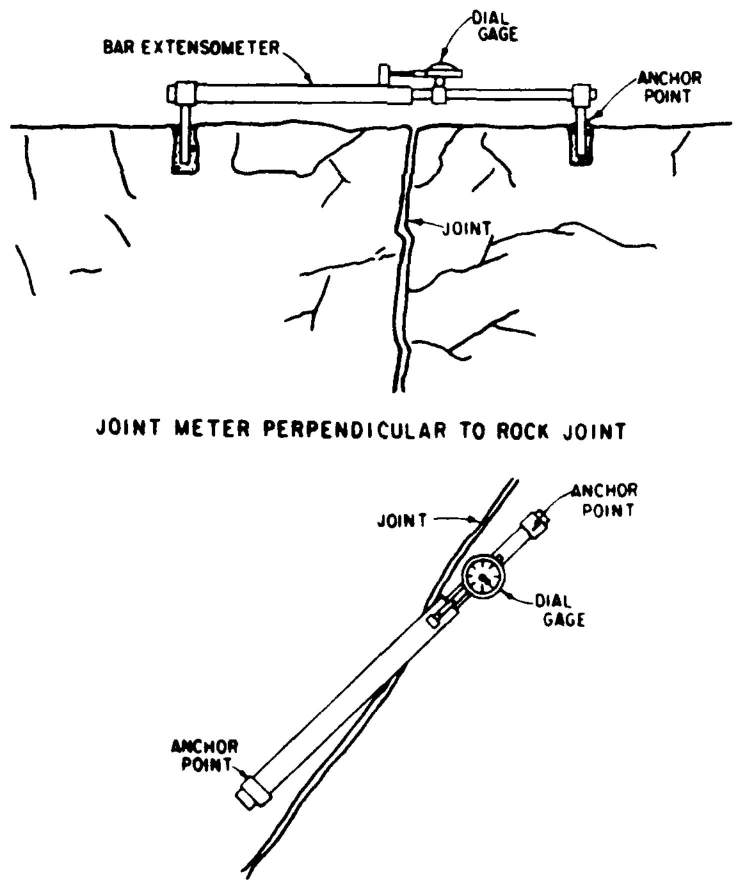
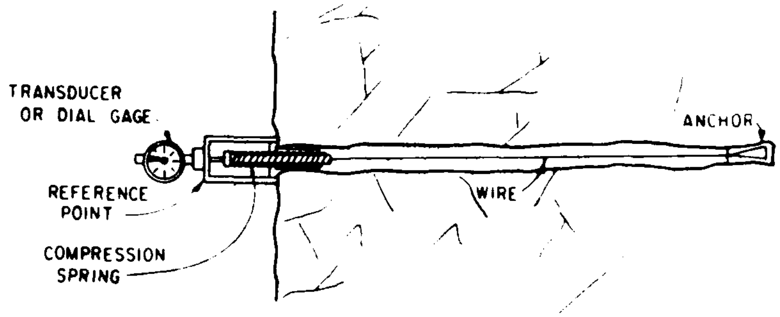


FIG. 5 Joint Meters



SPRING TENSIONED WIRE EXTENSOMETER

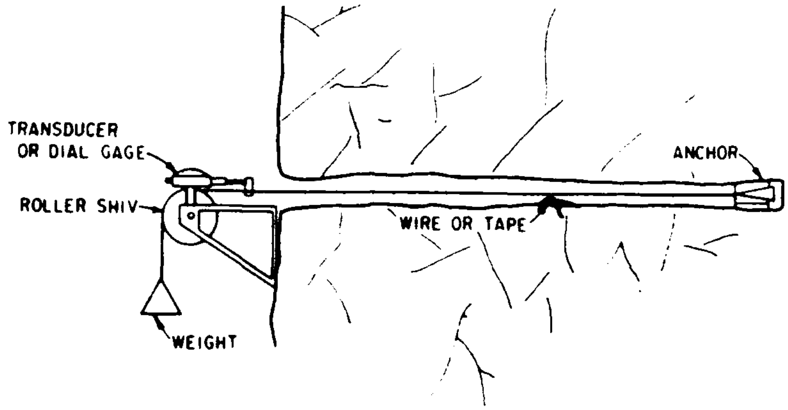


FIG. 6 Wire Extensometers

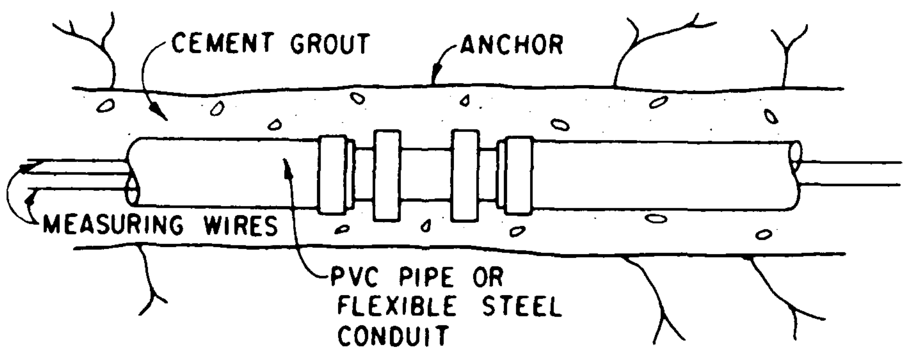
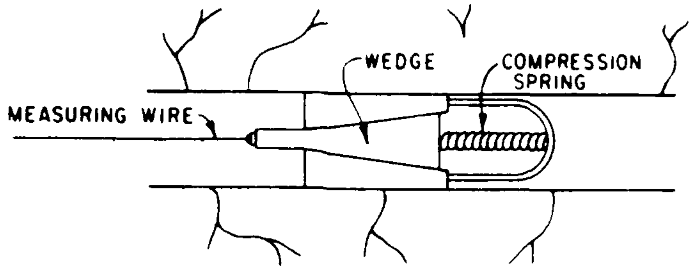
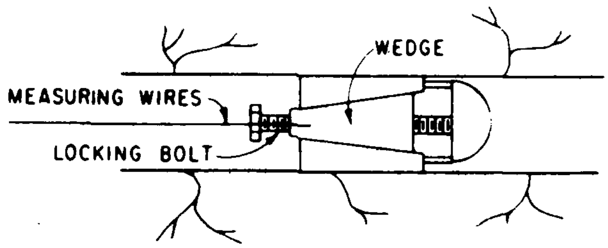


FIG. 7 Grouted Anchor System

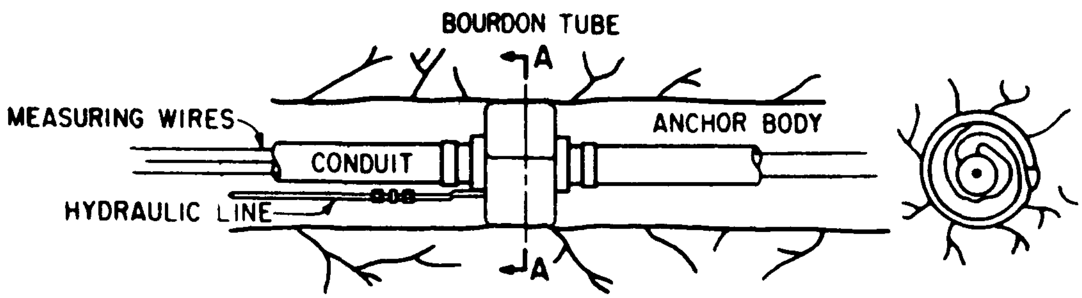


SELF-LOCKING WEDGE ANCHOR

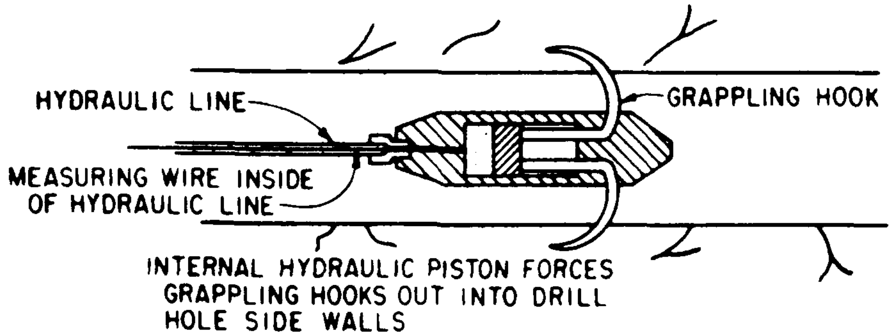


MECHANICAL-LOCKING WEDGE ANCHOR

FIG. 8 Wedge Anchors



BOURDON TUBE HYDRAULIC ANCHOR SECTION A-A



PISTON OR GRAPPLING HOOK HYDRAULIC ANCHOR

FIG. 9 Hydraulic Anchors

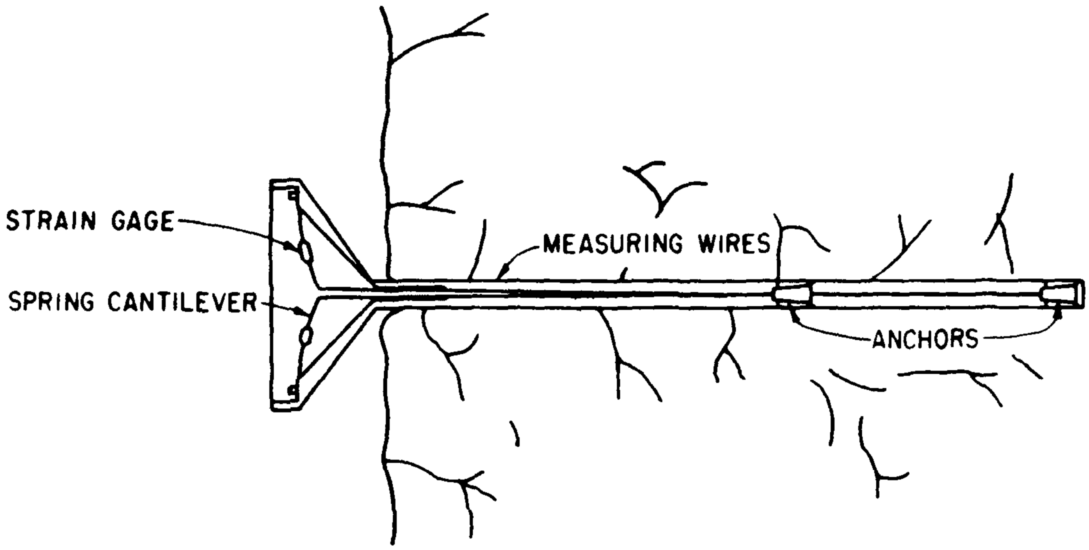
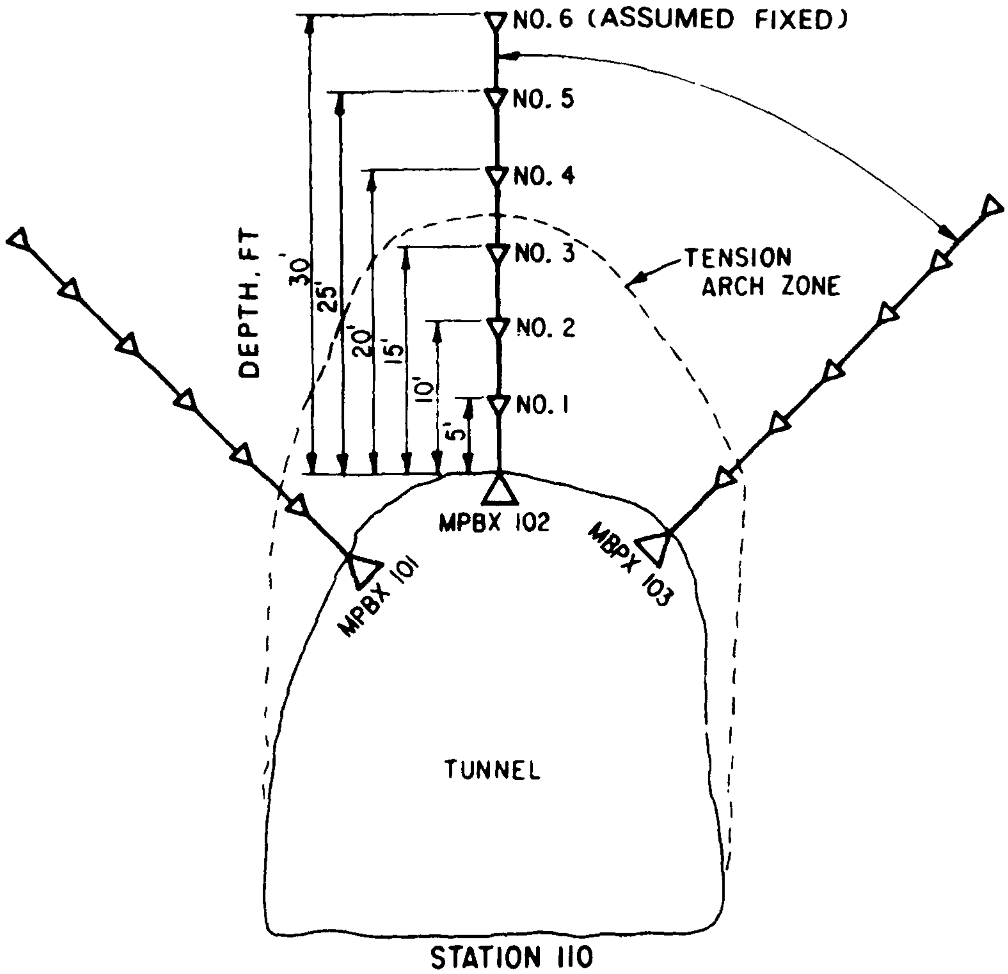


FIG. 10 Extensometer Using Strain-Gaged Spring Cantilevers



NOTE—Include such features as shear zones, rock bolts, and the like.

FIG. 11 Typical Extensometer Installation for a Tunnel Using Three 6-Point Extensometers with Anchors Spaced at 5-ft (1.5 m)

DOUBLE-POSITION MECHANICAL EXTENSOMETER

EXTENSOMETER NO. DX4 STATION 66+03 LOCATION CROWN

DATE	TIME	DEPTH 30 FT.		DEPTH 6 FT.		COMMENTS
		READING 0.001 INCHES	DISPLACEMENT INCHES	READING 0.001 INCHES	DISPLACEMENT INCHES	
7/11/72	10:30	2.252	–	2.460	–	INITIAL READING
10/24	11:00	2.264	+.012	2.459	-.001	STAGE I -65+66
10/27	15:30	2.270	+.018	2.464	+.004	STAGE I -65+76
10/31	21:30	2.283	+.031	2.471	+.011	STAGE I -65+86
11/3	18:20	2.240		2.374		NEW ZERO STAGE I - 66+06
11/6	13:52	2.281	+.072	2.392	+.029	STAGE I -66+11
11/6	22:00	2.353	+.094	2.398	+.035	STAGE I -66+16
11/7	22:00	2.323	+.114	2.400	+.037	STAGE I -66+21
11/8	10:00	2.329	+.120	2.400	+.037	STAGE I -66+21
11/9	9.45	2.349	+.140	2.402	+.039	STAGE I -66+26
11/10	10:30	2.366	+.157	2.406	+.043	STAGE I -66+31
11/13	17.00	–	–	2.408	+.045	STAGE I -66+36
11/14	10:00	2.381	+.172	2.409	+.046	STAGE I -66+36
11/14	18:00	2.388	+.179	2.409	+.046	STAGE I - 65+41 2b-65+32

FIG. 12 Sample Computation and Data Summary Sheet for a Double-Postion Mechanical Extensometer

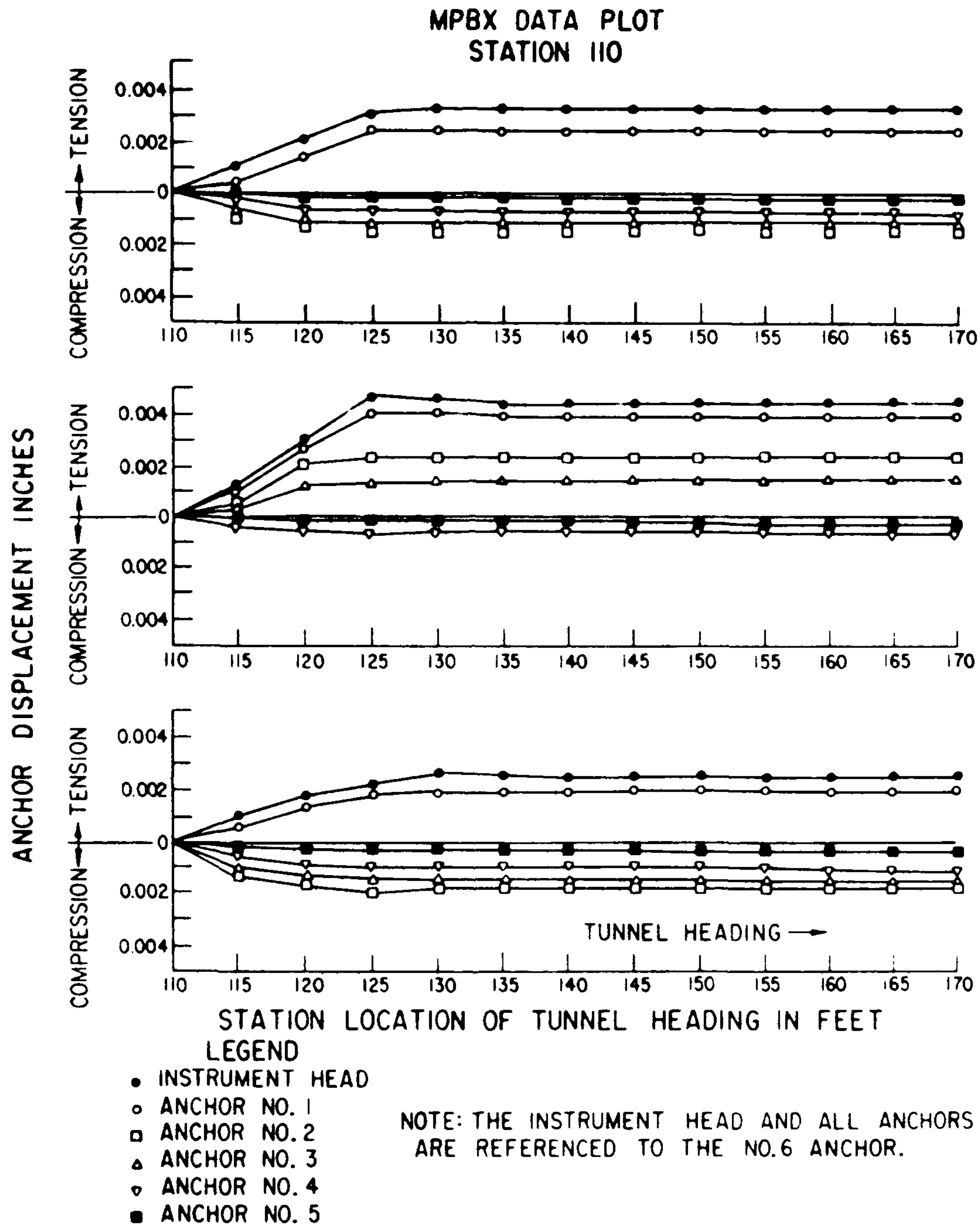


FIG. 13 Hypothetical Extensometer Data Plot

EXTENSOMETER SUPPLIERS AND MANUFACTURERS

Earth Science Research, Inc., 133 Mt. Auburn St., Cambridge, Massachusetts, 02138, USA.

Geonor, Norway, U. S. supplier, Slope Indicator Co.

Interfels International, Germany, North American supplier, Rocktest Ltd.

Irad Gage, 14 Parkhurst St., Lebanon, New Hampshire, 03766, USA.

Rocktest Ltd, 1485 Desaulneirs, Longeuil (Montreal), Quebec, Canada.

Peter Smith Instrumentation, England, North American supplier, Rocktest LTD.

Slope Indicator Co., 3668 Albion Place North, Seattle, Washington, 98103, USA.

Soil and Rock Instrumentation Inc., 30 Tower Road, Tower Office Park, Newton Upper Falls, Massachusetts 02164, USA.

Soil Test Inc., 2205 Lee St., Evanston, Illinois 60222, USA.

Terrametrics Inc., 16027 West 5th Avenue, Golden, Colorado 80401, USA.

LOAD CELLS

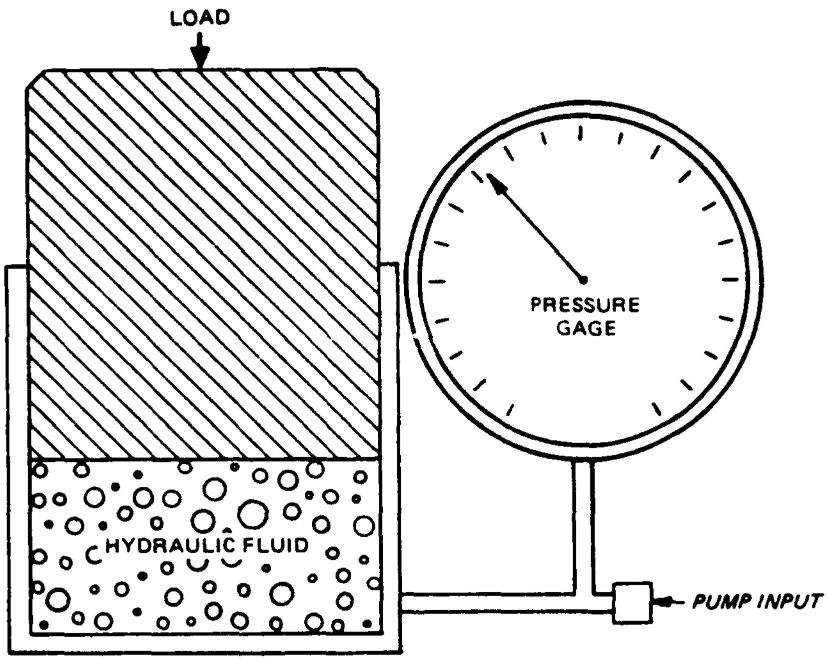
1. Scope

1.1 This method deals with load cells and their application in the field of rock mechanics and includes descriptions of various types, their construction, readout procedure, and data reduction.

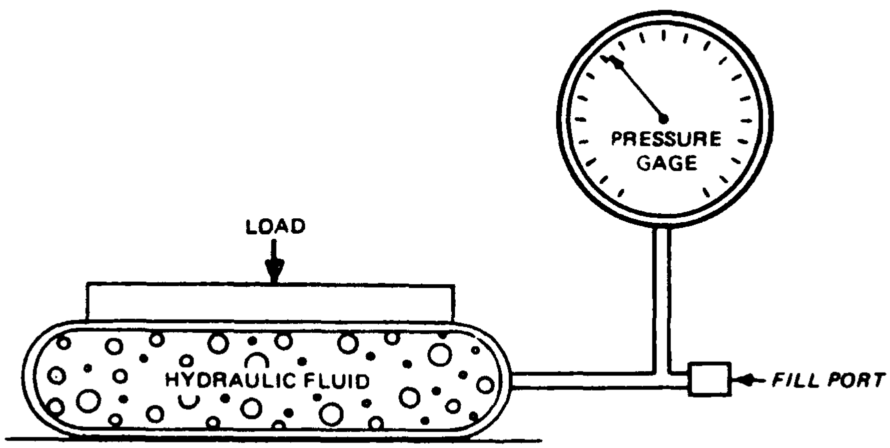
2. Apparatus

2.1 Load cells have been utilized for a wide variety of engineering applications. Such uses include tunnel support, rock bolt, and tieback load monitoring. The data obtainable from load cells can be used for safety monitoring and as an engineering aid for support design. Some of the more common types of load cells used in rock mechanics applications utilize one of four basic types of measurement systems: (1) hydraulic, (2) mechanical, (3) strain gage (this includes bonded foil, vibrating wire, and unbonded wire gages), and (4) photoelastic. Although the strain gage type load cell is the most commonly used, all types of load cells have certain advantages depending on their application.

2.1.1 Hydraulic Load Cells - Hydraulic load cells are basically a fluid-filled deformable chamber connected to a pressure gage or an electric pressure transducer. The load is transferred to the fluid by means of a piston, or in the case of the flat jack, deformation of the fluid confinement chamber (Fig. 1). Hydraulic load cells allow the user to preload the load member, such as rock bolt tiebacks, by applying an initial pressure to the fluid. It is often desirable to posttension rock bolts and tiebacks due to anchor slippage or shifting load distributions. Although most hydraulic load cells are of rugged construction, their application has been limited due to their physical size, poor load resolution, and temperature sensitivity. Hydraulic flat jacks have had the greatest application as earth pressure cells and concrete stress cells. Flat jacks have also been used in conjunction with other apparatus on radial jacking tests and in situ stress measurements.



PISTON HYDRAULIC LOAD CELL



FLAT JACK HYDRAULIC LOAD CELL

Fig. 1. Hydraulic load cells.

2.1.2 Mechanical Load Cells - The proving ring is the most common type of mechanical load cell but, due to its construction, has had very little application to field uses. The most commonly used mechanical load cells consist of an elastic disk element sandwiched between two plates. The disk deflects under load, changing the distance between the plates. The deflection is measured with a dial gage or suitable electronic transducer (Fig. 2). Although this type of load cell is relatively inexpensive to manufacture, it has had limited use because of its nonlinear calibration curve and restricted application. (This type of cell is generally designed to be used on rock bolts or tieback tendons.)

2.1.3 Strain-Gaged Load Cell

2.1.3.1 The strain-gaged load cell is by far the most commonly used for both field and laboratory applications. This type of cell is manufactured by a large number of geotechnical instrumentation suppliers. Most strain-gaged load cells consist of a metal cylindrical column. The column is loaded axially and the axial strain is measured with a suitable strain gage (Fig. 3). Bonded foil strain gages are used by most cell manufacturers because of their simplicity and availability, but the vibrating-wire and unbonded gages have also been proved to have distinct advantages. The bonded strain gage is a resistive element that undergoes a change in resistance when subjected to an axial strain. The gages are generally bonded to the load cell's cylindrical wall and connected together to form a Wheatstone bridge. The bonded gages are oriented in such a way as to cause a linear resistive imbalance proportional to the strain in the load cell. The unbalanced signal is amplified and observed on a galvanometer. Most strain gage readout equipment contain resistive balancing circuits that allow the operator to null the unbalanced signal with a potentiometer connected to a digital indicator. The load cells are calibrated to read load in pounds per readout digit (Fig. 4). The main advantages of the bonded strain-gaged load cells are simplicity of construction, relatively small physical size to load capacity ratio, direct reading requiring no summing, averaging or

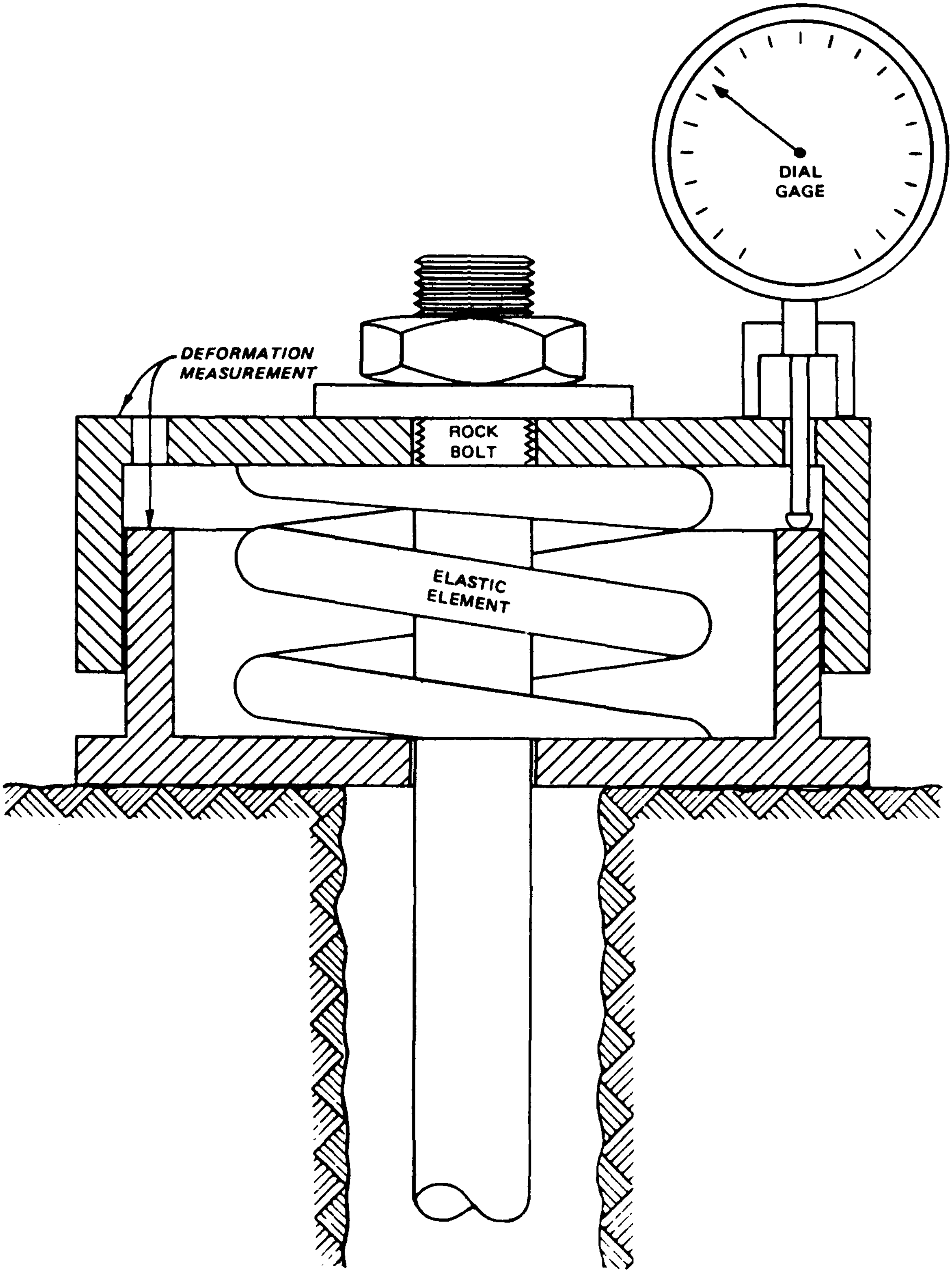


Fig. 2. Mechanical load cell.

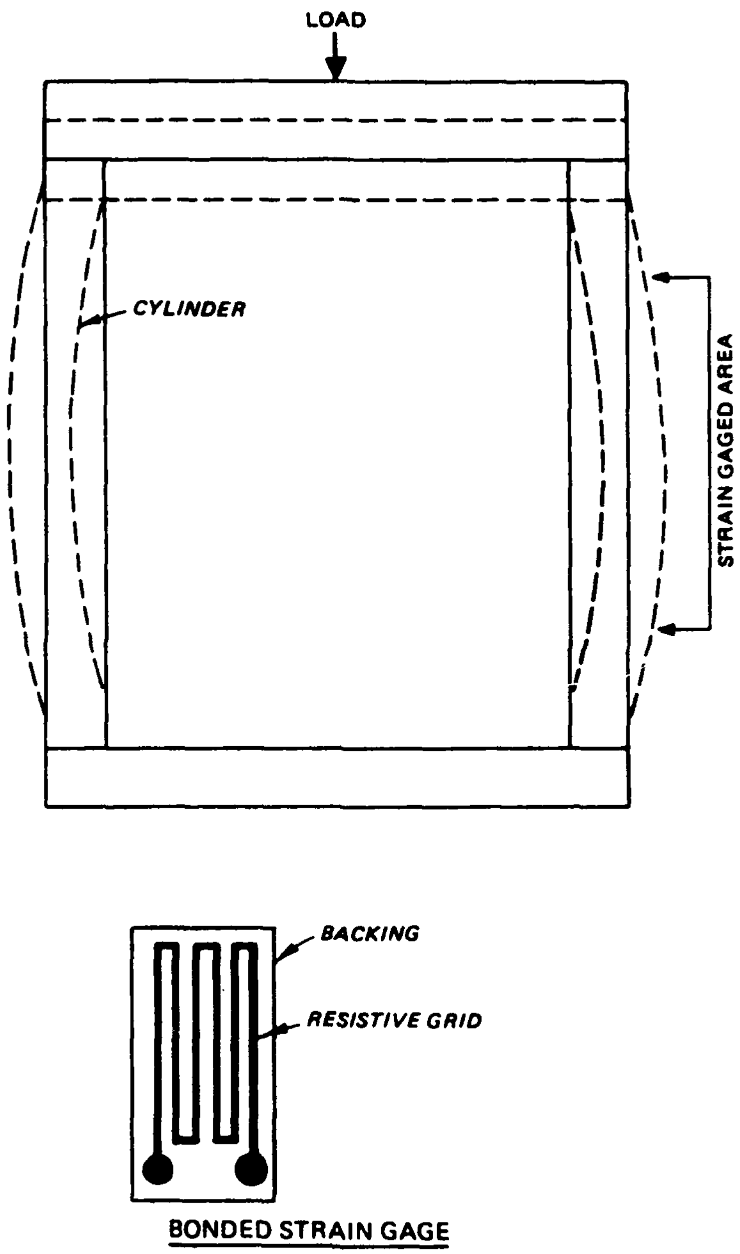


Fig. 3. Cylindrical column load cell.

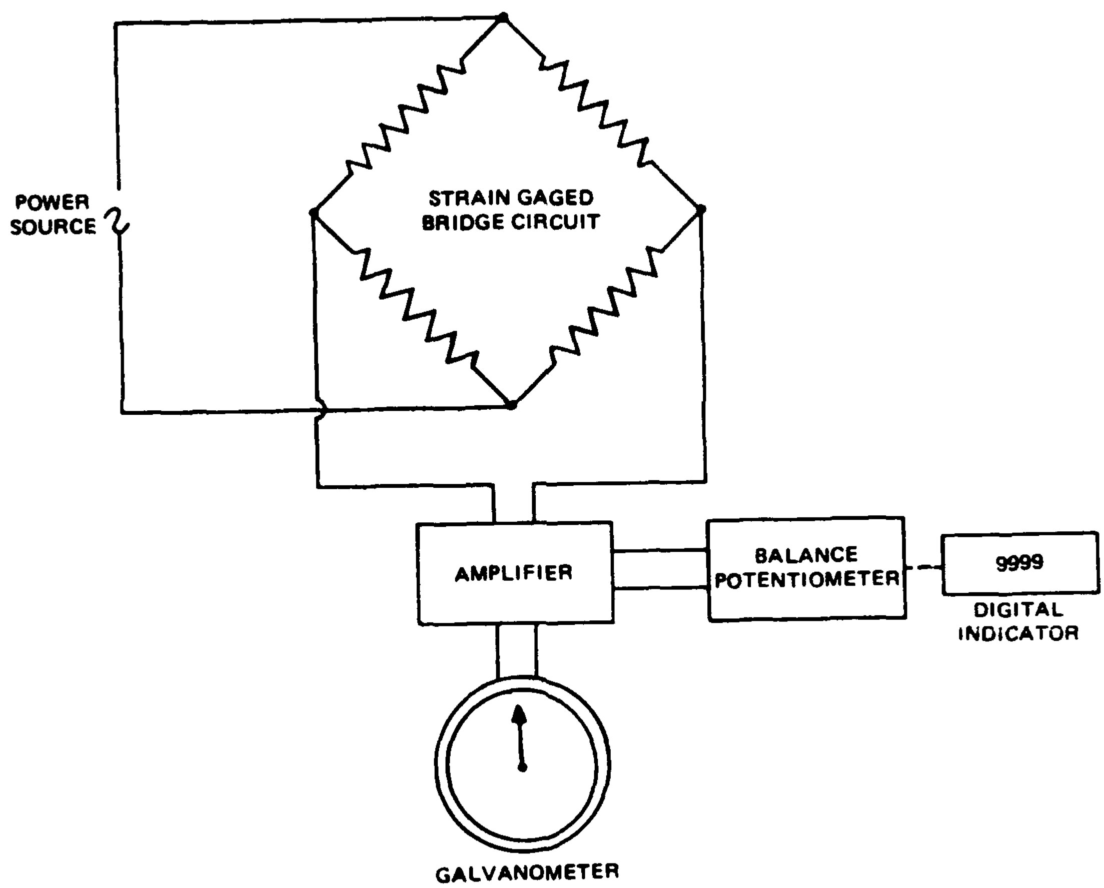


Fig. 4. Typical strain gage readout unit.

correction factors to obtain true load readings, and good temperature stability over a wide temperature range, thus allowing the load cells to be used in changing environmental conditions. The readout equipment is small, compact, and generally suitable for field use. The disadvantages of the bonded strain-gaged load cells are the extreme care required in waterproofing to prevent electrical leakage in the gaged circuit, the gage bonding technique required to assure long-term stability of the load cell, and the recalibration required when changing cable lengths due to lead wire load resistance changes and parasitic electrical signals.

2.1.3.2 The vibrating-wire load cell is generally constructed similar to the bonded strain gage cell. The load cells differ in the method of measuring the axial strain of the cell body. The vibrating-wire strain gage consists of a length of steel wire stretched between two posts extending from the cylinder wall (Fig. 5). The wire is pre-tensioned below its elastic limits when installed on the cell body. An electromagnet is placed near the wire, providing a method of plucking the wire when an electrical pulse excites the magnet. This pulse causes the wire to vibrate over the magnetic coil. The vibrating wire induces an electrical current in the electromagnet coil with a frequency equal to the frequency of the vibrating wire. The signal is then amplified by the readout unit and the frequency is determined by a frequency counter. As the load cell is subjected to load, the strain in the cylinder body reduces the tension on the vibrating wire, changing its frequency. This change in frequency per unit load is used to calibrate the load cell. Most load cells contain at least three vibrating-wire transducers placed at 120 deg around the periphery of the cell body. The vibrating wires are read separately and the readings averaged. This method of reading reduces errors caused by eccentric loading of the load cell. Some advantages of the vibrating-wire load cell are that the loads are read as a frequency, thus reducing the problems caused by ground leakage and poor signal cable condition. Long signal cables should not effect the frequency readings as long as the signal is not attenuated beyond the sensitivity of the readout equipment. The vibrating-wire load cell could also be read through radio telemetry systems, eliminating the need for an analog to frequency converter. Some of the disadvantages of the vibrating-wire load cells are their physical size, cost of manufacturing, poor temperature compensation, expensive and complicated readout equipment, and vulnerability to shock damage, causing zero shifts in load cell readings.

2.1.3.3 Load cells utilizing the unbonded strain gage have not found wide usage in load cell manufacturing. The unbonded strain gage employs the same principle as the bonded gage in that it consists of a wire made of a resistive material that, when strained, changes its resistance in proportion to the strain. The unbonded strain gage has more commonly been used in soil and concrete stress meters such as manufactured by the Carlson Company. The gages are mounted similar to the vibrating-wire gage and are generally employed in a bridge utilizing the same readout principles as the bonded gage (Fig. 6).

2.1.4 Photoelastic Load Cell - The photoelastic load cell consists of a cylindrical steel column with a hole drilled through its center diameter. A photoelastic element (optical glass) is inserted in this hole and locked in place. When polarized light passes through the optic glass, interference fringes can be observed if viewed through a polarizing filter. The number of fringes observed depends on the amount of stress in the optic glass. The load cell is calibrated by counting the interference fringes produced by a given load. Although this type of load cell is quite rugged and is a comparatively simple device, limited use has been made of it due to its coarse calibration and inability to be read from a remote location.

2.1.5 Manufacturers - Although there are a large number of companies manufacturing load cells for various applications, most of this equipment is not suitable for rock mechanics instrumentation.

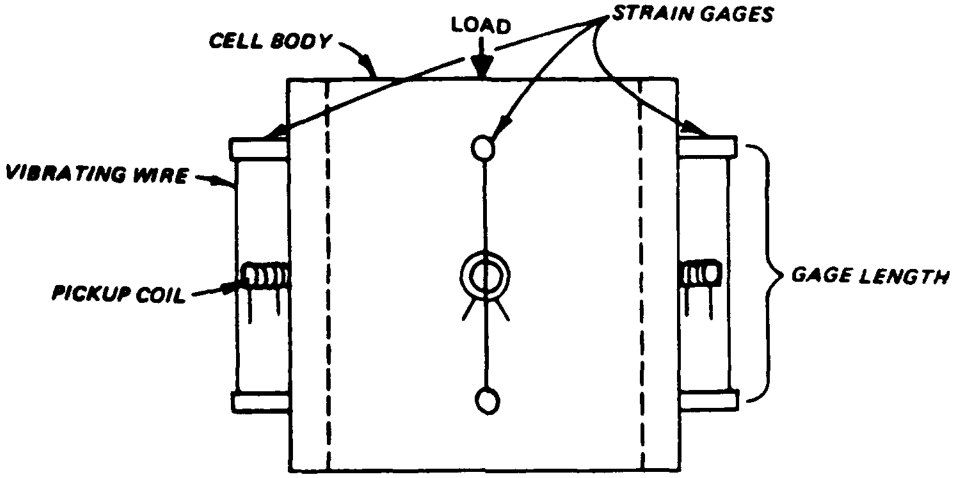


Fig. 5. Vibrating-wire load cell.

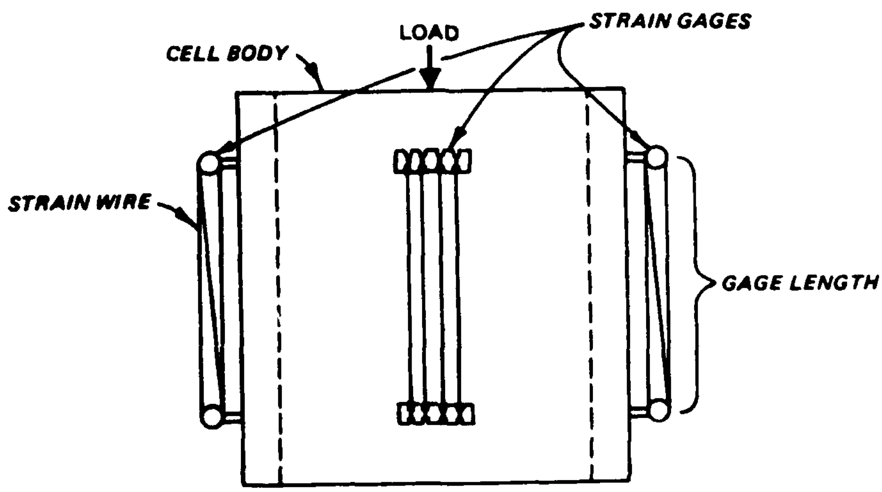


Fig. 6. Unbonded strain gage load cell.

The following is a partial list of geotechnical instrumentation suppliers that deal with the various types of load cells previously described:

Hydraulic load cells

Terrametrics, PJJ Machine Co., Interfels

Mechanical load cells

Terrametrics, Interfels, Norseman, Procep, Doboku Sokki, Strain sert

Strain-gaged load cells

Terrametrics, Soil Test, Telamac, Maihok, Geonor, Remote Systems

Photoelastic load cells

Terrametrics, Stress Engineering

3. Procedure

3.1 Although load cells have been employed for many diversified applications, the primary function in the field of rock mechanics has been for tunnel support load monitoring and rock bolt tension measurements. The following uses and methods of installation are typical for most applications.

3.1.1 Steel Arch Tunnel Support Instrumentation - The structural steel arch support set is the most commonly used tunnel support system. Measurements of the compressive loads actually being supported by the arch support provide a direct means of comparing actual loads with assumed support design loads. Load cells are generally installed under the base plates of arch-type sets; however, at times, it is desirable to include a crown load cell. In areas of squeezing or swelling ground invert struts may be used with load cells placed in them to measure the side loads. Close attention to the placement of blocking should be observed to assure proper load distribution on the arch supports (Fig. 7). The load cells should be installed on the set at the time of the set placement. It is desirable to place the instrumented sets as close to the blasting face as practical to measure the entire load history. Care must be taken to afford blast protection for the load

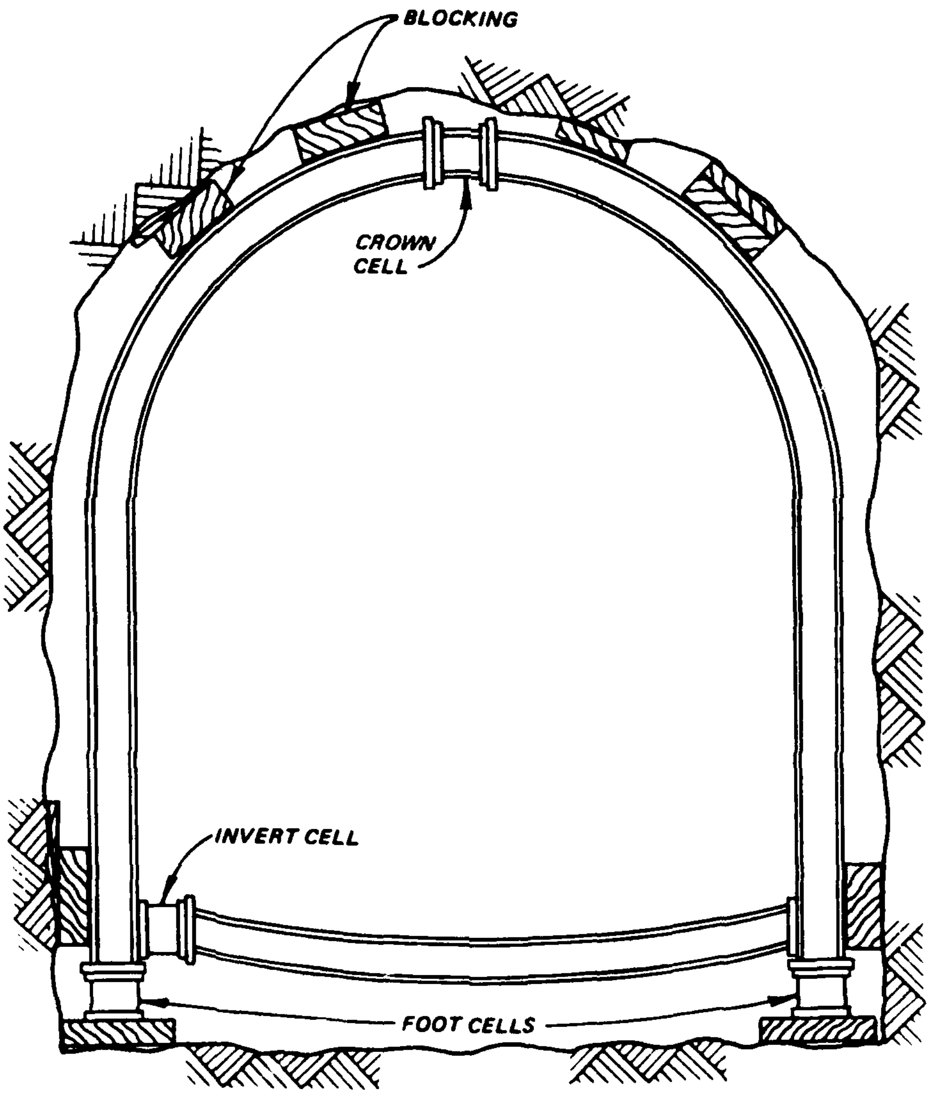


Fig. 7. Arch support load cell placement.

cells and signal cables when used near the blast face. This can be accomplished by running the signal cable inside the set flange facing away from the .unnel face. Steel shrouds can be welded to the sets to protect the load cells from fly rock (Fig. 8). The load cells selected should have a capacity greater than the yield strength of the arch supports. It is generally desirable to instrument at least three consecutive sets to reduce errors caused by anisotropy of the rock mass or nonuniform blocking on individual sets. Provisions should be made for the removal of the load cells after their portion of the tunnel has stabilized. This allows the load cells to be reused in a leap frog fashion, resulting in considerable instrumentation savings. Methods for removal of load cells on arch and circular sets are shown in Figs. 9 and 10. The load values obtained during a systematic instrumentation program provide a quantitative basis for reviewing the structural tunnel lining requirements for the final tunnel bore.

3.1.2 Rock Bolts and Tieback System Instrumentation - Rock bolts and tiebacks are often used to stabilize subsurface and surface excavations. In either case, the system usually incorporates steel rods or cables anchored at the base of a drill hole and tensioned to produce a compressive load along the axis of the drill hole. The actual loads acting on the bolt can be monitored (Fig. 11) by using a hollow core load cell acting as a washer at the collar of the bolt assembly. Calibrated torque wrenches have been widely used in rock bolting to produce the desired tension in the anchor tendon. Extensive tests have proven that such torque measurements can produce errors in the bolt tension as much as one to two times the indicated load. This variation can be caused by the condition of the threads on the bolt or anchor, dirt, rust, bending of the bolt, or anchor misalignment. Hydraulic jacks are often used in place of the torque wrench, especially in bolts or cables requiring high tensile loads. This system, where applicable, is far superior to the torque wrench but requires the use of specialized equipment for its adaptation. Actual loads can be monitored, using load cells, at the

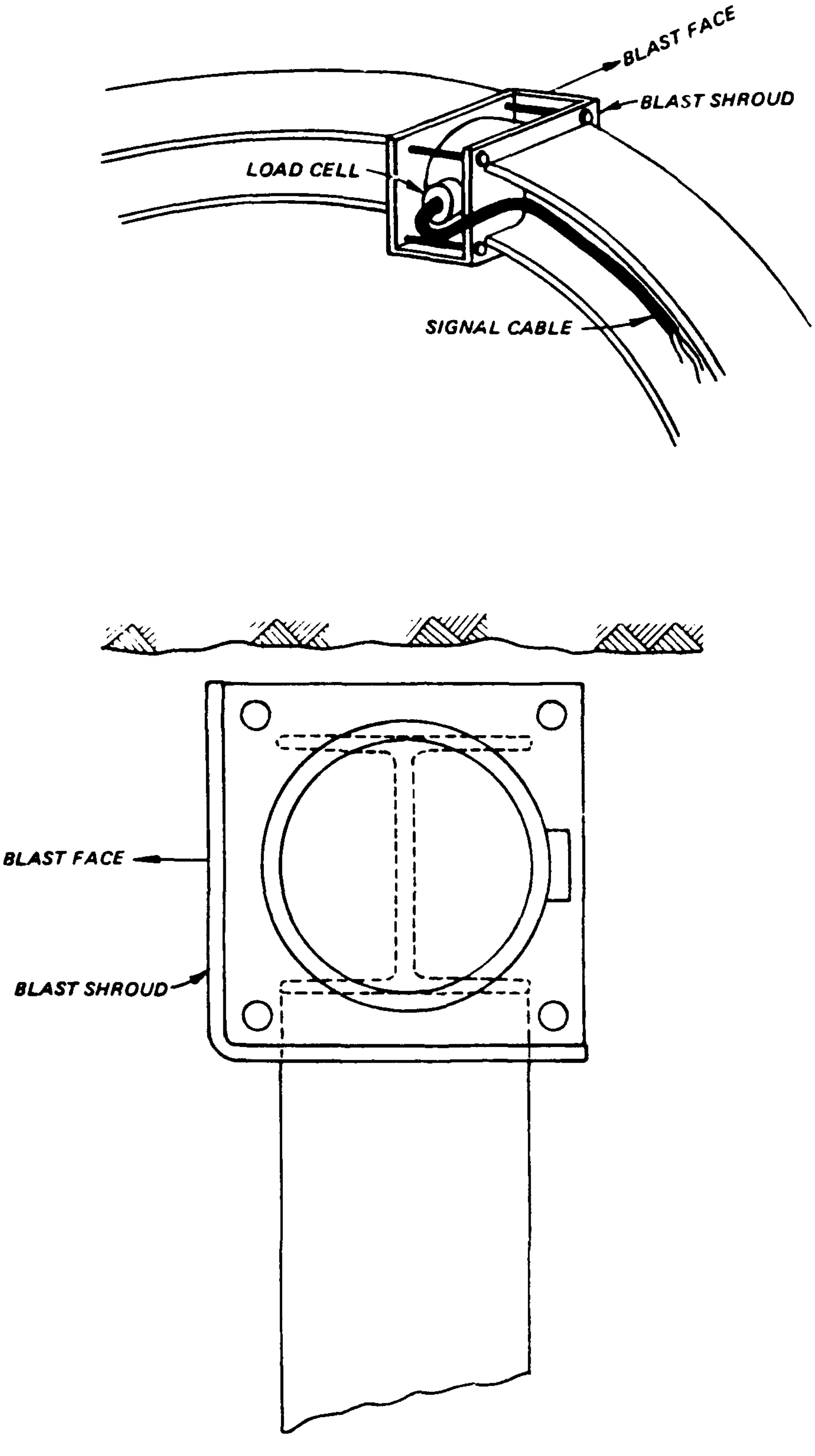


Fig. 8. Crown load cell blast protection

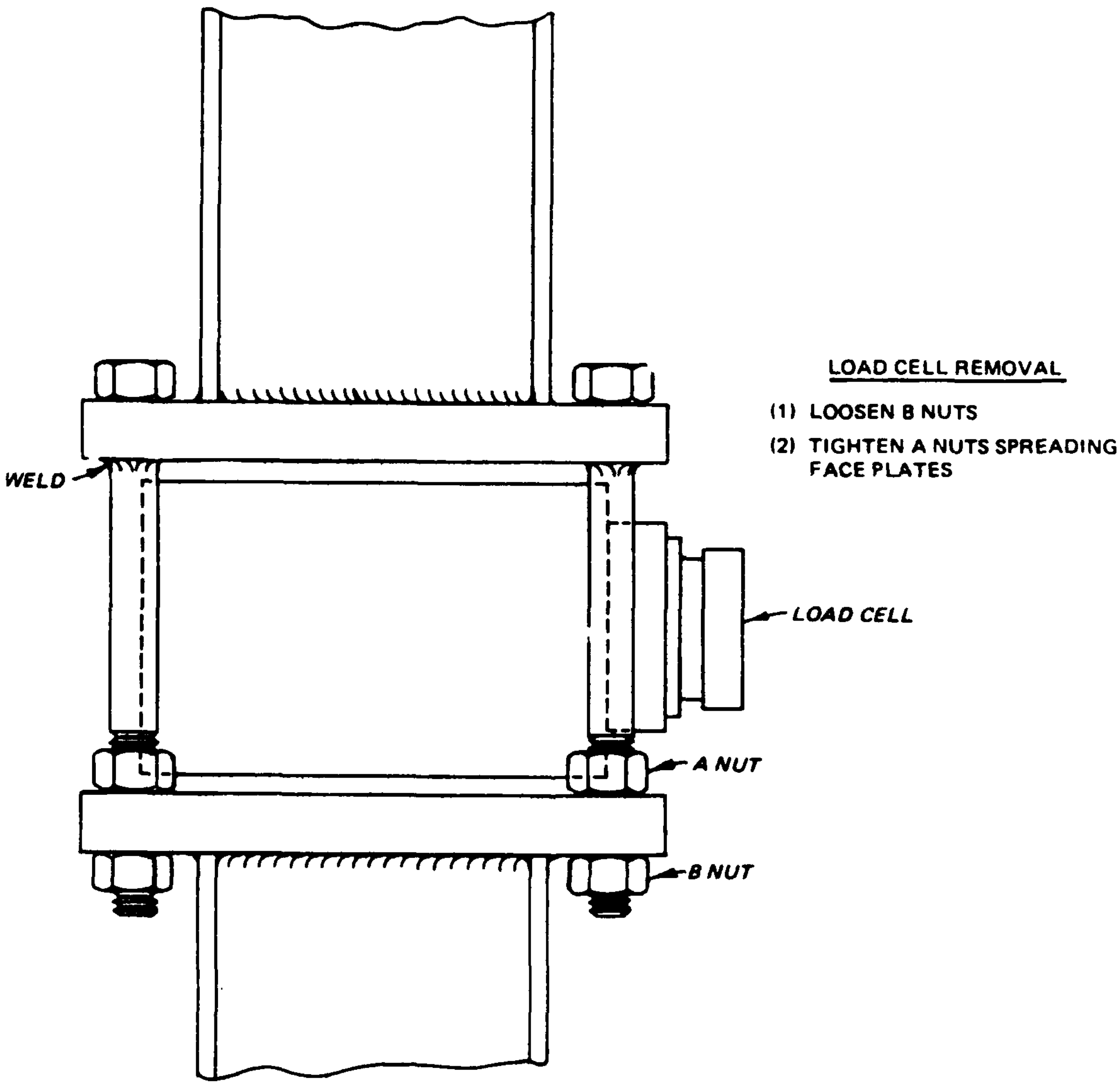


Fig. 9. Crown or springline load cell removal.

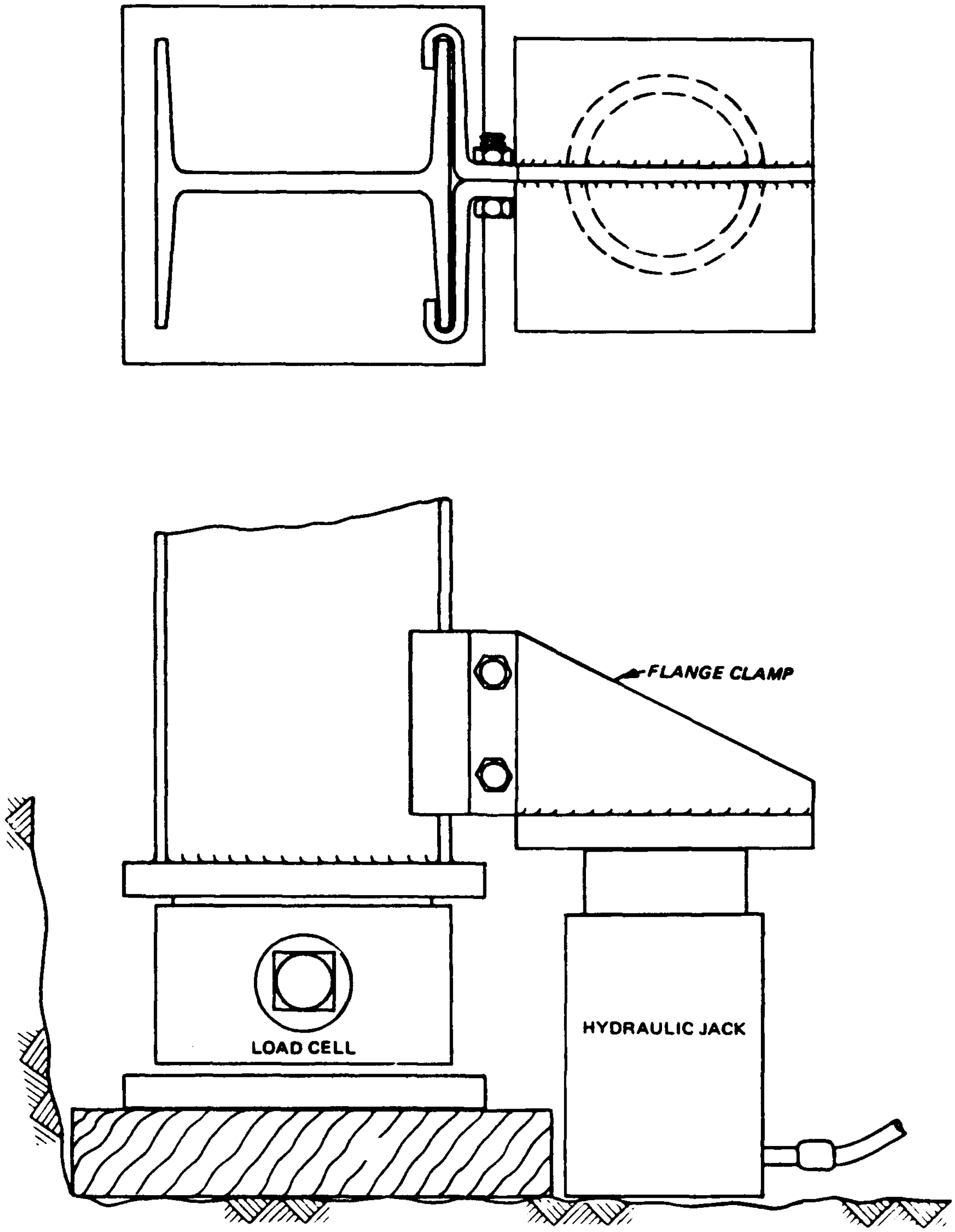


Fig. 10. Foot load cell removal.

time of installation and thereafter to plot the load history of the bolt system. It is desirable to use a number of load cells over a tunnel length equivalent to 2-1/2 tunnel diameters to obtain the average load being supported by the rock bolts. The load cell capacity should be greater than the yield strength of the rock bolts because the strain induced into the bolts at the time of blasting often exceeds their yield point. After tunnel stabilization the load cells can be removed and reused as the tunnel advances. The cells should be removed one at a time and the bolt retensioned to maintain stability of the tunnel section.

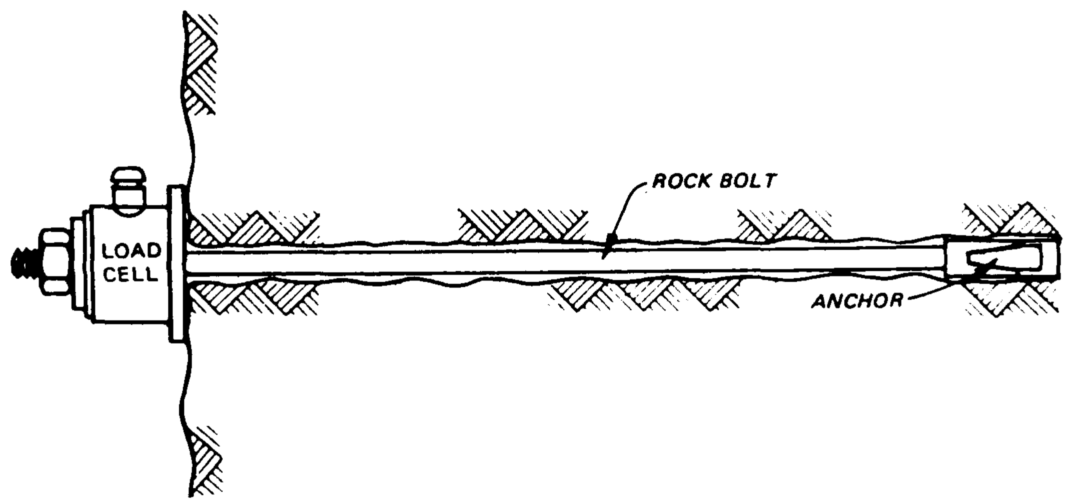


Fig. 11. Rock bolt load cell installation.

4. Data Reduction

4.1 Steel Arch Supports - As the tunnel excavation advances, the load normally being supported by the rock or soil removed in the excavated section is redistributed into the side walls and support system. At some point in time, depending on the tunnel advancement rate, material strength, and support system, the tunnel reaches a point of equilibrium. Support loads should be monitored during this stabilization period to determine support adequacy and future requirements.

Fig. 12 is a typical plot of load versus tunnel advancement. Note the decreasing frequency of readings as tunnel heading advances away from the load cells. The load is generally highest two to three diameters behind the tunnel face because the load has not stabilized or shifted to the side walls at this time. In this example the peak loading occurred when the tunnel heading was at station 130 (20 ft (6 m) or 2 tunnel diameters beyond the instrumental set) and equilibrium at station 110 was essentially achieved when the tunnel heading reached heading 148. A blocking diagram is often included with the data plot to help explain the stress distribution on the support member. A load versus time plot may also be used in some instances where varying time lags exist between tunnel heading advancements.

4.2 Rock Bolts - Rock bolt load data may be plotted in the same manner as the data for the arch support load cells; however, the indicated load should not decrease with time or tunnel advancement. By the use of the load plot the engineer can readily recognize anchor slippage, i.e., load loss or overload. A certain amount of anchor slippage is generally observed with bolts located near the blast face. When this occurs, the bolts should be retorqued to design specifications. The load plot will indicate stabilization versus tunnel advancement and can be used to determine a bolt torquing program for noninstrumented tunnel sections.

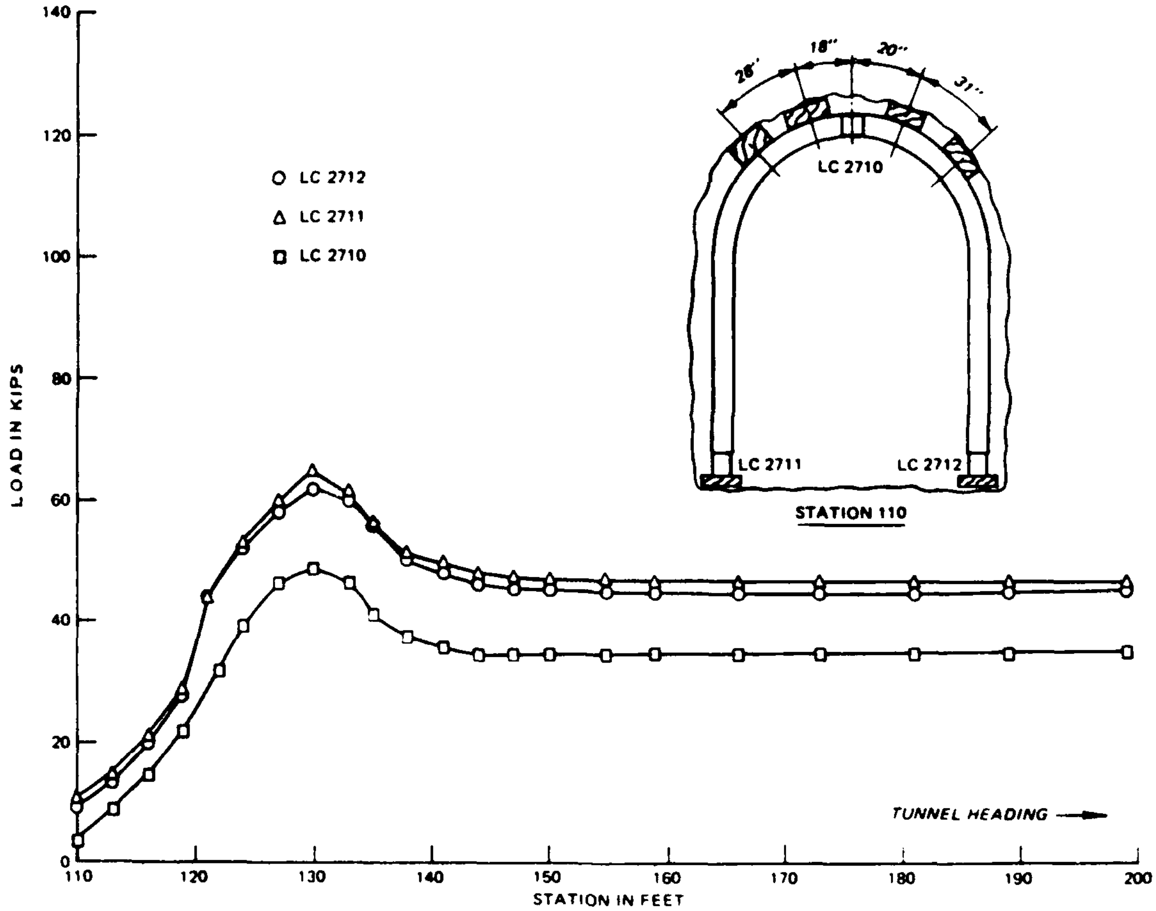


Fig. 12. Typical plot of load versus tunnel advancement.

BIBLIOGRAPHY

Able, J. F., "Tunnel Mechanics," Quarterly of the Colorado School of Mines, Vol 62, No. 2, 1967.

Cording, E. J., "Methods for Geotechnical Observations and Instrumentation in Tunneling," The National Science Foundation Research Grant GI-33644X, 1975.

Hartmann, B. E., "Rock Mechanics Instrumentation for Tunnel Construction," Terrametrics, Inc., Golden, Colorado, 1967.

Huie, J. S. and Lachel, D. J., "Warm Springs Project Instrumentation Program," U. S. Army Engineer Division, Missouri River, Omaha, Nebraska, 1973.

Lane, K. S., "Field Test Sections Save Cost in Tunnel Support," Underground Construction Research Council, Published by American Society of Civil Engineers, October 1975.

SUGGESTED METHOD OF DETERMINING ROCK BOLT TENSION USING A TORQUE WRENCH

1. Scope

1.1 This method can be used to apply a specified tension during rock bolt installation or to estimate loss of tension in a previously installed bolt. It can also be used to verify that anchor strength is greater than a specified value consistent with the maximum tension that can be applied with the wrench.

2. Apparatus

2.1 A torque wrench, preferably with a maximum applied torque indicator, capable of giving readings that are repeatable to 5 percent throughout the range of torques to be measured. It should be provided with sockets suitable for the nuts or bolt heads to be tested, should be used only for testing, and should be stored, together with its most recent calibration chart, in a dry place so as to preserve its accuracy of reading.

2.2 Equipment for calibrating the torque wrench (Fig. 1) including a rigidly fixed bolt head, a weight pan and weights, and a measuring tape.

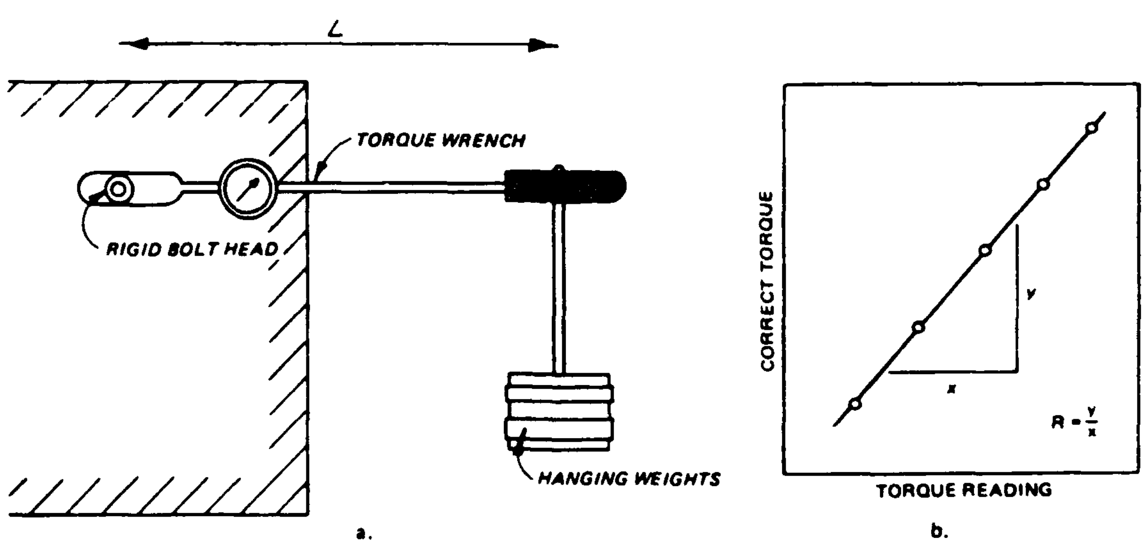


Fig. 1. Calibration of torque wrench.

2.3 Equipment for determining the relationship between tension and torque (Fig. 2), typically an installed rock bolt and faceplate assembly identical with that to be used in practice, and a hydraulic ram with handpump and pressure gage (to be used for tension measurement) or alternatively a rock bolt load cell. Tension should be measured with an accuracy better than 2 percent of the maximum reached in the test.

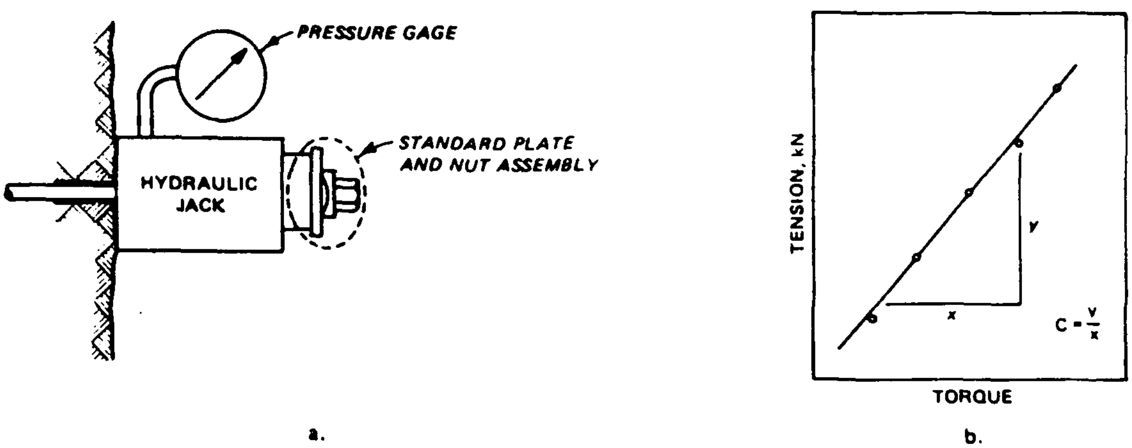


Fig. 2. Determination of ratio tension/torque.

3. Procedure

3.1 Calibration of the torque wrench should be accomplished as follows:

(a) With the wrench horizontal, the wrench socket is positioned on a rigid bolt head. A weight pan is suspended from the center of the wrench handle (Fig. 1) and weights are added. The torque reading is noted, also the weight of the pan together with the weights it contains. The procedure is repeated with increasing weights to obtain at least five torque readings covering the range of torques for which the wrench is to be used. The distance L between the center of the wrench handle and that of the bolt head is recorded.

(b) Correct torque values are calculated by multiplying the distance L by the applied weights. A graph is plotted of correct torque values against torque readings, and a straight line is fitted to the data points (Fig. 1b). The gradient of this line is measured, equal to the ratio R of correct torque divided by torque reading. Torque readings later obtained when using this wrench should be multiplied by the ratio R to obtain corrected values.

(c) Torque wrenches should be recalibrated at intervals not exceeding six months.

3.2 Determination of the ratio C of tension to torque is as follows:

(a) The load cell, or alternatively a hydraulic ram with the ram extended to 3/4 travel, is positioned concentrically and coaxially over the bolt to be tested, and the face nut is tightened to take up slack in the assembly (Fig. 2). Ram pressure should be increased to a nominally small value before the start of the test, and the pump valve firmly closed.

(b) The bolt diameter, state of lubrication, thread pitch, faceplate, and washers should be identical with the conditions expected in the actual rock bolt installation.

(c) Torque is applied in increasing increments to the nut, taking readings of torque and bolt tension. Torque application should be smooth and force should be applied through the center of the wrench handle only. At least five pairs of readings are required, covering the complete range of torques for which the wrench is to be used.

(d) A graph of tension versus torque is plotted, showing data points and a straight line fitted to these points. The gradient C of this line, the ratio of tension to torque, is measured (Fig. 2b).

(e) The ratio C is determined separately for each change in bolt diameter, thread pitch, and state of lubrication of for any other variation in the bolt/anchor/faceplace assembly that may result in a change in the tension/torque ratio.

3.3 Determination of bolt tension using the torque wrench is done as follows:

(a) If a torque wrench of the type that applies a preset torque is used, the torque setting should be increased in small increments until just sufficient to cause the face nut to rotate. The torque setting, the bolt identification, and the date are recorded.

(b) If a torque wrench with a maximum applied torque indicator is used, the torque may be applied steadily rather than in increments. Both types of torque wrench should be used with care to ensure that loading is smooth and that force is applied through the center of the wrench handle.

(c) Bolt tension is calculated using the correction R and a value of tension/torque ratio C determined for the identical bolt and faceplate assembly conditions using the method described in paragraph 3.2 above.

(d) An approximate check on minimum anchor strength may be obtained by applying an increasing torque, recording this torque as a function of number of rotations until no further torque can be applied, or until the anchor shows signs of failing.

4. Reporting of Results

4.1 The report should include diagrams and graphs as illustrated in Figs. 1 and 2, together with full details of:

(a) Torque wrench calibration; type of torque wrench, methods used for calibration, and results.

(b) Determination of the tension/torque ratio C; methods used and results obtained.

(c) The rock bolts tested; types, locations, dates installed, rock characteristics, methods used for drilling and installation, appearance, and condition of the faceplate assembly at time of testing.

(d) The method used for tension determination; tabulated values of bolt identification, applied torque to cause rotation of the nut, corresponding bolt tension, and any other observations pertinent to the test results.

4.2 If the method is used as a check on minimum anchor strengths, data should be included in the form of graphs of torque versus nut rotation, with scales converted to show bolt tension versus displacement. The report may compare these results with an arbitrary acceptable performance established by previous extensive testing. The complete bolt tension versus displacement curve should be considered when making such a comparison.

SUGGESTED METHOD FOR MONITORING ROCK BOLT TENSION USING LOAD CELLS

1. Scope

1.1 This method is for monitoring changes in tension that occur in a rock bolt over an extended period of time following installation (Note 1).

NOTE 1—Tension may fall below that applied at the time of rock bolt installation due to loosening and slip either of the anchor or of the faceplate assembly, for example as a result of rock creep, anchor corrosion, fretting of rock from beneath the faceplate, or blasting vibration. Tension may also either rise or fall as a result of bulk rock dilation or contraction associated with the progress of nearby excavation.

2. Apparatus

2.1 Rock bolt load cells should be used to monitor tension in approximately one bolt in ten of the support system to be studied (special considerations may require the instrumentation of a greater or lesser percentage of bolts). The load cells may, for example, be of mechanical, photoelastic, hydraulic, or electric type, depending on requirements of cost and accuracy. The cells should have reversible and preferably linear calibrations (see paragraph 3) and should incorporate a spherical seating or other provision to ensure that load transfer and measurement are reproducible. They should be capable of withstanding the effects of nearby blasting, water, and dust over long periods of time.

3. Procedure

3.1 Calibration of load cells should be accomplished as follows:

(a) Calibration is required when selecting a suitable type of load cell, and each load cell to be installed should be individually calibrated before use.

(b) Short-term calibration of each cell is performed in a testing laboratory by increasing the load in increments, taking readings of 'observed' and 'true' load values (Note 2). Tension is released and incremental readings are taken during unloading. A further cycle of loading and unloading is made, and a graph plotted showing data points and curves fitted to these points (Fig. 1).

NOTE 2—Details of the calibrating devices and their precision should be included in the report.

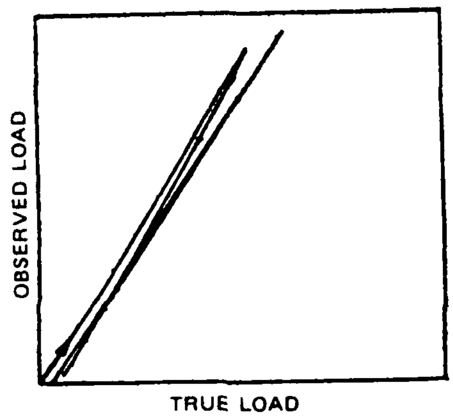


Fig. 1. Load cell calibration

(c) A check should also be made on the stability of readings over extended periods of time. Tension is increased to a value approximately equal to that to be measured on site, and is maintained at this value for as long a time as is practical. Any 'drift' in reading is noted. The effect of water on the cell should be observed, also the effect of coupling and uncoupling any electrical connections.

3.2 Installation and monitoring shall be as follows:

(a) Load cells are installed on selected rock bolts at the time of installation of the support system. Care should be taken to ensure that spherical seatings are correctly positioned and lubricated. Bolts that have been instrumented should be clearly and permanently numbered, and may be painted for ease of recognition. Bolt length, diameter, and type of anchor should also be noted.

(b) Tension readings should be taken immediately following installation and again a few hours later. Further readings may be taken at intervals depending on the rate at which readings are changing. In the vicinity of an advancing face, for example, intervals between readings should be of the order of hours, whereas if steady values are recorded for bolts in inactive areas the intervals between readings may be increased to days or weeks. Each reading should be accompanied by a record of bolt number, location, and the date and time of observation.

4. Calculations

4.1 Bolt tension readings are corrected using the calibration charts. Graphs of bolt tension versus time are plotted for each bolt (Fig. 2). For comparison, the loss or gain of tension may be reduced to a percentage of the initial installed value.

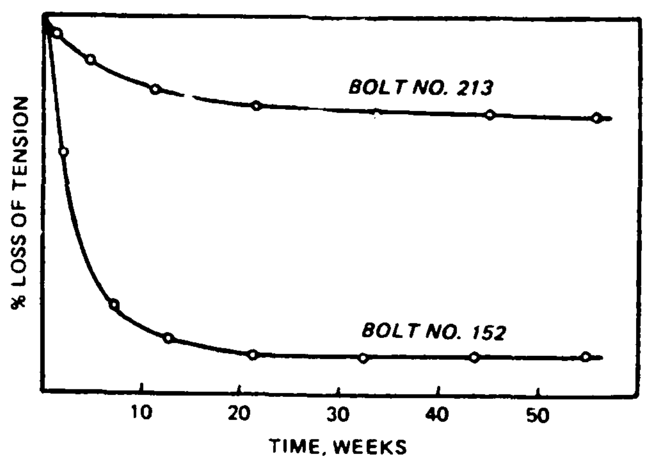


Fig. 2. Graph of bolt tension loss.

5. Reporting of Results

5.1 The report should include:

(a) Details of the load cells used, calibration methods, and results.

(b) Locations of the rock bolts monitored.

PART II. IN SITU STRENGTH METHODS

B. In Situ Strength Tests

SUGGESTED METHOD FOR IN SITU DETERMINATION OF DIRECT SHEAR STRENGTH

(International Society for Rock Mechanics)

1. Scope

1.1 This test measures peak and residual direct shear strengths as a function of stress normal to the sheared plane. Results are usually employed in limiting equilibrium analysis of slope stability problems or for the stability analysis of dam foundations (Notes 1-3).

NOTE 1—Direct shear strength can be determined in the laboratory (using the method described in RTH 203) if the plane to be tested is smooth and flat in comparison with the size of specimen, and if the specimen can be cut and transported without disturbance.

NOTE 2—Definitions (clarified in Figs. 5 and 6):

Peak shear strength - the maximum shear stress in the complete shear stress displacement curve.

Residual shear strength - the shear stress at which no further rise or fall in shear strength is observed with increasing shear displacement. A true residual strength may only be reached after considerably greater shear displacement than can be achieved in testing. The test value should be regarded as approximate and should be assessed in relation to the complete shear stress-displacement curve.

Shear strength parameters c and ϕ - respectively, the intercept and angle to the normal stress axis of a tangent to the shear strength-normal stress curve at a normal stress that is relevant to design (see Fig. 6).

NOTE 3—The measured peak strength can be applied directly to full-scale stability calculations only if the same type and size of roughness irregularities are present on the tested plane as on a larger scale. If this is not the case, the true peak strength should be obtained from the test data using appropriate calculations (for example, Patton, F. D., 1966, Proc. 1st Int. Cong. Rock Mech. ISRM, Lisbon, Vol 1, pp. 509-512;

Ladanyi, B. and Archambault, G., 1970. In "Rock Mechanics - Theory and Practice," (W.II. Somerton, ed.), AIME, New York, pp. 105-125; Barton, N. R., 1971, Proc. Symp. ISRM, Nancy, Paper 1-8).

1.2 The inclination of the test block and system of applied loads are usually selected so that the sheared plane coincides with a plane of weakness in the rock (e.g., a joint, plane of bedding, schistosity, or cleavage), or with the interface between soil and rock or concrete and rock (Note 4).

NOTE 4—Tests on intact rock (free from planes of weakness) are usually accomplished using laboratory triaxial testing. Intact rock can, however, be tested in direct shear if the rock is weak and if the specimen block encapsulation is sufficiently strong.

1.3 A shear strength determination should preferably comprise at least five tests on the same test horizon with each specimen tested at a different but constant normal stress.

1.4 In applying the results of the test, the pore water pressure conditions and the possibility of progressive failure must be assessed for the design case as they may differ from the test conditions.

2. Apparatus

2.1 Equipment for cutting and encapsulating the test block, rock saws, drills, hammer and chisels, formwork of appropriate dimensions and rigidity, expanded polystyrene sheeting or weak filler, and materials for reinforced concrete encapsulation.

2.2 Equipment for applying the normal load (e.g., Fig. 1) including:

(a) Flat jacks, hydraulic rams, or dead load of sufficient capacity to apply the required normal loads (Note 5).

NOTE 5—If a dead load is used for normal loading, precautions are required to ensure accurate centering and stability. If two or more hydraulic rams are used for either normal or shear loading, care is needed to ensure that they are identically matched and are in exact parallel alignment. Each ram should be provided with a spherical seat. The travel of rams and particularly of flat jacks should be sufficient to accommodate the full anticipated specimen displacement. A normal displacement of ±5-10 mm may be expected, depending on the clay content and roughness of the shear surface.

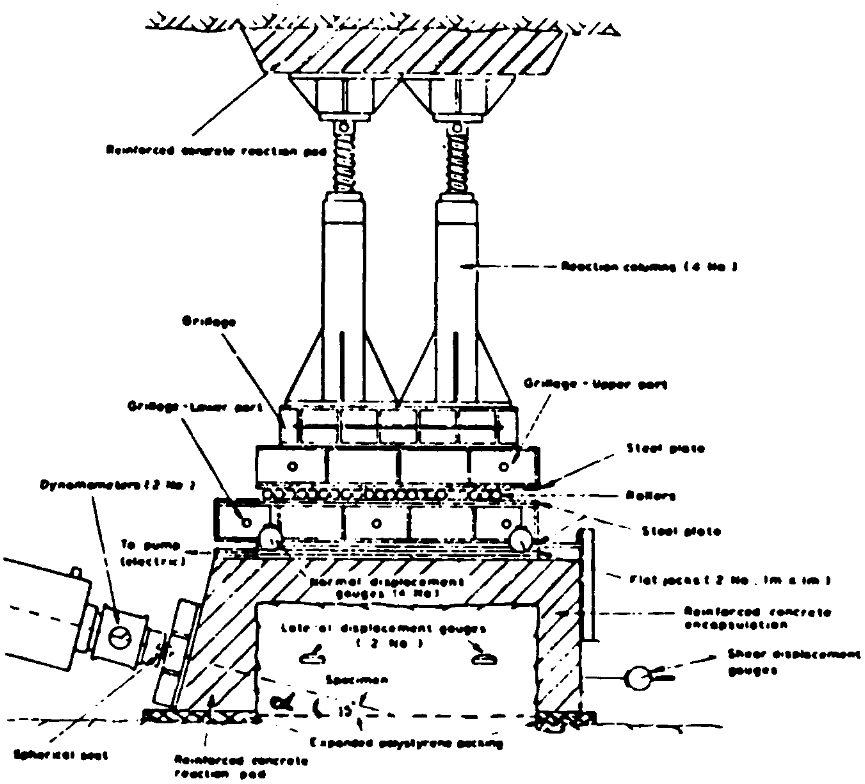


Fig. 1. Typical arrangement of equipment for in situ direct shear test.

(b) A hydraulic pump if used should be capable of maintaining normal load to within 2 percent of a selected value throughout the test.

(c) A reaction system to transfer normal loads uniformly to the test block, including rollers or a similar low friction device to ensure that at any given normal load, the resistance to shear displacement is less than 1 percent of the maximum shear force applied in the test. Rock anchors, wire ties, and turnbuckles are usually required to install and secure the equipment.

2.3 Equipment for applying the shear force (e.g. Fig. 1) including:

(a) One or more hydraulic rams (see Note 5) jacks of adequate total capacity with at least 70-mm travel.

(b) A hydraulic pump to pressurize the shear force system.

(c) A reaction system to transmit the shear force to the test block. The shear force should be distributed uniformly along one face of the specimen. The resultant line of applied shear forces should pass through the center of the base of the shear plane (Note 6) with an angular tolerance of ±5 deg.

NOTE 6--The applied shear force may act in the plane of shearing so that the angle α is 0 (Fig. 1). This requires a cantilever bearing member to carry the thrust from the shear jacks to the specimen. If a method is used where the shear force acts at some distance above the shear plane, the line of action of the shear jacks should be inclined to pass through the center of area of the shear plane. The angle for a specimen 700 by 700 by 350 mm approximates to 15 deg depending on the thickness of encapsulation. Tests where both shear and normal forces are provided by a single set of jacks inclined at greater angles to the shear plane are not recommended, as it is then impossible to control shear and normal stresses independently.

2.4 Equipment for measuring the applied forces including one system for measuring normal force and another for measuring applied shearing force with an accuracy better than ±2 percent of the maximum forces reached in the test. Load cells (dynamometers) or flat jack pressure measurements may be used. Recent calibration data applicable to the range of testing should be appended to the test report. If possible, the gages should be calibrated both before and after testing.

2.5 Equipment for measuring shear normal, and lateral displacements:

(a) Displacements should be measured (e.g., using micrometer dial gages (Note 7)) at eight locations on the specimen block or encapsulating material, as shown in Fig. 2.

NOTE 7—The surface of encapsulating material is usually insufficiently smooth and flat to provide adequate reference for displacement gages, and glass plates may be cemented to the specimen block for this purpose. These plates should be of adequate size to accommodate movement of the specimen. Alternatively, a tensioned wire and pulley system with gages remote from the specimen can be used. The system as a whole must be reliable and conform with specified accuracy requirements. Particular care is needed in this respect when employing electric transducers or automatic recording equipment.

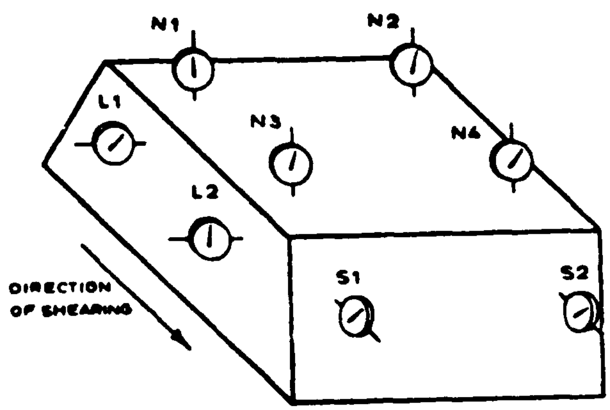


Fig. 2. Arrangement of displacement gages (S1 and S2 for shear displacement L1 and L2 for lateral displacement, N1-N4 for normal displacement.

(b) The shear displacement measuring system should have a travel of at least 70 mm and an accuracy better than 0.1 mm. The normal and lateral displacement measuring systems should have a travel of at least 20 mm and an accuracy better than 0.05 mm. The measuring reference system (beams, anchors, and clamps) should, when assembled, be sufficiently rigid to meet these requirements. Resetting of gages during the test should be avoided if possible.

3. Procedure

3.1 Preparation:

(a) The test block is cut to the required dimensions (usually 700 x 700 x 350 mm) using methods that avoid disturbance or loosening of the block (Notes 8 and 9). The base of the test block should coincide with the plane to be sheared and the direction of shearing should correspond if possible to the direction of anticipated shearing in the full-scale structure to be analyzed using the test results. The block and particularly the shear plane should, unless otherwise specified, be retained as close as possible to its natural in situ water content during preparation and testing, e.g., by covering with saturated cloth. A channel approximately 20 mm deep by 80 mm wide should be cut around the base of the block to allow freedom of shear and lateral displacements.

NOTE 8--A test block size of 700 x 700 x 350 mm is suggested as standard for in situ testing. Smaller blocks may be permissible, for example if the surface to be tested is relatively smooth; larger blocks may be needed when testing very irregular surfaces. The size and shape of test block may for convenience be adjusted so that faces of the block coincide with natural joints or fissures; this minimizes block disturbance during preparation. Irregularities that would limit the thickness or emplacement of encapsulation material or reinforcement should be removed.

NOTE 9--It is advisable, particularly if the test horizon is inclined at more than 10-20 deg to the horizontal, to apply a small normal load to the upper face of the test specimen while the sides are cut, to prevent premature sliding and also to inhibit relaxation and swelling. The load, approximately 5-10 percent of that to be applied in the test, may for example be provided by screw props or a system of rock bolts and crossbeams and should be maintained until the test equipment is in position.

(b) A layer at least 20 mm thick of weak material (e.g. foamed polystyrene) is applied around the base of the test block, and the remainder of the block is then encapsulated in reinforced concrete or similar material of sufficient strength and rigidity to prevent collapse or significant distortion of the block during testing. The encapsulation formwork should be designed to ensure that the load bearing faces of the encapsulated block are flat (tolerance ±1 mm) and at the correct inclination to the shear plane (tolerance ±2 deg).

(c) Reaction pads, anchors, etc., if required to carry the thrust from normal and shear load systems to adjacent sound rock, must be carefully positioned and aligned. All concrete must be allowed time to gain adequate strength prior to testing.

3.2 Consolidation:

(a) The consolidation stage of testing is to allow pore water pressures in the rock and filling material adjacent to the shear plane to dissipate under full normal stress before shearing. Behavior of the specimen during consolidation may also impose a limit on permissible rate of shearing (see paragraph 3.3(c)).

(b) All displacement gages are checked for rigidity, adequate travel, and freedom of movement, and a preliminary set of load and displacement readings is recorded.

(c) Normal load is then raised to the full value specified for the test, recording the consequent normal displacements (consolidation) of the test block as a function of time and applied loads (Figs. 3 and 4).

DIRECT SHEAR TEST DATA SHEET

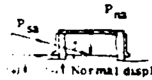
Client: Project Concrete Dam Location: Alcântara Loc. No.: Block No.:

TEST BLOCK SPECIFICATION See drawings & photographs Nos.:

General rock description, index properties and water conditions
Phyllite, sound to moderately weathered Normal conditions

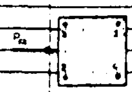
Description and index properties of surface to be sheared Type:
Dip. Dip direction; Roughness; Filling & alteration.
Persistance; Spacing of set; Surface dimensions; 0.70 x 0.70 Initial area A: 0.490 m^2

FORCES



$P_n = P_{na} + P_{sa} \sin\alpha$
$P_s = P_{sa} \cos\alpha$
$\sigma_n = P_n / A$
$\tau = P_s / A$
$\alpha = 15°$ (nominal)

1 Time elapsed (min)	2 Applied normal force P_n Reading	Force (kN)	3 Normal displacement Δ_n Reading 1	2	3	4	Average (mm)	4 Applied shear force P_s Reading	Force (kN)	5 Shear displacement Δ_s Reading 1	2	Average (mm)	6 Contact area A (corrected) m^2	7 P_{na} (kN)	8 σ_n (MPa)	9 P_{sa} (kN)	10 τ (MPa)
10		198	0.100	0.070	0.130	0.070			0	0	0	-	0.490	196		0	
35		233	0.130	0.065	0.140	0.090			137	0.05	0.05	0.05		•		142	
48		270	0.050	0.065	0.285	0.290			275	0.55	0.35	0.45		•		284	
64		306	-0.200	0.010	0.435	0.495			412	1.35	1.10	1.22		•		426	
87		343	-0.710	-0.205	0.600	0.720			548	2.55	2.30	2.42		•		568	
109		380	-1.165	-0.445	0.680	0.850			686	3.90	3.50	3.70		•		710	
131		417	-1.675	-0.615	0.710	0.970			824	5.15	4.60	4.88		•		853	
154		453	1.965	-0.745	0.720	1.050			961	6.10	5.50	5.80		•		995	
172		490	-2.245	-0.880	0.720	1.105			1098	7.20	6.50	6.85		•		1137	
189		527	-2.480	-1.055	0.695	1.165			1235	8.20	7.40	7.80		-		1279	
206		564	-2.750	-1.205	0.640	1.165			1373	9.45	8.45	8.95		•		1421	
234		601	-3.075	-1.505	0.465	1.100			1510	11.00	10.00	10.50		•		1563	
252		637	-3.350	-1.830	0.280	0.910			1647	12.45	11.40	11.92		•		1705	
264		674	-3.675	-2.185	0.050	0.720			1764	14.00	12.80	13.40		•		1847	
278		711	-4.005	-2.685	-0.290	0.360			1922	15.55	14.40	14.98		•		19[illegible]9	
289		748	-4.585	-3.125	-0.890	-0.020			2059	17.60	16.45	17.02		•		2132	
293	Rupture	784	-4.975	-3.375	-1.250	-0.290			2196	20.00	19.55	19.78		•		2274	



Calibration data

Remarks

Tested by: Dates:
Checked by: Date:

Fig. 3. Example layout of direct shear test data sheet.

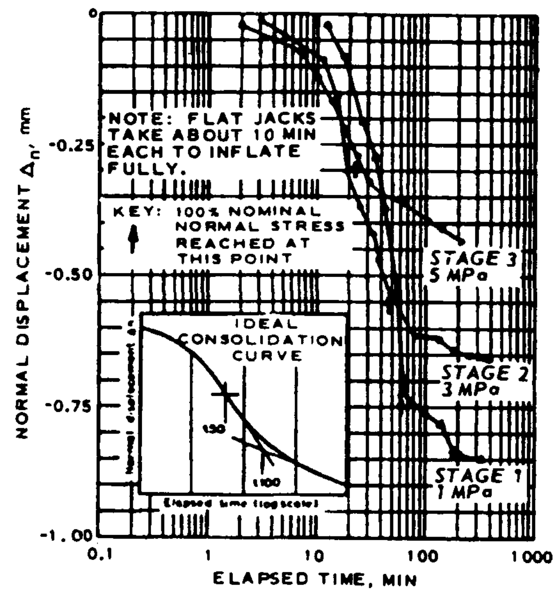


Fig. 4. Consolidation curves for a three-stage direct shear test, showing the construction used to estimate t_{100}.

(d) The consolidation stage may be considered complete when the rate of change of normal displacement recorded at each of the four gages is less than 0.05 mm in 10 minutes. Shear loading may then be applied.

3.3 Shearing:

(a) The purpose of shearing is to establish values for the peak and residual direct shear strengths of the test horizon. Corrections to the applied normal load may be required to hold the normal stress constant; these are defined in paragraph 4.5.

(b) The shear force is applied either in increments or continuously in such a way as to control the rate of shear displacement.

(c) Approximately 10 sets of readings should be taken before reaching peak strength (Figs. 3 and 5). The rate of shear displacement should be less than 0.1 mm/min in the 10-minute period before taking a set of readings. This rate may be increased to not more than 0.5 mm/min between sets of readings provided that the peak strength itself is adequately recorded. For a 'drained' test, particularly when testing clay-filled discontinuities, the total time to reach peak strength should exceed 6 t_{100} as determined from the consolidation curve (see paragraph 4.1 and Fig. 4) (Note 10). If necessary, the rate of shear should be reduced on the application of later shear force increments delayed to meet this requirement.

NOTE 10—The requirement that total time to reach peak strength should exceed 6 t_{100} is derived from conventional soil mechanics consolidation theory (for example Gibson and Henkel, Geotechnique 4, p 10-11, 1954) assuming a requirement of 90 percent pore water pressure dissipation. This requirement is most important when testing a clay-filled discontinuity. In other cases it may be difficult to define t_{100} with any precision because a significant proportion of the observed "consolidation" may be due to rock creep and other mechanisms unrelated to pore pressure dissipation. Provided the rates of shear specified in the text are followed, the shear strength parameters may be regarded as having been measured under conditions of effective stress ("drained conditions").

(d) After reaching peak strength, readings should be taken at increments of from 0.5-5 mm shear displacement as required to adequately define the force-displacement curves (Fig. 5). The rate of shear displacement should be 0.02-0.2 mm/min in the 10-minute period before a set of readings is taken and may be increased to not more than 1 mm/min between sets of readings.

(e) It may be possible to establish a residual strength value when the specimen is sheared at constant normal stress and at least four consecutive sets of readings are obtained which show not more than 5 percent variation in shear stress over a shear displacement of 1 cm (Note 11).

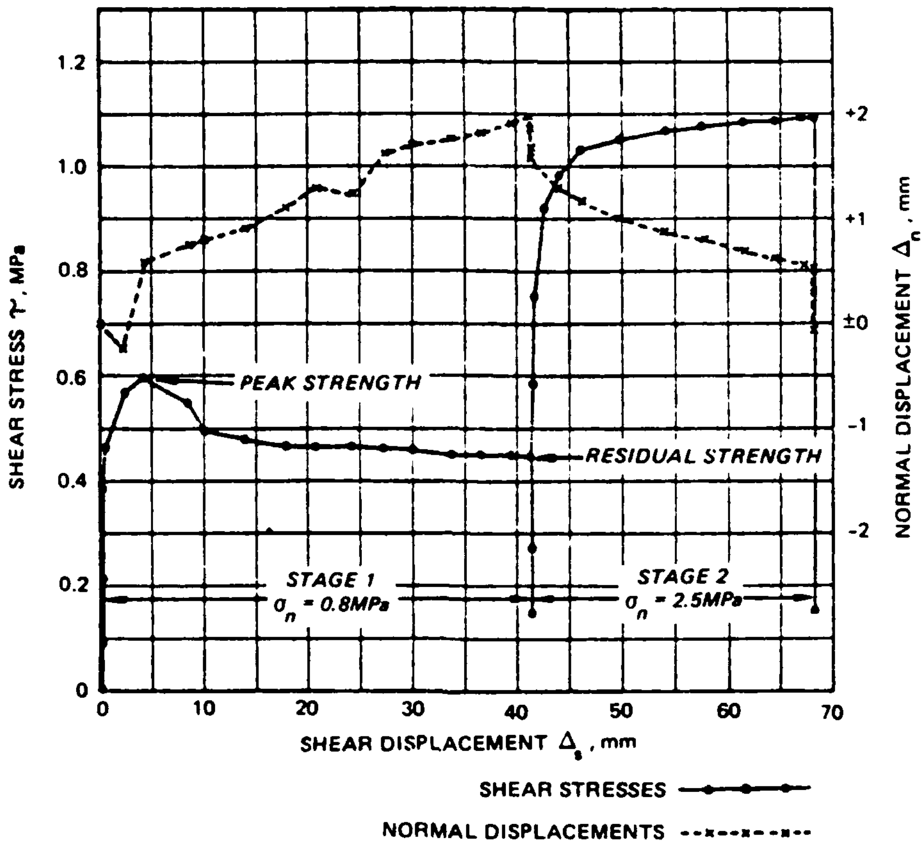


Fig. 5. Shear stress - displacement graphs.

NOTE 11—An independent check on the residual friction angle should be made by testing in the laboratory two prepared flat surfaces of the representative rock. The prepared surfaces should be saw-cut and then ground flat with No. 80 silicon carbile grit.

(f) Having established a residual strength, the normal stress may be increased or reduced (Note 12) and shearing continued to obtain additional residual strength values. The specimen should be reconsolidated under each new normal stress (see paragraph 3.2(d)) and shearing continued according to criteria given in 3.3(c) to 3.3(e).

NOTE 12--The normal load should when possible be applied in increasing rather than decreasing stages. Reversals of sh ar direction or resetting of the specimen block between normal load stages, sometimes used to allow a greater total shear displacement than would otherwise be possible, are not recommended because the shear surface is likely to be disturbed and subsequent results may be misleading. It is generally advisable, although more expensive, to use a different specimen block

(g) After the test, the block should be inverted, photographed in color, and fully described (see paragraph 5.1). Measurements of the area, roughness, dip, and dip direction of the sheared surface are required, and samples of rock, infilling, and shear debris should be taken for index testing.

4. Calculations

4.1 A consolidation curve (Fig. 4) is plotted during the consolidation stage of testing. The time t_{100} for completion of "primary consolidation" is determined by constructing tangents to the curve as shown. The time to reach peak strength from the start of shear loading should be greater than 6 t_{100} to allow pore pressure dissipation (see Note 10).

4.2 Displacement readings are averaged to obtain values of mean shear and normal displacements Δ_s and Δ_n. Lateral displacements are recorded only to evaluate specimen behavior during the test, although if appreciable they should be taken into account when computing corrected contact area.

4.3 Shear and normal stresses are computed as follows:

$$\text{Shear stress} \quad \tau = \frac{Ps}{A} = \frac{Psa \cos\alpha}{A}$$

$$\text{Normal stress} \quad \sigma_n = \frac{Pn}{A} = \frac{Pna + Psa \sin\alpha}{A}$$

where Ps = total shear force; Pn = total normal force

Psa = applied shear force; Pna = applied normal force

α = inclination of the applied shear force to the shear plane (if $\alpha = 0$, $\cos\alpha = 1$ and $\sin\alpha = 0$).

A = area of shear surface overlap (corrected to account for shear displacement)

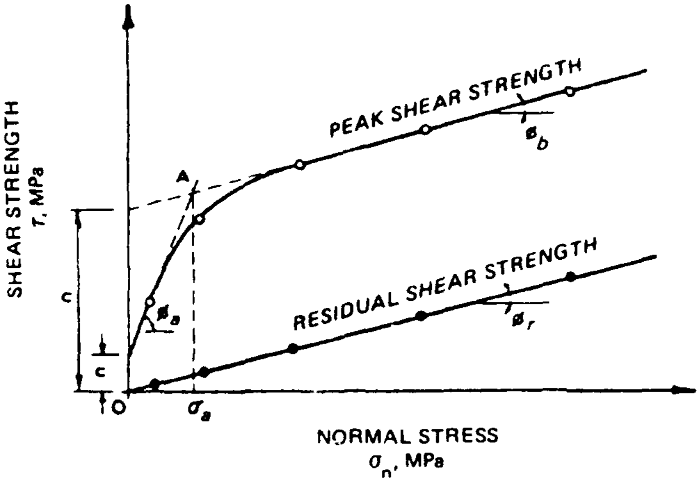
If α is greater than zero, the applied normal force should be reduced after each increase in shear force by an amount Psa sin α in order to maintain the normal stress approximately constant. The applied normal force may be further reduced during the test by an amount

$$\frac{\Delta_s \text{ (mm)} \cdot Pn}{700}$$

to compensate for area changes.

4.4 For each test specimen graphs of shear stress (or shear force) and normal displacement versus shear displacement are plotted (Fig. 5), annotated to show the nominal normal stress and any changes in normal stress during shearing. Values of peak and residual shear strengths and the normal stresses and shear and normal displacements at which these occur are abstracted from these graphs (Note 2).

4.5 Graphs of peak and residual shear strengths versus normal stress are plotted from the combined results for all test specimens. Shear strength parameters ϕ_a, ϕ_b, ϕ_r, c', and c are abstracted from these graphs as shown in Fig. 6.



ϕ_r = residual friction angle

ϕ_a = apparent friction angle below stress σ_a; point A is a break in the peak shear strength curve resulting from the shearing off of major irregularities on the shear surface. Between points O and A, ϕ_a will vary somewhat; measure at stress level of interest. Note also that $\phi_a = \phi_u + i$, where ϕ_u is the friction angle obtained for smooth surfaces of rock on rock and angle i is the inclination of surface asperities.

Fig. 6. Shear strength - normal stress graph.

ϕ_b = apparent friction angle above stress level σ_a (Point A); note that ϕ_a will usually be equal to or slightly greater than ϕ_r and will vary somewhat with stress level; measure at the stress level of interest^r.

c' = cohesion intercept of peak shear strength curve; it may be zero.

c = apparent cohesion at a stress level corresponding to ϕ_b.

5. Reporting of Results

5.1 The report should include the following:

(a) A diagram, photograph, and detailed description of test equipment and a description of methods used for specimen preparation and testing. (Reference may be made to this "suggested method," stating only departures from the prescribed techniques).

(b) For each specimen, a full geological description of the intact rock, sheared surface, filling, and debris preferably accompanied by relevant index test data (e.g., roughness profiles and Atterberg limits, water content, and grain-size distribution of filling materials).

(c) Photographs of each sheared surface together with diagrams giving the location, dimensions, area, dip, and dip direction and showing the directions of shearing and any peculiarities of the blocks.

(d) For each test block, a set of data tables, a consolidation graph, and graphs of shear stress and normal displacement versus shear displacement (e.g. Figs. 3, 4, and 5). Abstracted values of peak and residual shear strengths should be tabulated with the corresponding values of normal stress and shear and normal displacement.

(e) For the shear strength determination as a whole, graphs and tabulated values of peak and residual shear strengths versus normal stress, together with derived values for the shear strength parameters (e.g. Fig. 6).

RTH 323-80

SUGGESTED METHOD FOR DETERMINING THE STRENGTH OF A ROCK BOLT ANCHOR (PULL TEST)

(International Society for Rock Mechanics)

1. Scope

1.1 This test is intended to measure the short-term strength of a rock bolt anchor installed under field conditions (Note 1). Strength is measured by a pull test in which bolt head displacement is measured as a function of the applied bolt load to give a load-displacement curve. The test is usually employed for selection of bolts and also for control on the quality of materials and installation methods (Note 2).

NOTE 1—It is essential to test anchors under realistic field conditions. It is, however, permissible to select safe and convenient test locations provided that the rock and the installation methods are identical with those encountered in full-scale utilization of the bolts. If the rock is schistose for example, test holes should be drilled at the same angle to the schistosity as anticipated for bolt utilization. If rock conditions are variable, the rock should be classified and tests conducted in rock of each class.

NOTE 2—The test is intended to measure anchor performance and this is possible only if the bolt, threads, nuts, and other components are stronger than the anchor. In some circumstances it may be desirable to reinforce the bolt or thread for purposes of anchor evaluation. Otherwise, if the bolt is consistently weaker than the anchor, it may be preferable to replace the field test with quality control of bolts and other components in a testing laboratory. Laboratory control testing may also be required as a supplement to field testing for evaluation of components, e.g. for their corrosion resistance, quality of materials, and consistency of dimensions.

1.2 At least five tests are required to evaluate an anchor in a given set of rock and installation conditions. The tests are destructive and should not in general be made on bolts that form part of the actual rock support system.

2. Apparatus

2.1 Equipment for installing the test anchors, including:

(a) Equipment for drilling and cleaning the drillhole, conforming to the manufacturers' specifications for optimum performance of the anchor provided that these are compatible with field conditions (Note 3).

NOTE 3--Manufacturers' specifications for hole dimensions and method of installation should be checked for compatibility with site operational limitations before testing and if compatible should be closely followed in the tests.

(b) Equipment for inspection and measurement of the drillhole, anchors, and bolts, e.g., a lamp, steel tape, internal and external calipers, and equipment for measuring the quantity of grout if used.

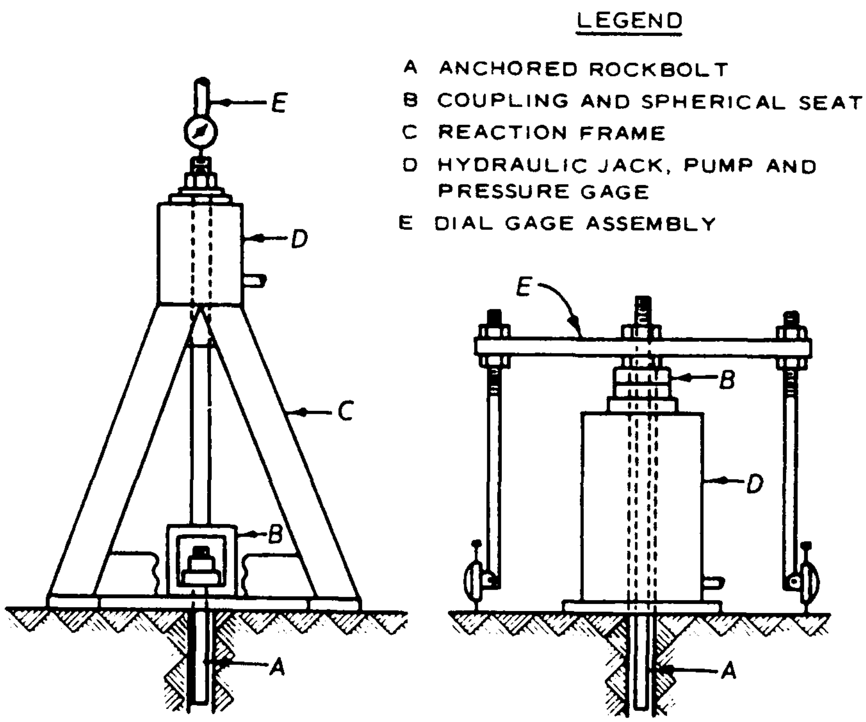
(c) Standard rock bolt assemblies as supplied by manufacturers of the bolts including anchors to be tested, grout and materials for grout injection if required, and equipment for installing the bolts in the manner recommended by the manufacturers (Note 3).

2.2 Equipment for applying the bolt load, e.g. as in Fig. 1, including:

(a) A hydraulic jack with hand pump and pressure hose capable of applying a load greater than the strength of both the anchor and the bolt to be tested and with travel of at least 50 mm.

(b) Equipment for transferring the load from the jack to the bolt (Note 4). A spherical seating, bevelled washers, and/or wedges under the jack are required to ensure that the applied load is coaxial with the bolt.

NOTE 4--Some types of anchors must essentially be tensioned during their installation, and these must be tested using a suitable coupling unit and bridging framework to carry load from the jack to the bolt (Fig. 1a). Whenever possible, however, anchors should be tested without pretensioning of the bolt, in which case a center-hole jack installed over the bolt may be used (Fig. 1b). The arrangement shown in Fig. 1a may also be used to test selected anchors in an operational support system at some time after their installation, provided this does not endanger the support as a whole. The percentage of initially applied bolt tension remaining at the time of test may be estimated from the load required to just loosen the faceplate and washers.



a. b.

Fig. 1. Rock bolt testing equipment.

2.3 Equipment for measuring load and displacement, including:

(a) A load measuring device, e.g., a load cell or a hydraulic pressure gage connected to the pump and calibrated in load units. Measurement should be accurate to 2 percent of the maximum load reached in the test. The device should include a maximum load indicator.

(b) Equipment for measuring the axial displacement of the bolt head (travel at least 50 mm and accurate to 0.05 mm (Note 5)). For example, a single dial gage measuring directly onto the bolt head may be used; alternatively, the displacement may be obtained as an average from two or three gages spaced equidistant from the bolt as shown in Fig. 1b.

NOTE 5--When testing anchors that in operation are intended to provide a reaction for external loads (e.g. holding anchors for cranes, suspension cables), the test equipment should be designed so that no test reaction forces are applied closer than one bolt length from the anchor drillhole.

3. Procedure

3.1 Site Preparation:

(a) The test site or sites are selected to ensure that rock conditions are representative of those in which the bolts are to operate (see Note 1).

(b) Holes are drilled as specified and at locations convenient for testing (Notes 1 and 3). The rock face surrounding each hole should be firm and flat and the hole should be perpendicular to the face (±5 deg).

(c) Drillholes and anchor materials are inspected before installation to ensure that they conform to specifications. Preliminary data, e.g., the measured dimensions of the drillhole, bolt, and anchor and the type and condition of rock at the test location, are recorded on a data sheet (e.g. Fig. 2).

(d) Bolts are installed in the specified manner (Note 3), recording essential details such as the installation torque (if any) (Note 4) and the date and time of installation.

ROCK ANCHOR TEST RESULT SHEET TEST No.

Date of installation: ____________ Date of test ____________

PROJECT: ____________ Location: ____________

ANCHOR: Type ______ Length: ______ Installation Torque: ______

ROCK: Classification ______ Fracture spacing: ______ Strength ______

BOLT: Diameter: ______ Strength: ______ Length: ______ Untensioned length: ______

HOLE: Diameter: ______ Length: ______ Orientation & roughness: ______

Pump Pressure	Bolt tension	Displacement readings					REMARKS
		Reading	Displacement	Reading	Displacement	Average	

TEST RESULTS: Maximum pull force: ______

Displacement at maximum pull force: ______ Max. displacement in test: ______

Nature of failure or yield: ______

Other remarks: ______

TESTED BY: ______ CHECKED BY: ______

Fig. 2. Rock bolt test data sheet.

3.2 Testing:

(a) The loading equipment is assembled, taking care to ensure that the direction of pull is axial to the bolt, that the equipment sits firmly on the rock, and that no part of the bolt or grout column will interfere with the application or measurement of load during the test (Note 5).

(b) An initial arbitrary load not greater than 5 kN (500 kgf) is applied to take up slack in the equipment. The displacement equipment is assembled and checked (Note 6).

NOTE 6--The displacement measuring system should be securely mounted and dial gages should be located on firm flat rock; glass or metal plates can, if necessary, be cemented to the rock to provide smooth measuring surfaces perpendicular to the bolt. All measuring equipment must be checked and calibrated at regular intervals to ensure that the standards of accuracy required by this "suggested method" are maintained.

(c) The anchor is tested by increasing the load until a total displacement greater than 40 mm has been recorded, or until the bolt yields or fractures if this occurs first.

(d) Readings of load and displacement are taken at increments of approximately 5-kN (500-kgf) load or 5-mm displacement, whichever occurs first. The rate of load application should be in the range 5-10 kN/min. Readings are taken only after both load and displacement have stabilized. The times required for stabilization should be recorded.

4. Calculations

4.1 Total displacement values are computed as the test progresses by subtracting initial readings from the incremental readings, taking averages if more than one gage is used.

4.2 The test data are plotted graphically as shown in Fig. 3. Anchor strength, defined as the maximum load reached in the test provided that the bolt itself does not yield or fail, is recorded on this graph. If the bolt yields or fails, the load 'X' at which this occurs is recorded, and the anchor strength is specified as "unknown but greater than 'X'".

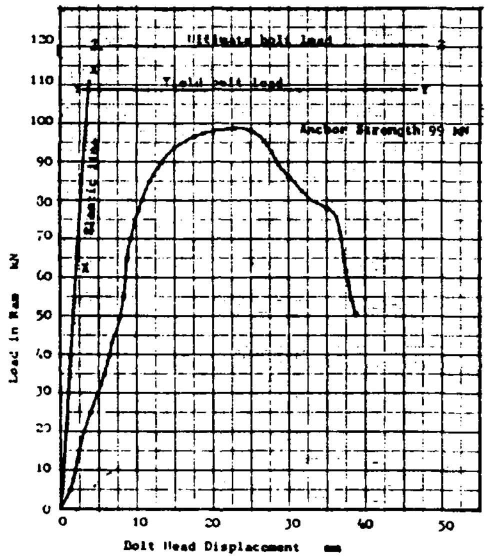


Fig. 3. Example of anchor test results graph.

4.3 The elastic elongation of the bolt at a given applied load may be calculated as

$$\text{Elongation at load P is equal to } \frac{P \cdot L}{A \cdot E}$$

where L is the tensioned ungrouted length of bolt + 1/3 the grouted length + length of extension bar used; A is the cross-sectional area of the bolt; and E is the modulus of elasticity of the bolt steel.

A straight line X-X is constructed to pass through this point and the origin of the load-displacement graph (Fig. 3). Straight lines Y-Y and Z-Z are constructed at the specified yield and ultimate loads of the bolt. Comparison of the actual test curve with these three lines allows independent assessment of anchor and bolt behavior.

4.4 For the evaluation of grouted anchors, the results of several tests should be abstracted and presented graphically to show the influence of grout cure time and bonded length on anchor strength (e.g. Fig. 4).

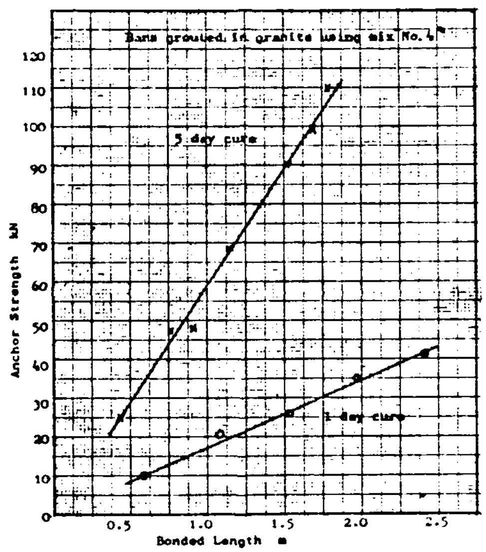


Fig. 4. Graph showing influence of bond length and cure time on the strength of anchors.

5. Reporting of Results

5.1 The report should include the data sheets and graphs illustrated in Figs. 2-4 together with full details of:

(a) Rock in which the anchors were tested.

(b) The anchors and associated equipment.

(c) The drillholes, including length, diameter, method of drilling, straightness, cleanness and dryness, and orientation.

(d) The method and time of installation.

(e) The method and time of testing.

(f) The nature of failure and other observations pertinent to the test results.

5.2 If required, the report may also compare performance of the anchors tested with an arbitrary acceptable performance established by previous extensive testing. Anchor strength, total displacements, and displacement per increment of load should be considered when making this comparison.

SUGGESTED METHOD FOR DEFORMABILITY AND STRENGTH DETERMINATION USING AN IN SITU UNIAXIAL COMPRESSIVE TEST

1. Scope

1.1 This method of test is intended to measure the strength and deformability of large in situ specimens of weak rock such as coal. The test results take into account the effect of both intact material behavior and the behavior of discontinuities contained within the specimen block.

1.2 Since the strength of rock is dependent on the size of the test specimen, it is necessary to test specimens by increasing size until an asymptotically constant strength value is found (Note 1). This value is taken to represent the strength of the rock mass. It can, for example, be applied to the design of mine pillars provided that the constraining effect of the roof and floor is taken into consideration.

NOTE 1—Bieniawski, Z. T. and Van Heerden, W. L., "The Significance of Large-Scale In Situ Tests," Int. J. Rock Mech. Min. Sci., Vol 1, 1975.

2. Apparatus

2.1 Preparation equipment, including:

(a) Equipment for cutting rectangular specimen blocks from existing underground mine pillars or exposed faces, e.g., a coal cutting machine, pneumatic chisel, and other hand tools. No explosives are permitted.

2.2 A loading system consisting of:

(a) Hydraulic jacks or flat jacks to apply a uniformly distributed load to the complete upper face of the specimen. The loading system should be of sufficient capacity and travel to load the specimen to failure.

(b) A hydraulic pumping system to supply oil at the required pressure to the jacks, the pressure being controlled to give a constant rate of displacement or strain rather than a constant rate of stress increase (Note 2).

NOTE 2—Experience has shown that deformation-controlled loading is preferable to stress-controlled loading because it results in a more stable and thus safer test. One way to achieve uniform deformation of the specimen is to use a separate pump for each jack and to set the oil delivery rate of each pump to the same value. Standard diesel fuel injection pumps have been found suitable and are capable of supplying pressures up to 100 MPa. The delivery rate of these pumps can be set very accurately.

2.3 Equipment to measure applied load and strain in the specimen, including:

(a) Load measuring equipment, e.g., electric, hydraulic, or mechanical load cells, to permit the applied load to be measured with an accuracy better than ±5 percent of the maximum in the test.

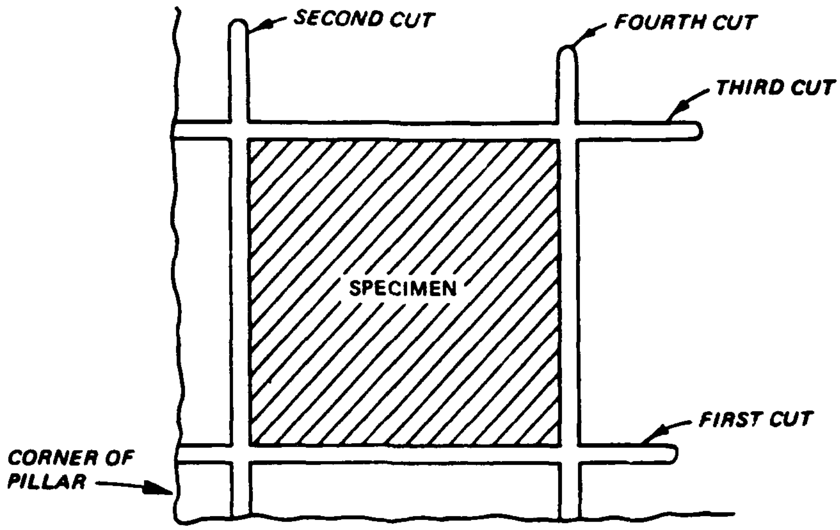
(b) Dial gage or similar displacement measuring devices with robust fittings to enable the instruments to be mounted so that the strain in the central third of each specimen face is measured with an accuracy better than $\pm 10^{-5}$. Strain is to be measured in the direction of applied load, also in a perpendicular direction if Poisson's ratio values are to be determined.

2.4 Equipment to calibrate the loading and displacement measuring systems, the accuracy of calibration to be better than the accuracies of test measurement specified above.

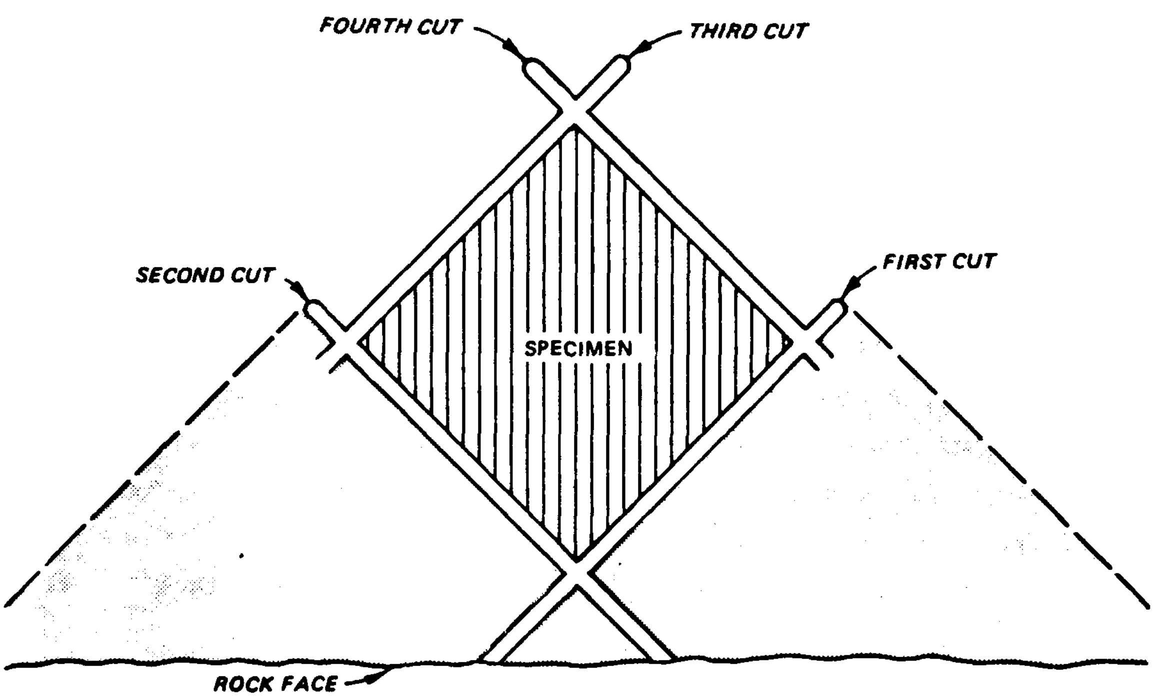
3. Procedure

3.1 Preparation:

(a) Specimens of the required dimensions (Note 3) are cut either from the corners of existing pillars or from exposed rock faces (Fig. 1). Loose and damaged rock is first removed. Vertical cuts are then made, e.g. as shown in Fig. 1, to form the vertical faces of



a. SEQUENCE OF VERTICAL CUTS TO SEPARATE
A SPECIMEN FROM THE CORNER OF A PILLAR



NOTE: ROCK IN THE SHADED REGIONS TO BE REMOVED
BEFORE MAKING THIRD AND FOURTH CUTS.

b. SEQUENCE OF VERTICAL CUTS TO SEPARATE
A SPECIMEN FROM A ROCK FACE

Fig. 1. Vertical cuts to separate specimen from the corner of a pillar and from a rock face.

the specimen. A horizontal cut is made to form the top face of the specimen. Loose rock is removed and the specimen trimmed to final size using hand tools.

NOTE 3—Specimen dimensions cannot be specified because these depend very much on the rock properties, e.g., the thickness of strata and the ease with which specimens can be prepared. It is recommended that a number of tests should be done with a specimen size of about 0.5 m and that the size of subsequent specimens should be increased until an asymptotically constant strength value is reached.

(b) The specimen is cleaned and inspected, recording in detail the geological structure of the block and of the reaction faces above and below. Specimen geometry, including the geometry of defects in the block, should be measured and recorded with an accuracy better than 5 mm. Photographs and drawings should be prepared to illustrate both geological and geometric characteristics.

(c) A concrete block, suitably reinforced, is cast to cover the top face of the specimen (Fig. 2). The thickness of this block should be sufficient to give adequate strength under the full applied load. The top face of the block should be flat to within ±5 mm and parallel to within ±5 deg with the basal plane of the block.

(d) Rock is removed from above the specimen to make space for the loading jacks, the rock being cut back to a stratum of sufficient strength to provide safe reaction. Generally, a concrete reaction pad must be cast to distribute the load on the roof and to prevent undue deformation and movement of the jacks during the test (Note 4). The lower face of the reaction block should be flat to within ±5 mm and parallel to within ±5 deg with the upper face of the specimen block. All concrete should be left to harden for a period of not less than 7 days.

NOTE 4—If a suitably designed concrete cap to the specimen is not employed, the corners and sides of the specimen will often fail before the central portion. The corner jacks will then cease to operate, and the test results will be suspect. The concrete cap should if possible be designed to ensure that the stress distributions in the top and bottom thirds of the specimen are nearly identical.

(e) The loading jacks and pumps are installed and checked to ensure that they operate as intended. Load and displacement measuring equipment is installed and checked. All measuring instruments should be calibrated both before and after each test series.

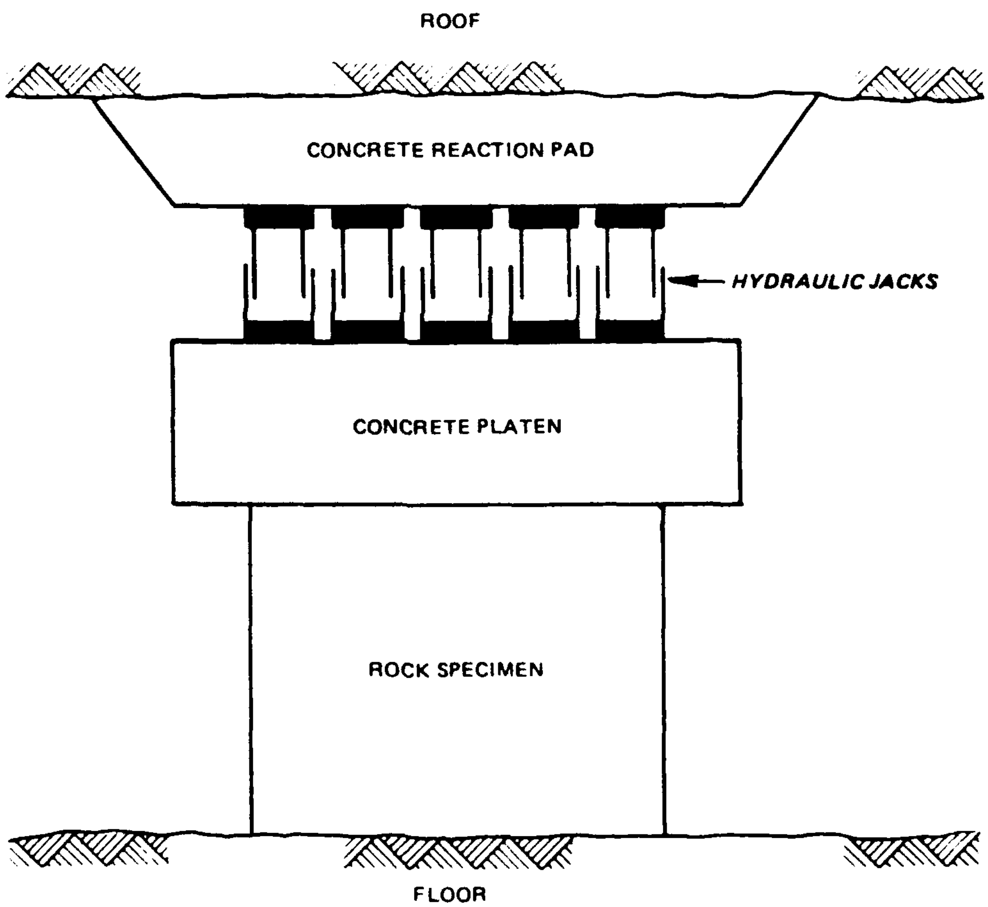


Fig. 2. Testing arrangement.

3.2 Testing:

(a) An initial load of approximately one-tenth of the estimated full test load is applied, and the jacks are checked to ensure that each is in firm contact with the specimen block. Displacement measuring equipment is again checked to ensure that it is rigidly mounted and is functioning correctly. Zero readings of load and displacement are taken.

(b) The specimen load is then increased by applying the same slow and constant oil delivery to each jack. The rate of specimen strain should be such that a displacement rate of between 5 and 15 mm per hour is recorded at each of the four faces of the specimen block.

(c) Readings of applied load and displacements are recorded at intervals such that the load-displacement or stress-strain curve can be adequately defined. There should be not less than ten points on this curve, evenly spaced from zero to the failure load.

(d) Unless otherwise specified, the test is to be terminated when the specimen fails. Specimen failure is indicated by a drop in hydraulic pressure to less than one-half the maximum applied, or by disintegration of the specimen to an extent that the loading system becomes inoperative or the test dangerous to continue. The mode of specimen failure is recorded, and a sketch is made of all failure cracks.

4. Calculations

4.1 The uniaxial compressive strength of the specimen shall be calculated by dividing the maximum load carried by the specimen during the test by the original cross-sectional area of the specimen.

4.2 Young's modulus for the specimen shall, unless otherwise specified, be calculated as the tangent modulus E_{t50} at one-half the uniaxial compressive strength. This modulus is found by drawing a tangent to the stress-strain curve at 50 percent maximum load, the gradient of this tangent being measured as E_{t50}. The construction and calculations used in deriving this and any other modulus values should be shown on the stress-strain curve.

4.3 If a number of specimens of different shape and/or size are tested, the trends in strength values due to shape and size effects should be plotted graphically, e.g., as shown in Fig. 3.

5. Reporting of Results

5.1 The report should include the following information:

(a) A diagram showing details of the locations of specimens tested, the specimen numbering system used, and the situation of each specimen with respect to the geology and geometry of the site.

(b) Photographs, drawings, and tabulations giving full details of the geological and geometrical characteristics of each specimen, preferably including index test data to characterize the rock. Particular attention should be given to a detailed description of the pattern of joints, bedding planes, and other discontinuities in the specimen block.

(c) A description, with diagrams, of the test equipment and method used. Reference may be made to this "suggested method," noting only the departures from recommended procedures.

(d) Tabulated test results, including recorded values of load and displacements together with all derived data, calibration results, and details of all corrections applied.

(e) Graphs showing load versus displacement or stress versus strain, including points representing all recorded data and a curve fitted to these points. The uniaxial compressive strength value should be shown, together with all constructions used in determining Young's modulus and other elastic parameters. The mode of specimen failure should be shown diagrammatically and described.

(f) Summary tables and graphs giving the values of uniaxial compressive strength and Young's modulus, and showing how these values vary as a function of specimen shape and size and the character of the rock tested.

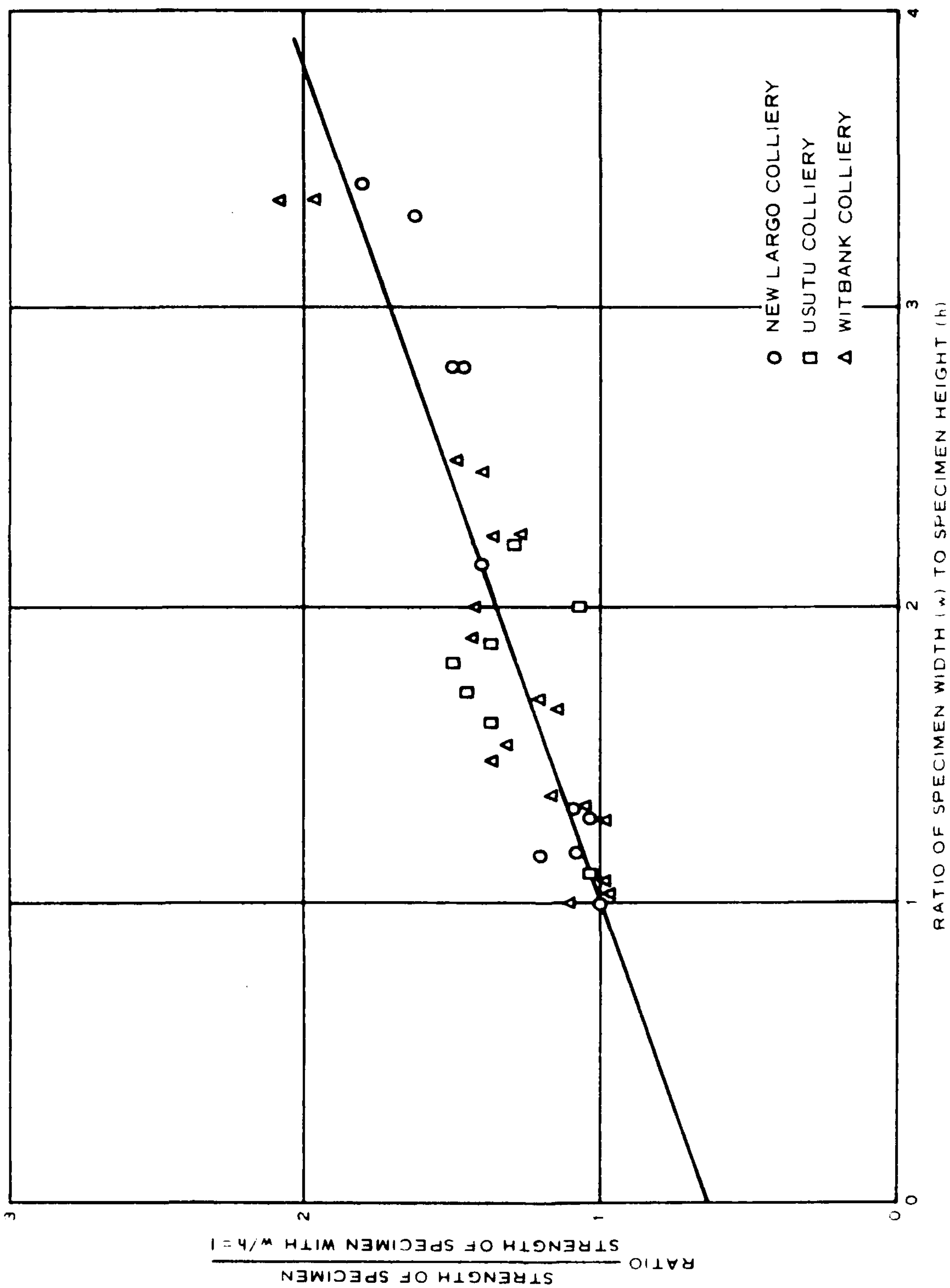


Fig. 3. Example showing the representation of strength data in dimensionless form.

RTH 325-89

Suggested Method for Determining Point Load Strength

SCOPE

1.(a) The Point Load Strength test is intended as an index test for the strength classification of rock materials. It may also be used to predict other strength parameters with which it is correlated, for example uniaxial tensile and compressive strength.[1]*

(b) The test measures the Point Load Strength Index ($I_{s(50)}$) of rock specimens, and their Strength Anisotropy Index ($I_{a(50)}$) which is the ratio of Point Load Strengths in directions which give the greatest and least values.

(c) Rock specimens in the form of either core (the *diametral* and *axial* tests), cut blocks (the *block* test), or irregular lumps (the *irregular lump* test) are broken by application of concentrated load through a pair of spherically truncated, conical platens.[2] Little or no specimen preparation is needed.

(d) The test can be performed with portable equipment or using a laboratory testing machine, and so may be conducted either in the field or the laboratory.

APPARATUS

2. The testing machine (Fig. 1) consists of a loading system (for the portable version typically comprising a loading frame, pump, ram and platens), a system for measuring the load P required to break the specimen, and a system for measuring the distance D between the two platen contact points (but see 5(e) below).

Loading system

3.(a) The loading system should have a platen-to-platen clearance that allows testing of rock specimens in the required size range. Typically this range is 15–100 mm so that an adjustable clearance is needed to accommodate both small and large specimens.

(b) The loading capacity should be sufficient to break the largest and strongest specimens to be tested.[3]

(c) The test machine should be designed and constructed so that it does not permanently distort during repeated applications of the maximum test load, and so that the platens remain co-axial within $\pm$ 0.2 mm throughout the testing. No spherical seat or other non-rigid component is permitted in the loading system. Loading system rigidity is essential to avoid problems of slippage when specimens of irregular geometry are tested.

(d) Spherically-truncated, conical platens of the standard geometry shown in Fig. 2 are to be used. The 60° cone and 5 mm radius spherical platen tip should meet tangentially.[3] The platens should be of hard material such as tungsten carbide or hardened steel so that they remain undamaged during testing.

Fig. 1. Photograph of portable point load test machine.

Load measuring system

4.(a) The load measuring system, for example a load cell or a hydraulic pressure gauge or transducer connected to the ram, should permit determination of the failure load P required to break the specimen and should conform to the requirements (b) through (d) below.

(b) Measurements of P should be to an accuracy of $\pm$ 5% P or better, irrespective of the size and strength of specimen that is tested.[4,5]

(c) The system is to be resistant to hydraulic shock and vibration so that the accuracy of readings is not adversely affected by repeated testing.

(d) Failure is often sudden and a maximum load indicating device is essential so that the failure load is retained and can be recorded after each test.

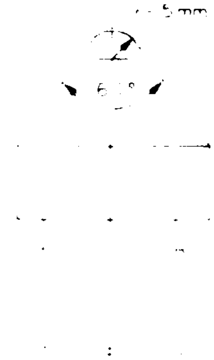


Fig. 2. Platen shape and tip radius.

* Superscript numbers refer to Notes at the end of the text.

Distance measuring system

5.(a) The distance measuring system, for example a direct reading scale or displacement transducer, is to permit measurement of the distance D between specimen-platen contact points and should conform with requirements (b) through (d) below.

(b) Measurements of D should be to an accuracy of $\pm 2\%$ D or better irrespective of the size of specimen tested.

(c) The system is to be resistant to hydraulic shock and vibration so that the accuracy of readings is not adversely affected by repeated testing.

(d) The measuring system should allow a check of the "zero displacement" value when the two platens are in contact, and should preferably include a zero adjustment.

(e) An instrument such as calipers or a steel rule is required, to measure the width W of specimens for all but the diametral test.

PROCEDURE

Specimen selection and preparation

6.(a) A test sample is defined as a set of rock specimens of similar strength for which a single Point Load Strength value is to be determined.

(b) The test sample of rock core or fragments is to contain sufficient specimens conforming with the size and shape requirements for diametral, axial, block or irregular lump testing as specified below.

(c) For routine testing and classification, specimens should be tested either fully water-saturated or at their natural water content.

Calibration

7. The test equipment should be periodically calibrated using an independently certified load cell and set of displacement blocks, checking the P and D readings over the full range of loads and displacements pertinent to testing.

The diametral test

8.(a) Core specimens with length/diameter ratio greater than 1.0 are suitable for diametral testing.

(b) There should preferably be at least 10 tests per sample, more if the sample is heterogeneous or anisotropic.

(c) The specimen is inserted in the test machine and the platens closed to make contact along a core diameter, ensuring that the distance L between the contact points and the nearest free end is at least 0.5 times the core diameter (Fig. 3a).

(d) The distance D is recorded $\pm 2\%$.

(e) The load is steadily increased such that failure occurs within 10–60 sec, and the failure load P is recorded. The test should be rejected as invalid if the

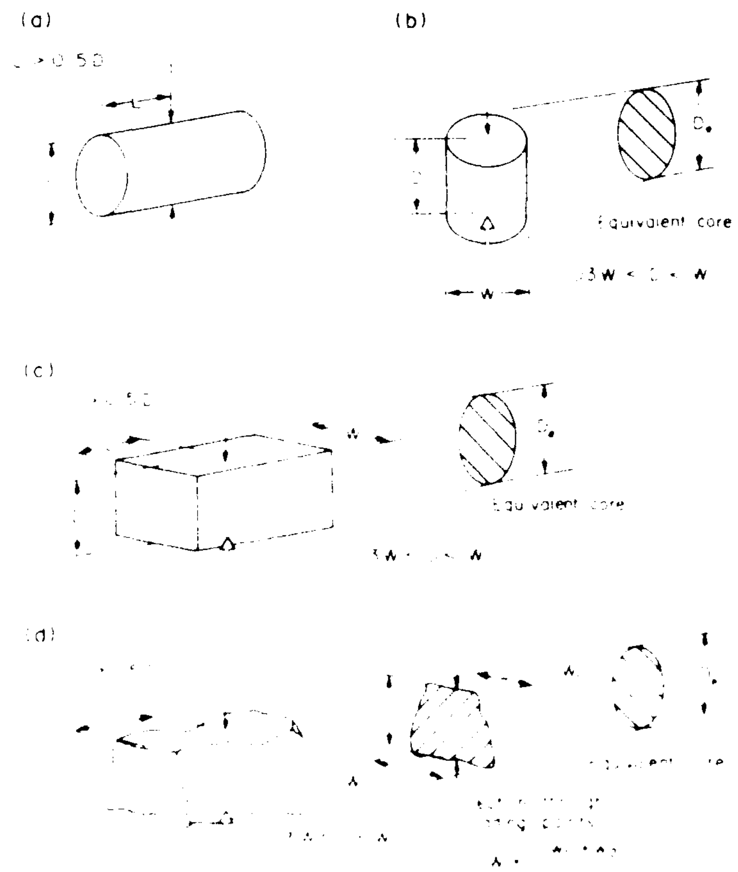


Fig. 3. Specimen shape requirements [illegible]

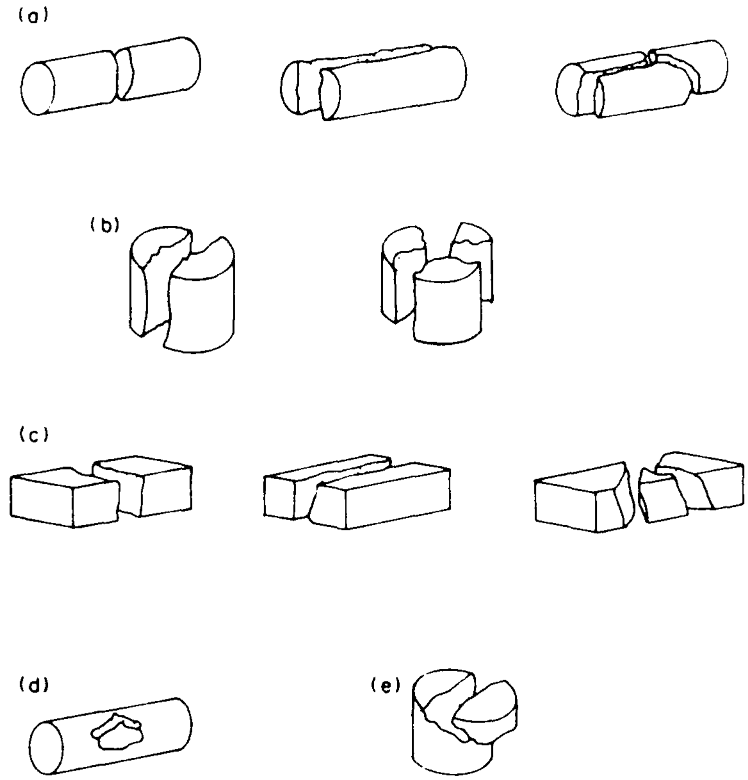


Fig. 4. Typical modes of failure for valid and invalid tests. (a) Valid diametral tests. (b) valid axial tests. (c) valid block tests. (d) invalid core test. (e) invalid axial test.

fracture surface passes through only one loading point (Fig. 4d).

(f) The procedure (c) through (e) above is repeated for the remaining specimens in the sample.

The axial test[2]

9.(a) Core specimens with length/diameter ratio of 0.3–1.0 are suitable for axial testing (Fig. 3b). Long pieces of core can be tested diametrally to produce suitable lengths for subsequent axial testing (provided that they are not weakend by this initial testing); alternatively, suitable specimens can be obtained by saw-cutting or chisel-splitting.

(b) There should preferably be at least 10 tests per sample, more if the sample is heterogeneous or anisotropic.[7]

(c) The specimen is inserted in the test machine and the platens closed to make contact along a line perpendicular to the core end faces (in the case of isotropic rock, the core axis, but see paragraph 11 and Fig. 5).

(d) The distance D between platen contact points is recorded $\pm 2\%$.[6] The specimen width W perpendicular to the loading direction is recorded $\pm 5\%$.[9]

(e) The load is steadily increased such that failure occurs within 10–60 sec. and the failure load P is recorded. The test should be rejected as invalid if the fracture surface passes through only one loading point (Fig. 4e).

(f) The procedures (c) through (e) above are repeated for the remaining tests in the sample.

The block and irregular lump tests

10.(a) Rock blocks or lumps of size 50 ± 35 mm and of the shape shown in Fig. 3(c) and (d) are suitable for the block and the irregular lump tests. The ratio D/W should be between 0.3 and 1.0, preferably close to 1.0.

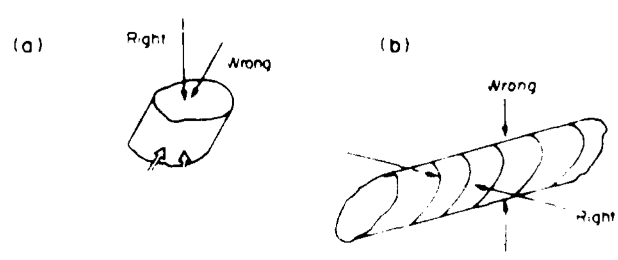


Fig. 5. Loading directions for tests on anisotropic rock

The distance L (Fig. 3c and d) should be at least $0.5W$. Specimens of this size and shape may be selected if available or may be prepared by trimming larger pieces by saw-or chisel-cutting.

(b) There should preferably be at least 10 tests per sample, more if the rock is heterogeneous or anisotropic.[7]

(c) The specimen is inserted in the testing machine and the platens closed to make contact with the smallest dimension of the lump or block, away from edges and corners (Fig. 3c and d).

(d) The distance D between platen contact points is recorded $\pm 2\%$. The smallest specimen width W perpendicular to the loading direction is recorded $\pm 5\%$. If the sides are not parallel, then W is calculated as $(W_1 + W_2)/2$ as shown in Fig. 3d.[6] This smallest width W is used irrespective of the actual mode of failure (Figs 3 and 4)

(e) The load is steadily increased such that failure occurs within 10–60 sec. and the failure load P is recorded. The test should be rejected as invalid if the fracture surface passes through only one loading point (see examples for other shapes in Fig. 4d or e).

(f) The procedure (c) through (e) above is repeated for the remaining tests in the sample.

Anisotropic rock

11. (a) When a rock sample is shaly, bedded, schistose or otherwise observably anisotropic it should be tested in directions which give the greatest and least strength values, which are in general parallel and normal to the planes of anisotropy.

(b) If the sample consists of core drilled through the weakness planes, a set of diametral tests may be completed first, spaced at intervals which will yield pieces which can then be tested axially.

(c) Best results are obtained when the core axis is perpendicular to the planes of weakness, so that when possible the core should be drilled in this direction. The angle between the core axis and the normal to the weakness planes should preferably not exceed 30°.

(d) For measurement of the I_s value in the directions of least strength, care should be taken to ensure that load is applied along a single weakness plane. Similarly when testing for the I_s value in the direction of greatest strength, care should be taken to ensure that the load is applied perpendicularly to the weakness planes (Fig. 5).

(e) If the sample consists of blocks or irregular lumps, it should be tested as two sub-samples, with load applied firstly perpendicular to, then along the observable planes of weakness.[10] Again, the required minimum strength value is obtained when the platens make contact along a single plane of weakness

CALCULATIONS

Uncorrected point load strength

12. The Uncorrected Point Load Strength I_s is calculated as P/D_e^2 where D_e, the "equivalent core diameter", is given by:

$$D_e^2 = D^2 \text{ for diametral tests;}$$

$$= 4A/\pi \text{ for axial, block and lump tests;}$$

and

$A = WD$ = minimum cross sectional area of a plane through the platen contact points.[6]

Size correction

13.(a) I_s varies as a function of D in the diametral test, and as a function of D_e in axial, block and irregular lump tests, so that a size correction must be applied to obtain a unique Point Load Strength value for the rock sample, and one that can be used for purposes of rock strength classification.

(b) The size-corrected Point Load Strength Index $I_{s(50)}$ of a rock specimen or sample is defined as the value of I_s that would have been measured by a diametral test with $D = 50$ mm.

(c) The most reliable method of obtaining $I_{s(50)}$, preferred when a precise rock classification is essential, is to conduct diametral tests at or close to $D = 50$ mm. Size correction is then either unnecessary ($D = 50$ mm) or introduces a minimum of error. The latter is the case, for example, for diametral tests on NX core, $D = 54$ mm. This procedure is not mandatory. Most point load strength testing is in fact done using other sizes or shapes of specimen. In such cases, the size correction (d) or (e) below must be applied.

(d) The most reliable method of size correction is to test the sample over a range of D or D_e values and to plot graphically the relation between P and D_e^2. If a log–log plot is used the relation is generally a straight line (Fig. 6). Points that deviate substantially from the straight line may be disregarded (although they should not be deleted). The value of P_{50} corresponding to $D_e^2 = 2500$ mm^2 ($D_e = 50$ mm) can then be obtained by interpolation, if

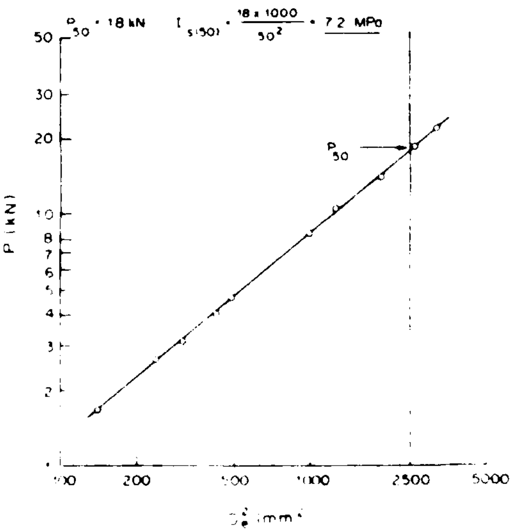


Fig. 6. Procedure for graphical determination of $I_{s(50)}$ from a set of results at D_e values other than 50 mm.

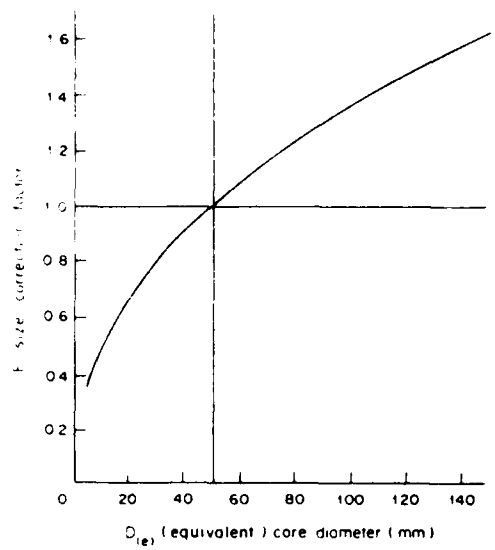


Fig. 7. Size correction factor chart.

necessary by extrapolation, and the size-corrected Point Load Strength Index calculated as $P_{50}/50^2$.

(e) When neither (c) nor (d) is practical, for example when testing single sized core at a diameter other than 50 mm or if only a few small pieces are available, size correction may be accomplished by using the formula:

$$I_{s(50)} = F \times I_s$$

The "Size Correction Factor F" can be obtained from the chart in Fig. 7,[11] or from the expression:

$$F = (D_e/50)^{0.45}$$

For tests near the standard 50 mm size, very little error is introduced by using the approximate expression:

$$F = \sqrt{(D_e/50)}$$

(f) The size correction procedures specified in this paragraph have been found to be applicable irrespective of the degree of anisotropy I_a and the direction of loading with respect to planes of weakness, a result that greatly enhances the usefulness of this test.

Mean value calculation

14.(a) Mean values of $I_{s(50)}$ as defined in (b) below are to be used when classifying samples with regard to their Point Load Strength and Point Load Strength Anisotropy Indices.

(b) The mean value of $I_{s(50)}$ is to be calculated by deleting the two highest and lowest values from the 10 or more valid tests, and calculating the mean of the remaining values. If significantly fewer specimens are tested, only the highest and lowest values are to be deleted and the mean calculated from those remaining.[1]

Point load strength anisotropy index

15. The Strength Anistropy Index $I_{a(50)}$ is defined as the ratio of mean $I_{s(50)}$ values measured perpendicular and parallel to planes of weakness, i.e. the ratio of greatest to least Point Load Strength Indices. $I_{a(50)}$ assumes values close to 1.0 for quasi-isotropic rocks and higher values when the rock is anisotropic.

REPORTING OF RESULTS

16. Results for diametral tests, axial tests, block tests and irregular lump tests, and for tests perpendicular and parallel to planes of weakness should be tabulated separately (see typical results form, Fig. 8). The report should contain calibration data for the test machine and at least the following information for each sample tested:

(a) The sample number, source location and rock type, and the nature and *in situ* orientation of any planes of anistropy or weakness.

(b) Information on the water content of the rock at the time of testing.

(c) Information on which specimens were loaded parallel (//), perpendicular (⊥), or at unknown or random directions with respect to planes of weakness.

(d) A tabulation of the values of P, D, (W, D_e^2 and D_e if required), I_s, (F if required) and $I_{s(50)}$ for each specimen in the sample.

(e) For all isotropic samples, a summary tabulation of mean $I_{s(50)}$ values.

(f) For all anisotropic samples, a summary tabulation of mean $I_{s(50)}$ values for sub-samples tested perpendicular and parallel to the planes of weakness, and of the corresponding $I_{a(50)}$ values.

NOTES

1. When first introduced, the point load strength test was used mainly to predict uniaxial compressive strength which was then the established test for general-purpose rock strength classification. Point load strength now often replaces uniaxial compressive strength in this role since when properly conducted it is as reliable and much quicker to measure. $I_{s(50)}$ should be used directly for rock classification, since correlations with uniaxial compressive strength are only approximate. On average, uniaxial compressive strength is 20–25 times point load strength, as shown in Fig. 9. However, in tests on many different rock types the ratio can vary between 15 and 50 especially for anisotropic rocks, so that errors of up to 100% are possible in using an arbitrary ratio value to predict compressive strength from point load strength. The point load strength test is a form of "indirect tensile" test, but this is largely irrelevant to its primary role in rock classification and strength characterization. $I_{s(50)}$ is approximately 0.80 times the uniaxial tensile or Brazilian tensile strength.

2. Of the four alternative forms of this test, the diametral test and the axial test with saw-cut faces are the most accurate if performed near the standard 50 mm size, and are preferred for strength classification when core is available. Axial test specimens with saw-cut faces can easily be obtained from large block samples by coring in the laboratory. Specimens in this form are

Sample Details	Point Load Test	Date 17/11/83

1 block sample from Gamblethorpe Opencast site.

Fine grained pale grey Coal Measures sandstone with numerous coaly streaks along horizontal bedding planes.

Specimens 1-6 chisel cut blocks, air-dried 2 weeks;
7-10 sawn blocks, air-dried 2 weeks;
11-15 cores, air-dried 2 weeks;
16-20 cores, air-dried 2 weeks;
- tested in laboratory.

No.	Type	W (mm)	D (mm)	P (kN)	D_e^2 (mm^2)	D_e (mm)	I_s	F	I_s (50)
1	i ⊥	30.4	17.2	2.687	666	25.8	4.03	0.75	~~3.03~~
2	i ⊥	16	8	0.977	163	12.8	5.99	0.54	3.24
3	i ⊥	19.7	15.6	1.962	391	19.8	5.02	0.66	3.31
4	i ⊥	35.8	18.1	3.641	825	28.7	4.41	0.765	3.46
5	i ⊥	42.5	29	6.119	1569	39.6	3.90	0.875	3.49
6	i ⊥	42	35	7.391	1872	43.3	3.95	0.935	~~3.69~~
7	b ⊥	44	21	4.600	1176	34.3	3.91	0.84	3.29
8	b ⊥	40	30	5.940	1528	39.1	3.88	0.89	3.46
9	b ⊥	19.5	15	2.040	372	19.3	5.48	0.655	~~3.59~~
10	b ⊥	33	16	2.87	672	25.9	4.27	0.75	~~3.20~~
11	d //	-	49.93	5.107	-	-	-	-	2.05
12	d //	-	49.88	4.615	-	-	-	-	1.85
13	d //	-	49.82	5.682	-	-	-	-	~~2.29~~
14	d //	-	49.82	4.139	-	-	-	-	~~1.67~~
15	d //	-	49.86	4.546	-	-	-	-	1.83
16	d //	-	25.23	1.837	-	-	2.89	0.74	2.14
17	d //	-	25.00	1.891	-	-	3.02	0.735	2.22
18	d //	-	25.07	2.118	-	-	3.37	0.735	~~2.48~~
19	d //	-	25.06	1.454	-	-	2.32	0.735	~~1.70~~
20	d //	-	25.04	1.540			2.46	0.735	1.81

d = diametral;
a = axial;
b = block;
i = irregular lump test;
⊥ = perpendicular;
// = parallel to planes of weakness.

Mean I_s (50) ⊥	3.38
Mean I_s (50) //	1.98
I_a (50)	1.71

Fig. 8. Typical results form.

particularly suitable when the rock is anisotropic and the direction of weakness planes must be noted.

3. Loads of up to 50 kN are commonly required for the larger hard rock specimens. The maximum specimen size that can be tested by a given machine is determined by the machine's load capacity, and the smallest by the machine's load and distance measuring sensitivity. Tests on specimens smaller than $D = 25$ mm require particular precautions to ensure that the measuring sensitivity is sufficient. The range of required test loads should be estimated before testing, from approximate assumed strength values, to ensure that the load capacity and sensitivity of the equipment are adequate. It may be necessary to change the load measuring gauge or load cell, or to test smaller or larger specimens to conform with the capacity of available equipment or with the accuracy specifications for this test.

4. The conical platen design is intended to give standardized penetration of softer specimens. When testing is confined to hard rocks and small (less than 2 mm) penetrations the conical design is unimportant provided that the tip radius remains at the standard 5 mm. For such testing the platen can be manufactured by embedding a hard steel or tungsten carbide ball in a

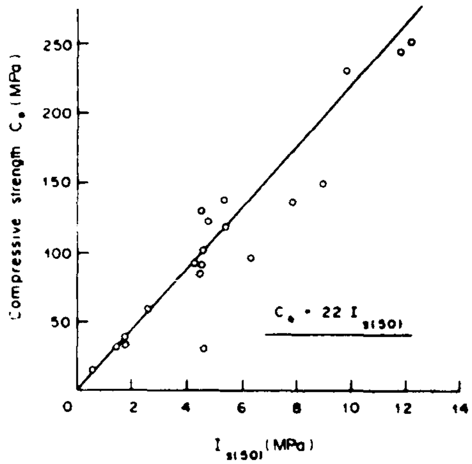


Fig. 9 Example of correlation between point load and uniaxial compressive strength results.

softer metal base of any geometry that will ensure that only the platen tip is in contact with the rock.

5. If a quick-retracting ram is used to reduce the delay between tests, either the ram return spring force and ram friction should together be less than about 5% of the smallest load to be measured during testing, or an independent load cell rather than an oil pressure gauge should be used for load determination. These forces can be significant when testing weaker and smaller specimens.

6. If significant platen penetration occurs, the dimension D to be used in calculating point load strength should be the value D' measured at the instant of failure, which will be smaller than the initial value suggested in paragraphs 8(d), 9(d) and 10(d). The error in assuming D to be its initial value is negligible when the specimen is large or strong. The failure value may always be used as an alternative to the initial value and is preferred if the equipment allows it to be measured (for example by electrical maximum-indicating load and displacement measurement). When testing specimens that are smaller than 25 mm, such as rock aggregate particles, equipment with electrical readout is usually necessary to obtain the required measuring accuracy, and should be designed to record D' at failure. Measurements of W or D made perpendicular to the line joining the platens are not affected and are retained at their original values. The value of D_e for strength calculation can then be found from:

$$D_e^2 = D \times D' \quad \text{for cores}$$

$$D_e^2 = \frac{4}{\pi}(W \times D') \quad \text{for other shapes}$$

7. Because this test is intended primarily as a simple and practical one for field classification of rock materials, the requirements relating to sample size, shape, numbers of tests etc. can when necessary be relaxed to overcome practical limitations. Such modifications to procedure should however be clearly stated in the report. It is often better to obtain strength values of limited reliability than none at all. For example, rock is often too broken or slabby to provide specimens of the ideal sizes and shapes, or may be available in limited quantities such as when the test is used to log the strength of drill core. In core logging applications, the concept of a "sample" has little meaning and tests are often conducted at an arbitrary depth interval, say one test every 1 m or 3 m depending on the apparent variability or uniformity of strength in the core and on the total length of core to be strength-logged.

8. As for all strength tests on rocks, point load strength varies with the water content of the specimens. The variations are particularly pronounced for water saturations below 25%. Oven dried specimens, for example, are usually very much stronger than moist ones. At water saturations above 50% the strength is less influenced by small changes in water content, so that tests in this water content range are recommended unless tests on dry rock are specifically required.

All specimens in a sample should be tested at a similar and well-defined water content, and one that is appropriate to the project for which the test data are required. Field testing of chisel-cut samples, not affected by drilling fluids, offers a method for testing at the *in situ* water content. If possible, numerical values should be given for both water content and degree of saturation at the time of testing. The ISRM Suggested Method for Water Content Determination should be employed. Whether or not water content measurements can be made, the sample storage conditions and delay between sampling and testing should be reported.

9. Some researchers argue in favour of measuring W as the minimum dimension of the failure surface after testing rather than of the specimen before failure (the German standard for this test is an example). Point load strengths computed using the two alternative W definitions may differ slightly. The minimum specimen dimension alternative has been adopted in this Suggested Method mainly because it is quicker and easier to measure, particularly in the field when fragments of broken specimens are easily lost.

10. Commonly the shortest dimension of naturally occurring anisotropic rock lumps is perpendicular to the weakness planes.

11. The size correction factor chart (Fig. 7) is derived from data on cores tested diametrally and axially and from tests on blocks and irregular lumps, for rocks of various strengths, and gives an averaged factor. Some rocks do not conform to this behaviour, and size correction should therefore be considered an approximate method, although sufficient for most practical rock classification applications. When a large number of tests are to be run on the same type of rock it may be advantageous to first perform a series of tests at different sizes to obtain a graph of load vs D_e^2 as in Fig. 6. If the slope of such a log–log graph is determined as "n", the size correction factor is then $(D_e/50)^m$ where $m = 2(1 - n)$. This can either be calculated directly or a chart constructed.

12. Mean results for small populations are generally better measures when the extreme values are not included in the calculation.

Received 10 *June* 1984.

BIBLIOGRAPHY

1. Franklin J. A., Broch E. and Walton G. Logging the mechanical character rock. *Trans. Instn Min. Metall.* **80**, A1-A9 (1971); and Discussion **81**, A43-A51 (1972).
2. Broch E. and Franklin J. A. The point-load strength test. *Int. J. Rock Mech. Min. Sci.* **9**, 669–697 (1972).
3. Bieniawski Z. T. The point-load test in geotechnical practice. *Engng Geol.* **9**, 1-11 (1975).
4. Boisen B. P. A hand portable point load tester for field measurements. *Proc. 18th U.S. Symp. on Rock Mechanics*, pp. 1–4. Keystone, Colorado (1977).
5. Broch E. Estimation of strength anisotropy using the point load test. *Int J. Rock Mech. Min. Sci. & Geomech. Abstr.* **20**, 181-187 (1983).
6. Brook N. A method of overcoming both shape and size effects in point load testing. *Proc. Conf. on Rock Engineering*, pp. 53–70. Univ. of Newcastle, England (1977).
7. Brook N. Size correction for point load testing. Technical Note. *Int. J. Rock Mech. Sci. & Geomech. Abstr.* **17**, 231-235 (1980).
8. Fitzhardinge C. F. R. Note on point load strength test. *Aust. Geomech. J.* **G8**, 53 (1978).
9. Forster I. R. Influence of core sample geometry on the axial point load test. Technical Note. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr* **20**, 291-295 (1983).
10. Gartung E. Empfehlung Nr. 5 des Arbeitskreises 19—Versuchstechnik Fels—der Deutschen Gesellschaft für Erd- und Grundbau e.V. Punktlastversuche an Gesteinsproben. *Die Bautechnik* **59(1)**, 13-15 (1982).
11. Greminger M. Experimental studies of the influence of rock anisotropy on size and shape effects in point load strength testing. Technical Note. *Int. J Rock Mech Min Sci & Geomech Abstr* **19**, 241-246 (1982).
12. Guidicini G., Nieble C. M. and Cornides A. T. Analysis of point load test as a method for preliminary geotechnical classification of rocks. *Bull. Int. Ass. Engng Geol.* **7**, 37-52 (1973).
13. Haramy K. Y., Morgan T. A. and DeWaele R. E. A method for estimating coal strengths from point load tests on irregular lumps. USBM, Denver Research Center, Progress Rept 10028, 31pp (1981).
14. Hassani F. P., Scoble M. J. and Whittaker B. N. Application of the point load index test to strength of rock, and proposals for a new size correction chart. *Proc 21st U.S. Symp. on Rock Mechanics*, pp. 543-556. Rolla, Missouri (1980).
15. International Society for Rock Mechanics. Suggested method for determining the point load strength index. ISRM (Lisbon, Portugal), Committee on Field Tests, Document No. 1, pp. 8-12 (1972).
16. Lajtai E. Z. Tensile strength measurement and its anisotropy measured by point-and line-loading of sandstone. *Engng Geol.* **15**, 163-171 (1980).
17. Pells P. J. N. The use of the point load test in predicting the compressive strength of rock materials. *Aust Geomech. J* **G5**, 54–56 (1975).
18. Peng S. S. Stress analysis of cylindrical rock discs subjected to axial double point-load. *Int J Rock Mech. Min Sci. & Geomech. Abstr.* **13**, 97-101 (1976).
19. Read J. R. L., Thornton P. N. and Regan W. M. A rational approach to the point load test. *Proc 3rd Aust. N.Z. Conf. on Geomechanics*, Vol. 2, pp. 35–39. Wellington (1980).
20. Reichmuth D. R. Point load testing of brittle materials to determine tensile strength and relative brittleness. *Proc. 9th U.S. Symp. on Rock Mechanics*, Colorado (1968).
21. Robins P. J. The point load test for concrete cores. *Mag. Concr Res.* **32**, 101-111 (1980).
22. Wijk G. Some new theoretical aspects of indirect measurements of the tensile strength of rocks. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* **15**, 149-160 (1978).
23. Wijk G. The point load test for the tensile strength of rock. *Geotech. Testing J.*, pp. 49-54 (June, 1980).

PART II. IN SITU STRENGTH METHODS

C. Determination of In Situ Stress

DETERMINATION OF IN SITU STRESS BY THE OVERCORING TECHNIQUE

1. Scope

1.1 The equipment required to perform in situ stress tests using the U. S. Bureau of Mines' three-component borehole deformation gage is described.

1.2 The test procedure and method of data reduction are described, including the theoretical basis and assumptions involved in the calculations. A section on troubleshooting equipment malfunctions is included.

1.3 The procedure herein described was taken from the Bureau of Mines Information Circular No. 8618 dated 1974 [8.1] as were many of the figures. Some modifications based on field experience are incorporated. This test method can be used from the surface or from underground openings. Good results can be expected using this technique in massive or competent rock. Difficulties will be encountered if tests are attempted in fissile or fractured rock.

2. Test Equipment

2.1 Instrumentation

2.1.1 The three-component borehole deformation gage (BDG) is shown in Fig. 1. It is designed to measure diametral deformations during overcoring along three diameters 60 deg apart in a plane perpendicular to the walls of a 1-1/2-in.-diameter borehole. The measurements are made along axes referred to as the U_1, U_2, and U_3 axes. Accessories required with the gage are special pliers, 0.005- and 0.015-in.-thick brass piston washers, and silicone grease (Fig. 1).

2.1.2 Three Vishay P350A or equal strain indicators are required. (Alternatively, one indicator with a switching unit may be used or one unit may be used in conjunction with manual wire changing to obtain readings from the three axes.) These units have a full range digital readout limit of 40,000 indicator units. A calibration factor must be obtained for each axis to relate indicator units to microinches deflection. The calibration factor for each axis will change proportionally with the gage factor used. Normally, a gage factor of 0.40 gives a good balance between range and sensitivity. Figure 2 shows a strain indicator, calibration jig, and a switching unit.

2.1.3 The shielded eight-wire conductor cable transmits the strain measurements from the gage to the strain indicators. The length of cable required is the depth to the test position from the surface plus about 30 ft to reach the strain indicators.

2.1.4 The orientation and placement tools consist of:

(a) The placement tool or "J slot" shown in Fig. 3.

(b) Placement rod extensions as shown in Fig. 3.

(c) The orientation tool or "T handle," also shown in Fig. 3.

(d) The scribing tool is used to orient the core for later biaxial testing. It consists of a bullet-shaped stainless steel head attached to a 3-ft rod extension. Projecting perpendicular from the stainless steel head is a diamond stud. The stud is adjusted outward until a snug fit is achieved in the EX hole so that a line is scratched along the borehole wall as the scribing tool is pushed in.

(e) The Pajari alignment device is inserted into the hole to determine the inclination. It consists of a floating compass and an automatic locking device which locks the compass in position before retrieving it.

2.1.5 The calibration jig (Fig. 2) is used to calibrate the BDG before and after each test.

2.1.6 The biaxial chamber is used to determine Young's Modulus of the retrieved rock core. A schematic of the apparatus is shown in Fig. 4.[8.2]

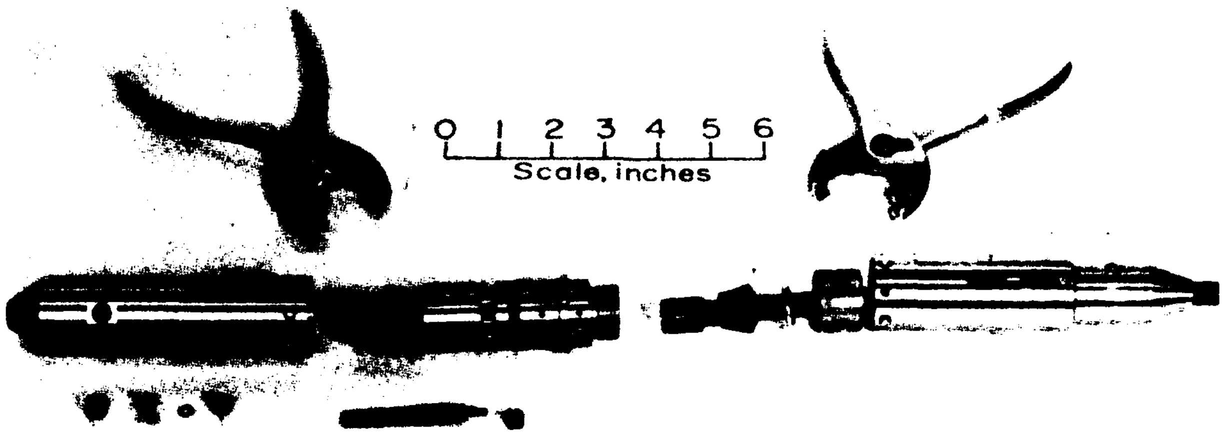


Fig. 1. Special pliers, the Bureau of Mines' three-component borehole gage, a piston, disassembled piston and washer, and a transducer with nut.

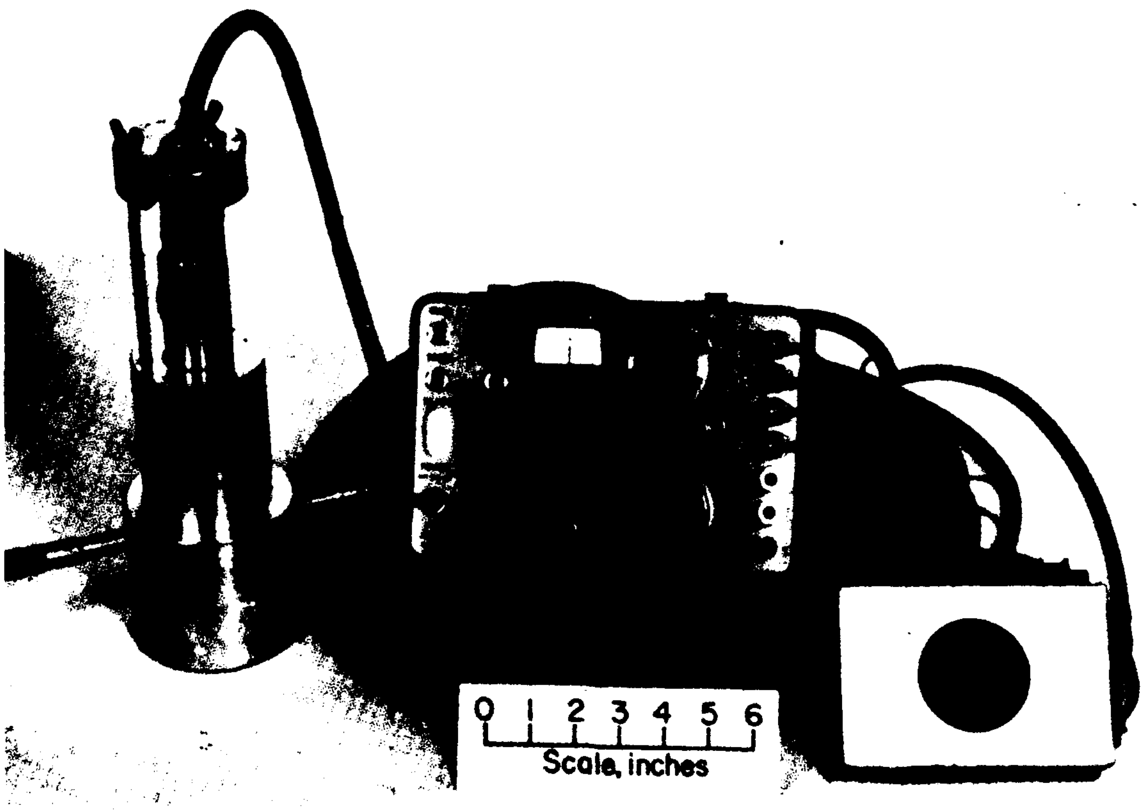


Fig. 2. The calibration device (left side) and a switching unit (right side).

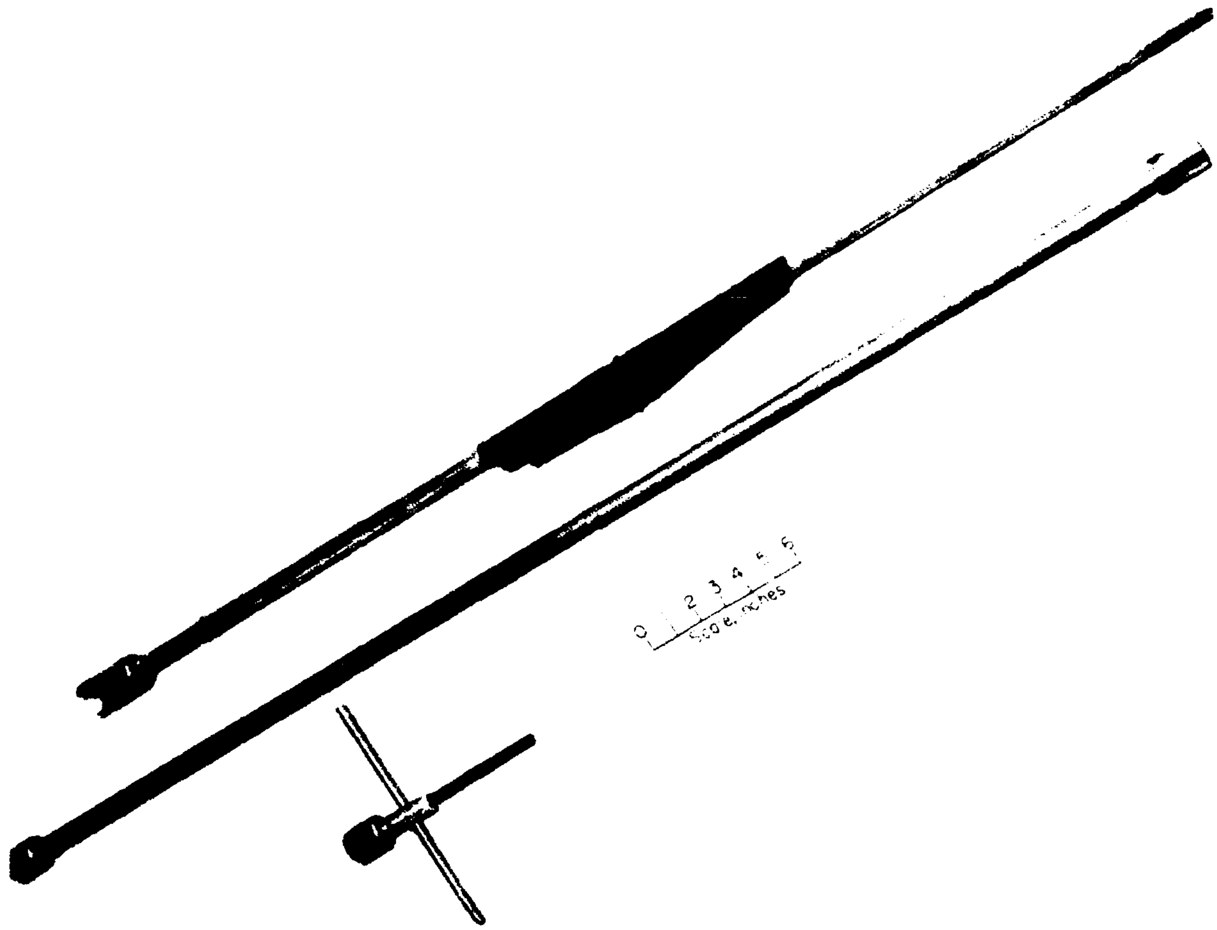
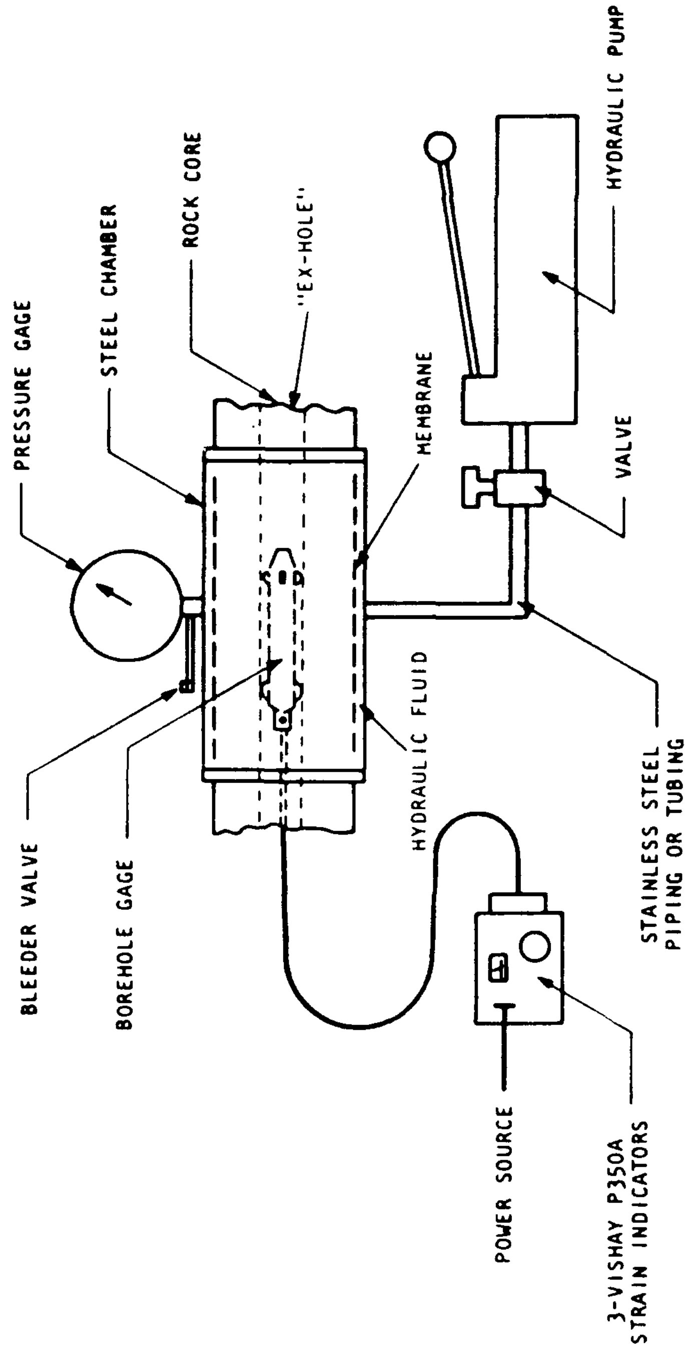


Fig. 3. Placement and retrieval tool.



SCHEMATIC : BIAXIAL TEST APPARATUS

Fig. 4

2.2 Drilling Equipment

2.2.1 A drill with a chuck speed ranging down to 50 rpm should be used. Achievement of this lower end speed will usually require a gear reduction.

2.2.2 An EWX single-tube core barrel, 2 ft long, is required (Fig. 5).

2.2.3 An EX diamond bit is required (Fig. 5).

2.2.4 A reamer is used with the EX bit for the 1-1/2-in.-diameter pilot gage hole (Fig. 5).

2.2.5 Two stabilizers are required. One should be 5-1/2 in. OD by 4 in. long and one should be 2-3/4 in. OD by 4 in. long. Each should have an inner concentric hole slightly larger than the OD of AX casing. Grooves should be cut into the stabilizers to allow water to pass through. The stabilizers are slipped over opposite ends of a 6 ft length of AX flush joint casing and secured with set screws. The stabilizer can be made from hard plastic stock. This assembly fits inside the 5-ft-long, 6-in.-diameter core barrel with the larger stabilizer at the bottom and the smaller stabilizer projecting into the NW casing. As overcoring proceeds, the stabilizer assembly is pushed upward into the NW casing.

2.2.6 An EWX rod stabilizer should be made that will slide over the EWX rod and fit inside the NW casing to align the EX bit with the hole in the center of the stabilizer in the 6-in. core barrel. Cut grooves along the outer edges of the stabilizer to allow water to flow through.

2.2.7 EW drill rods are used with the EX bit. The required length is dependent on the test depth.

2.2.8 NW casing is used with the 6-in. core barrel and bit. The required length will depend on test depth. An adapter is required to couple the NW casing to the 6-in. core barrel.

2.2.9 A water swivel (Fig. 6) is required with a 1/2-in. hole for the conductor cable to pass through and a plug is required to fit the hole when the gage is not being used.

2.2.10 A 6-in.-diameter starter barrel 1 ft long with a detachable 1-1/2-in.-diameter pilot shaft in the center (Fig. 7) is required. The pilot shaft should extend about 5 in. beyond the diamonds of the starter barrel. This barrel is used to center the 6-in.-diameter hole over an initial 1-1/2-in.-diameter hole at the face. The barrel and pilot shaft are not needed for vertical or near vertical holes.

2.2.11 An EW core barrel to replace the pilot shaft should be cut to extend 1 in. beyond the starter barrel. When the bit and reamer are attached, the unit is used to drill a 1-1/2-in.-diameter starter hole 4 in. deep at the end of a 6-in. horizontal hole. This piece of equipment is not needed in vertical holes.

2.2.12 A standard 6-in. diamond drill bit or a 6-in. thin wall masonry bit is used for overcoring.

2.2.13 A core breaker, at least 2-1/2 in. wide and hardened, to fit the EW rod (Fig. 8) is required.

2.2.14 A 6-in. core shovel (Fig. 8) to fit an EW rod is needed for retrieving core from horizontal holes.

2.2.15 A 6-in. core puller (Fig. 8) approximately 18 in. long to fit an EW drill rod is sometimes needed for retrieving core from vertical holes. The core puller is made from a used 6-in. core barrel. A 5/8-in.-thick steel plate is welded to the end of the barrel with an EW rod welded in the center. Three 1-1/2-in.-diameter holes on 120-deg centers are drilled into the plate to allow water to pass through. Four U cuts 90 deg apart are made on the front of the barrel. The rectangular pieces of metal inside the U cuts are pushed in slightly to grip the core. This is an optional piece of equipment. Normally, the core can be retrieved inside the 5-ft barrel.

2.2.16 A 6-in.-diameter core barrel 5 ft long is required.

2.2.17 A high-capacity water pump and hose are required.

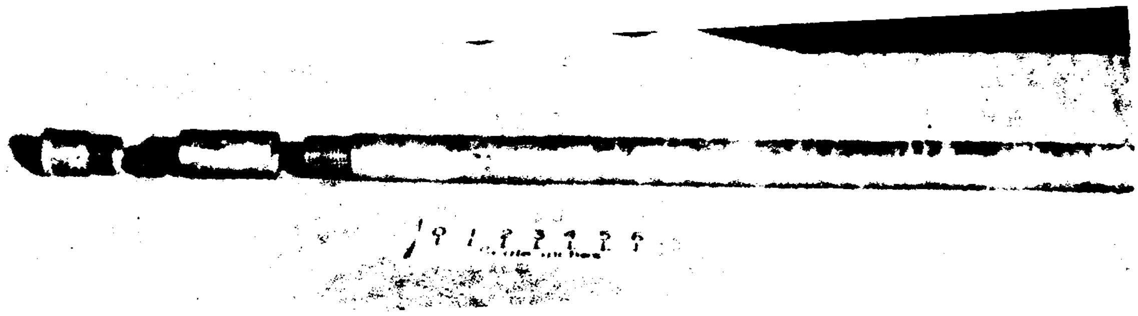


Fig. 5. EX size bit with core spring, reamer, and 2-ft EWX core barrel.

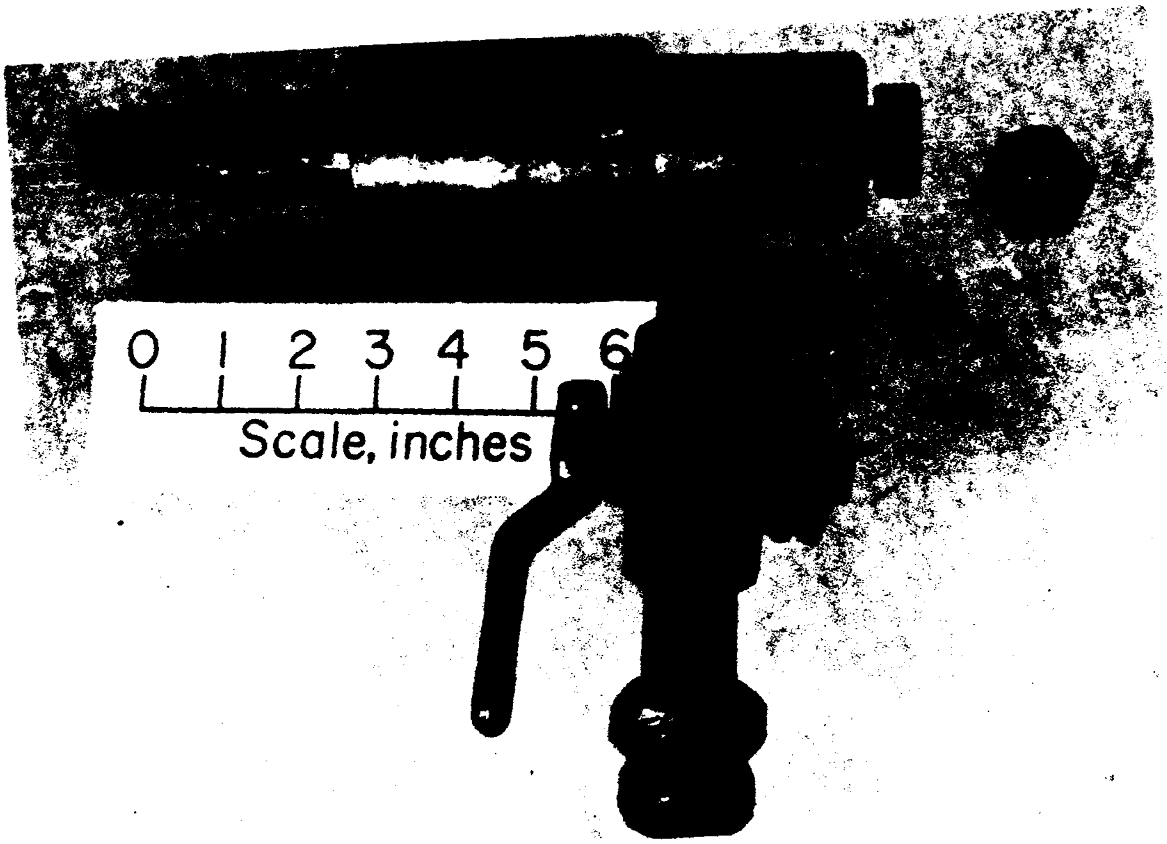


Fig. 6. Water swivel with solid plug. Plug at right used during overcoring.

Fig. 7. Six-inch-diameter starter barrel with pilot and expander head adapted for EW drill rod.

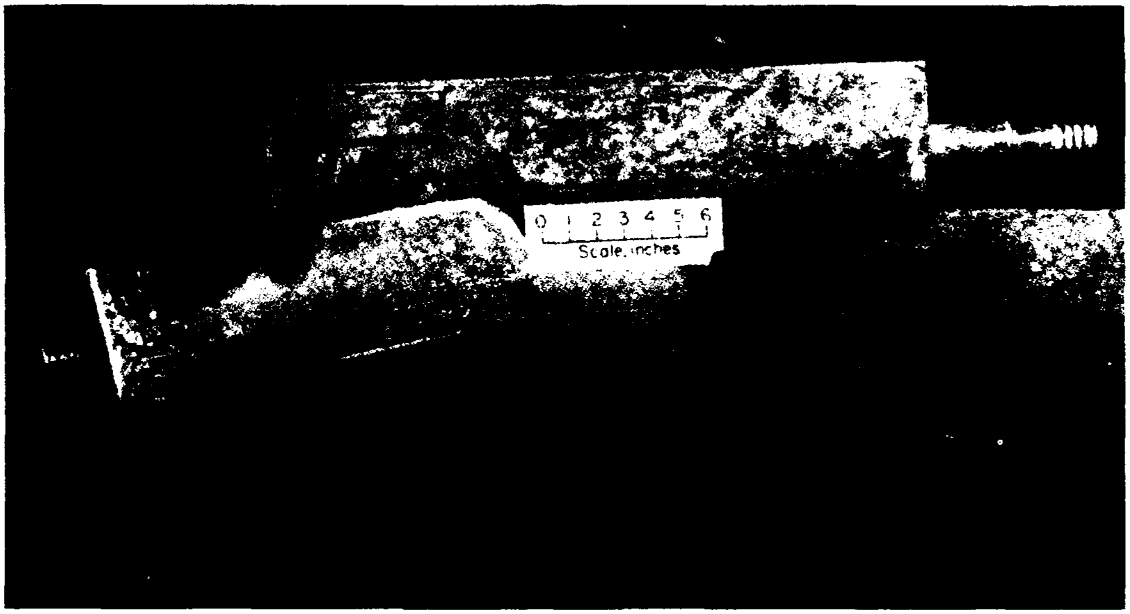


Fig. 8. Core breaker (lower right), core shovel (lower left), and core puller (center).

2.2.18 One clothesline pulley is required.

2.3 Miscellaneous Equipment - This field operation requires a good set of assorted hand tools which should include a soldering iron, solder and flux, pliers, pipe wrenches, adjustable wrenches, end wrenches, screwdrivers, allen wrenches, a hammer, electrical tape, a yardstick and carpenter's rule, chalk, and a stopwatch.

3. Overcoring Test Procedure

3.1 The procedure for determining in situ stress can be divided into three phases: (a) strain relief measurements in situ, (b) determination of Young's Modulus of the rock by recompression in a biaxial chamber, and (c) computation of stress.

3.2 Reliable information on subsurface rock conditions is essential for good test results. For this reason it is advisable to drill an exploratory NX size hole within a few feet of the desired test location.

3.3 Horizontal holes should be started 5 deg upward from horizontal to facilitate removal of water and cuttings. An EX pilot hole is first drilled about 4 in. into the rock. Attach the 6-in. starter barrel and pilot shaft and start the 6-in. hole. Remove the starter barrel, attach the regular 6-in. bit and barrel, and extend the 6-in. hole to within 12 in. of the desired test depth. Proceed to step 3.6.

3.4 When testing in vertical holes, first wash bore through the overburden and core about 5 ft into bedrock. Case the hole with 8-in. casing and grout in place.

3.5 Drill a 6-in.-diameter hole to within 12 in. of the desired test depth. Retrieve the core and go back down the hole with the stabilizers in place in the core barrel.

3.6 Insert the EX bit and reamer coupled to the 2-ft EX core barrel and EWX rods with the EWX stabilizer in place. Drill 2 ft of EX hole.

3.7 Retrieve the EX core and inspect. Insert the scribing tool coupled to the rod extensions. When the scribing tool reaches the stabilizers, it will have to be shoved through. It will then be at the top of the EX hole. Attach the orientation handle and orient the scribe mark as desired. Shove the scribe straight down the hole. (Note: If the scribe cannot be pushed down the hole, the diamond stud is projecting too far; adjust it inward. If the scribe feels loose the stud must be adjusted to project further.) When the scribe hits the bottom of the EX hole, slowly pull it back up along the same scribe mark. If joints or fractures intersect the borehole walls, they can often be detected by a subtle vibration as the diamond stud crosses them. If joints or fractures are detected, extend the hole and try again.

3.8 When the EX hole has been scribed, remove the scribing tool.

3.9 The BDG must now be calibrated. It should be calibrated before and after each test.

3.9.1 Grease all gage pistons with a light coat of silicone grease and install them in gage.

3.9.2 Place the gage in the calibration jig as shown in Fig. 2 with the pistons of the U_1 axis visible through the micrometer holes of the jig. Tighten the wing nuts.

3.9.3 Install the two micrometer heads and lightly tighten the set screws.

3.9.4 Set the strain indicators on "Full Bridge", center the balance knob, and set the gage factor to correspond to the anticipated in situ range and sensitivity requirements. (If high stress conditions are anticipated, sensitivity will have to be compromised to gain the required range.) A lower gage factor results in higher sensitivity. The gage factor used should be the same for calibration, in situ testing, and modulus tests.

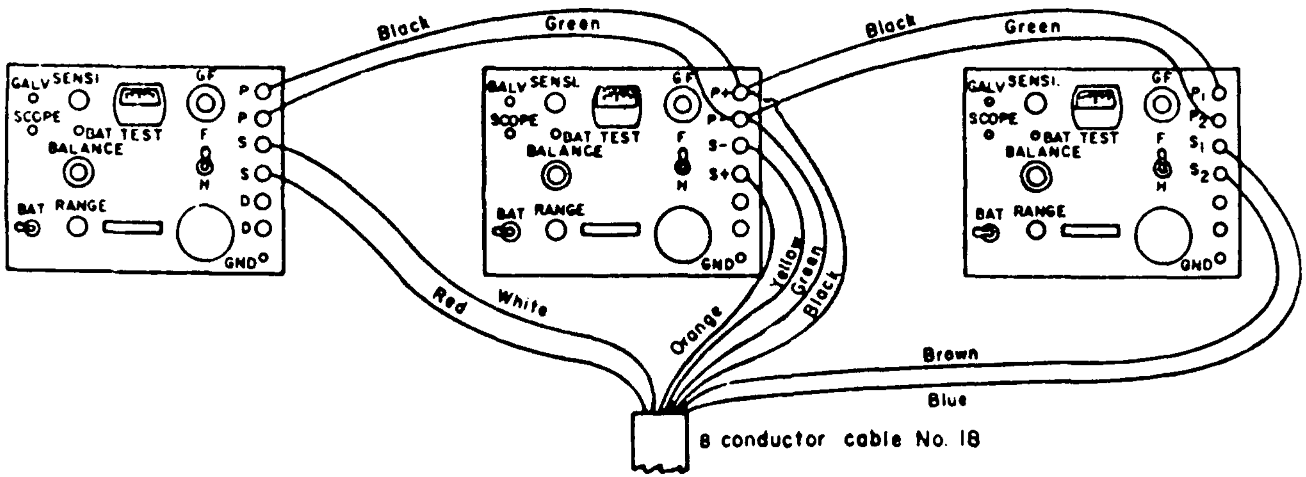
3.9.5 Wire the gage to the indicators as shown in Fig. 9 or to a switching and balance unit and one indicator.

3.9.6 Balance the indicator using the "Balance" knob.

3.9.7 Turn one micrometer in until the needle of the indicator just starts to move. The micrometer is now in contact with the piston. Repeat with the other micrometer.

3.9.8 Rebalance the indicator.

Sensitivity knob turn full clockwise.
Balance knob: put mid-range (5 turns of the 10-turn potentiometer).
Bridge switch: Switch to full (F).



Note. Hook black and green wires to indicator No. 2 and use two other wires (No. 18 or No. 20) to common P+ and P- (or P_1 and P_2) of all three indicators.

Fig. 9. Wire hookup to model P-350 strain indicators.

3.9.9 Record this no load indicator reading for the U_1 axis.

3.9.10 Turn in each micrometer 0.0160 in. (a total of 0.0320-in. displacement).

3.9.11 Balance the indicator and record the reading and the deflection.

3.9.12 Wait two minutes to check the combined creep of the two transducers. Creep should not exceed 20 μ in./in. in two minutes.

3.9.13 Record the new reading.

3.9.14 Back out each micrometer 0.0040 in. (a total of 0.0080 in.).

3.9.15 Balance and record.

3.9.16 Continue this procedure with the same increments until the initial point on the micrometer is reached. This zero displacement reading will be the zero displacement reading for the second run.

3.9.17 Repeat steps 3.9.10 through 3.9.16.

3.9.18 Loosen the wing nuts and rotate the gage to align the pistons of the U_2 axis with the micrometer holes.

3.9.19 Retighten the wing nuts.

3.9.20 Repeat steps 3.9.6 through 3.9.17.

3.9.21 Loosen wing nuts and align pistons of U_3 axis with micrometer holes. Repeat the calibration procedure followed for the U_1 and U_2 axes.

3.9.22 To determine the calibration factor for each axis:

(a) Subtract the zero displacement strain indicator readings (last reading of each run) from the indicator readings for each deflection to establish the differences.

(b) Subtract the difference in indicator units at 0.0080-in. deflection from the difference in indicator units at 0.0320-in. deflection.

(c) Divide the difference in deflections (0.0240 in.) by the corresponding difference in indicator units just calculated to obtain the calibration factor for that axis.

(d) Repeat for the second cycle and take the average as the calibration factor.

(e) The following is an example of the calibration for one axis, calibrated at a gage factor of 0.40.

CALIBRATION OF AXIS U_1

	Displacement in.	Indicator Reading	Difference μin./in.
Run 1	0	-693	---
	0.0320	+30,140	Wait 2 minutes
	0.0320	30,055	30,535
	0.0240	21,920	22,400
	0.0160	14,040	14,520
	0.0080	6,380	6,860
	0	-480	---
Run 2	0	-480	---
	0.0320	+30,034	Wait 2 minutes
	0.0320	29,980	30,430
	0.0240	21,914	22,364
	0.0160	13,975	14,425
	0.0080	6,335	6,785
	0	-450	---

$$\text{Calibration Factor} = K_1 = \frac{\text{Displacement}}{\text{Indicator Units}}$$

$$\text{For Run 1, } K_1 = \frac{32,000 - 8,000}{30,535 - 6,860} = \frac{24,000}{23,675} = 1.014$$

$$\text{For Run 2, } K1 = \frac{32,000 - 8,000}{30,430 - 6,785} = \frac{24,000}{23,645} = 1.015$$

Use $K_1 = 1.01$ μ inches per indicator unit

3.10 Remove the BDG from the calibration jig and disconnect the wires from the strain indicators. Tape the ends of the wires together.

3.11 Thread the conductor cable through the chuck and water swivel and over the clothesline pulley attached to the top of the derrick if testing from ground surface. Reconnect the wires to the strain indicators exactly as during calibration.

3.12 Take zero deformation readings for each axis and record on the Field Data Sheet (Fig. 10) in the row labeled "zero" and in the three columns labeled U_1, U_2, and U_3. If just one indicator is being used, use a switching unit. If a switching unit is not available, the wires must be changed for each axis. Check each axis by applying slight finger pressure to opposing pistons and releasing. The balance needle should deflect, then return to the balanced position. Make sure the correct axis is connected to the correct strain indicator.

3.13 Engage the orientation pins of the BDG with the placement tool using a clockwise motion. Secure the conductor cable with the wire retainer clip on the placement tool. Make sure the orientation pins of the BDG are aligned with the U_1 axis. Push the gage through the stabilizer tube and about 9 in. into the EX hole. With the gage at test depth, orient the U_1 axis along the scribe mark by turning clockwise. If the BDG feels too loose or too tight in the EX hole, it must be removed. If too tight, remove one washer from one piston of each axis and try again. If too loose, add one washer to one piston of each axis. To add or remove washers, pull the piston out with the special pliers, unscrew the two piston halves with two pairs of special pliers, add or remove washers, and screw piston halves back together. Be careful not to damage the O ring. Regrease O ring and reinstall piston in gage. Do this to only one piston in each diametral pair initially. If gage is still too tight or loose, repeat with remaining pistons.

3.14 With the gage installed at test depth and correctly oriented, check the bias of the gage on the strain indicators. The bias set on each component should be between 10,000 and 15,000 indicator units

Hole No. ________________ Date __________ Orientation: U_1 ________________

Gage No. ______________ Calibration Factor U_1 ________________

Gage factor _____________ U_2 ________________

True Bearing of Hole _____________ U_3 ________________

DEPTH			DEFORMATION			TIME			TEMP.		REMARKS
			INDICATOR READING								
Gage	Hole (+)		U_1	U_2	U_3	Gage Set	Overcore Start	Deformation Read	Rock	Water	
		Zero									
9"	Face	Bias									
	1/2"										
	1"										
	1 1/2"										
	2"										
	2 1/2"										
	7"										
	7 1/2"										
	8"										
	8 1/2"										
Pistons	9"										
	9 1/2"										
	10"										
	10 1/2"										
	11"										
	13"										
	13 1/2"										
	14"										
	14 1/2"										
	15"										
	15 1/2"										
	16"										
	16 1/2"										
	17										
	17 1/2"										
	18										

Note. Next relief would start at 18 inches and go to 36 inches and gage would be orientated at a depth of 27 inches.

Fig. 10. Field data sheet.

with a gage factor of 0.40 for overcoring strain relief tests. For recompression tests in the biaxial chamber, the bias should be between 4,000 and 8,000 indicator units with a gage factor of 0.40. With a gage factor of 1.50, the bias should be between 2300 and 3400 indicator units for overcoring tests and between 900 and 1800 indicator units for recompression tests. Care must be taken to avoid overloading the transducers. Maximum load on any component should not exceed 20,000 indicator units with a gage factor of 0.40 and 4560 units with a gage factor of 1.50.

3.15 Turn the placement tool counterclockwise approximately 60 deg to disengage it from the BDG and remove the tool. (When retrieving the BDG, the tool is lowered onto the orientation pins and turned 60 deg clockwise.)

3.16 Pull the slack conductor cable through the chuck and over the clothesline pulley. Avoid excess tension in the cable or the gage may be pulled out of the EX hole. Tie off the cable and close the drill. Couple the NW casing to the chuck adaptor.

3.17 Turn on water. Allow approximately 10 minutes for gage, water, and rock to reach temperature equilibrium. Obtain new zero readings for each axis.

3.18 With the 6-in. bit resting on the bottom of the hole, tape a yardstick to the drill stand with the end flush with the bottom of the truck bed. As overcoring proceeds, check the advance rate by timing the descent of the yardstick with a stopwatch. Alternatively, the exposed casing may be marked at 1/2-in. increments to regulate the advance rate.

3.19 Start overcoring at a penetration rate of 1/2 in. per 40 seconds and a chuck speed of 50 rpm. The stopwatch is used to calibrate the drill to this rate. Each 1/2-in. penetration should be signaled to the recorder who records the indicator readings for each axis on the field data sheet. Overcore approximately 12 to 18 in. at this rate. If the core breaks during overcoring, the needles on the strain indicators will fluctuate erratically or the cable will twist. If either happens, stop overcoring immediately and retrieve the gage and core. If overcoring is successfully completed, stop the drill and continue to take periodic readings, with the water still running, until no appreciable changes in readings are occurring. This may take only a few minutes or it may take 2 or 3 hours depending on the rock.

3.20 Disconnect the wires from the strain indicators and tape the end of the cable so the drill can be uncoupled and raised without applying excess tension to the cable.

3.21 Pull the cable end back through the water swivel and chuck.

3.22 Secure the cable to the placement tool with the retainer clip and insert the tool over the BDG. When the placement tool engages the pins on the BDG, turn the tool 60 deg clockwise to secure the BDG. Pull the BDG and cable out of the hole.

3.23 Remove the core barrel and NW casing.

3.24 Retrieve the core (if it was not brought up inside the barrel) using the core breaker and core puller (or shovel in horizontal holes).

3.25 Plot the change in indicator units versus inches overcored for each test as shown in Fig. 11.[8.2] Compare this plot with the plot of an idealized overcore test (Fig. 12[8.2]) to determine if the test was successful.

3.26 Repeat this procedure for each additional test.

4. Procedure for Determining Young's Modulus of Elasticity of the Rock Core

4.1 The retrieved rock core should be tested in a biaxial chamber (Fig. 4) as soon as conveniently possible after recovery to determine the modulus of elasticity.

4.2 Place the calibrated BDG in the EX hole in the core at the same point and orientation where the in situ test was performed (align the U_1 axis of the BDG with the scribe mark).

4.3 Slide the rubber membrane over the rock core and place in the biaxial chamber.

4.4 Record initial or zero readings for all axes.

4.5 Increase hydraulic pressure in increments up to the measured in situ strain level and unload in identical increments.

4.6 Record deformation readings for each axis at each loading and unloading increment.

4.7 Repeat steps 4.4 through 4.6 for a second cycle.

4.8 Plot the applied pressure versus diametral deformations for each axis as shown in Fig. 13.[8.2] To calculate the average modulus value, E, obtain the differences in deflections corresponding to the differences in applied pressures on the second unloading cycle and use Equation 5 in the calculations section (see paragraph 6).

4.9 This test procedure requires an intact piece of core at least 10-1/2 in. long.

4.10 Alternatively, the modulus may be obtained by testing the NX core from the same depth in the nearby exploratory NX hole in uniaxial compression using standard procedures described in ASTM Standard Method of Test for Elastic Moduli of Rock Core Specimens in Uniaxial Compression, ASTM Designation D3148-72. This test method is described in Section 201-80 of this handbook.

5. Troubleshooting Equipment Malfunctions

5.1 If balance on one or more indicators cannot be achieved.

5.1.1 Check wiring hookup against wiring diagram (Fig. 9). Make sure all connections are tight.

5.1.2 Check cable connector plug in BDG. Remove screws from placement end of gage, slide the end off, unscrew the knurled retaining cap, and check the plug connection. Push in firmly if loose.

5.1.3 Nonbalance may occur when too much tension has been applied to the conductor cable during gage retrieval. Sometimes in a vertical hole, cuttings or rock fragments drop into the 1-1/2-in. hole on top of the gage, making it impossible to hook the placement and retrieval tool onto the gage pins. A tendency always exists to try to retrieve the gage by pulling on the cable. Do this only as a last resort. Instead, snap the core off and bring it up with the gage inside if overcoring has been successfully completed.

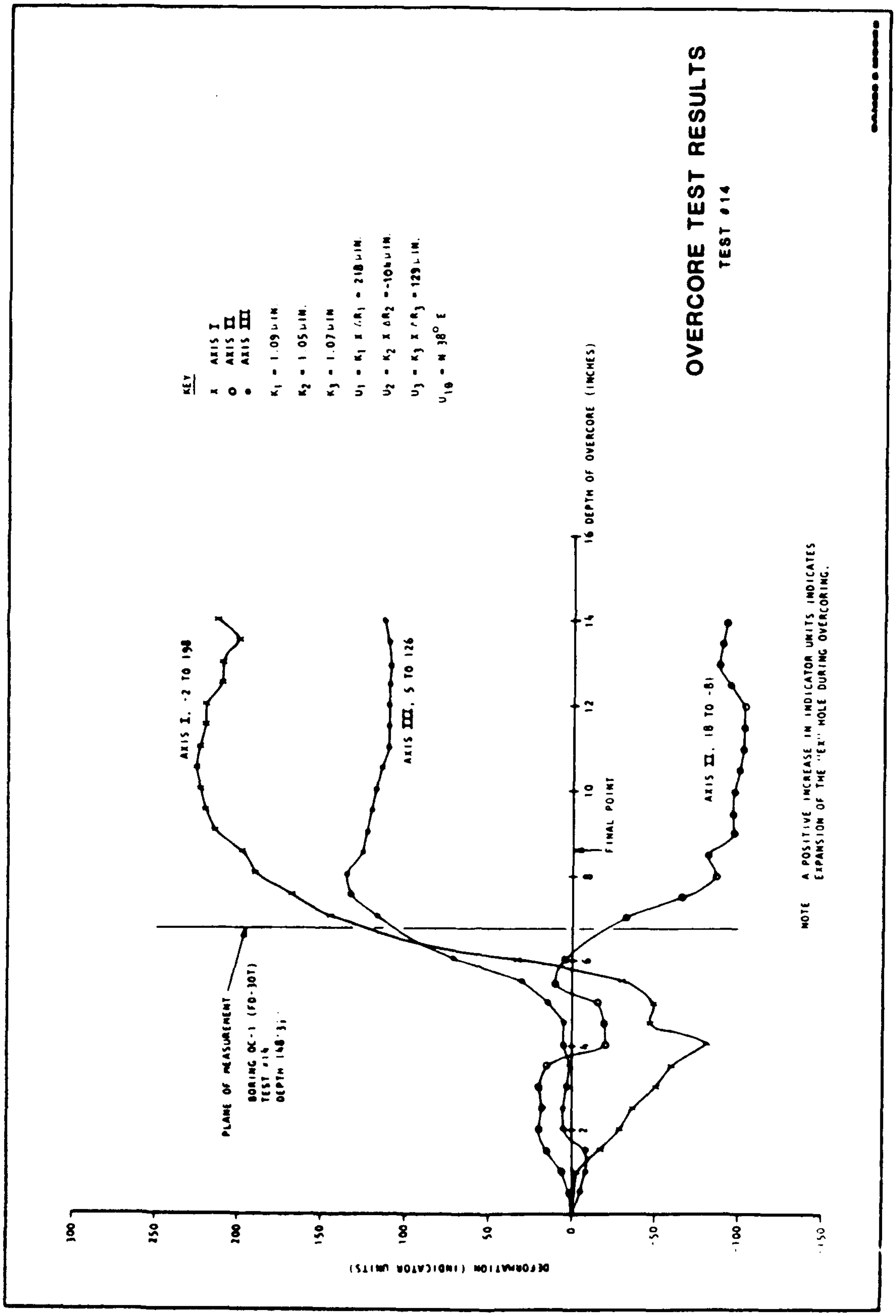

OVERCORE TEST RESULTS
TEST #14
KEY
x AXIS I
o AXIS II
• AXIS III
K1 = 1.09 μIN.
K2 = 1.05 μIN.
K3 = 1.07 μIN
U1 = K1 x ΔR1 = 218 μIN.
U2 = K2 x ΔR2 = -106 μIN.
U3 = K3 x ΔR3 = 129 μIN.
U1θ = N 38° E
AXIS I, -2 TO 198
AXIS III, 5 TO 126
AXIS II, 18 TO -81
PLANE OF MEASUREMENT
BORING OC-1 (FD-30T)
TEST #14
DEPTH 148'3"
FINAL POINT
DEPTH OF OVERCORE (INCHES)
DEFORMATION (INDICATOR UNITS)
300
250
200
150
100
50
0
-50
-100
-150
2
4
6
8
10
12
14
16
NOTE A POSITIVE INCREASE IN INDICATOR UNITS INDICATES EXPANSION OF THE "EX" HOLE DURING OVERCORING.

Fig. 11

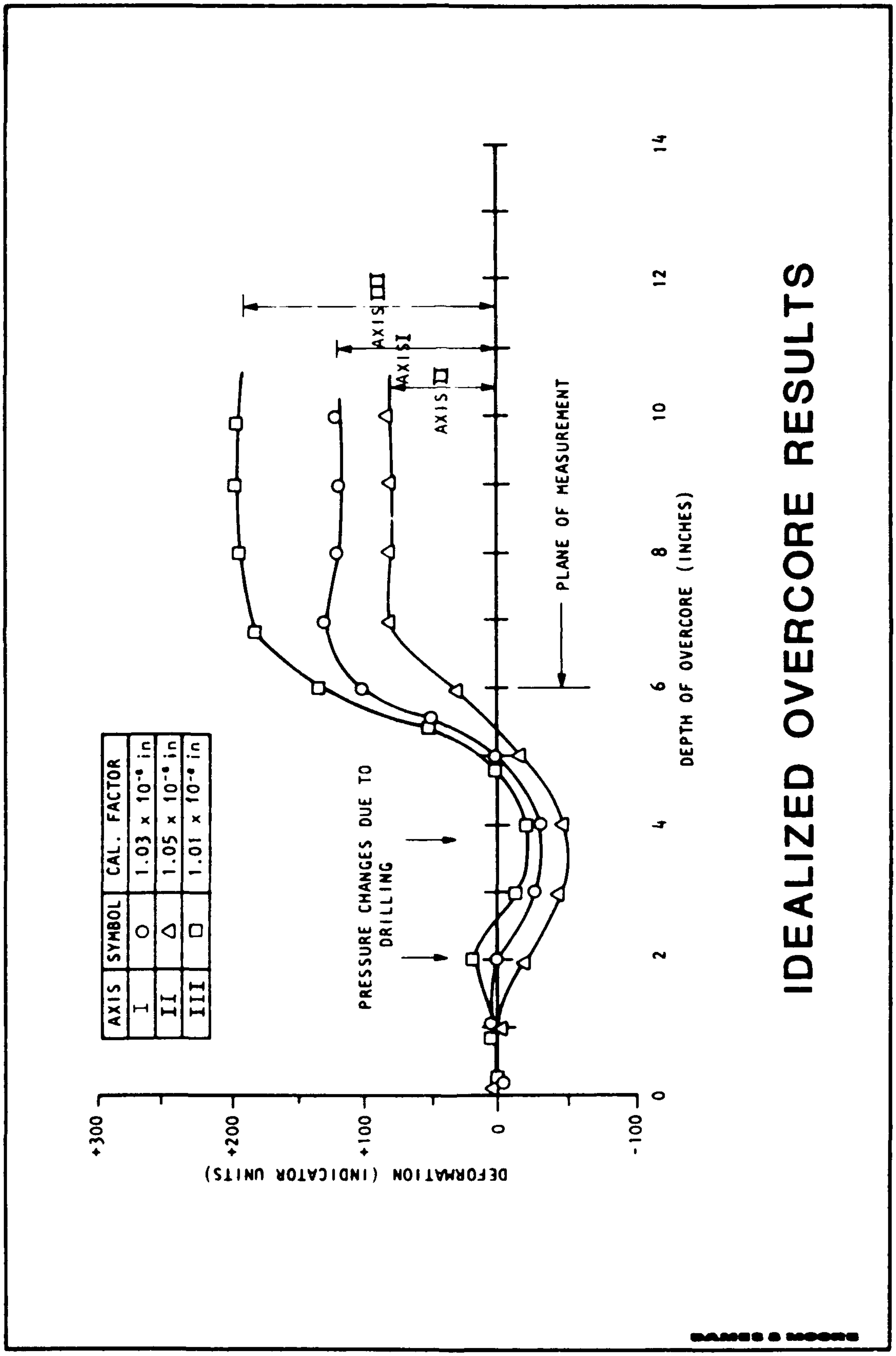
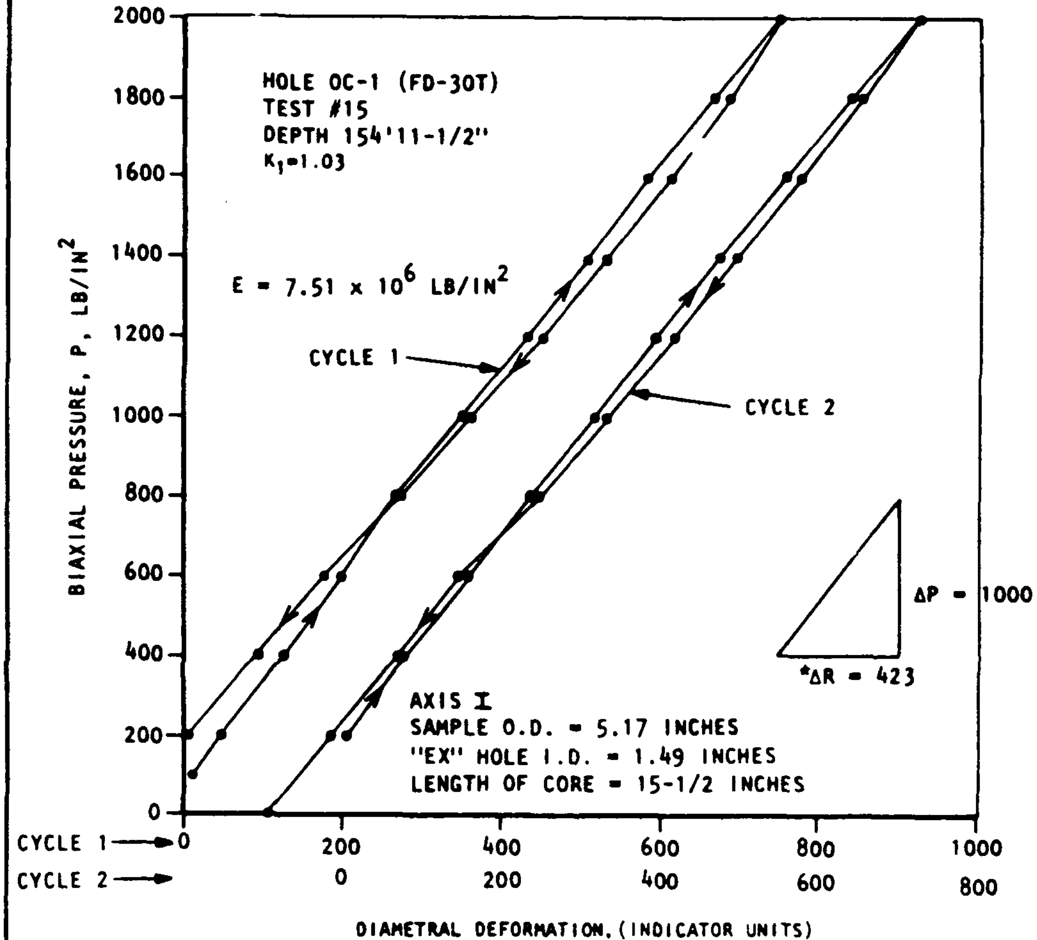

IDEALIZED OVERCORE RESULTS
AXIS | SYMBOL | CAL. FACTOR
I | ○ | 1.03 x 10^-6 in
II | △ | 1.05 x 10^-6 in
III | □ | 1.01 x 10^-6 in
PRESSURE CHANGES DUE TO DRILLING
PLANE OF MEASUREMENT
AXIS III
AXIS I
AXIS II
DEFORMATION (INDICATOR UNITS)
+300
+200
+100
0
-100
DEPTH OF OVERCORE (INCHES)
0
2
4
6
8
10
12
14
DAMES & MOORE

Fig. 12



BIAXIAL TEST RESULTS

TEST #15

AXIS I

*ΔR = CHANGE IN INDICATOR UNITS CORRESPONDING TO CHANGE IN APPLIED PRESSURE ΔP.

5.2 If core breaks during overcoring:

5.2.1 If the indicators suddenly start to fluctuate erratically or the needles indicate maximum deflection, the core has probably broken. This situation may also be indicated by twisting of the conductor cable. If this situation occurs, stop the test immediately.

5.2.2 Disconnect the NW casing from the chuck and insert the retrieval tool, but leave the 6-in. bit on the bottom. If retrieval tool will slide over BDG pins, retrieve the BDG. If the retrieval tool will not slide over the pins a piece of rock has probably fallen in on top of it.

5.2.3 If overcoring has proceeded past the end of the gage, snap the core off below the gage and bring core and gage up inside the core barrel. Keep light tension on conductor cable as gage is brought up. If the core does not come up with the barrel, use the core puller.

5.2.4 If overcoring has not proceeded past the end of the gage, try gently tugging on the cable to free the BDG. If this fails, tug sharply on the cable. It will snap off, usually at the connector plug. Retrieve the cable and resume overcoring several inches past the gage (Note 3).

NOTE 3—Snapping the cable off is a drastic measure but is a necessary trade-off to retrieve the gage in working condition. Never raise the 6-in. bit until you are certain that the bit has overcored past the gage. Then snap off core and bring the bit, core, and gage up.

5.3 If one or more elements become insensitive on indicators:

5.3.1 If elements become insensitive to deflection of the pistons or unresponsive to turning of the indicator dial, or the needles drift, water has probably caused a short at the strain gage connections or at the cable connector plug.

5.3.2 Remove the pistons with the special pliers and check for moisture. If moisture is present, dry area thoroughly, check O rings for damage, and replace them if necessary. Apply thin coat of silicone grease to O rings before reinserting pistons.

5.3.3 Check the cable connector plug by removing the upper gage case. Unscrew the retaining cap and check for moisture on plug. Dry plug and surrounding area and grease cable where it passes through retainer cap and rubber grommet. Reassemble gage.

5.4 If one component does not balance anywhere on the indicator dial or balances intermittently:

5.4.1 This situation indicates a disconnected wire or a cold solder joint. Remove the borehole gage case and check all wires and connections, including the cable connector plug. Solder where needed and reassemble gage.

5.5 If indicators are sensitive to touch:

5.5.1 If indicator needles deflect when the units are touched, it is usually a result of prolonged use in a damp environment. Use plastic or other insulating material underneath the indicators as a moisture barrier. Store indicators in a dry place when not in use to allow them to dry out.

6. Calculations

6.1 To determine the secondary principal stress magnitudes and directions it is usually convenient to assume the rock is a linearly elastic isotropic material.

6.2 The diametric deflections are obtained for each of the three axes of measurement by multiplying the indicator reading differences by the appropriate calibration factor. These values, U_1, U_2, and U_3, along with the calculated modulus of elasticity, form the basis for evaluating the maximum and minimum stresses acting in a plane perpendicular to the borehole walls. These stresses are principal stresses only when the borehole is parallel to the third principal stress, which is not always the case for vertical boreholes. However, under the conditions of more or less homogeneous, gently dipping rocks of low relief, it can be assumed that the third principal stress is vertical and equal to the overburden stress and that the maximum and minimum stresses perpendicular to the borehole walls are in fact principal stresses. For other conditions and different borehole orientations this assumption would be invalid, so consideration must be given to the special features and conditions appropriate to the individual site. Fairhurst[8.3] presents a solution for the state of stress in a transversely isotropic medium, that is, one for which the elastic properties are constant in any direction in a given plane but change in directions that intersect the plane. Fairhurst states that the assumption of elastic isotropy can result in errors in the computed stresses of as much as 50 percent in cases where the rock is anisotropic or "transversely isotropic" (typical of rocks such as shale and gneiss).

6.3 The overburden stress, σ_v, is equal to the average density of the overlying material, γ, multiplied by the depth of overburden, H, or

$$\sigma_v = \gamma H \qquad (1)$$

6.4 The secondary principal stresses are calculated as shown below, using equations based on a plane stress analysis of an elastic, isotropic thick-walled cylinder as discussed by Obert and Duvall.[8.4]

$$P_c = \frac{E}{6d}\left[U_1 + U_2 + U_3 + \frac{\sqrt{2}}{2}\left\{(U_1 - U_2)^2 + (U_2 - U_3)^2 + (U_3 - U_1)^2\right\}^{1/2}\right] \qquad (2)$$

$$Q_c = \frac{E}{6d}\left[(U_1 + U_2 + U_3) - \frac{\sqrt{2}}{2}\left\{(U_1 - U_2)^2 + (U_2 - U_3)^2 + (U_3 - U_1)^2\right\}^{1/2}\right] \qquad (3)$$

where U_1, U_2, U_3 are measurements of diametral deformations along three axes 60 deg apart. Deformation is positive for increasing diameter during overcoring.

P_c is the maximum normal stress, psi.*

Q_c is the minimum normal stress, psi.*

E is the modulus of elasticity, psi, and d is the "EX" hole diameter.

6.5 The orientation of the principal stress axis is given by

$$\theta_p = 1/2 \text{ arc tan } \frac{\sqrt{3}\,(U_2 - U_3)}{2U_1 - U_2 - U_3}. \qquad (4)$$

where θ_p is the angle from the U_1 axis (positive in a counterclockwise direction) to the major principal stress.

U_1, U_2, U_3 are the diametral deformations.

The angle θ_p could have two values 90 deg apart. The correct angle can be determined using the following rules. For a 60 deg rosette (angular measurements positive in the counterclockwise direction and all angles measured from U_1 to P_c):

6.5.1 If $U_2 > U_3$, θ_p lies between 0° and $+90^\circ$ or -90° and -180°.

6.5.2 If $U_2 < U_3$, θ_p lies between 0° and -90°.

6.5.3 If $U_2 = U_3$, and if: (a) $U_1 > U_2 = U_3$, $\theta_p = 0^\circ$

(b) $U_1 < U_2 = U_3$, $\theta_p = \pm 90^\circ$.

6.6 The modulus of elasticity is calculated using the biaxial test results in the equation

$$E_i = \frac{(4ab^2)\ (\Delta Pi)}{(b^2 - a^2)\ \Delta Ui} \qquad (5)$$

---

* Positive values of P_c and Q_c indicate compressive stresses.

where E_i is the modulus of elasticity, psi

a is the diameter of the "EX" hole, inches

b is the radius of the core, inches

ΔPi is the change in applied pressure, psi

ΔUi is the diametral deformation, in inches corresponding to the change in applied pressure

i is the direction of the axis

7. Reporting Results

7.1 The report shall include:

7.1.1 A description of the test site, including a general area map, characteristic features, and the type and depth of rock at which tests were performed.

7.1.2 The test procedure and apparatus used in the field should be described, including any innovations in procedure or apparatus. The assumptions and equations used in calculating stresses should be presented.

7.1.3 For each test the sample should be described, including depth and rock type and description of joints present. Young's Modulus should be tabulated and a plot of the biaxial test should be presented, as in Fig. 13, along with a plot of the overcoring test results, as in Fig. 11. Calibration factors should be tabulated for each test. The maximum and minimum secondary principal stresses and the orientation should be tabulated for each test.

7.1.4 Any difficulties or unusual circumstances encountered should be noted.

8. References

8.1 Hooker, Verne E., and Bickel, David L., "Overcoring Equipment and Techniques Used in Rock Stress Determination," U. S. Bureau of Mines Information Circular 8618, Denver Mining Research Center, Denver, Colorado, 1974.

8.2 Nataraja, Mysore, "In Situ Stress Measurements, Park River Project," Miscellaneous Paper No. S-77-22, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi, 1977.

8.3 Fairhurst, Charles, "Methods of Determining In Situ Rock Stresses at Great Depths," Technical Report No. 1-68, U. S. Army Corps of Engineers, Missouri River Division, Omaha, Nebraska, 1968.

8.4 Chert, L., and Duvall, W. I., "Rock Mechanics and the Design of Structures in Rock," John Wiley and Sons, Inc., New York, 1967.

SUGGESTED METHOD FOR DETERMINING STRESS BY OVERCORING A PHOTOELASTIC INCLUSION

Scope

1. (a) This test determines in situ stress by sensing strain transmitted during overcoring from the rock to a stiff inclusion coupled in a drill hole.[1] The translation of strain to stress is accomplished by calibration of the inclusion at known stresses. The hard inclusion is sometimes described as a stressmeter.

(b) Maximum and minimum stresses are measured in the plane normal to the drill hole axis. At least three tests in separate drill holes will generally be needed to estimate directions and magnitudes of principal stresses.

(c) The method is relatively inexpensive.

(d) Rock anisotropy and fine dependent deformations complicate interpretation, and may lead to serious errors if ignored.

(e) Another overcoring method measures strain of the drillhole rather than the response of a hard inclusion, See RTH-341.

Apparatus

2. Equipment for drilling a 3-in. emplacement hole in any direction.[2]

3. Equipment for overcoring in any direction at the diameter of 9-in. Included is an axial stabilizer or other device for maintaining concentricity with the smaller center hole.

4. Optical transducer assembly (Figure 1) consisting of:

(a) Hard disk inclusion of glass or other photoelastic material, with axial hole.

(b) Waterproof light source positioned beyond the inclusion, powered by an outside source. Leads from the outside source pass axially through the inclusion.

(c) Means of mechanically or chemically bonding the inclusion to the hole wall.

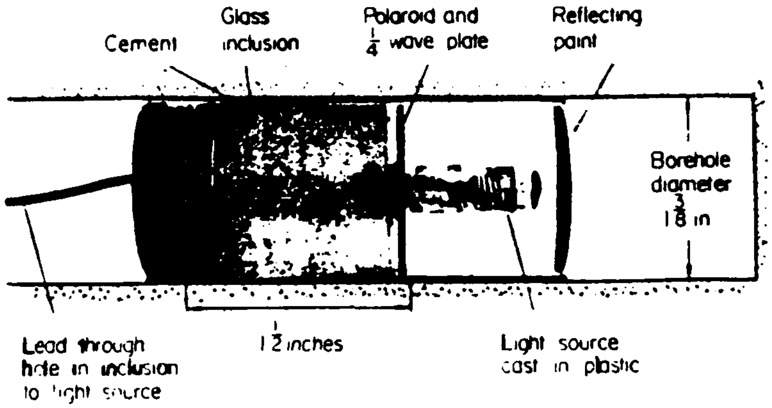


Figure 1. Example of photoelastic stressmeter.

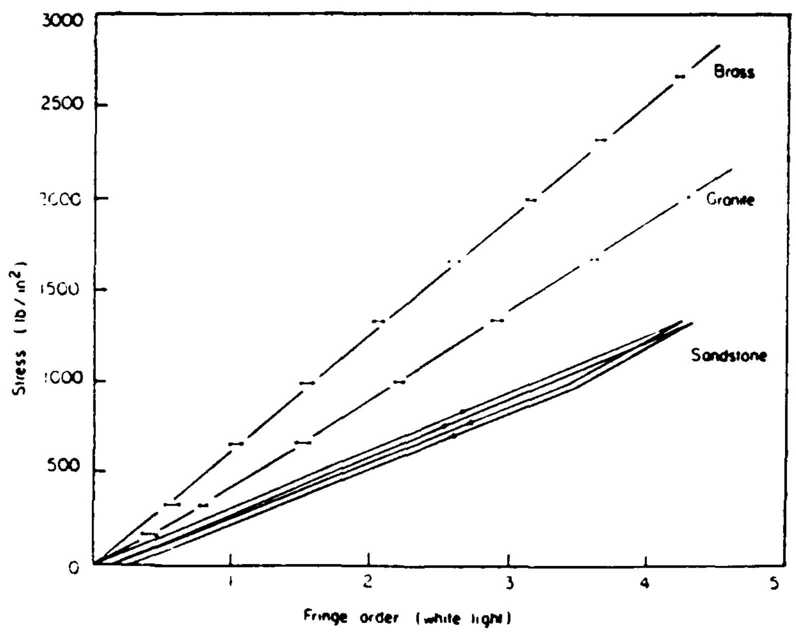


Figure 2. Example calibration curves for photoelastic glass inclusion in three materials.

(d) Analyzing and polarizing plates supplemented with telescope for distant viewing.

5. Tools for positioning and wedging or bonding the inclusion.

6. Calibrating equipment consisting of:

(a) Block of material having modulus like rock of interest and containing a 3-in. hole.

(b) Loading machine and recorder.

Procedure

7. Preparation

(a) The inclusion is calibrated by mounting the complete assembly in the test block and subjecting the block to known loads. Calibration curves are prepared (Figure 2).

(b) The site is selected emphasizing hard, homogeneous rock with no fractures in the volume to be overcored.

(c) If testing will be below the water table, the test volume should be dewatered and kept dry during testing.[3]

(d) The 3-in. hole should be cored to the full depth for all testing and the core should be logged and inspected to confirm suitability of the test intervals.[4]

(e) The optical transducer assembly is emplaced in the 3-in. hole and the hard inclusion is bonded to the side wall. There should be at least 9-in. of open hole between collar and inclusion and inclusion and far end.

8. Testing:

(a) The assembly is checked, a pretest observation of birefringence is made and documented, and lead lines are temporarily disconnected from the outside.[5]

(b) The 9-in. overcore hole is drilled precisely along the same axis to a depth 9-in. beyond the hard inclusion. The drill is withdrawn.

(c) Lead lines are reconnected.[5]

(d) Birefringence is observed (Figure 3 and 4) and is recorded photographically and/or by accurate drawing. A vertical reference should be established. Retrievable portions of the assembly are withdrawn.

(e) The inclusion and polarizing plate are usually not retrieved.

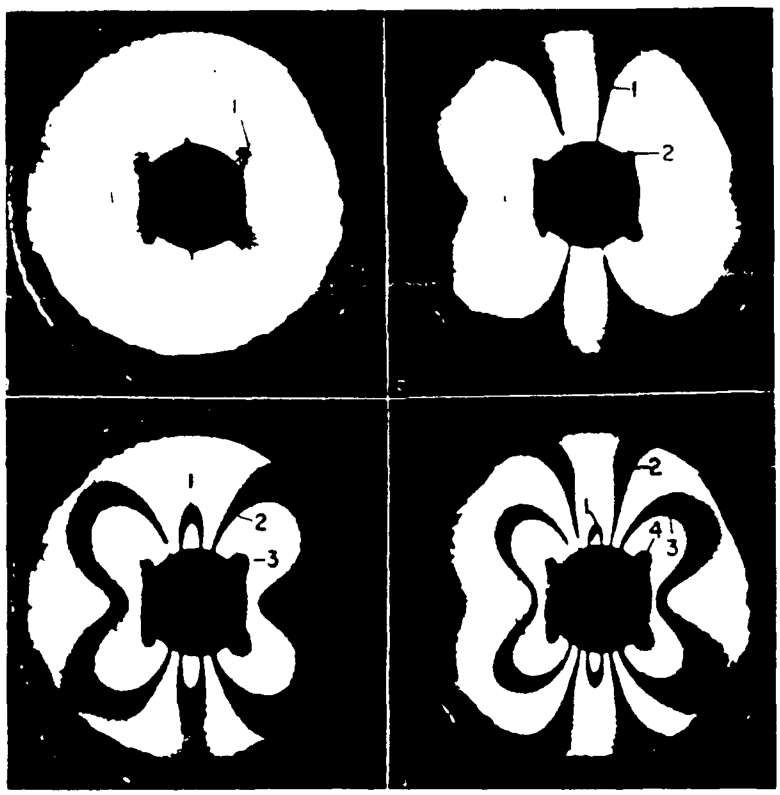


Figure 3. Patterns displayed by the photoelastic disk under increasing uniaxial loading

(f) Where another test is needed at greater depth, the overcored rock stub is broken and removed to reopen the 3-in. hole.

Calculations

9. The glass inclusion with hole along its axis forms a biaxial gage displaying birefringence patterns which identify strain in the glass and, by calibration, stress in the rock. The following characteristics are diagnostic (Figures 3 and 4):

(a) The axes of symmetry of the signal identify the principal stress directions.

(b) The direction in which the signal moves with increase of stress, and the presence of isotropic points in biaxial fields, identifies the major principal stress direction.

(c) The fringe order at a selected point of reference on the pattern is measured to give the major principal stress directly in terms of a calibration factor for any particular principal stress ratio (Figure 1).

(d) The ratio between major and minor principal stresses is indicated approximately by the shape of the signal and precisely by the measured distance between two isotropic points on the major axis.

(e) The manner in which the optical pattern changes, when the analyzer of the polariscope is operated in the process of taking the measurement, identifies whether the measured stress is tensile or compressive.

10. Calculations based on elastic theory are unnecessary in following the procedure presented in paragraph 9. However, the calibration must be well founded in theory. Fortunately, the effects are similar for all materials of great stiffness[1] and even metal may be substituted for site rock during calibration.

Reporting

11. The report should include the following:

(a) Position and orientation of test.

(b) Logs and other geological descriptions of rock near the test. Geological structure and elastic properties are particularly important.

(c) Record of test activities.

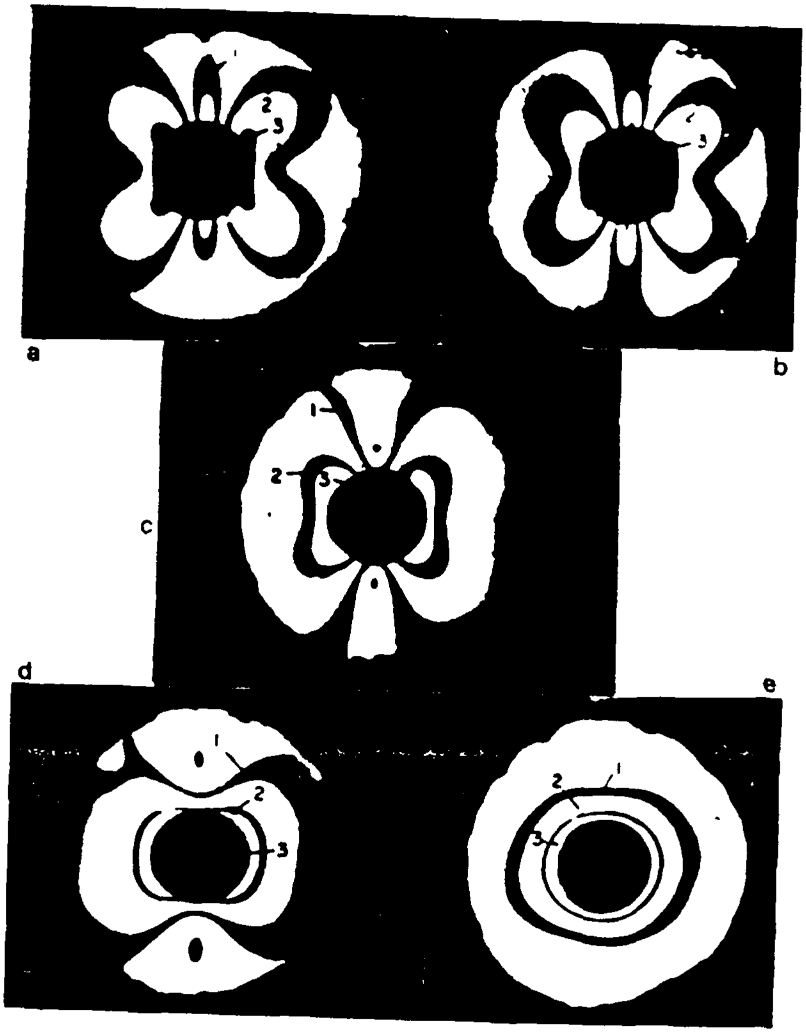


Figure 4. Patterns displayed by photoelastic disk in various stress states, all at third fringe order.

(d) Accurate drawings or photographs of birefringence patterns including that prior to overcoring.

(e) Biaxial stresses indicated from calibration curves.

(f) Description of calibration investigations and resultant curves. Where several inclusions are calibrated at once, a reference to description elsewhere may be sufficient.

Notes

[1]The stress induced in a stiff inclusion will be about 1.5 times the comparable stress in the host rock provided the elastic modulus of the inclusion exceeds that of host by a factor of 5 or more (Roberts 1968).

[2]Diamond core drilling is recommended for obtaining the necessary close tolerance between wall and inclusion.

[3]The test is most commonly conducted in tunnel walls where water is usually not a major problem.

[4]The 9-in. hole is sometime started first and advanced beyond any disturbed zone along the excavated surface.

[5]If the light is retrievable, it is withdrawn temporarily for overcoring.

References

1. Roberts, A., "The Measurement of Strain and Stress in Rock Masses," Chapter 6 in Rock Mechanics in Engineering Practice, John Wiley & Sons, New York, 1968.

PART II: IN SITU STRENGTH METHODS

D. Determination of Rock Mass Deformability

SUGGESTED METHOD FOR DETERMINING ROCK MASS DEFORMABILITY USING A PRESSURE CHAMBER

Scope

1. (a) This test determines the deformability of a rock mass by subjecting the cylindrical wall of a tunnel or chamber to hydraulic pressure and measuring the resultant rock displacements. Elastic or deformation moduli are calculated in turn.

(b) The test loads a large volume of rock so that the results may be used to represent the true properties of the rock mass, taking into account the influence of joints and fissures. The anisotropic deformability of the rock can also be measured.

(c) The results are usually employed in the design of dam foundations and for the proportioning of pressure shaft and tunnel linings.

(d) Two other methods are available for tunnel-scale deformability. See RTH-366 and RTH-367 to compare details. Potentially large impacts, especially in terms of cost, of variations at this scale justify the separation of methods.

(e) This method reflects practice described in the references listed at the end.

Apparatus

2. Equipment for excavating and lining the test chamber including:

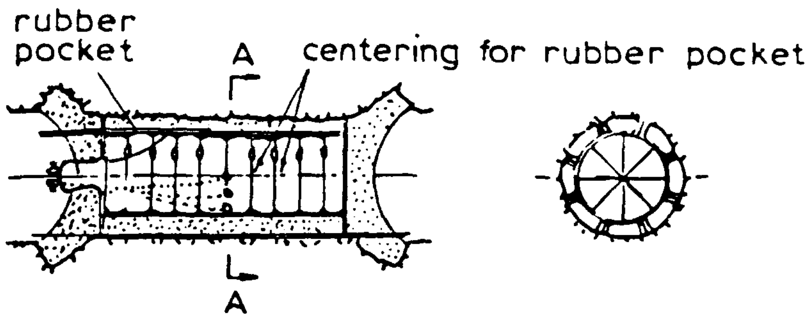
(a) Drilling and blasting materials or mechanical excavation equipment.[1*]

(b) Materials and equipment for lining the tunnel with concrete or flexible membrane.

3. A reaction block (Figure 1) usually comprising a tunnel bulkhead of sufficient strength and rigidity to resist the force applied by the pressurizing fluid. The bulkhead must also be waterproof. Reusable bulkheads are possible where several sites are to be investigated.

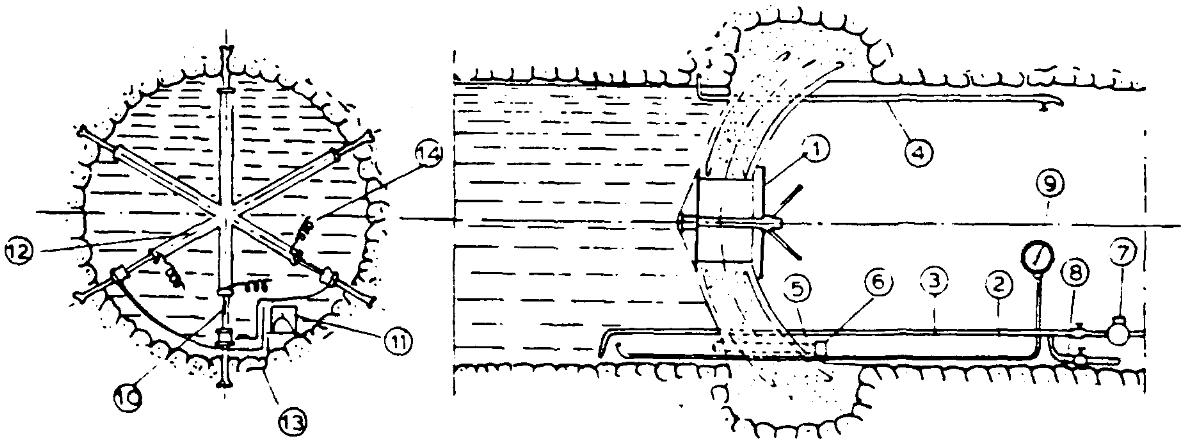
4. Loading equipment to apply the hydraulic pressure to the inner face of the lining including:

*Numbers refer to NOTES at end of the text.



(a) (b)

(A)



1. manhole
2. pressure gauge tube.
3. water inlet.
4. air outlet
5. cable admission tube
6. cable tube seal
7. water meter.
8. water outlet
9. pressure gauge.
10. vibrating meter
11. air pressure equalising chamber.
12. invar rods
13. air pressure equalising tube.
14. cable

(B)

Figure 1. Example Pressure Chamber.

(a) A hydraulic pump capable of applying the required pressure and of holding this pressure constant within 5% over a period of at least 24 hr. together with all necessary hoses, connectors and fluid.[2]

(b) Water seals to contain the pressurized water behind the bulkhead and along any vulnerable seams and boundaries of the membrane. Special water seals are also required to allow the passage of extensometer rods through the lining and instrumentation lines through the bulkhead. Pressurized water should not be allowed to escape into the rock since this may greatly affect the test results.

5. Load measuring equipment comprising one or more hydraulic pressure gages or transducers of suitable range and capable of measuring the applied pressure with an accuracy better than ±2%.

6. (a) Displacement measuring equipment to monitor rock movements radial to the tunnel with a precision better than 0.01 mm. Equipment may be extensometers, vibrating wire deflectometers, joint meters, or multipurpose strain meters. Changes in tunnel diameter have been measured with spring-loaded invar wires using LVDT's to sense. All measuring systems must be waterproof and designed to stand the pressure to which they are to be subjected.

Procedure

7. Preparation

(a) The test chamber location is selected taking into account the rock conditions, particularly the orientation of the rock fabric elements such as joints, bedding, and foliation in relation to the orientation of the proposed tunnel or opening for which results are required.

(b) The test chamber is excavated to the required dimensions.[1,3] Where possible, the dead end of the tunnel serves as one end of the test chamber, thus eliminating the need for a second bulkhead.

(c) The geology of the chamber is recorded and specimens taken for index testing as required.

(d) Extensometer holes are accurately marked and drilled, ensuring no interference between loading and measuring systems. Directions of measurement should be chosen with regard to the rock fabric and any other anisotropy.

(e) The bulkhead is installed.

(f) The chamber is lined with flexible, impermeable membrane. To protect this lining from sharp irregularities, the wallrock is first covered with thin reinforced concrete walling, gypsum board, or plaster. Rubber membrane may be stapled and vacuumed into place and the whole sprayed with rubber.

(g) Measuring equipment is installed and the equipment is checked. For multiple position extensometers the deepest anchor may be used as a reference provided it is situated at least 2 chamber diameters from the lining. Alternatively the measurements may be related to a rigid reference beam passing along the axis of the chamber and anchored of not less than 1 chamber diameter from either end of the chamber.

(h) Check water seals for leakage, if necessary by filling and pressurizing the chamber. Leaks are manifested as anomalous pressure decay and visible seepage through the bulkhead.

8. Testing

(a) The test is carried out in at least three loading and unloading cycles, a higher maximum pressure being applied at each cycle.[2]

(b) For each cycle the pressure is increased at an average rate of 0.05 MPa/min to the maximum for the cycle, taking not less than 3 intermediate sets of load-displacement readings in order to define a set of pressure-displacement curves.

(c) On reaching the maximum pressure for the cycle the pressure is held constant (±2% of maximum test pressure) recording displacements as a function of time until approximately 80% of the estimated long-term displacement has been recorded. Each cycle is completed by reducing the pressure to near-zero at the same average rate, taking a further three sets of pressure-displacement readings.

(d) For the final cycle the maximum pressure is held constant until no further displacements are observed. The cycle is completed by unloading in stages taking readings of pressure and corresponding displacements.

(e) The test equipment is then dismantled.[3]

Calculations

9. (a) The value of E is calculated from

$$E = \frac{p_i a^2}{r(U_r)}(1 + \nu)$$

where:

E = modulus of elasticity,

P_i = internal pressure,

a = radius to rock face - assuming circular chamber,

r = radius to point here deflection is measured,

U_r = change in radius due to pressure, and

ν = Poisson's ratio

and rock is regarded as linearly elastic.

(b) Where only surface deflections are measured, the equation is reduced to:

$$E = \frac{P_i a}{U_r} (1 + \nu)$$

Reporting

10. The report should include the following:

(a) Drawings, photographs, and detailed description of the test equipment, chamber preparation, lining, and testing.

(b) Geological plans and section of the test chamber showing features that may affect the test results.

(c) Tabulated test observations together with graphs of displacement versus applied pressure and displacement versus time at constant pressure for each of the displacement measuring locations.

(d) Transverse section of the test chamber showing the total and plastic displacements resulting from the maximum pressure. The orientations of significant geological fabrics should be shown on this figure for comparison with any anisotropy of test results. Calculated moduli should be shown also.

NOTES

[1]The recommended diameter is 2 m., with a loaded length about 5 m. The chamber should be excavated with as little disturbance as possible. Material disturbed by blasting may need to be removed since it tends to produce moduli lower than found at depth. However, blast effects are representative if the test results are applied directly as a "model" test to the case of a blasted full-scale tunnel.

[2]Maximum test pressure varies from 1.5 to 4.0 MPa.

[3]To assess the effectiveness of grouting, a second, adjacent chamber may be prepared. Grouting is carried out after completion of testing in the ungrouted chamber, and the equipment is then transferred to the grout chamber.

References

1. Clark. G. B., "Deformation Moduli of Rocks," in Testing Techniques for Rock Mechanics, *Special Technical Publication* 402, American Society for Testing and Materials, 1966, pp. 133-172.

2. Stagg, K. G., "In Situ Tests on the Rock Mass," in Rock Mechanics in Engineering Practice, John Wiley & Sons, New York, 1968, pp. 125-156.

3. Lama, R. D., and Vutukuri, V. S., Handbook on Mechanical Properties of Rock Vol. III, Trans Tech Publications, 1978, 406 pp.

4. International Society for Rock Mechanics, "Suggested Method for Measuring Rock Mass Deformability Using a Radial Jacking Test," International Journal of Rock Mechanics and Mining Sciences, V. 16, 1979, pp. 208-214.

PRESSUREMETER TESTS IN SOFT ROCK

1. Scope

The pressuremeter test consists of lowering an inflatable cylindrical probe into a predrilled borehole, expanding the probe laterally against the borehole wall, and recording the increase in size of the probe and associated pressure within the probe. This method covers the procedure for testing in soft rocks.

2. Principle of the Method

The pressuremeter probe is placed in the ground by lowering it into a predrilled hole. Once the probe is placed at the desired depth, the pressure in the probe is increased in equal increments and the associated increase on probe volume is recorded. The test is terminated if yielding in the rock becomes large. This procedure is repeated at the desired depth intervals but not closer than the length of the probe to the previously tested zone. A pressure-volume curve is plotted and a pressuremeter modulus is calculated.

3. Apparatus

3.1 The pressuremeter consists of two basic components: the probe and the pressure regulator-volumeter. Various sizes of probes are available to accommodate different borehole diameters. The probe consists of a light, flexible inner sheath and heavy, durable outer sheath, as shown schematically in Fig. 1. The inner sheath is pressurized with a liquid (water) through ports in the brass cylinder. The outer sheath is pressurized with a gas (normally dry nitrogen) through ports at each end of the cylinder. During testing, the outer sheath is kept at a pressure slightly less than that within the inner sheath. Normally, a pressure differential of 30 to 45 psi is maintained between sheaths since differences greater than 45 psi could possibly cause a rupture of the inner sheath. The bursting strength of the outer sheath depends on the deformability of the material being tested, but pressures in the range of 1000 to 1500 psi can often be obtained unless the surrounding medium has deformed excessively.

3.2 The purpose of the double-sheath arrangement is to simulate plane strain conditions. All volume change measurements are made within the inner sheath, although pressure is distributed along the entire length of the outer sheath; thus, end effects are greatly reduced.

3.3 The probe pressure is controlled by the pressure regulator-volumeter. The change of volume of the probe caused by the applied probe pressure is also monitored by this device. The probe is connected to the pressure regulator-volumeter by means of a coaxial tube, the inner tube being filled with liquid and the outer tube with gas.

4. Calibration

4.1 The probe must be calibrated to correct for its compressibility and inertia (Fig. 2). The compressibility of the sheaths, the fluid, and the coaxial tubing is determined by placing the probe into a rigid container, such as a pipe, and measuring the pressure-volumeter relationship. During a field test, the volume increase caused by the compressibility (V_p) of the probe system is deducted from that recorded by the volumeter at the corresponding field test pressure.

4.2 The inertia of the system is determined by inflating the instrument with no confining pressure and again determining the pressure-volume relationship. The pressure (P_p) required to inflate the probe to a given volume under no confinement can then be deducted from the (field) recorded pressure (plus pressure due to the head of water), which results in the true pressure exerted on the borehole wall during a field test.

4.3 Corrections for temperature changes and head losses due to circulating fluid are usually small and may be disregarded in routine tests.

4.4 Hydrostatic pressure (P_y) existing in the probe due to the column of fluid in the testing equipment must be determined before each test. This is accomplished by measuring the test depth (H) and multiplying the unit weight of the test fluid by the distance from the probe to the pressure gauge. This pressure must be added to the pressure readings obtained on the readout device.

5. Procedure

5.1 Drilling of the borehole must be performed in such a manner as to cause the least possible disturbance to the walls of the borehole and produce an adequate hole diameter for testing. The hole is advanced to the test level and cleaned of any debris or cuttings.

5.2 With the probe still at the surface, the fluid circuit valve open, and without applying pressure, an accurate setting of the zero volume reading (V_o) is accomplished by adjusting the water level in the instrument to zero. The volume circuit is then closed to prevent any further change in the volume of the measuring circuit. The probe is lowered to the test depth in this condition. Failure to close the valve will result in probe expansion as it is lowered into the hole. The test depth is determined as the depth to the midpoint of the probe.

5.3 Once the probe is positioned, the volume circuit is opened and the probe allowed to equalize under the hydrostatic head. Since the probe and inner coaxial tube are initially water-filled, a pressure equal to the head of water is exerted on the borehole walls at the beginning of each test. During loading, the pressure is increased in approximately 30-psi increments by controlling the pressure regulator valve. Volume measurements are recorded at lapsed times of 15, 30, and 60 sec after each pressure increase. The 60-sec readings are used for the modulus calculations. Typically, relatively large volume changes occur at low pressures as the probe is seated against the borehole wall; thus, the test usually indicates hardening response until the seating pressure p_o is reached (Fig. 3). This is followed by an essentially linear response range up to some pressure p_e. Above p_e, the curve again becomes nonlinear, but softening, as the material around the borehole begins to fail.

5.4 Once the maximum loading has been reached, or upon reaching the maximum expanded volume of the probe, the test is terminated; the probe is deflated to its original volume and withdrawn or repositioned in the hole at the next test depth. Cyclic testing may be performed when required by alternately inflating and deflating the probe.

6. Calculation

6.1 Calculate the pressure transmitted to the rock by the probe from the pressure readings as follows:

$$P = P_g + P_\gamma - P_p$$

where

P = pressure exerted by the probe on the rock (psi)

P_g = pressure reading on control unit (psi)

P_γ = hydrostatic pressure between control unit and probe (psi)

P_p = pressure correction due to inertia of instrument (psi)

For determination of P_p see paragraph 4.2. The pressure P_γ shall be the hydrostatic pressure as follows:

$$P = H \times \gamma_t$$

where

H = vertical distance from probe to pressure gauge (ft)

γ_t = unit weight of measuring fluid in instrument (lb/ft^3)

6.2 Calculate the increase in volume of the probe from the volume readings. The corrected increase in volume of the probe is calculated as follows:

$$V = V_R - V_P$$

V = Corrected volume increase of probe (cm^3)

V_R = Volume reading on readout device (cm^3)

V_P = Volume correction (cm^3)

The volume correction, V_P, shall be determined as outlined in paragraph 4.1.

6.3 Plot the pressure-volume increase curve by entering the corrected volume of the ordinate and the corrected pressure on the abscissa. Connect the points by a smooth curve. This curve is the corrected pressuremeter test curve and is used in the determination of the test results (Fig. 3).

7. Modulus Interpretations

If it is assumed that the material surrounding the pressuremeter probe behaves in a linear elastic manner and that the theory of thick-walled cylinders is applicable, then the applied internal pressure increment p and tangential and radial stress increments $\Delta\sigma_\theta$ and $\Delta\sigma_r$, respectively, are related by

$$-\Delta\sigma_\theta = \Delta\sigma_r = \Delta p$$

for compression positive. The volumetric strain $\Delta v/v$ of a unit length of the borehole probe may be related to the radial strain ϵ_r in the material by

$$\frac{\Delta v}{v} = \frac{\pi(r + \Delta r)^2 - \pi r^2}{\pi r^2} \approx \frac{2\Delta r}{r} \approx 2\epsilon_r$$

where v is the average total volume per unit length of the deformed borehole. From Hooke's law

$$\varepsilon_r = \frac{1}{E}(\sigma_r - \nu\sigma_\theta)$$

where ν is Poisson's ratio and E is Young's modulus. Thus

$$E = \frac{v\Delta p}{\Delta v} 2(1 + \nu)$$

or

$$G = \frac{E}{2(1 + \nu)} = \frac{v\Delta p}{\Delta v}$$

The above equations may be used to interpret the Young's modulus, E, from the pressuremeter tests only if Poisson's ratio is independently determined or assumed. The last equation indicates that the pressuremeter results can be used as a direct measure of shear modulus, G.

8. Limitations

The accuracy of the pressuremeter test is dependent in part upon the stiffness of the material being tested. For stiffer materials, the determination of the instrument compressibility (e.g., apparent change in volume per unit pressure when the probe is completely restrained from external volume change) is important since an increasing proportion of the measured volume response results from the instrument and not the material. The effect of any uncertainties in the instrument stiffness S_I of the pressuremeter (S_I is the reciprocal of the compressibility) on the predicted shear modulus is shown on Fig. 4. As seen in the figure, shear modulus calculations for materials in which the ratio of the instrument stiffness to shear modulus (S_I/G) is small will be less accurate than when the ratio is large, if the instrument compressibility at the time of the test is not accurately known.

9. Report

For each pressuremeter test a data form similar to Fig 5 shall be used.

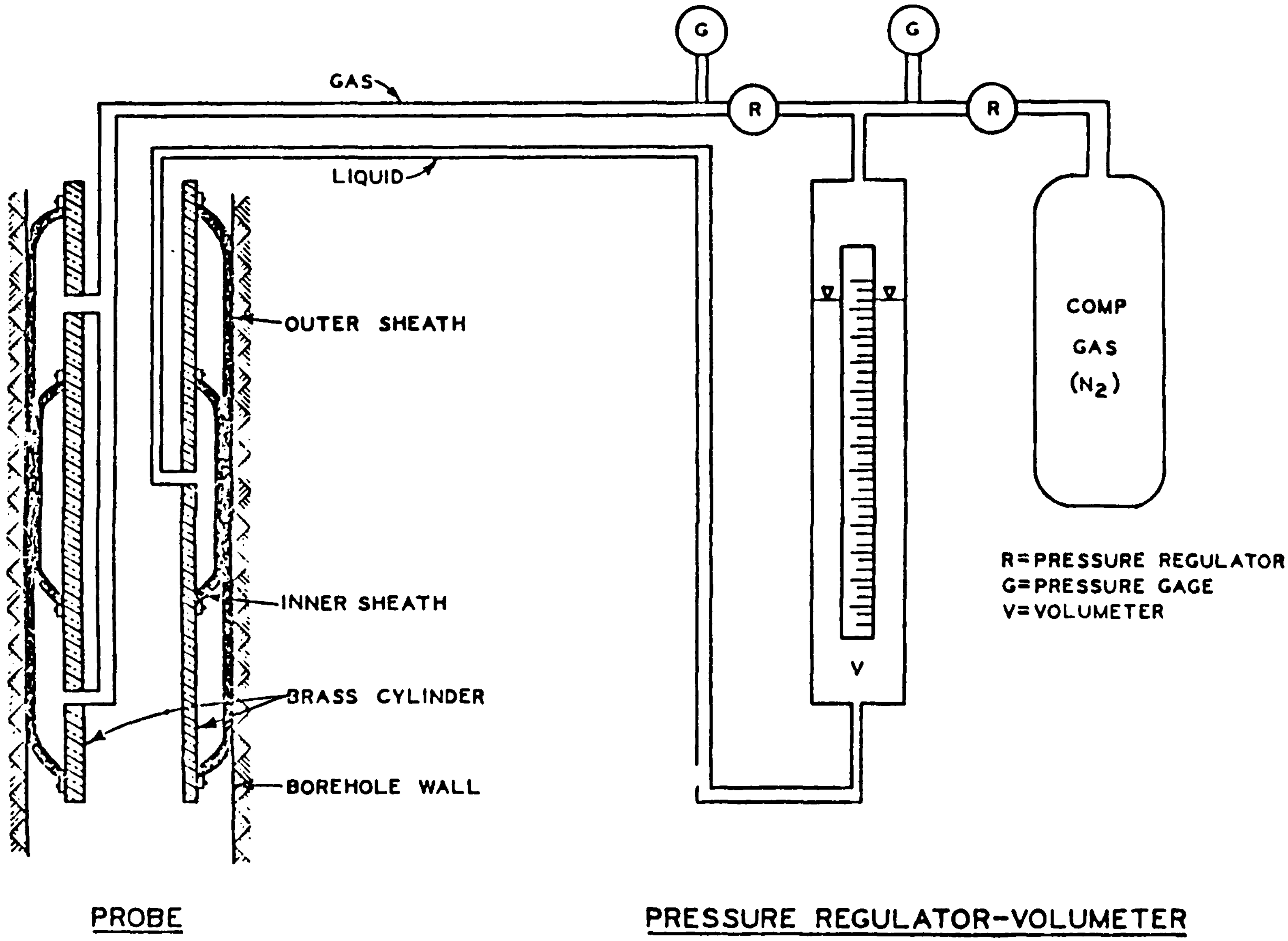


Fig. 1. Pressuremeter Schematic

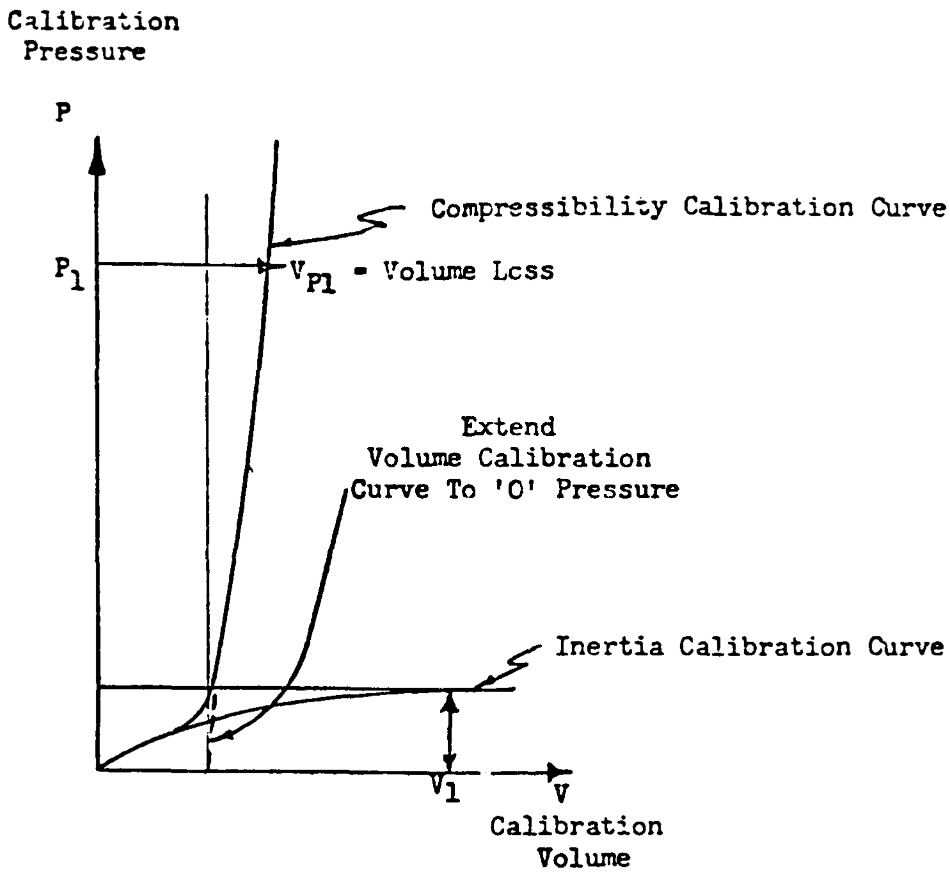


Figure 2. Calibration for Volume & Pressure Corrections.

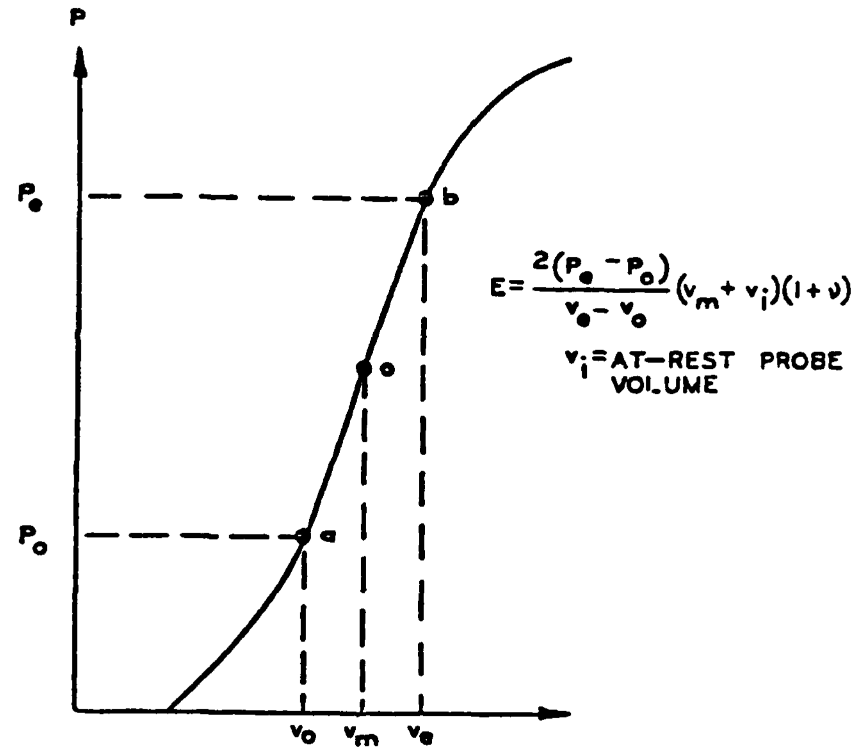


Fig. 3 Idealized pressure-volume relationship

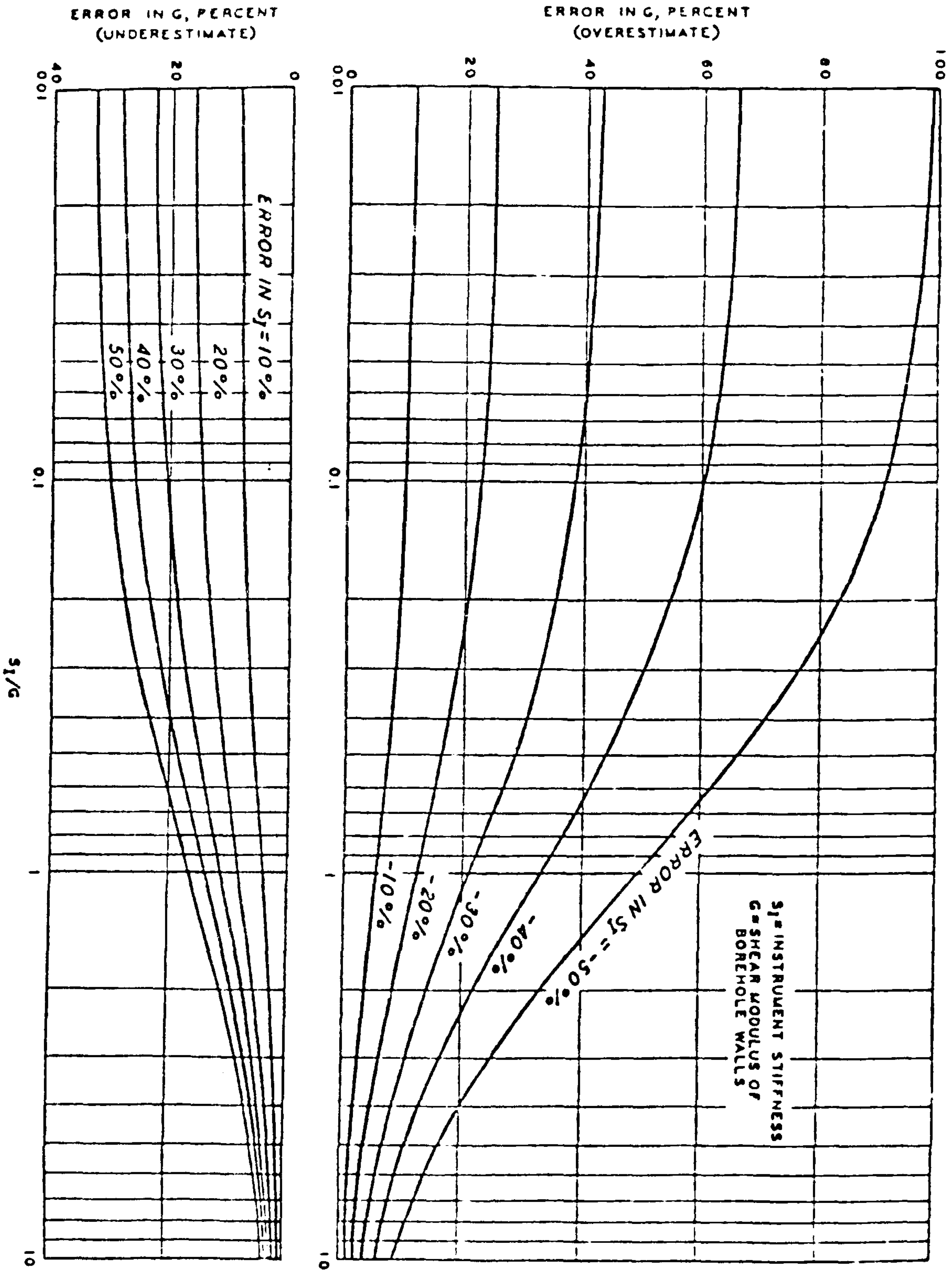


Fig. 4 Uncertainty in predicted shear modulus versus instrument stiffness

RTH 362-89

SITE	DATE	BORING NO.	DEPTH	OPERATOR	PAGE ___ of ___
PROBE SIZE & TYPE	REMARKS				

TIME	LOAD PSI	VOLUME - CM^3			CREEP (30-60) SEC	DIFF (60 SEC)	REMARKS
		15 SEC	30 SEC	60 SEC			

Fig 5: Data Sheet

SUGGESTED METHOD FOR DETERMINING ROCK MASS DEFORMABILITY USING A HYDRAULIC DRILLHOLE DILATOMETER

Scope

1. (a) This test determines the deformability of a rock mass by subjecting a section of drillhole to hydraulic pressure and measuring the resultant wall displacements.[1] Elastic moduli and deformation moduli are calculated in turn.

(b) The results are employed in design of foundations and underground construction.

(c) The dilatometer is self-contained and tests are relatively inexpensive compared to similar tests at a larger scale.[2] Also, the wall is damaged only minimally by the drilling of the hole and usually remains representative of the undisturbed rock condition. These advantages, however, come at a sacrifice of representation of the effects of joints and fissures which are usually spaced too widely to be fully represented in the loaded volume around the drillhole.

(d) This method reflects practice described in the references at the end.

(e) Another type of dilatometer for drillholes transmits pressure to the rock through mechanical jacks: See RTH-368.

Apparatus

2. Drilling equipment to develop access hole in a given orientation without disturbing the wallrock.[3]

3. A drillhole dilatometer similar to that in Figure 1, which consists of:

(a) stainless steel cylinder

(b) Rubber (neoprene) jacket surrounding the steel cylinder and sealed at both ends to confine pressurized fluid between the jacket and the steel cylinder

(c) Two end plugs containing pipes, electric wires, and relief valves.

(d) Linear differential transformers oriented along different diameters of the drillhole (commonly four at 45-degree sectors). Deflections as large as 5 mm are measured, commonly with accuracy of about 0.001 mm.

4. Hydraulic pump capable of applying the required pressure and of holding this

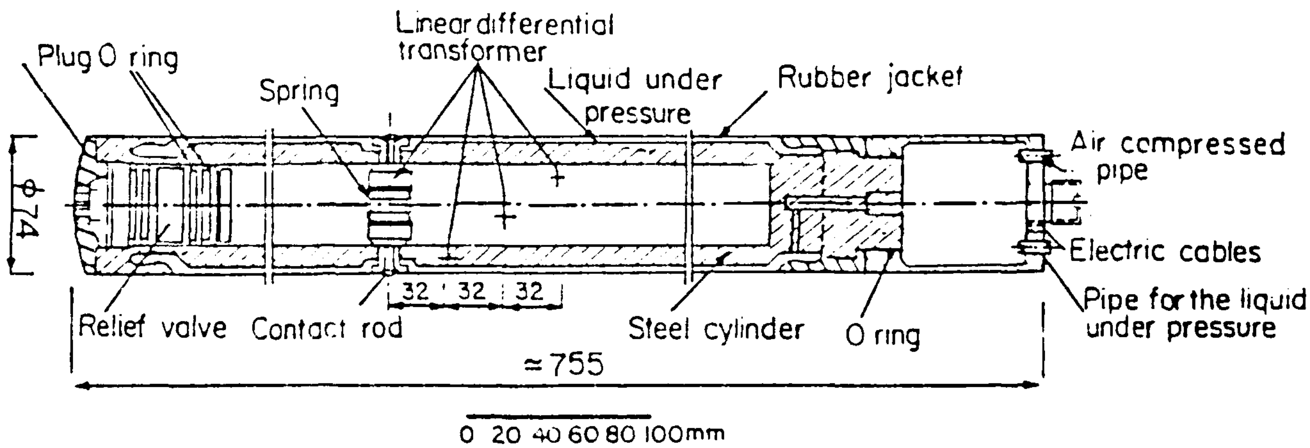


Figure 1. Example hydraulic drillhole dilatometer.

pressure constant within 5% for a period of at least 2 hr. together with all necessary hoses, connectors, and fluid.

5. Hydraulic pressure gages or transducers of suitable range and capable of measuring the applied pressure with accuracy better than 2%.

Procedure

6. Preparation

(a) The positions for testing are planned with due regard to the location of drilling station and the rock conditions to be investigated. The effects of geological structure and fabric are particularly important.

(b) The hole is drilled and logged. The log is studied for possible modifications in positions for testing. Multiple testing positions in one hole should be separated by at least 0.5 m.

(c) The dilatometer is assembled and inserted into the hole, commonly using an attachable pole to position and rotate and taking special care with trailing lines.

(d) The rods of the linear differential transformers are seated against the wall.

7. Testing

(a) The dilatometer is pressurized in increased stages, with pressure released between stages. Typically, the stage pressures are 25, 50, 75, and 100 percent of the planned maximum of the complete test.[4]

(b) Pressure is increased at a rate of 0.5 MPa/min. or less.

(c) On reaching the planned pressure for the stage, the pressure is held constant for at least 1 min. to detect and define nonelastic deformation. Each stage is completed by releasing pressure at a prescribed rate up to 0.5 MPa/min.

(d) The test history is documented with no less than four sets of measurements during pressure increase and two during pressure decrease. Supplementary notes are necessary to describe any complexities not otherwise revealed (such as nonelastic deformation).

(e) The pressure is released and fluid withdrawn. The rods of the linear differential transformers are retracted away from the wall and the dilatometer is removed from the hole.

Calculations

8. For presumed quasi elastic conditions, an elastic modulus is calculated from

$$E = \frac{p\ a}{U_r}(1 + \nu)$$

where

p = fluid pressure

a = hole radius

U_r = change in radius

ν = Poisson's ratio

Where permanent deformation (nonelastic) occurs also, that portion of U_r should be excluded from the equation.

Reporting

9. The report should include for each test or all tests together the following:

(a) Position and orientation of the test, presented numerically, graphically, or both ways.

(b) Logs and other geological descriptions of rock near the test. The structural details are particularly important.

(c) Tabulated test observations together with graphs of displacement versus applied pressure and displacement versus time at constant pressure for each of the displacement measuring devices (e.g. linear differential transformers).

(d) Transverse section of hole showing the displacements resulting from the pressure in all orientations tested. Calculated moduli are indicated also.

Notes

[1] In very deformable rocks, the diametral strain can also be determined indirectly from changes in volume of pressurizing fluid. See RTH-362 for that procedure.

[2] See RTH-361, -366, and -367 for similar test at tunnel scale.

[3] Diamond core drilling is recommended for obtaining the necessary close tolerance when using dilatometer only slightly smaller than the hole and displacement measuring devices with very limited stroke.

[4] Typically, the maximum pressure is about 15 MPa.

References

Lama, R. D., and Vutukuri, V. S., Handbook on Mechanical Properties of Rock, Vol.III, TransTech Publications, 1978, 406 pp.

Stagg, K. G., "In Situ Tests on the Rock Mass," in Rock Mechanics in Engineering Practice, John Wiley & Sons, New York, 1968, pp. 125-156.

RTH-364-89

SUGGESTED METHOD FOR DETERMINING ROCK MASS DEFORMABILITY BY LOADING A RECESSED CIRCULAR PLATE

Scope

1. (a) This test determines the deformability characteristics of a rock mass by loading a flat surface at the end of a drill hole or other recess and measuring the resultant displacement of that surface. Elastic or deformation moduli are calculated as well as time dependent (creep) properties.

(b) Several depth horizons may be tested from the same setup using a large-diameter drill to advance between tests. The direction of loading necessarily coincides with the drill hole axis, usually near-vertical, so that no information can be obtained regarding rock anisotropy.

(c) Plate bearing tests are commonly used to provide information for the design of foundations.

(d) This method is a modification of practice suggested by the International Society for Rock Mechanics. See Reference. That previous version provides details not included here.

(e) Another method, differing in the loading system, is available for comparison and consideration in RTH-365.

Apparatus

2. Equipment for drilling, cleaning, and preparing a test hole at least 500 mm[1*] in diameter. The hole may need casing. A reamer and other special tools are useful in flattening the bearing surface (± 5 mm) perpendicular to the hole axis (± 3°) and for removing water and debris.

3. Core drill for taking samples to at least 3 m below the bearing surface, the diameter to be less than 10% that of the bearing plate.

* Numbers refer to NOTES at the end of the text.

4. Circular bearing plate of diameter at least 500 mm and sufficiently rigid to distort by not more than 1 mm under the test conditions (Figures 1 and 2)[2].

5. Hollow loading column to transmit the applied force centrally to the test plate without detrimental buckling.

6. Hydraulic jack and reaction anchors such that:

(a) Loads can be varied throughout the required range and can be held constant to within 2% of a selected value for a period of at least 24 hr.

(b) The travel of the jack is greater than the sum of anticipated displacements of the plate and reaction beam.

(c) Reaction anchors are located further than 10 test hole diameters from the bearing plate.

7. Equipment (load cell or proving ring) to measure the applied load with an accuracy better than ± 2% of the maximum reached in the test.

8. Equipment to measure axial displacement of the plate[3] with accuracy better than 0.05 mm. The reference anchors should be at least 10 test hold diameters from the loading plate and reaction anchors.

Procedure

9. Preparation

(a) The site is selected to allow testing at the actual foundation level with loading in the direction of foundation loading. Alternatively rock considered typical of anticipated conditions may be tested. Attention should be given to locations for reaction and reference anchors and to ground water and other influential conditions.

(b) Test hole and anchor holes are drilled and logged. The test hole is cased if necessary for stability throughout the test.

(c) Exploratory core is taken to a depth of at least 3 m below the proposed test horizon, and the choice of horizon confirmed or modified.

(d) Ground water encountered in the test hole should be lowered by pumping from well points surrounding the test area or otherwise during installation of the bearing plate.

(e) The bearing surface is trimmed, one or more layers of mortar or plaster are placed, and the bearing plate installed before the last layer has set. The delay between excavation of the surface and installation of the plate should not exceed 12 hr.[4]

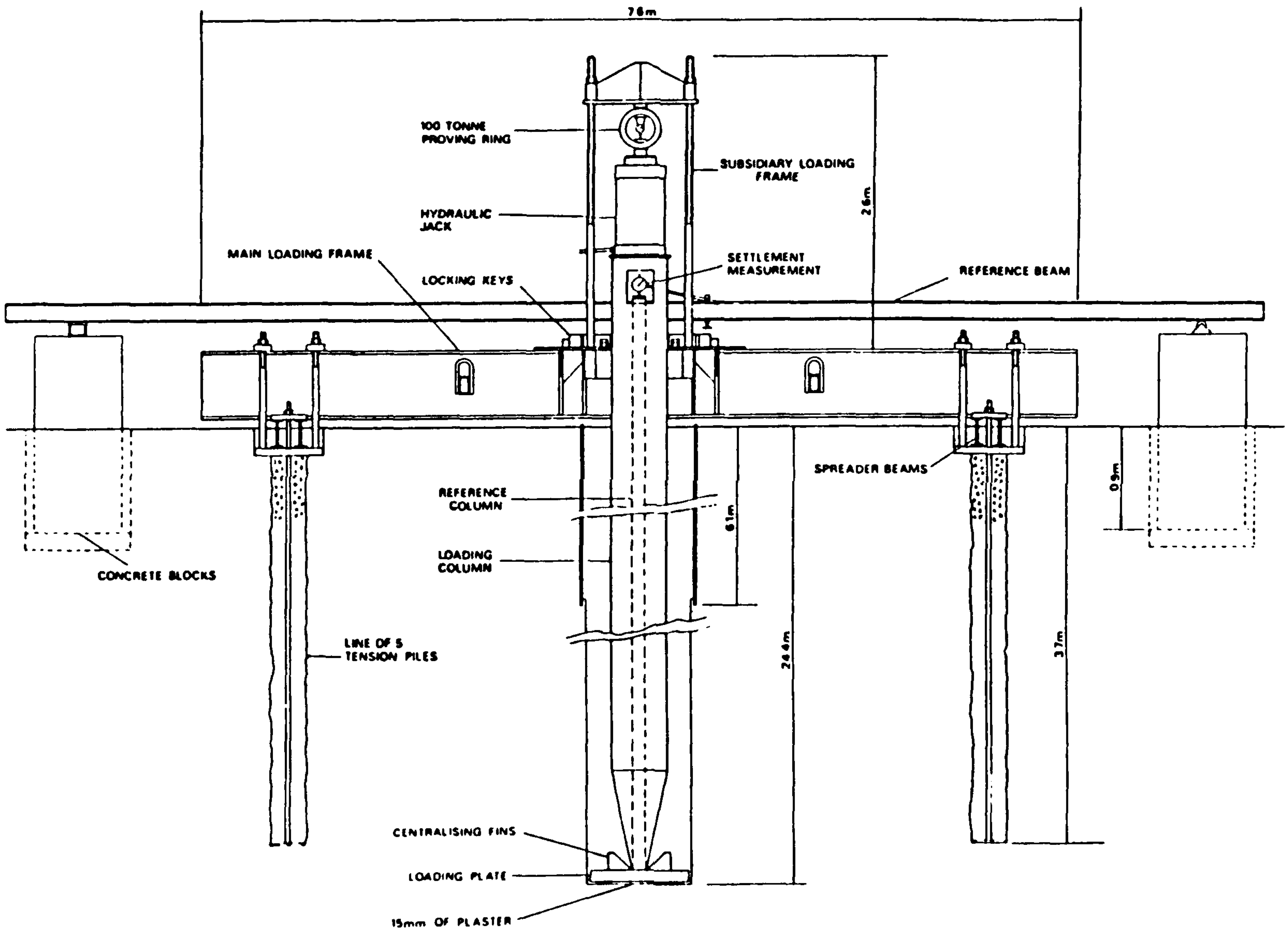


Figure 1. Example plate-loading equipment.

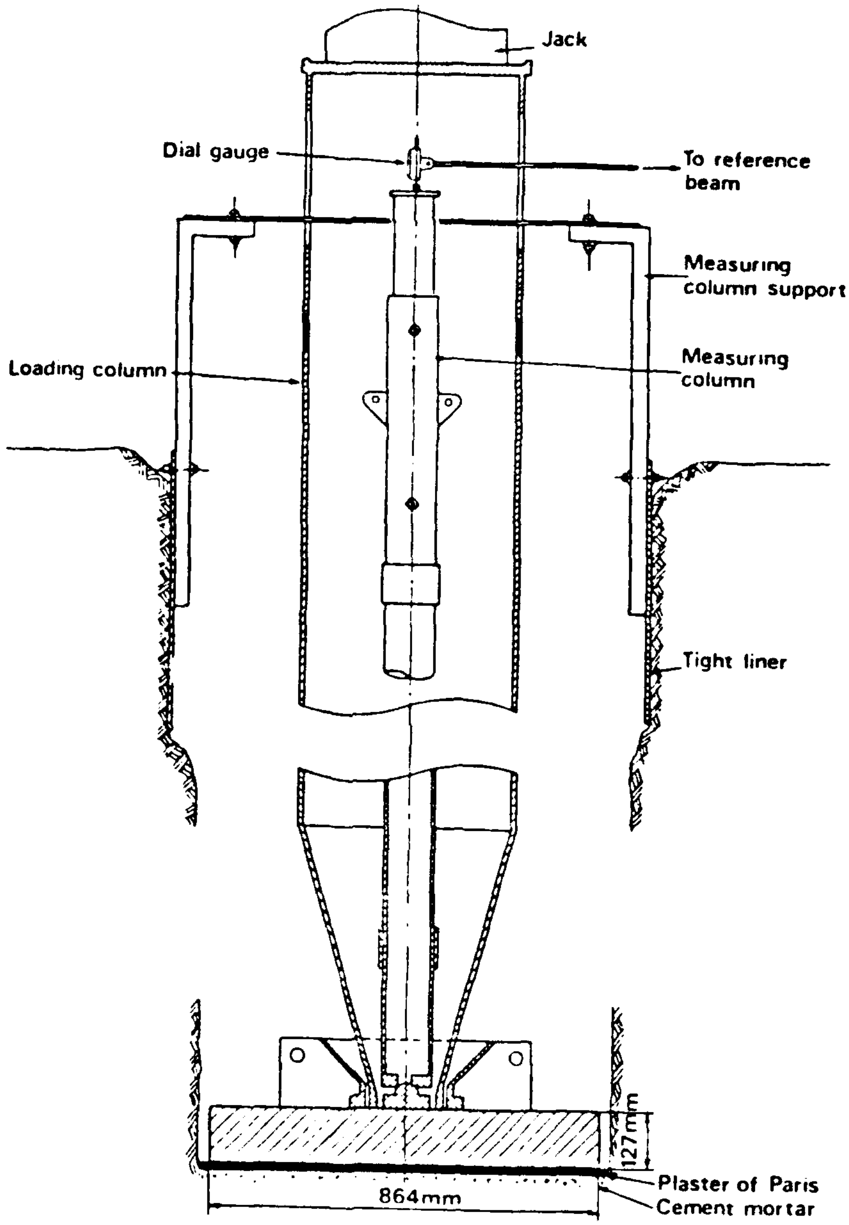


Figure 2. Details of plate test equipment.

(f) Reaction and reference anchors are installed 10 or more diameters away and the equipment assembled and checked. A small seating load (approximately 5% of the maximum test value) is applied and held until the start of testing.

(g) The water table should be allowing to return to its normal elevation before the start of testing.

10. Testing

(a) With the seating load applied, load and displacement should be observed and recorded over a period not less than 48 hours to establish datum values and to assess variations due to ambient conditions.[5]

(b) Loads and load increments to be applied during the test should be selected to cover a range 0.3-1.5 q_o, where q_o is the stress intensity produced by the proposed structure.[6]

(c) Load is increased in not less than five approximately equal increments to a maximum of approximately 1/3 the maximum for the test. At each increment the load is held constant (±3%) and plate displacement recorded intermittently until it stabilizes.[6] The procedure is continued for decreasing load increments until the seating load is again reached.

(d) Procedure (c) is repeated for maximum cycle loads of approximately 2/3 and 3/3 the maximum for the test.

(e) The equipment is removed from the test hole and further tests may be carried out on deeper horizons by re-drilling in the same hole.

Calculations

11. (a) Graphs are plotted of incremental settlement (or uplift in the case of unloading) against the logarithm of time (Figure 3).

(b) Bearing pressure versus settlement curves are plotted for each test (Figure 4).

(c) Deformqtion modulae may be determined using tangents to the pressure-settlement curves as follows:

$$E = \frac{dq}{d\rho} \frac{\pi}{4} D(1 - \nu^2).I_c$$

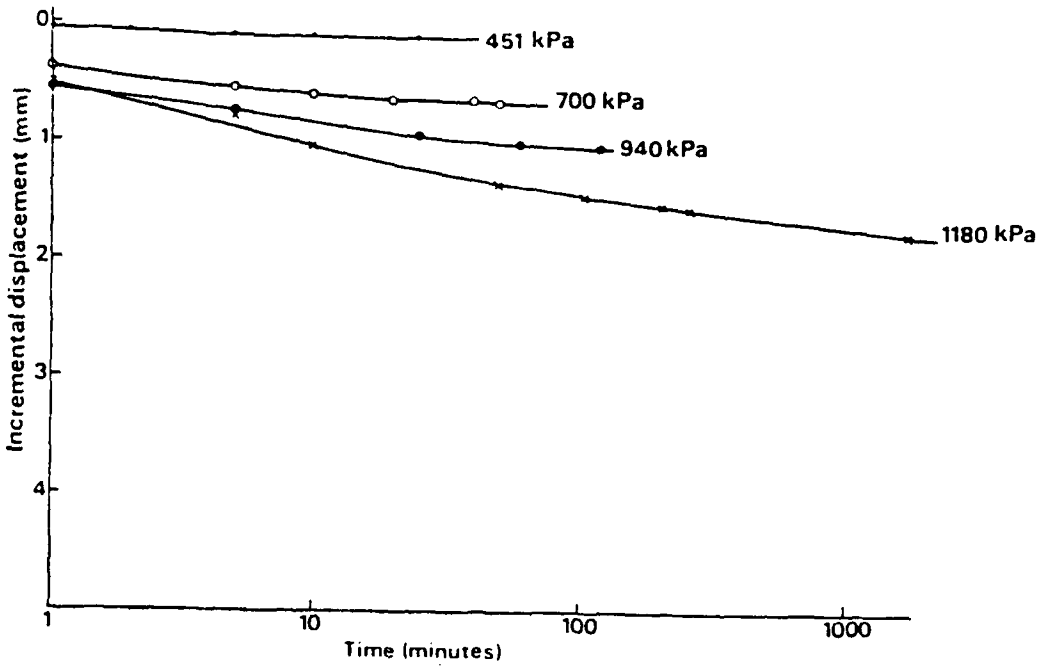


Figure 3. Typical relations between small displacement and time for various load intensities.

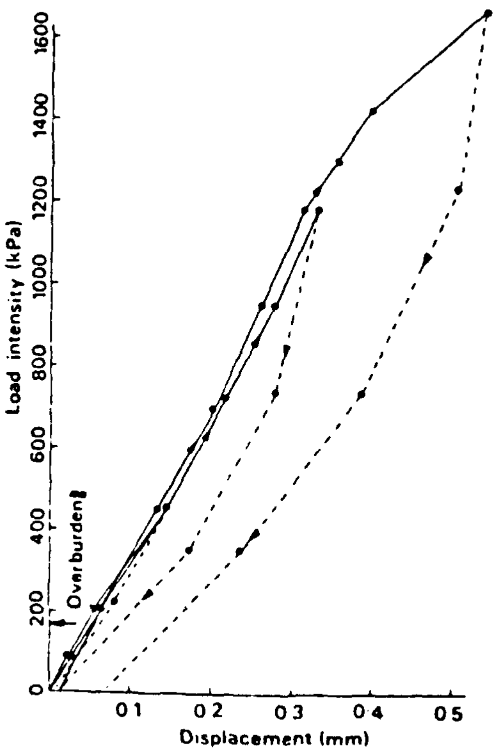


Figure 4. Example plate test results.

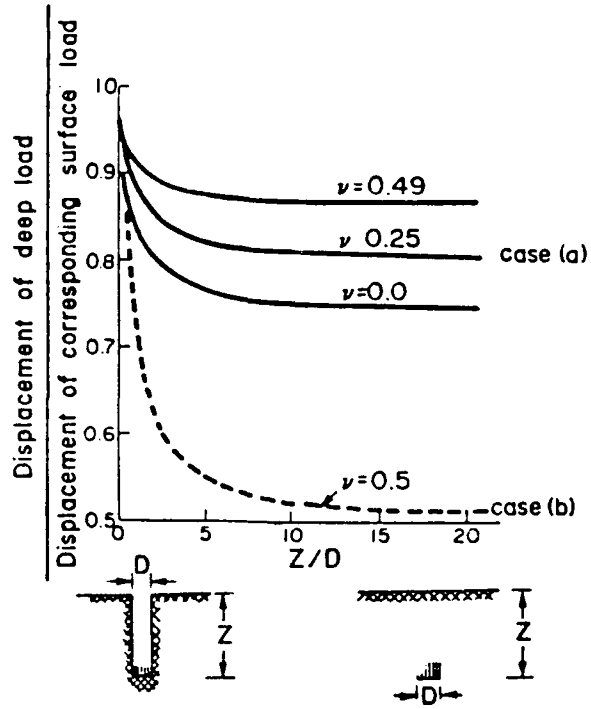


Figure 5. I_c factors for deep loading.

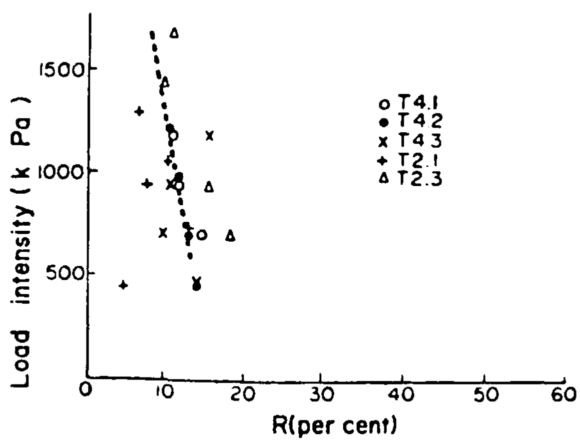


Figure 6. Relation between load intensity and creep ratio R.

where

q is applied pressure

p is settlement

D is plate diameter

ν is Poisson's ratio

I_c is a depth correction factor from Figure 5

(d) A time-dependent parameter R (known as the creep ratio) is determined for each load increment. The parameter R is defined as the incremental settlement per cycle of log time divided by the total overall settlement due to the applied pressure. The relationship between R and applied pressure may be presented graphically (Figure 6).

Reporting

12. The report should include the following

(a) Diagrams and detailed descriptions of the test equipment and methods used for drilling, preparation, and testing.

(b) Plans and sections showing the location of tests in relation to the generalized topography, geology, and ground-water regime.

(c) Detailed logs and descriptions of rock at least 3 m above and below each tested horizon.

(d) Tabulated test results, graphs of displacement versus time for each load increment, and graphs of load versus displacement for the test as a whole.

(e) Derived values of deformability parameters, together with details of methods and assumptions used in their derivation. Variations with depth in the ground should also be shown graphically as 'deformability profiles' superimposed on the log of the test hole.

Notes

[1] The test hole should preferably be of sufficient diameter to allow manual inspection, and preparation of the bearing surface. Otherwise preliminary coring is needed to provide adequate samples of ground conditions.

[2] Steel plate unreinforced by webs, should be at least 20 mm thick for a diameter of 500 mm.

[3]If required, the displacement of rock at any level below the bearing plate may be monitored, using rods passing through a hole in the center of the plate and rigidly anchored in the exploratory drillhole.

[4]Particularly when testing weaker rocks there will be rebound, loosening, and possibly swelling associated with excavation of the bearing surface and changes in ground water conditions.

[5]Small fluctuations in displacement are likely to result from changes in the ground water regime, temperature, and other environmental effects.

[6]At higher applied loads the displacement may not completely stabilize in a reasonable time; a criterion that readings should continue until the rate of displacement is less than 2% of the incremental displacement per hour may be used. This criterion may be modified to suit the purpose of the test. The final increment in any one cycle should be held for as long as practical if the displacement is still increasing.

Reference

International Society of Rock Mechanics, "Suggested Method for Field Deformability Determination Using a Plate Test Down a Borehole," International Journal of Rock Mechanics and Mining Sciences, v. 16, 1979, pp. 202 - 208

RTH 365-80

Authorized Reprint from
Special Technical Publication 554

Bureau of Reclamation Procedures for Conducting Uniaxial Jacking Tests

D. L. Misterek,[1] *E. J. Slebir,*[1] *and J. S. Montgomery* [1]

REFERENCE: Misterek, D. L., Slebir, E. J., and Montgomery, J. S., **"Bureau of Reclamation Procedures for Conducting Uniaxial Jacking Tests,"** *Field Testing and Instrumentation of Rock, ASTM STP 554,* American Society for Testing and Materials, 1974, pp. 35–51.

ABSTRACT: The Bureau of Reclamation's present method of conducting uniaxial jacking tests incorporates the desirable features and techniques developed over many years of testing. Successful tests yielding usable data are attributed to thorough pre-test geologic exploration; careful site preparation; special drilling and explosive excavation techniques; and well-planned procedures for installation and removal of specialized equipment. Nonengineering factors, such as good contractor-test team relationship, also play a critical role in a successful test program. The process where hydraulic flatjacks are used to apply desired loads to prepared rock surfaces within a tunnel is described. Instrumentation to measure resulting displacement is referenced within the rock mass and is independent of the loading system. Details of all phases of conducting the tests are presented. In addition, a discussion is given of some of the problems, solutions, and philosophies that evolved while tests were being planned or conducted. This discussion is of value to those conducting *in situ* jacking tests or those preparing standards for the tests.

KEY WORDS: rocks, uniaxial tests, geologic investigations, logging (recording), mapping, drilling equipment, instruments, installing, evaluation, tests, jacking

The Bureau of Reclamation has performed a variety of *in situ* tests in recent years. The most frequently utilized, because of economy and usable data, is the uniaxial jacking test. This test determines how foundation rock reacts to controlled loading and unloading cycles and provides data on deformation moduli, creep, rebound, and set.

[1] Engineers and civil engineering technician, Rock Mechanics Section, Division of General Research, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colo. 80225.

Previously published articles [2,3] provide a general discussion of the uniaxial jacking test procedures together with some typical graphs and explanations of data obtained. In consonance with the purpose of this symposium, this paper updates that information and explains in more detail the sequential procedures utilized to accomplish the test.

Test Site Selection

Prior to selecting a test site location, all available surface and subsurface geologic data are compiled and analyzed. A three-dimensional portrayal of these data is prepared, either in plotted form or by construction of a plastic model.

Exploratory tunnels are generally driven into the rock mass to further investigate geologic conditions and to provide access to test site locations. Ideally, these tunnels are oriented parallel to the horizontal projection of the resultant thrust that the proposed structure will exert on the foundation. Test adits are driven from the tunnels so that test loads applied to surfaces of the adit will be oriented in the same direction as loads from the proposed structure. To provide additional data on possible variations in properties of the rock mass, test loads may also be applied normal and parallel to geologic structure. Therefore, sufficient mapping and probe drilling are performed to delineate the geology in the test adit. After reviewing all available information, the precise location of a test site is designated by a team of geologists and engineers who must determine that the test location is representative of conditions to be found in a significant portion of the rock mass. In this case, significance is determined by the relative influence on the deformability of total rock mass. That is, a small volume of relatively deformable material may be as significant to overall rock mass deformation as a much larger volume of more competent material. In making this evaluation, a number of factors must be considered. These include: (1) spatial orientation and stress intensity of the loads to be transmitted to the rock mass by the proposed structure, (2) the various types of material found in the rock mass and the relative volume and location of each, (3) spatial orientation of rock structure, that is, bedding, foliation, jointing, etc., and its relationship to applied loads from the structure, and (4) fracture density of the rock mass.

[2] Wallace, G. B., Slebir, E. J., and Anderson, F. A. in *Determination of the* In Situ *Modulus of Deformation of Rock, ASTM STP 477*, American Society for Testing and Materials, 1970, pp. 3-26.

[3] Wallace, G. B., Slebir, E. J., and Anderson, F. A. in *Eleventh Symposium on Rock Mechanics*, University of California, Berkeley, Calif., 16-19 June 1969, American Institute of Mining, Metallurgical, and Petroleum Engineers, 1970, pp. 461-498.

Preparation of the Test Site

After site selection, the area is prepared for testing by removing blast-damaged material using pneumatic chipping hammers and drills. Although only a 34.2 in. (87 cm) diameter area of rock is actually loaded during a test, an area approximately 5 ft (1.52 m) in diameter is prepared to reduce the restraining influence of the surrounding rock. If the rock cannot be removed with pneumatic tools and blasting is required, the wire-lath system, shown schematically in Fig. 1, has proved to be successful. Four wooden laths are installed in drill holes located approximately 4.5 ft (1.37 m) from the center of the area to be loaded. Wires are strung from lath to lath forming a square reference plane over the test area. Percussion-drilled blasting holes, 1⅝ in. (4.13 cm) in diameter, are driven to terminate in a common plane at a predetermined depth parallel to the wire-lath reference plane. The holes are spaced on 4 to 5 in. (10.16 to 12.7 cm) centers to ensure shearing of rock along a flat plane during blasting. Starting with the shallowest blasting holes, the test area is excavated in three to five separate shots using either 4 in. (10.16 cm) of 70 percent dynamite tamped at the bottom of each hole or several windings of detonating cord (18 in.) (44.7 cm) placed at the bottom of each hole and detonated with electric blasting caps. This procedure produced a test area with minimum blast damage and minor final cleanup.

Drilling for Instrumentation

After two diametrically opposite test areas have been prepared, a 20-ft (6.10-m) deep Nx (3-in. (7.6-cm) diameter) hole is drilled into the center of each area. Care must be exercised to ensure the two holes are aligned coaxially. This is essential for proper alignment of the test equipment and to allow accurate measurements to be made between the centroids of the loaded surfaces. The upper hole is drilled first to minimize debris entering the lower hole. After a hole is completed, a special bit is used to produce a 5-in. (12.7-cm) diameter counterbored recess at the collar of the hole. This step ensures a good coupling between the rock and the extensometer installed in the hole to measure deformation. Additional details are presented in the section on instrument installation. Figure 2 shows a test area with the instrument hole and counterbored recess complete.

Maximum core recovery with ensured correct orientation is important for the geologic and structural evaluation of the test site. To accomplish these objectives, two procedures are involved.

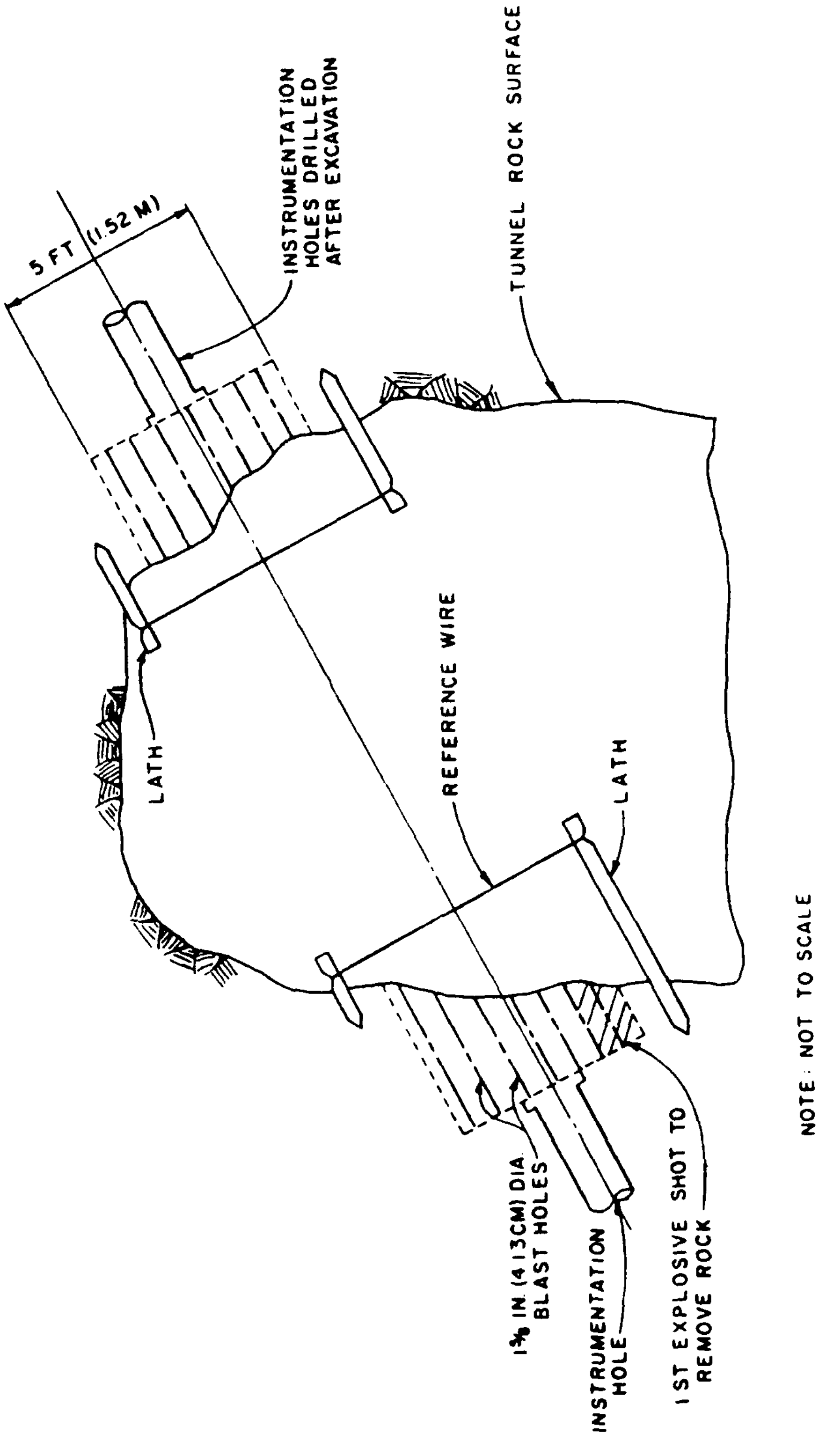


FIG. 1—*Wire lath system.*

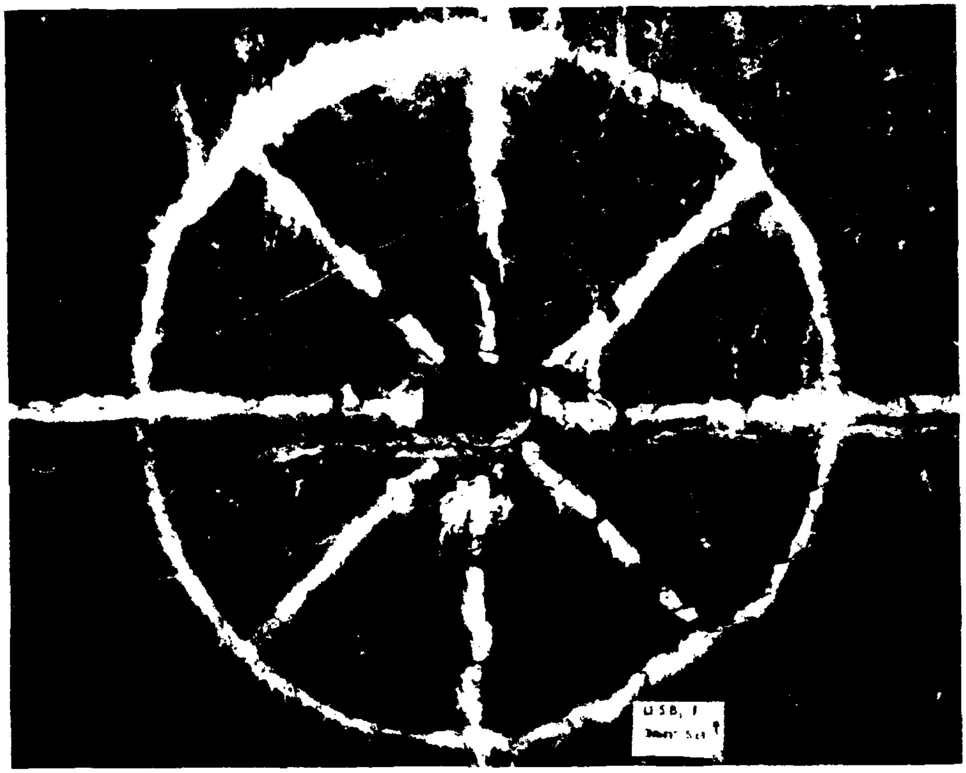


FIG. 2—*Instrumentation hole with counterbored recess. (Note the termination of blasting holes on the final rock surface.)*

The first procedure utilizes a split tube inner core barrel to obtain better quality core specimens. This inner barrel keeps the core from rotating while the outer barrel transfers torque and thrust to the diamond cutting bit. After a drill run, the split tube barrel is retrieved and placed in a horizontal position, the end fastening rings are removed, and half of the split inner tube is replaced by a cardboard split tube. The assembly is then rotated 180 deg so the cardboard half cylinder is on the bottom supporting the core. The remaining piece of the inner barrel is then removed.

The second procedure involves the use of a clay pot impression for orienting recovered core. At the end of a drill run, a special pot made of drill casing and loading poles is filled with oil base clay, oriented, and lowered to the bottom of the hole. An impression of the hole bottom is matched with the next core run.

Logging and Mapping at the Test Site

The core from the two instrumentation holes is logged to determine rock type, strike and dip of foliation, bedding and joints, whether breaks (joints) are mechanical or natural, weathering and alteration, plus any other pertinent features. After the core has been inspected, the drill holes are logged using a TV borehole camera. Prior to logging, the holes are washed and, if possible, filled with water to give a clearer picture. Borehole conditions observed during the TV examination are compared with core logs to verify the frequency, location and magnitude of significant geologic features. (*Note:* The Bureau of Reclamation already had the TV camera and appurtenant equipment for use in other work. A borescope-type device can be utilized for this phase of the work.) Next the prepared test areas are measured and mapped, and a geologic cross section through the test axis is prepared. Finally, the test site location in relation to the established survey of the tunnel is determined.

Measurement Locations

After comparing TV and rock core logs to verify existing geologic conditions, a composite drill hole log is assembled and used to select locations for instrumentation. Locations within each hole are selected to receive mechanically expandable anchors to serve as reference points for displacement measurements. Figure 3 shows some typical anchor locations for various geologic conditions. Some points are located to indicate strain in solid rock, others to span joint or fracture systems, and some to span lithology changes. Results of early tests [2,3] show that most of the significant portion of deformation occurs in the first several feet of rock nearest the applied load. It was also noted that displacement of the loaded surface is not always symmetrical with respect to the center of the loaded area; that is, displacement on one side of the center might be greater than on the other side. These two conditions led to the development of a semi-standard arrangement of anchor locations. Instead of utilizing seven anchors, the bottom three anchors have been replaced by a single anchor located within a few inches of the bottom of the hole and fitted with three metering rods. The reason for this procedure is presented in the section on testing. The remaining four anchors are normally located within the upper half of the drill hole. In general, careful logging of the borehole allows anchors to be placed in competent rock rather than within undesirable fractured or jointed zones.

Instrumentation Installation

After the anchor depths in each instrument hole are established, a 3-in. (7.6-cm) diameter stainless steel borehole collar sleeve is inserted into each hole. The outside flange of the sleeve fits into the counterbored

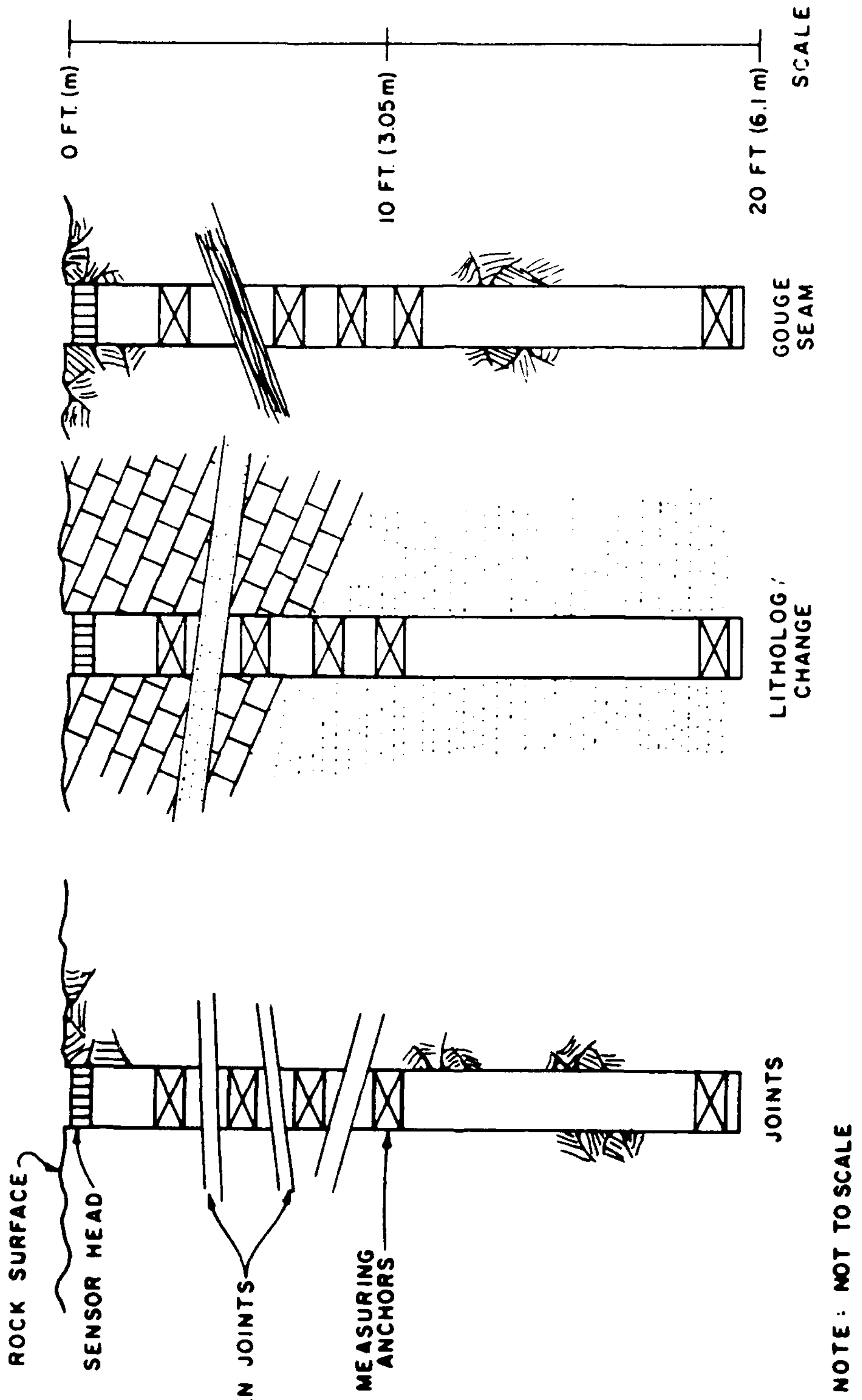


FIG. 3—*Typical anchor locations.*

recess previously described and is covered with mortar to secure it to the rock. Figure 4 shows a test area with the sleeve installed. After the mortar has set, a final inspection is made to ensure that the hole is clear of all obstructions. At this point, the Retrievable Borehole Extensometer (REX-7P) can be installed. The REX-7P is a seven-position extensometer, which, except for lead wires to the readout device, is installed entirely within the instrumentation hole. The instrument, developed by the Bureau of Reclamation, has been granted U.S. Patent No. 3,562,916. Detailed drawings, installation, and operating instructions are presented in the patent documents and will be summarized only briefly in this paper.

The instrument consists of anchors installed at depth within the borehole, a sensor head including seven linear variable differential transformers (LVDT) located at the collar of the borehole, and thin-wall stainless steel metering rods relating the movement of the anchors to that of the sensor head. The device has a sensitivity and repeatability of 100 $\pm$ μin. (0.00025 cm) and reliably measures rock movement to 200 $\pm$ μin.

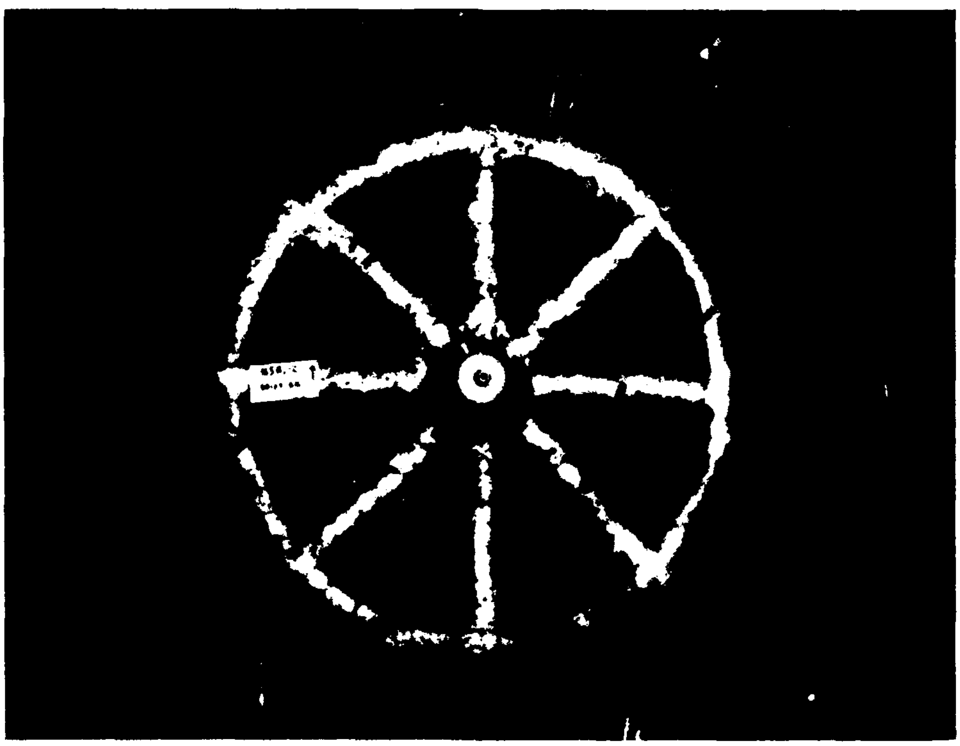


FIG. 4—*Instrumentation hole with borehole collar sleeve installed and cover cap in place. (Note fitting for tunnel diameter gage at center of cover cap and exit tube for lead wires on upper left of sleeve.)*

(0.0005 cm). Each anchor, with its metering rod attached, is installed by expanding it radially against the drill hole with a placing tool. Each subsequent anchor is installed at a lesser depth than the previous one and has holes which allows metering rods from previously set anchors to pass through. The sensor head containing the LVDT's is attached to the borehole collar sleeve after all anchors and metering rods are in place. After all LVDT's are set, the lead wires are covered with plastic tubing to protect them during placement of the concrete bearing pad.

Finally, a cover cap is screwed over the borehole collar sleeve. The cap contains fittings which serve as reference points for the tunnel diameter gage which is inserted between the two diametrically opposite REX-7P's to provide a redundant measurement of total displacement. All measurements with the REX-7P are referenced within the borehole and, therefore, within the rock mass. This arrangement of referencing all measurements within the rock mass eliminates the necessity of monitoring the deformation of various other components of equipment during load applications and release.

Equipment Installation

The complete setup of the test equipment is shown schematically in Fig. 5. To handle the various components of equipment, a hoist supported by steel pins inserted in holes drilled in the vicinity of the test site is used. A timber platform is used to assure proper alignment of test equipment. Installation of equipment is greatly facilitated if the platform surface is accurately located. It should be just far enough away from the center line axis of the instrumentation holes so that minimal amounts of wedges and shims are sufficient to align the equipment into final position. This final placement will ensure that the center line of the test equipment coincides with the axis of the instrumentation holes. With the platform in place, wood blocking is positioned against the lower rock test area. The test equipment is then assembled upward from the blocking, starting with the baseplate with four concave bearing shoes attached. Four screws with convex ends are then positioned against the bearing shoes. Four sets of columns with their connecting plates are placed on the screws. The top plate is then installed, and the total unit is forced together by chain come-alongs attached to the base and top plates. The blocking on the lower end is then removed as the unit is supported by the platform. Flatjacks 34.2 in. (87 cm) in diameter with a 2-in. (5.08-cm) diameter hole in the center and particle board separators are fitted on both base and top plates. Although only one flatjack is required on each end of the test setup for load

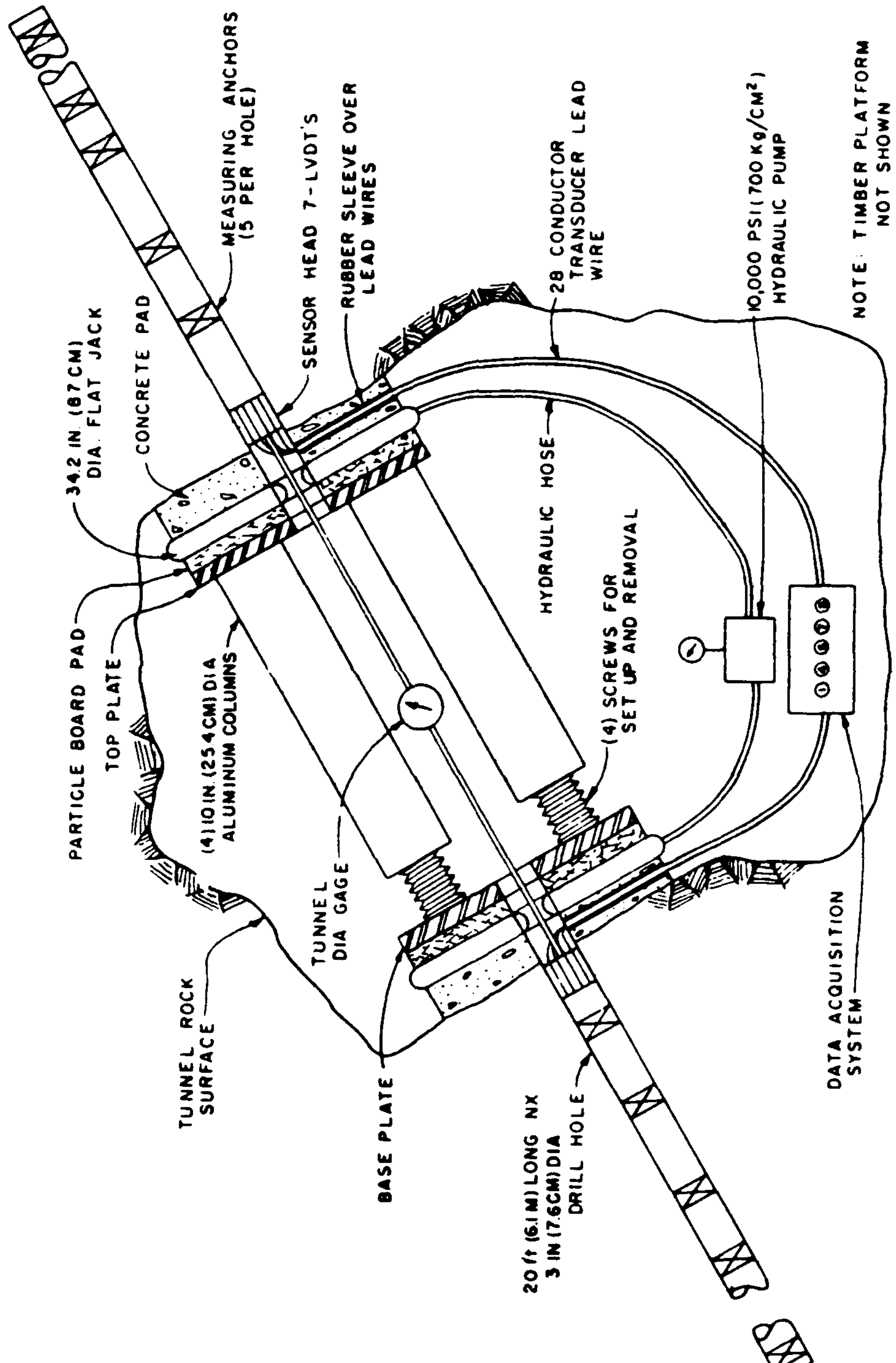


FIG. 5—*Uniaxial jacking test.*

application, an additional jack may be inserted to guard against loss of the entire test setup in case one flatjack was to rupture or if large deformations are anticipated. The flatjack and particle board assembly is held against the base and top plates by special "C" clamps to take up any slack and keep jacks in alignment. A bolt with a hemispherical gage point for the tunnel diameter gage is inserted through the center hole in the base and top plates, flatjacks, and particle board and screwed into each borehole collar cap. The bolt is isolated from the concrete bearing pad placement by a rubber sleeve.

The space between the flatjack asembly and the rock surface is formed and filled with small aggregate concrete (¼ to ⅜ in.) (0.63 to 0.95 cm). Placing the bottom pad first and then securing the test setup from moving, prevents misaligning the equipment while placing the top concrete pad. After 12 h, the forms are stripped so that the rock surface interface with the concrete pad can be observed. A concrete pad with forming material removed is shown in Fig. 6.

The hydraulic system is composed of a 1½-hp electric pump (10 000-psi (700-kg/cm^2) maximum output pressure), high-pressure hose with miscellaneous fittings, and a 2000-psi (140-kg/cm^2) laboratory test gage. This system provides pressure to the hydraulic flatjacks, which, because the jacks have been calibrated previously in the laboratory, results in a known load being applied to the rock.

Testing

The uniaxial jacking test loading and unloading procedures have been presented in detail in previous publications [2,3] and are only briefly reiterated here. Loads are applied in 200-psi (14-kg/cm^2) increments from 200 to 1000 psi (14 to 70 kg/cm^2) with periods of no load between each incremental loading. Frequently, loads are maintained for a 48-h period followed by a 24-h period of no load. However, this should not be considered a standard, for in some rock, very little deformation occurs after 12 to 18 h of loading. In these cases, considerable time can be saved if deformation characteristics of the rock mass are monitored to determine the point in time at which the creep rate becomes insignificant and that time is then used as a terminal point for a load or unload cycle.

While the test is in progress, deformation as indicated by the REX-7P transducers and by the tunnel diameter gage are recorded at intervals ranging from 15 min to 2 h. The REX-7P measures displacements between the bottom anchor and three points 120 deg apart at the collar of the hole. This provides a method of measuring tilting that might occur

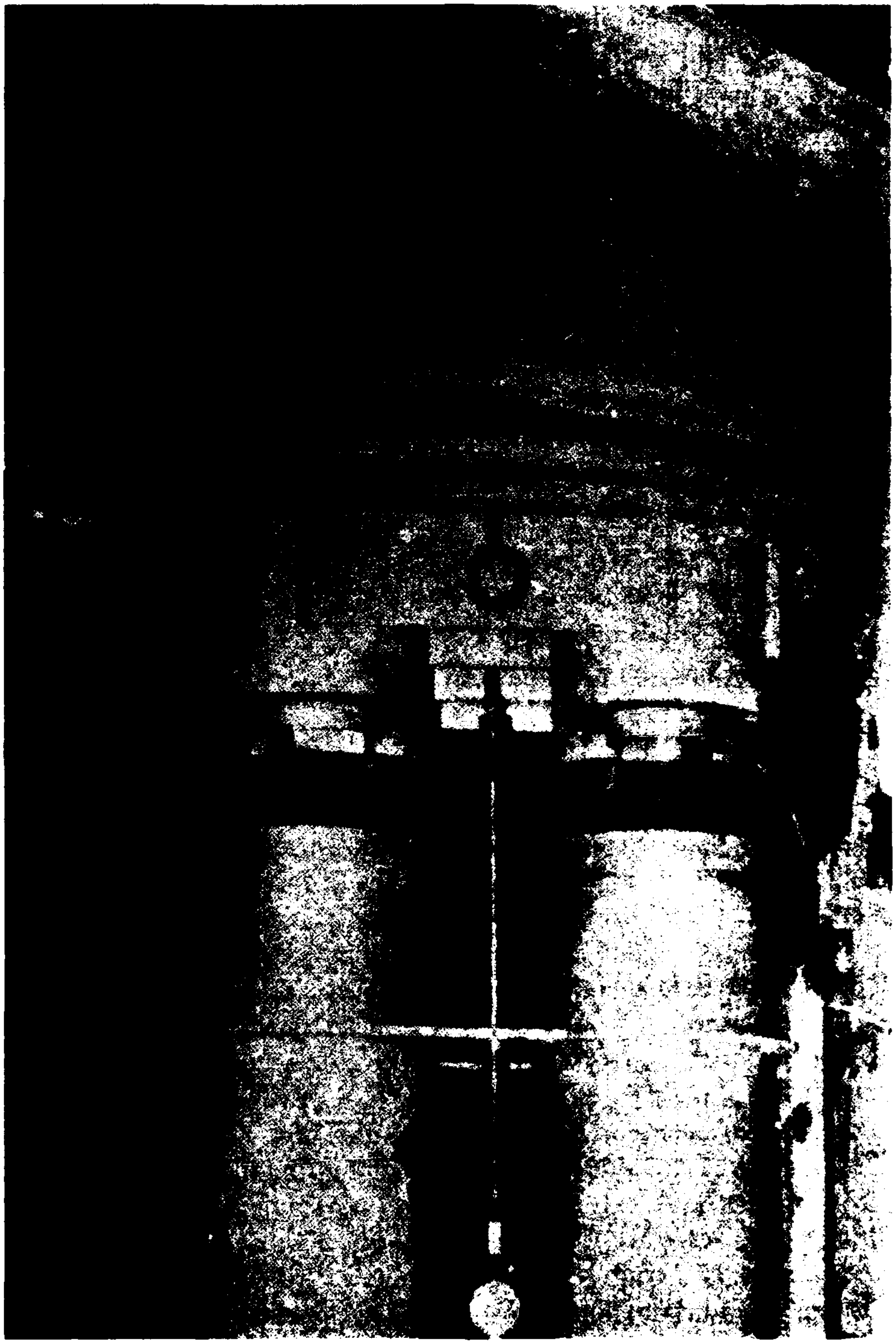


FIG. 6—*Concrete bearing pad for horizontal uniaxial jacking test.*

at the collar of the hole due to differential surface rock deformation. The surface deformation and the remaining measurements are then adjusted for any tilt. The tunnel diameter gage provides a measurement of total displacements between the two loaded rock surfaces. The policy of recording frequent readings and of making redundant measurements (the tunnel diameter gage) has contributed to our confidence in the reliability of measured displacements.

Removal of Equipment and Instrumentation

After the test is completed, the installation procedure is reversed and the entire test setup is retrieved for subsequent use. The centers of the two concrete bearing pads are then partially excavated with pneumatic chipping hammers to allow access for the removal of the REX-7P's. Finally, the concrete pads are shot down with a small explosive charge so as not to create a safety hazard by having them released by the action of gravity at some later date.

Philosophies, Problems, and Solutions

The procedures and comments presented are a result of approximately 30 tests performed over a 4-year period. Almost all tests were conducted by the same individuals; therefore, the overall experience, comments, and conclusions expressed in this paper are those of a limited number of individuals.

In situ jacking tests are nearly always conducted in tunnels where the test environment is anything but ideal. Frequently, access to the test site is difficult, lighting is less than desired, the source of power is unstable or unreliable, and, most serious, dampness in the tunnel can play havoc with the electronic instrumentation used to gather data. These conditions place an extra burden on the test team. Thorough advance planning, careful selection of instrumentation, adherence to the best instrumentation procedures, and built-in redundancy in the loading and data gathering systems become essential features of an *in situ* test program.

The complete sequence of events previously discussed involves the help and cooperation of a contractor and his miners, drillers, and other personnel. Bureau of Reclamation engineers and technicians have found that a general explanation to all involved contractor personnel of what, how, and why certain sequences of the site preparation and equipment installation are to be performed prior to starting the work will result in a more successful test.

Although precautions are taken to minimize the damaging effects of blasting and surface preparation of the test area, the damage cannot be eliminated completely. This condition leads to the question of how to evaluate the results to arrive at a deformation modulus which best represents the behavior of the rock mass when subjected to loads from the structure to be built. That is, should an effort be made to determine a modulus in which the effect of the damaged surface material is eliminated—either by the type of displacement measurements made or by a mathematical treatment of the data? In the case of a concrete dam (which is the usual structure for which the Bureau of Reclamation conducts *in situ* jacking tests), a similar blast-damaged or somewhat disturbed zone is created by the excavation for the keyways and foundation. This zone is undoubtedly much deeper than the disturbed zone at a test site. However, the load from the structure is also much greater, spread over a larger area and, therefore, affects the rock mass to a much greater depth. (So although there is no assurance that the relative effects of the two disturbed zones are equal or even comparable, they have been at least considered.) In the absence of specific information indicating the desirability of a different approach, the effect of the disturbed zone is included in the calculation of deformation modulus for the jacking tests.

Although *in situ* jacking tests are usually conducted in tunnels, data from the tests are converted to deformation moduli by utilizing the mathematical theory for a load applied to the boundary of a semi-infinite solid, as shown on Fig. 7. As there is little physical resemblance between a tunnel and a semi-infinite solid, a potential source of significant error may exist. Unless open cracks in the tunnel arch, invert, or walls allow free movement, deformation of the loaded area will be resisted because a portion of the force will be required to deform the adjacent tunnel surfaces. An analytical study has been made to evaluate the magnitude of the possible error. It was concluded that the distance between the loaded area and a restraining tunnel surface should be at least equal to the radius of the loaded area so that any restraint would not affect calculated moduli significantly.

Either of two philosophies can be embraced in regard to load distribution and location of displacement measurements for *in situ* jacking tests. In one case, displacements are measured at or beneath the surface of the loaded area. At this location, elastic theory indicates that the magnitude of the displacement is highly dependent on the distribution of the applied load. In the second case, displacements are measured at or beneath the rock surface at a point some distance from the loaded area. For these locations, displacement is dependent on the total force applied and the distance from the resultant of the force to the point of measurement, but

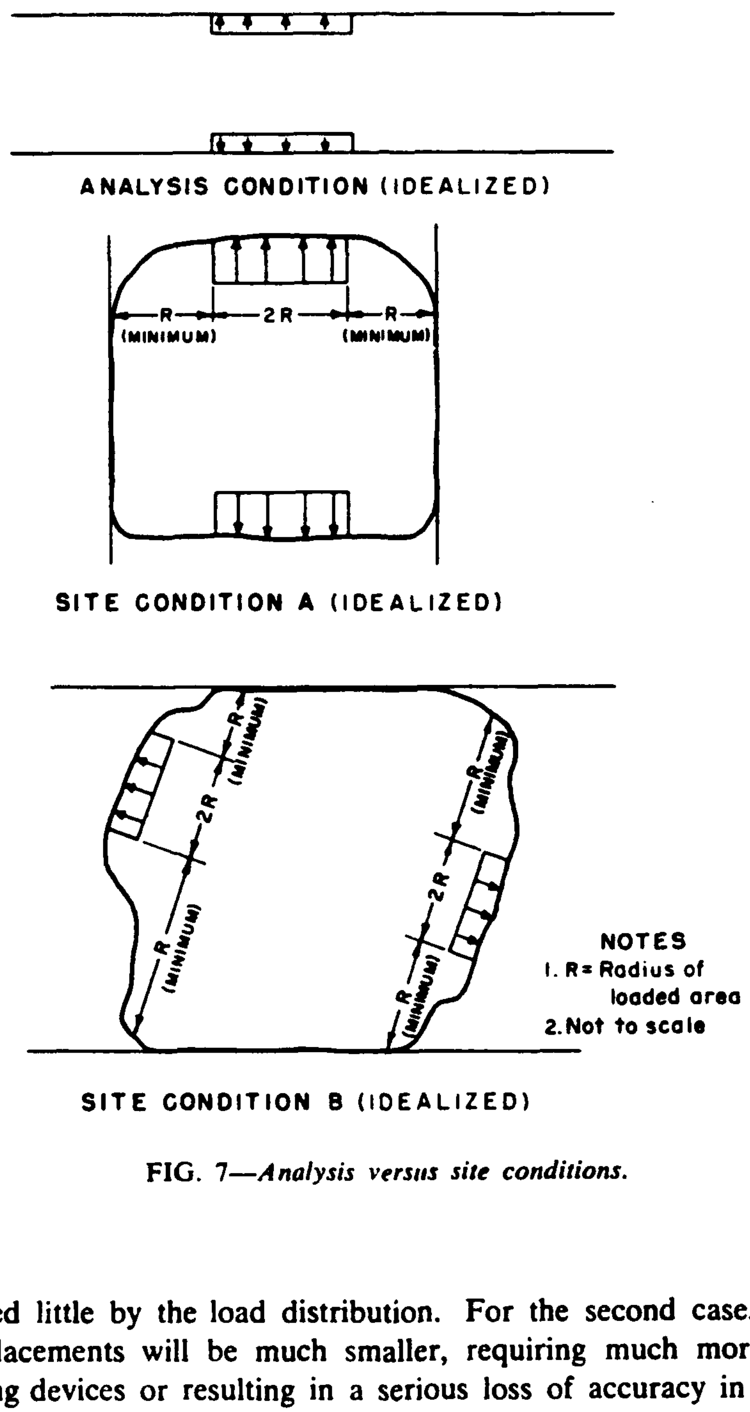


FIG. 7—*Analysis versus site conditions.*

is affected little by the load distribution. For the second case, however, the displacements will be much smaller, requiring much more sensitive measuring devices or resulting in a serious loss of accuracy in calculated moduli. For the Bureau of Reclamation tests, the first philosophy is used, and every effort is made to ensure that the applied load is uniformly distributed.

Following are some of the specific problems encountered during Bureau of Reclamation tests; some have explicit solutions, others lend themselves only to recommendations or suggestions:

1. *Preparing test site*—Obtaining an understanding between contractor's crew and test engineers on the importance of careful treatment of rock during all excavation phases. Define the work carefully in specifications so that the contractor is aware that test site excavation will be more time consuming and more costly than when merely advancing a tunnel. Maintain constant liaison with the shift foreman who supervises the excavation crew.

2. *Drilling of instrumentation holes and retrieving maximum undisturbed core*—Because of his past experience with unit price contracts, a contractor frequently has a tendency to have his crew drill for footage against time. To ensure the necessary high quality workmanship, define work carefully in specifications and also state that an owner's representative must be present during drilling operations. Develop liaison with drillers and emphasize that good "undisturbed" core recovery is important for instrumentation location. (A compliment to individuals involved in a good site preparation or drill hole and core recovery will usually result in a repeat performance.)

3. *Line power*—During a test, a separate source of line power is desirable for electronic instrumentation. Even though voltage regulators are utilized, periodic high power consumption by the contractor can cause problems.

4. *Metal to concrete contact*—Any component of the measuring system coming in contact with concrete should not be made of aluminum. Experience has shown that when fresh concrete is placed in contact with aluminum, a chemical reaction can occur. Under conditions at a test site, this reaction can have an adverse effect on the accuracy of deformation measurements.

5. *Traffic through test area*—Avoid letting traffic pass the test setup. Additional probe drilling, heading advance, or other activities beyond test site location should be scheduled to take place before or after the jacking tests. The test equipment occupies the majority of the adit opening, and hauling other equipment through the test site area containing all the required instrumentation and electronics may result in the loss of valuable data.

Conclusions

1. The equipment and procedures used by the Bureau of Reclamation to conduct *in situ* jacking tests produce satisfactory results.

2. Thorough pre-test geologic exploration is essential to ensure that the site selected is representative of the rock mass.

3. The position of the loaded area within the tunnel should be selected to prevent tunnel side walls from significantly affecting deflections.

4. Careful site preparation is essential for good test results.

5. Since *in situ* jacking tests are nearly always conducted in areas of adverse environmental conditions, the best of instrumentation procedures and a redundancy in the loading and measuring systems are important.

6. Although standardization of *in situ* jacking tests is a desirable goal, it is a goal that may be difficult to achieve. Variations in the needs, available resources, and design philosophy of the user may preclude a universally acceptable standard.

SUGGESTED METHOD FOR DETERMINING ROCK MASS DEFORMABILITY USING A MODIFIED PRESSURE CHAMBER

Scope

1. (a) This test determines the deformability of a rock mass by subjecting the cylindrical wall of a tunnel or chamber to hydraulic pressure and measuring the resultant displacements. Elastic moduli or deformation moduli are calculated in turn.

(b) The test loads a large volume of rock so that the results may be used to represent the true properties of the rock mass, taking into account the influence of joints and fissures. The anisotropic deformability of the rock can also be measured.

(c) The results are usually employed in the design of dam foundations and for the proportioning of pressure shaft and tunnel linings.

(d) Two other methods are available for tunnel-scale deformability. See RTH-361 and RTH-367 to compare details. Potentially large impacts, especially in terms of cost, of variations at this scale, justify the detailing of each method separately.

(e) This method reflects practice described in the reference at the end.

Apparatus

2. Equipment for excavating and lining the test chamber including:

(a) Drilling and blasting materials or mechanical excavation equipment.[1]

(b) Materials and equipment for lining the tunnel with concrete or flexible membrane.[2]

3. A reaction frame (Figure 1) composed of a set of steel rings of sufficient strength and rigidity to resist the force applied by pressurizing fluid must also act as a waterproof membrane.

4. Loading equipment to apply a uniformly distributed radial pressure to the lining including:

(a) A hydraulic pump capable of applying the required pressure and of holding this pressure constant within 5% over a period of at least 24 hr. together with all necessary hoses, connectors, and fluid.[3]

*Numbers refer to NOTES at the end of the text.

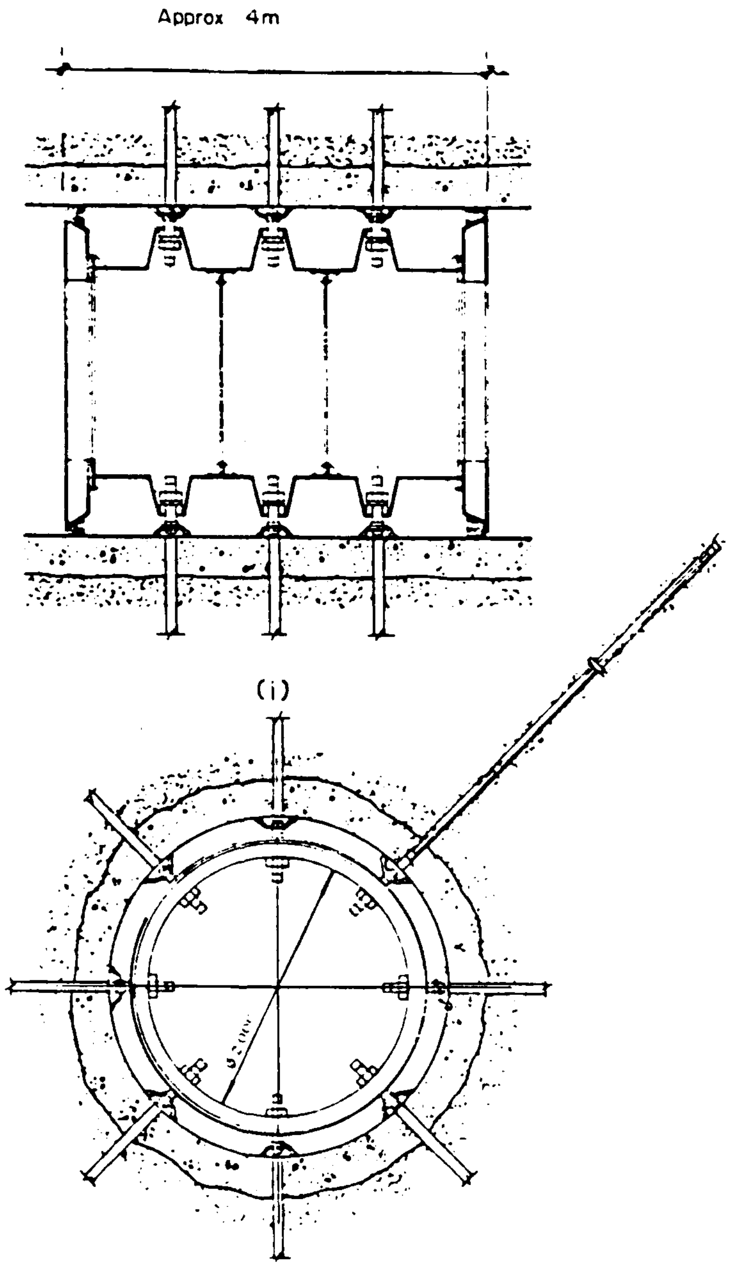


Figure 1. Example partial pressure chamber loading system.

(b) Water seals to contain the pressurized water between the lining and the reaction frame. Special water seals are also required for extensometer rods passing through the lining and reaction frame: pressurized water should not be allowed to escape into the rock since this will greatly affect the test results.

5. Load measuring equipment comprising one or more hydraulic pressure gages or transducers of suitable range and capable of measuring the applied pressure with an accuracy better than $\pm 2\%$.

6. (a) Displacement measuring equipment to monitor rock movements radial to the tunnel with a precision better than 0.01 mm. Single or multiple position extensometers are suggested but joint meters and other measuring devices are also available.

Procedure

7. Preparation

(a) The test chamber location is selected taking into account the rock conditions, particularly the orientation of the rock fabric elements such as joints, bedding, and foliation in relation to the orientation of the proposed tunnel or opening for which results are required.

(b) The test chamber is excavated to the required dimensions.[1]

(c) The geology of the chamber is recorded and specimens taken for index testing as required.

(d) The test section is lined with concrete.[2]

(e) The reaction frame and loading equipment are assembled.

(f) Holes for extensometers or other measuring devices are accurately marked and drilled, ensuring no interference between loading and measuring systems. Directions of measurement should be chosen with regard to the rock fabric and any other anisotropy.

(g) Measuring equipment is installed and checked. For multiple position extensometers, the deepest anchor may be used as a reference provided it is situated at least 2 chamber diameters from the lining. Alternatively the measurements may be related to a rigid reference beam passing along the axis of the chamber and anchored not less than 1 diameter from either end of the test section.

(h) Check water seals for leakage, if necessary by filling and pressurizing the hydraulic chamber. Leaks are manifested as anomalous pressure decay and visible seepage through the reaction frame.

8. Testing

(a) The test is carried out in at least three loading and unloading cycles, a higher maximum pressure being applied at each cycle.

(b) For each cycle the pressure is increased at an average rate of 0.05 MPa/min to the maximum for the cycle, taking not less than 3 intermediate sets of load-displacement readings in order to define a set of pressure-displacement curves.

(c) On reaching the maximum pressure for the cycle the pressure is held constant ($\pm 2\%$ of maximum test pressure) recording displacements as a function of time until approximately 80% of the estimated long-term displacement has been recorded. Each cycle is completed by reducing the pressure to near-zero at the same average rate, taking a further three sets of pressure-displacement readings.

(d) For the final cycle the maximum pressure is held constant until no further displacements are observed. The cycle is completed by unloading in stages taking readings of pressure and corresponding displacements.

Calculations

9. (a) The value of deformation modulus is calculated as follows

$$E = \frac{P_i \ a^2}{r(U_r)}(1 + \nu)$$

where:

E = modulus of deformation

P_i = internal pressure

a = radius to rock face - assuming circular chamber,

r = radius to point here deflection is measured,

U_r = change in radius due to pressure, and

ν = Poisson's ratio.

(b) The elastic modulus is sometimes represented by using only the portion of U_r which is recovered upon unloading.

Reporting

10. The report should include the following:

(a) Drawings, photographs, and detailed description of the test equipment, chamber preparation, lining, and testing.

(b) Geological plans and section of the test chamber showing features that may affect the test results.

(c) Tabulated test observations together with graphs of displacement versus applied pressure and displacement versus time at constant pressure for each of the displacement measuring locations.

(d) Transverse section of the test chamber showing the total and plastic displacements resulting from the maximum pressure. The orientations of significant geological fabrics should be shown on this figure for comparison with any anisotropy of test results. Calculated moduli should be shown also.

NOTES

[1]The recommended diameter is 2.5 m, with a loaded length equal to this diameter. The chamber should be excavated with as little disturbance as possible. Material disturbed by blasting may need to be removed since it tends to produce moduli lower than found at depth. However blast effects are representative if the test results are applied directly as a "model" test to the case of a blasted full-scale tunnel.

[2]When testing only the rock, the lining should be segmented so that it has negligible resistance to radial expansion; in this case the composition of the lining is relatively unimportant, and it may be of either shotcrete or concrete. Alternatively when it is required to test the lining together with the rock, the lining should not be segmented and its properties should be modeled according to those of the prototype.

[3]Maximum hydraulic pressure varies from 5 to 10 MPa.

Reference

International Society for Rock Mechanics, "Suggested Method for Measuring Rock Mass Deformability Using a Radial Jacking Test," International Journal of Rock Mechanics and Mining Sciences, v. 16, 1979, pp. 208-214.

SUGGESTED METHOD FOR DETERMINING ROCK MASS DEFORMABILITY USING A RADIAL JACK CONFIGURATION

Scope

1. (a) This test determines the deformability of a rock mass by subjecting the cylindrical wall of a tunnel or chamber to uniformly distributed radial jack loading and measuring the resultant rock displacements. Elastic or deformation moduli are calculated in turn.

(b) The test loads a large volume of rock so that the results may be used to represent the true properties of the rock mass, taking into account the influence of joints and fissures. The anisotropic deformability of the rock can also be measured.

(c) The results are usually employed in the design of dam foundations and for the proportioning of pressure shaft and tunnel linings.

(d) Two other methods are available for tunnel-scale deformability. See RTH-361 and RTH-366 to compare details. The large impacts, especially in terms of cost, of variations at this scale justify the separation of methods.

(e) This test is expensive to perform, and therefore, should only be used in cases where the information to be gained is of critical importance to the success or failure of the project. Laboratory tests, together with plate bearing and borehole jack tests and seismic surveys may provide adequate estimates of deformability at less cost.

(f) This method largely follows the method in the ISRM and ASTM references listed at the end of this method.

Apparatus

2. Equipment for excavating and lining the test chamber including:

(a) Drilling and blasting materials or mechanical excavation equipment.[1*]

*Numbers refer to NOTES at then end of the text.

(b) Concreting materials and equipment for lining the tunnel, together with strips of weak jointing material for segmenting the lining.[2]

3. A reaction frame (Figure 1) composed of a set of steel rings of sufficient strength and rigidity to resist the force applied by flat jacks. The frame must be provided with smooth surfaces; hardwood planks are usually inserted between the flat jacks and the steel rings.

4. Loading equipment to apply a uniformly distributed radial pressure to the inner face of the concrete lining, including:

(a) A hydraulic pump capable of applying the required pressure and of holding this pressure constant within 5% over a period of at least 24 hr together with all necessary hoses, connectors, and fluid.

(b) Flat jacks, designed to load the maximum of the full circumference of the lining, with sufficient separation to allow displacement measurements, and with a bursting pressure and travel consistent with the anticipated loads and displacements.

5. Load measuring equipment comprising one or more hydraulic pressure gages or transducers[3] of suitable range and capable of measuring the applied pressure with an accuracy better than ±2%.

6. Displacement measuring equipment to monitor rock movements radial to the tunnel with a precision better than 0.01 mm. Single or multiple position extensometers are suggested but joint meters and other measuring devices are also available. Measured movements must be related to fixed reference points, outside the zone of influence of the test section.

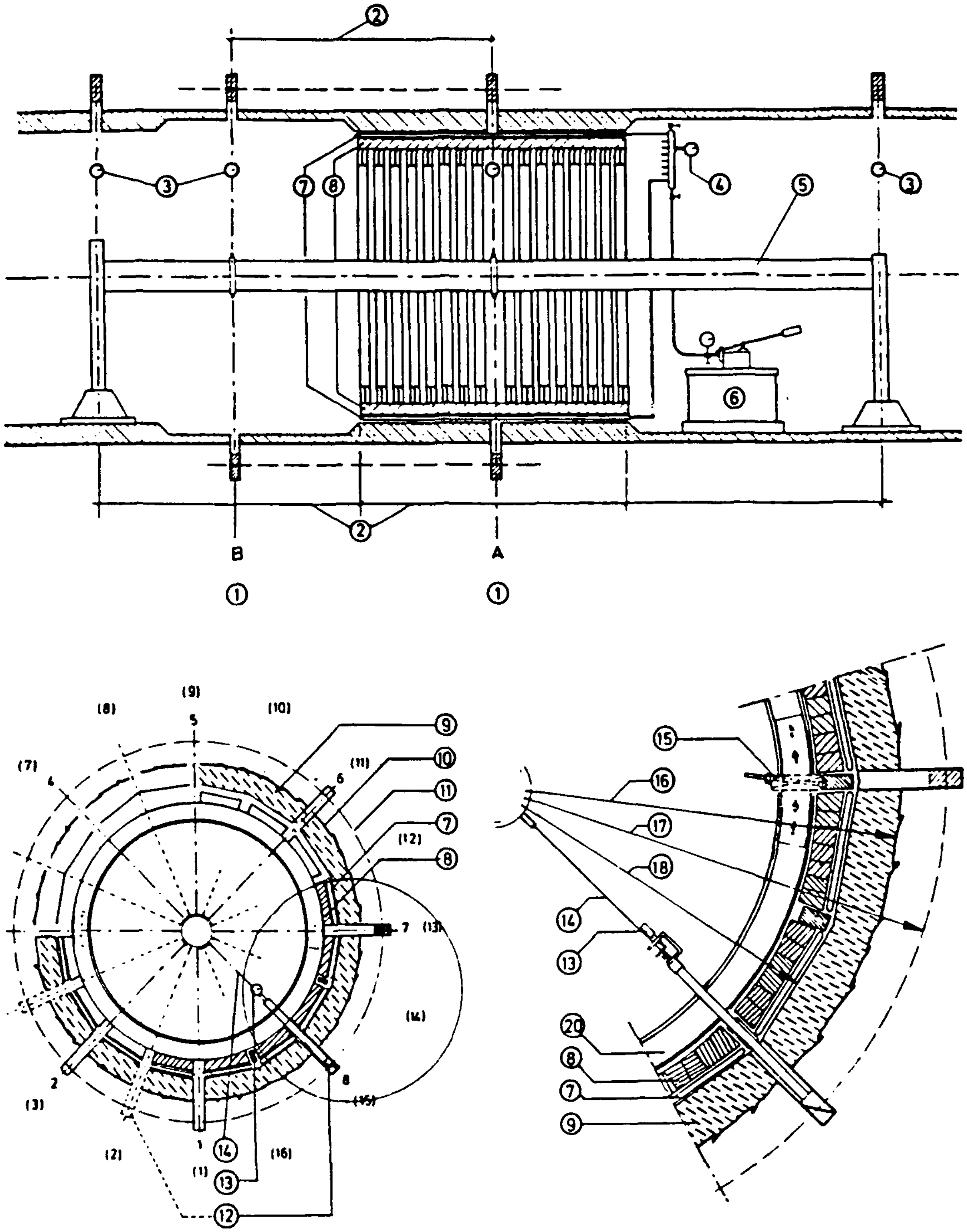
Procedure

7. Preparation

(a) The test chamber location is selected taking into account the rock conditions, particularly the orientation of the rock fabric elements such as joints, bedding and foliation in relation to the orientation of the proposed tunnel or opening for which results are required.

(b) The test chamber is excavated to the required dimensions.[1,4] Generally, the test chamber is about 3 m diam and 9 m long or longer.

(c) The geology of the chamber is mapped and recorded and specimens taken for testing as required, e.g. laboratory strain gaged uniaxial and triaxial testing.



1. Measuring profile. 2. Distance equal to the length of active loading. 3. Control extensometer. 4. Pressure gage. 5. Reference beam. 6. Hydraulic pump. 7. Flat jack. *. Hardwood lagging. 9. Shotcrete. 10. Excavation diameter. 11. Measuring diameter. 12. Extensometer drill holes. 13. Dial gage extensometer. 14. Steel rod. 15. Expansion wedges. 16. Excavation radius. 17. Measuring reference circle. 18. Inscribed Circle. 19. Rockbolt anchor. 20. Steel ring.

Figure 1. Radial jacking test schematic.

(d) The chamber is lined with concrete.[2]

(e) The reaction frame and loading equipment are assembled.

(f) Holes for extensometers or other measuring devices are accurately marked and drilled, ensuring no interference between loading and measuring system. Locations of radial measurement should be chosen with regard to the rock fabric and any other anisotropy. These holes should be continuously cored and all core carefully logged. These holes may be drilled before the test section is lined, but this complicates the lining formwork.

(g) Measuring equipment is installed and checked. With multiple position extensometers the deepest anchor may be used as a reference provided it is situated at least 2 chamber diameters from the lining. Alternatively the measurements may be related to a rigid reference beam passing along the axis of the chamber and anchored not less than 1 diameter from either end of the test section (Figure 1).

8. Testing

(a) The test is carried out in at least three loading and unloading cycles, a higher maximum pressure being applied at each cycle.[5] Maximum test pressures are typically about 1000 psi (7 MPa), but should correspond to expected actual loads.

(b) For each cycle the pressure is increased at an average rate of 0.05 to 0.7 MPa/min to the maximum for the cycle. Three to 10 or more intermediate sets of load-displacement readings are taken for each load increment to define a set of pressure-displacement curves (e.g. Figure 2). Data acquisition may be automated.

(c) On reaching the maximum pressure for the cycle the pressure is held constant (±2% of maximum test pressure) while recording displacements as a function of time until approximately 80% of the estimated long-term displacement has been recorded (Figure 3).[6] Each cycle is completed by reducing the pressure to near-zero at the same average rate, taking a further three sets of pressure-displacement readings.

(d) For the final cycle the maximum pressure is held constant for 24 hr of displacement are observed to evaluate creep.[6] The cycle is completed by unloading in stages while taking readings of pressure and corresponding displacements.

(e) The test equipment is then dismantled and moved.[4]

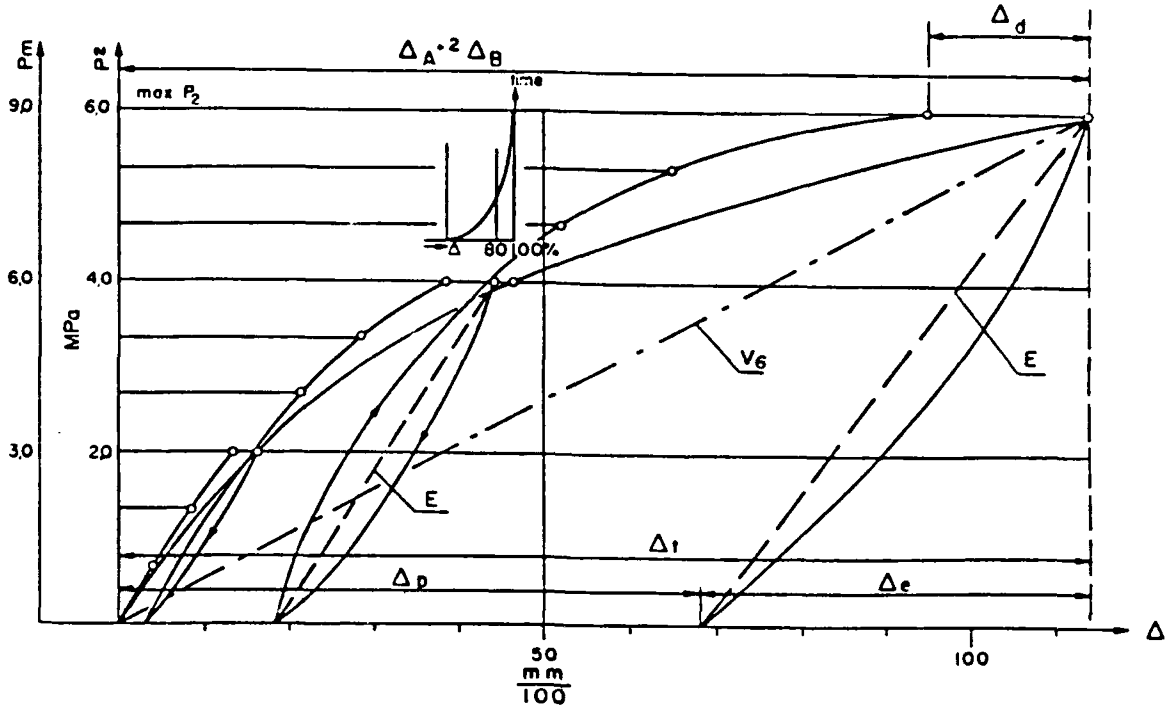


Figure 2. Typical pressure-displacement curves.

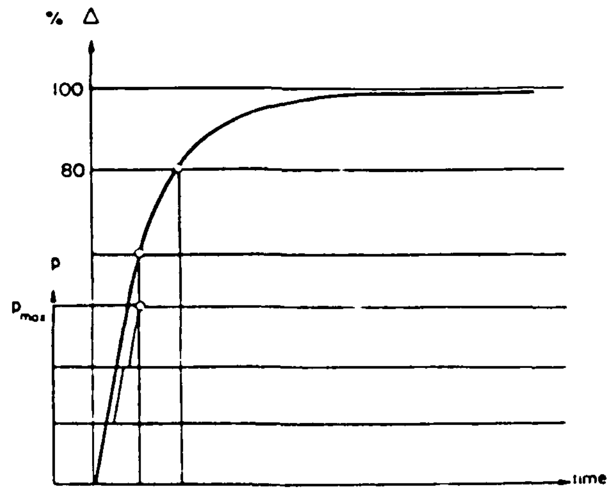


Figure 3. Typical displacement-time curves at constant applied pressure.

Calculations

9. (a) A solution is given only for the case of a single measuring circle with extensometer anchors immediately behind the lining. This solution, which also assumes linear-elastic behavior for the rock, is usually adequate in practice although it is possible to analyze more complex and realistic test configurations using for example finite element analysis.

(b) The load applied through the flat jacks are first corrected to given an equivalent distributed pressure p_1 on the test chamber lining.

$$p_1 = \frac{\Sigma b}{2 \cdot \pi \cdot r_1} p_m$$

p_1 = distributed pressure on the lining at radius r_1

p_m = manometric pressure in the flat jacks

b = flat jack width (see Figure 4)

r_1 = inner radius of lining

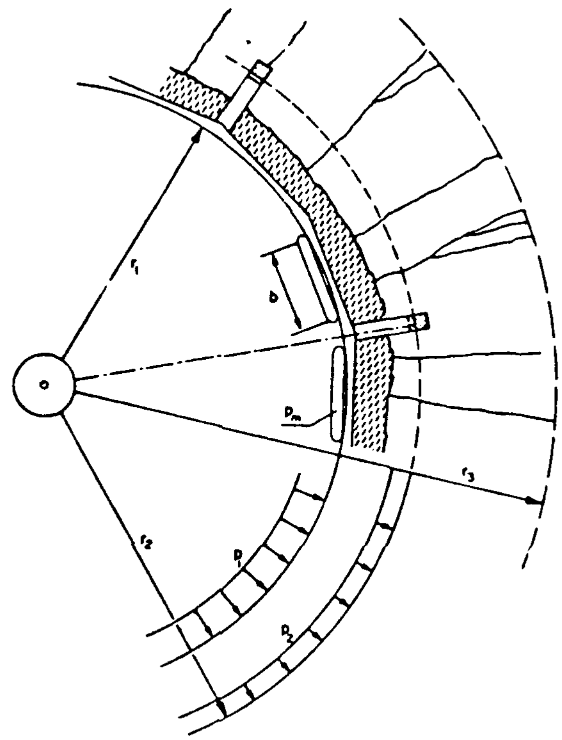
The equivalent pressure p_2 at a "measuring radius" r_2 just beneath the lining is calculated, this radius being outside the zone of irregular stresses beneath the flat jacks and the lining and loose rock.

$$p_2 = \frac{r_1}{r_2} \cdot p_1 = \frac{\Sigma b}{2 \cdot \pi \cdot r_2} \cdot p_m.$$

(c) Superposition of displacements (Figure 5) for two "fictitious" loaded lengths is used to approximate the equivalent displacements for an "infinitely long test chamber,"[7] based on the measured displacements of the relatively short test chamber with respect to its diameter.

$$\Delta = \Delta A_1 + \Delta A_2 + \Delta A_3 = \Delta A_1 + 2 \cdot \Delta B_1$$

(d) The result of the long duration test (Δ_d) under maximum pressure (max p_2) is plotted on the displacement graph (Figure 2). Test data for each cycle are proportionally corrected to give the complete long-term pressure-displacement curve. The elastic and plastic components of the total



$$p_m \cdot \Sigma b = p_1 \cdot 2 \cdot r_1 \cdot \pi$$

$$p_1 = \frac{p_m \cdot \Sigma b}{2 \cdot \pi \cdot r_1}$$

$$p_2 = p_1 \cdot \frac{r_1}{r_2}$$

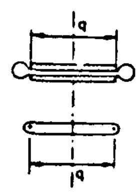


Figure 4. Schematic of loading with symbols used in calculations.

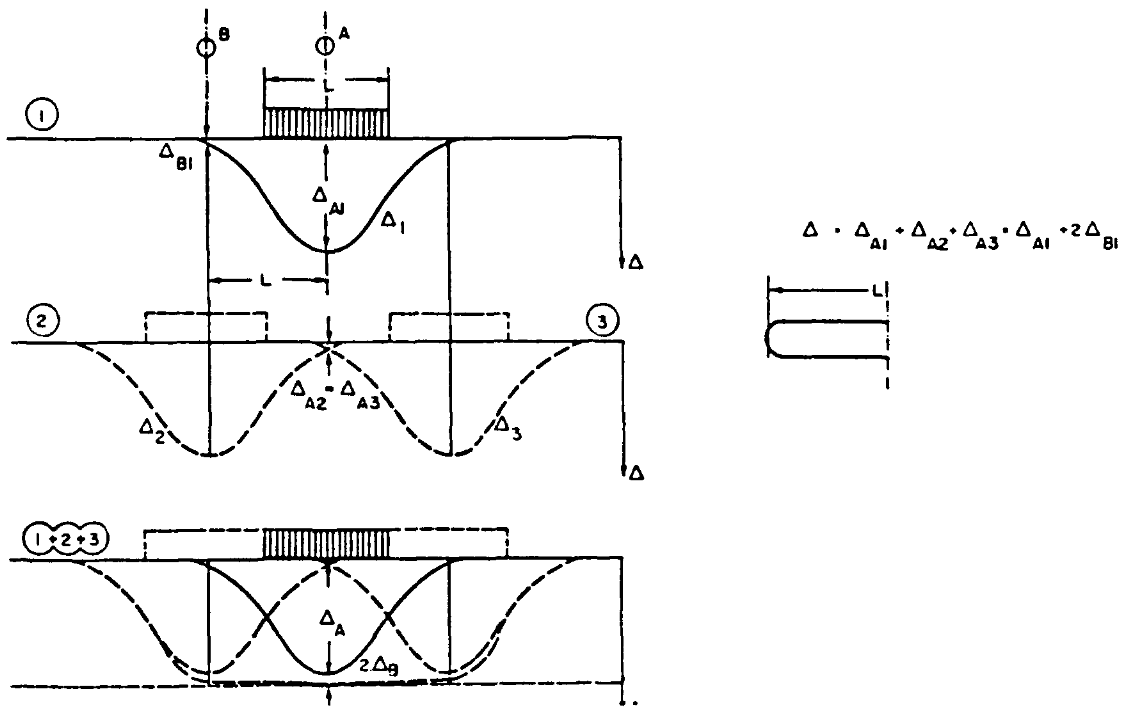


Figure 5. Method of superpositioning of displacements to eliminate end effects.

deformation are obtained graphically from the plotted deformation measurements at the final unloading:

$$\Delta_t = \Delta_p + \Delta_e$$

(e) The elastic modulus E and the deformation modulus V are obtained from the pressure-displacement graphs (Figure 2) using the following formulas based on the theory of elasticity:

$$E = \frac{p_2 \cdot r_2}{\Delta_e} \cdot \frac{(\nu + 1)}{\nu}$$

$$V = \frac{p_2 \cdot r_2}{\Delta_t} \cdot \frac{(\nu + 1)}{\nu}$$

where p_2 is the maximum test pressure and ν is an estimated value for Poisson's ratio.

(f) Alternatively to (e) above, the moduli of undisturbed rock may be obtained taking into account the effect of a fissured and loosened region by using the following formulas:

$$E = \frac{p_2 \cdot r_2}{\Delta_e} \quad \frac{\nu + 1}{\nu} + \ln \frac{r_3}{r_2}$$

$$V = \frac{p_2 \cdot r_2}{\Delta_t} \quad \frac{\nu + 1}{\nu} + \ln \frac{r_3}{r_2}$$

Where Δt is the total deformation, from all loading cycles, and Δe is the rebound deformation from the last loading cycle. Other variables are as previously defined. Where r_3 is the radius to the limit of the assumed fissured and loosened zone.

(g) As one example of the application of radial jack tests, the dimensions of pressure linings can be determined directly by graph. See Lauffer and Seeber 1961. However, such empirical applications should only be applied with caution and good judgement.

Reporting

10. The report should include the following:

(a) Drawings, photographs, and detailed description of the test equipment, chamber, chamber preparation, lining, and testing.

(b) Geological plan and section of the test chamber showing and describing features that may affect the test results. Logs of borings made for the extensometer installations, and indexed photographs of the rock cores.

(c) Tabulated test observations together with graphs of displacement versus applied pressure p_m or p_2, and displacement versus time at constant pressure for each of the displacement measuring locations including displacements at the rock to lining interface. Tabulated "corrected" values together with details of the corrections applied. See Figures 2 and 3 and Table 1 (graphs are usually drawn only for the maximum and minimum displacements).

(d) Transverse section of the test chamber showing the total plastic displacements resulting from the maximum pressure (Figure 6). The orientations of significant geological fabrics should be shown on this figure for comparison with any anisotropy of test results.

(e) Detailed test procedure actually used and any variations and reasons for these from the method described herein, as well as any pertinent or unusual observations.

(f) The graphs showing displacements as a function of applied pressure (Figure 2) should be annotated to show the corresponding elastic and deformation moduli and data from which these were derived.

(g) Equations and methods used to reduce and interpret results should be clearly presented, along with one worked out example.

(h) All simplifying assumptions should be listed, along with discussion of pertinent variations between assumptions and actual size conditions and their possible influence on the results measured. Any corrections should be fully documented.

(i) Results summary table, including test location, rock type, test pressure range, and modulus values for different depth increments around the chamber.

(j) Individual test summary tables for each measurement point.

(k) Results of complementary tests, such as laboratory modulus.

Table 1. Suggested Layout for Test Data Sheet

1	2	3	4	5	4 + 5	6	7	4 + 5 + 7	8	9
NR	time	p_2	Δ_A	Δ_B	$\Delta_A + \Delta_B$	Δ_d	Δ_d corr.	Δ_t	Δ_e	Δ_p
1						—	—			
2						—	—			
3a							—			
3b							—			
3c										
4										
5										
6a							—			
6b							—			
6c										
7										
8										
9a										
9*							—			

$$E = \frac{p_2 \cdot r_2}{\Delta_e} \cdot \frac{\mu + 1}{\mu} = \text{----------------}$$

$$V = \frac{p_2 \cdot r_2}{\Delta_t} \cdot \frac{\mu + 1}{\mu} = \text{----------------}$$

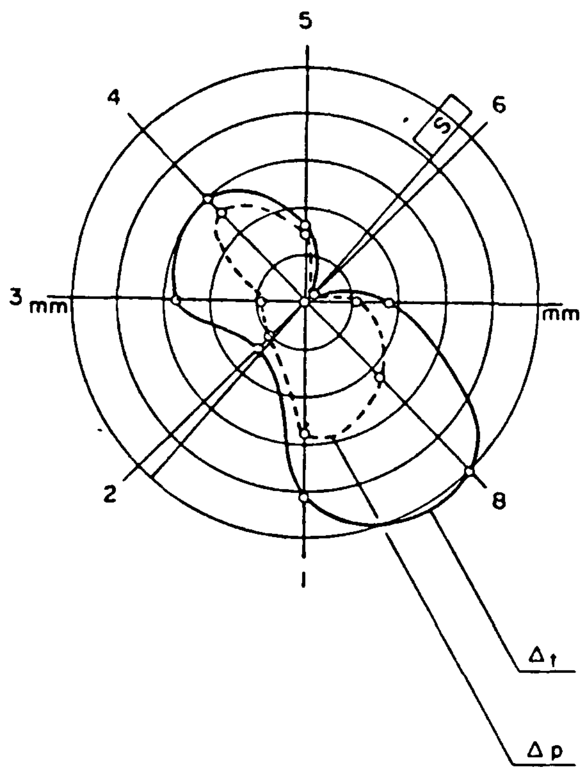


Figure 6. Typical illustration for showing displacement as a function of direction.

Notes

[1]The recommended diameters is 2.5 to 3 m, with a loaded length equal to 3 times this diameter. The chamber should be excavated with as little disturbance as possible, which implies the use of controlled blasting methods, such as line drilling or channel drilling. Alternatively, partial face mechanical excavation equipment may be used if available.

[2]When testing only the rock, the lining should be segmented so that it has negligible resistance to radial expansion; in this case the composition of the lining is relatively unimportant, and it may be of either shotcrete or concrete. Alternatively when it is required to test the lining together with the rock, the lining should not be segmented and its properties should be modeled according to those of the prototype.

[3]Measurements are usually made with mechanical gages. Particular care is required to guarantee the reliability of electric transducers and recording equipment.

[4]To assess the effectiveness of grouting, two test chambers may be prepared adjacent to each other. Grouting is carried out after completion of testing in the ungrouted chamber, and the equipment is then transferred to the grouted chamber. Alternatively the same chamber may be retested after grouting.

[5]Typically the maximum pressure applied in this test is 5 - 10 MPa.

[6]In the case of "creeping" rock it may be necessary to stop loading even though the displacements continue. Not less than 80% of the anticipated long-term displacement should have been reached.

[7]This superposition is made necessary by the comparatively short length of test chamber in relation to its diameter. Superposition is only strictly valid for elastic deformations but also give a good approximation if the rock is moderately plastic in its behavior.

References

International Society for Rock Mechanics, "Suggested Method for Measuring Rock Mass Deformability Using a Radial Jacking Test," International Journal of Rock Mechanics and Mining Sciences, v. 16, 1979, pp. 208-214.

American Society for Testing and Materials, 1986 Annual Book of ASTM Standards, Section 4, Construction; Volume 4.08 Soil and Rock; Building Stones, Standard D 4506-85, "Standard Test Method for Determining the In Situ Modulus of Deformation of Rock Mass Using a Radial Jacking Test."

Lauffer, H. and Seeber, G., "Design and Control of Linings in Pressure Tunnels and Shafts," 7th International Conference on Large Dams, 1961.

RTH-368-89

SUGGESTED METHOD FOR DETERMINING ROCK MASS DEFORMABILITY USING A DRILLHOLE-JACK DIALOMETER

Scope

1. (a) This test determines the deformability of a rock mass by subjecting a section of drill hole to mechanical jack pressure and measuring the resultant wall displacements. Elastic moduli and deformation moduli are calculated in turn.

(b) The results are employed in design of foundations and underground construction but are mostly used as semiquantitative index values revealing variability from point to point in a rock mass.

(c) The dilatometer is self-contained, and tests are relatively inexpensive compared to similar tests at a large scale.[1] Also, the wall is damaged only minimally by the drilling of the hole and usually remains representative of the undisturbed rock condition. These advantages, however, come at a sacrifice of representation of the effects of joints and fissures which are usually spaced too widely to be fully represented in the loaded volume around the drill hole.

(d) This method reflects practice described in the references at the end.

(e) Another type of dilatometer for drillholes transmits hydraulic pressure to the rock through a soft membrane. See RTH-363.

Apparatus

2. Drilling equipment to develop the access hole, in a given orientation without disturbing the wallrock.[2]

3. A drill hole-jack dilatometer similar to that in Figure 1, which consists of:

(a) Metal frame holding the other units and having parts and connections to external equipment.

(b) Two half-cylindrical, rigid steel plates of same curvature as wall of drillhole. Alternatively, the plates can be flexible as where they are faced with curved flat jacks as shown in Figure 2.

(c) Loading jacks or wedges functioning to drive the plates against the wall. A stroke of about 5 mm is needed beyond any requirements for seating the plates.

(d) Linear differential transformers oriented diametrically with potential resolution of 2 microns.

4. Hand-operated hydraulic pump and flexible hose and steel plumbing to withstand working pressure to 70 MPa.

5. Hydraulic pressure gages or transducer of suitable range and capable of measuring the applied pressure with accuracy better than 2 percent.

Procedure

6. Preparation

(a) The positions for testing are planned with due regard to the location of drilling station and the rock conditions to be investigated. The effects of geological structure and fabric are particularly important.

(b) The hole is drilled and logged. The log is studied for possible modifications in positions for testing. Multiple testing positions in one hole should not overlap but may join where two or more loading directions are to be distinguished.

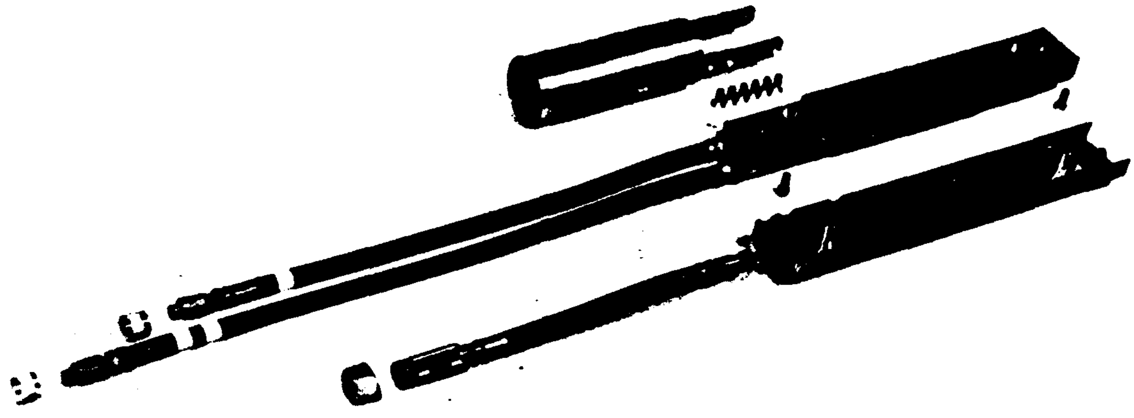
(c) Evaluate the texture and strength of the wall to confirm suitability of method. Otherwise, consider relocation or use of another dilatometer.

(d) The dilatometer is assembled and inserted into the hole, commonly using an attachable pole to position and rotate and taking special care with trailing lines.

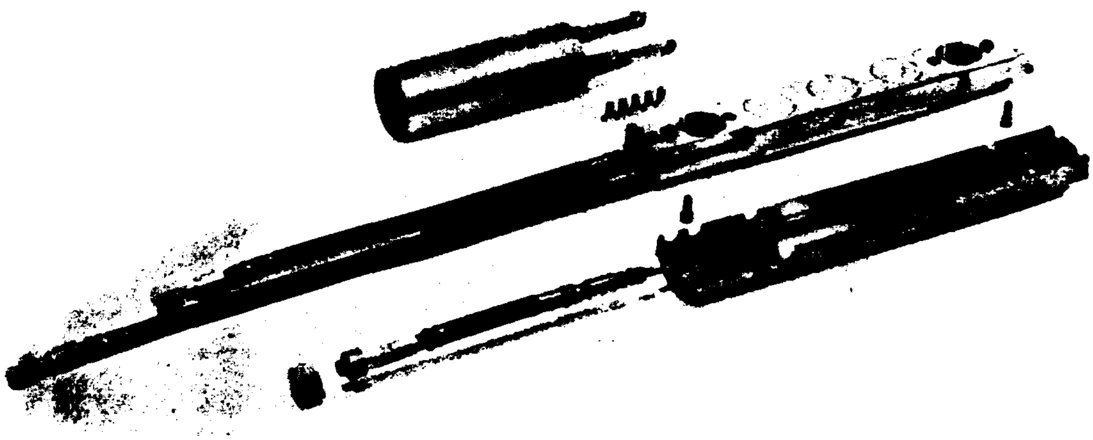
(e) The bearing plates are brought into initial contact with the wall with adjustments as necessary to minimize eccentricity and stress concentrations.

(f) The rods of the linear differential transformers are seated against the wall.

BOREHOLE TESTS



(a) Goodman jack model 52101 hard rock jack
(Courtesy Slope Indicator Co.).



(b) Goodman jack model 52102 soft rock jack
(Courtesy Slope Indicator Co.).

Figure 1. Example dilatometers using hard-faced jacks for loading. (Lama and Vukuturi, 1978)

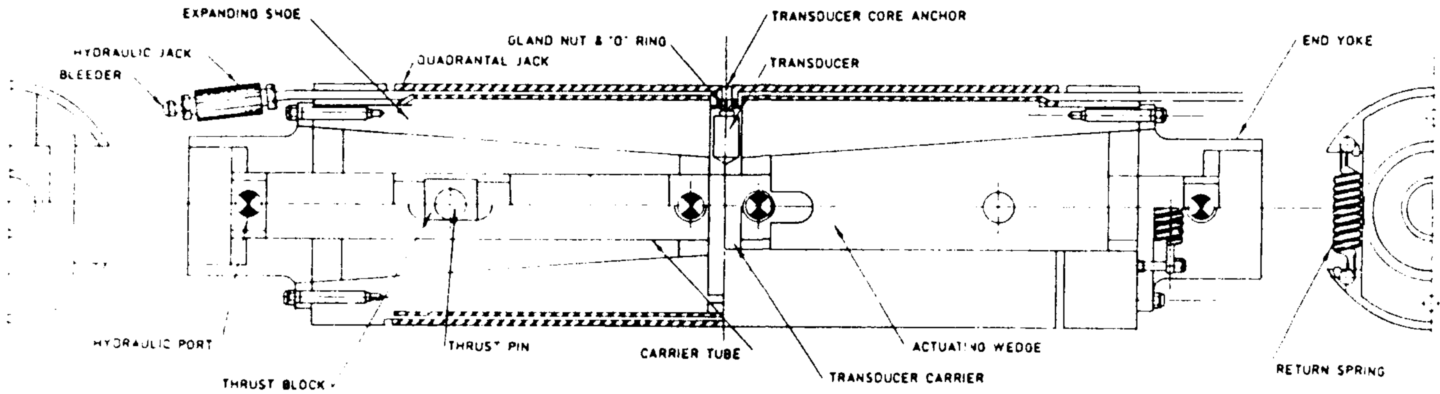


Figure 2. Example dilatometer using soft-faced flat jacks for loading. (Lama and Vukuturi, 1978)

7. Testing

(a) The dilatometer is pressurized in increased stages, with pressure released between stages. Typically the stage pressures are 25, 50, 75, and 100 percent of the planned maximum of the complete test.[3]

(b) Bearing pressure is increased at a rate of 0.5 MPa/min or less.

(c) On reaching the planned pressure for the stage, the pressure is held constant for at least 1 min to detect and define nonelastic deformation. Each stage is completed by releasing pressure at a prescribed rate up to 0.5 MPa/min.

(d) The test history is documented with four or more sets of measurements during pressure increase and two during pressure decrease. Supplementary notes are necessary to describe any complexities not otherwise revealed (such as nonelastic deformation).

(e) The hydraulic pressure is released from the loading jacks. The rods of the linear differential transformers are retracted. The loading plates are retracted from the wall and the dilatometer is removed from the hole.

Calculations

8. The axiosymmetrical relationship for elastic deformation does not apply directly for dilatometers with split loading. Modified expressions have been developed for the specific apparatus. The expression for the dilatometer in Figure 1 is

$$E = \frac{\Delta pd}{\Delta U_d} \cdot K(\nu, \beta)$$

where

Δp = pressure increment

ΔU_d = diametral displacement increment

d = diameter of hole

$K(\nu, \beta)$ = constant dependent on Poisson's ratio and angle of loaded arc β.

Such characteristics as $K(\nu, \beta)$ are supplied by the manufacturer since they are unique to each particular design.[4]

Where permanent deformation (nonelastic) occurs also, that portion of ΔU_d should be excluded from the equation for calculating modulus of elasticity. However, total deformation should be used to compute modulus of deformation, with the same equation.

Reporting

9. The report should include for each test or all tests together the following:

(a) Position and orientation of the test, presented numerically, graphically, or both ways.

(b) Logs and other geological descriptions of rock near the test. The structural details are particularly important.

(c) Tabulated test observations together with graphs of displacement versus applied pressure and displacement versus time at constant pressure for each of the displacement measuring devices (e.g., linear differential transformers).

(d) Transverse section of hole showing the displacements resulting from the pressure in all orientations tested. Calculated moduli are indicated also.

Notes

[1] See RTH-361, -366, -367 for similar test at tunnel scale.

[2] Diamond core drilling is recommended for obtaining the necessary close tolerance when using dilatometer only slightly smaller than the hole and displacement measuring devices with very limited stroke.

[3] Typically, the maximum pressure is about 15 MPa.

[4] Representation of the effects of a combination of complexities by this empirical factor $K(\nu,\beta)$ has been found unsatisfactory for most purposes except indexing (Heuze and Amadei 1985).

References

Lama, R. D., and Vutukuri, V. S., Handbook on Mechanical Properties of Rock Vol. III, TransTech Publications, 1978, 406 pp.

Stagg, K. G., "In Situ Tests on the Rock Mass," in Rock Mechanics in Engineering Practice, John Wiley & Sons, New York, 1968, pp 125-156.

Heuze, F. E., and Amadei, B., "The N-X-Borehole Jack: A Lesson in Trials and Errors," Int. Journal of Rock Mechanics and Mining Sciences, v. 22, No. 2, 1985, pp 105-112.

PART II. IN SITU TEST METHODS

E. Determination of Rock Mass Permeability

RTH 381-80

SUGGESTED METHOD FOR IN SITU DETERMINATION OF ROCK MASS PERMEABILITY USING WATER PRESSURE TESTS

1. Introduction

1.1 The water pressure test consists of the injection of water into a borehole at a constant flow rate and pressure. Water enters the rock mass along the entire length of borehole or along an interval of the borehole (test section) which has been sealed off by one or more packers (Fig. 1). Water pressure tests can be conducted in media above or below the groundwater table and anisotropic permeability can be estimated by orienting test boreholes in different directions. Permeability can be computed by assuming a continuous porous medium, or individual fissures and fissure sets within the rock mass can be considered.

2. Test Procedures and Interpretation

2.1 Test Layout and Setup - In many cases, initial exploration boreholes are routinely pressure tested prior to determining location and orientation of predominant fissure sets. Some boreholes should be specifically located for pressure testing as information concerning fissure networks is obtained. A pressure test affects a region, possibly within only a few feet of the borehole. Consequently, test boreholes should be as closely spaced as practicable. Extrapolation of test data between boreholes can be aided by determination of fissure continuity through examination of core logs or visual inspection of borehole walls with a borehole television or conventional type camera. Fault zones should be located and tested separately as they may be zones of exceptionally high or low permeability with respect to the surrounding region.

2.1.1 Where the scope of the exploration program will allow it, predominant fissure sets should be tested individually by orienting boreholes to intersect only the fissure set under investigation. Permeabilities of fissure sets can be combined to obtain overall directional permeabilities. Where fissures are numerous and randomly oriented, the borehole orientation should be perpendicular to the plane in which permeability is to be measured. The majority of pressure tests will be conducted in vertical boreholes; however, some test boreholes in other orientations are needed to estimate the anisotropy of the rock mass. In rock exploration programs, groups of inclined boreholes are generally needed to determine reliable estimates of joint set orientations. These boreholes could be pressure tested to aid in estimating anisotropic permeability.

2.1.2 Generally, a borehole should be tested at intervals along its length to determine a permeability profile. A knowledge of the characteristics and location of intersected fissures is desirable in choosing test section lengths. Such information can be obtained from core examination (Note 1). When possible, test section lengths should be chosen to isolate fissure sets. Where fissures are numerous, test lengths can be limited to 5 or 10 ft (1.5 or 3 m). Where fissures are infrequent, a longer test length may be utilized. In many cases, fissure networks may be considered too complex to require special care in selecting test section lengths. However, it is good practice to test in 5- to 10-ft (1.5- to 3-m) intervals to allow detection of localized high- or low-permeability zones.

NOTE 1--Visual inspection with a borehole television camera or film camera would be beneficial and should be used where economically feasible.

2.1.3 Intervals along the borehole length should be tested using either the single or double packer method. The single packer setup shown in Fig. 1 is used when testing as drilling progresses. This technique is advantageous because it reduces the amount of drill cuttings available for clogging fissures since each section is tested before being exposed to the cuttings produced by further drilling. Also, errors due to packer leakage are minimized since only one packer is used. The double packer method (Fig. 1) can be used to test or retest sections of previously drilled boreholes.

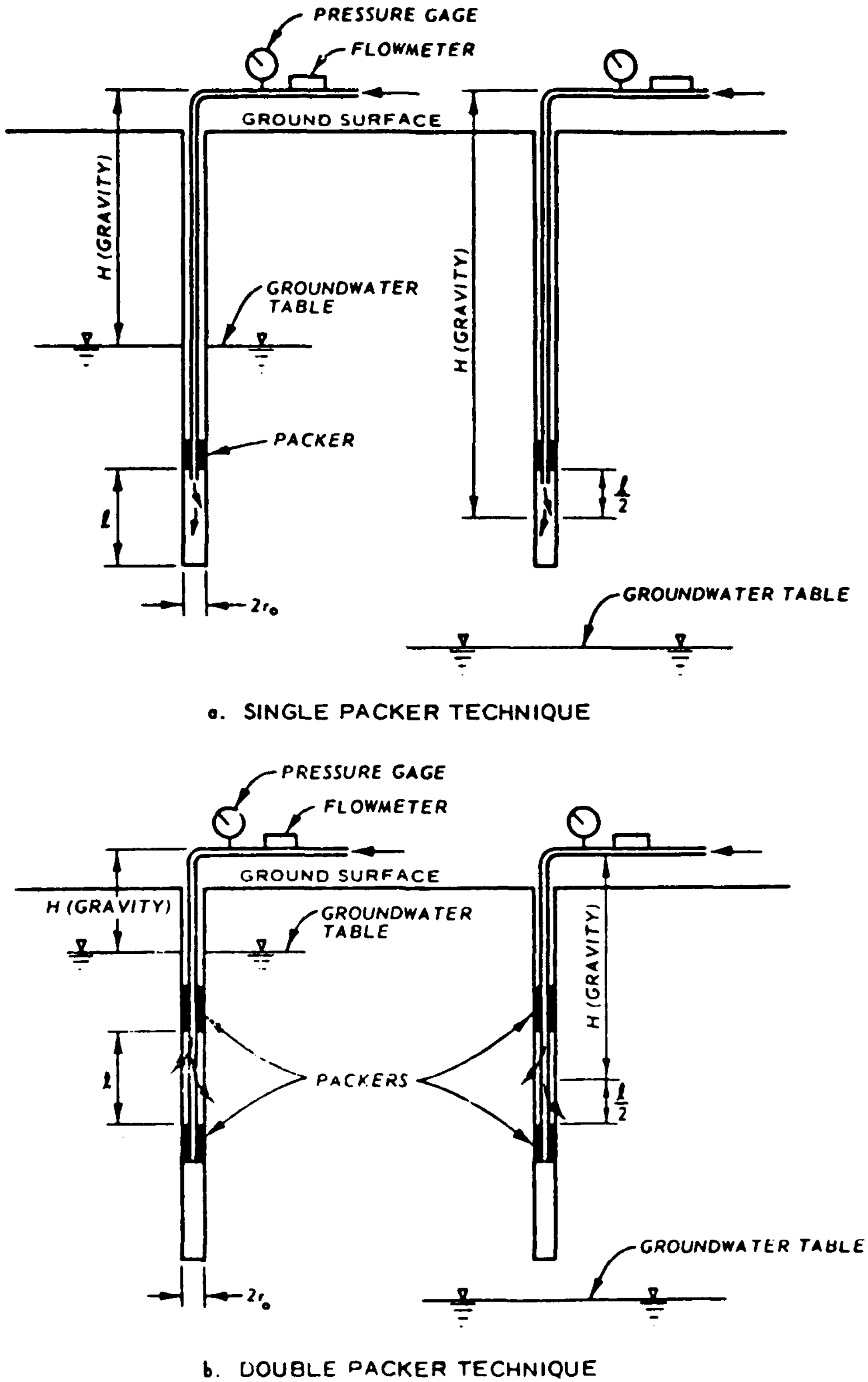


Fig. 1. Pressure test setups--pressure measured at the ground surface.

2.2 Drilling Operations - Prior to pressure testing, the borehole should be surged with water in an effort to remove some of the cuttings and dust. Reverse rotary drilling should be considered for use in boreholes drilled specifically for pressure testing. The removal of cuttings through the drill stem will minimize the clogging of rock fissures.

2.3 Test Equipment - The basic equipment consists of a water supply, pump, packers, flow pipe, and measuring devices. A pump with a minimum capacity of 50 gpm (3150 cu cm/sec) against a pressure of 100 psi (689.5 kPa) is recommended, and only clean water should be used. A progressing cavity-type positive displacement pump is recommended for pressure testing since it maintains a uniform pressure. The type and length of packer needed are dependent on the character of the rock mass to be tested. In most cases, the pneumatic packer will suffice; however, if problems arise, the cup leather or mechanical packers may be substituted. All packers should be at least 18 in. (450 mm) in length. The flow pipe should have a diameter as large as possible to reduce pressure losses between the ground surface and the test section.

2.3.1 Measuring devices are required for determination of volume flow rate and pressure. Flow rate is conveniently measured at the surface, and it is preferred that flow rate be measured continuously rather than averaged by measuring the volume of flow over a known period of time. Multiple gages may be required to measure flow rates ranging from less than 1 gpm (63 cu cm/sec) to as much as 50 gpm (3150 cu cm/sec).

2.3.2 It is recommended that pressure be measured directly within the test section by, for example, installation of an electric transducer. The transducer will also provide a measurement of the existing groundwater pressure at the level of testing. The transducer can be connected to a chart recorder and the initial groundwater pressure indicated as zero. Pressure changes recorded during testing would then be a direct measurement of the excess pressure which is needed in permeability calculations. In most cases, transducer systems will not be readily available. Consequently, pressure will be measured with surface gages. In these instances, pressure loss between the surface and test section must be considered.

2.3.3 When excess pressures are to be determined from surface gage readings, pressure loss between surface and test section must be estimated. Head loss during flow is generally caused by (a) friction, bends, constrictions, and enlargements along the flow pipe; and (b) exit from the flow pipe into the test section. The majority of pressure loss will be caused by friction. Friction losses are dependent on pipe roughness and diameter, and are directly proportional to the square of the flow velocity. Friction losses can be determined experimentally by laying the flow pipe on level ground and pumping water through it at several different velocities while measuring the gage pressure at two points along the pipe. The difference in the gage pressures is the friction loss over the distance between the gages. A plot of friction loss per unit length versus velocity can be obtained from the results. Friction losses can also be estimated from elementary fluid mechanics formulas, tables, and charts.

2.3.4 In most tests, pressure losses caused by pipe bends, constrictions, and enlargements will be insignificant; however, such losses can be checked from relationships given in elementary fluid mechanics textbooks or determined experimentally by pumping on the ground surface and measuring the pressure drop across critical portions of the flow pipe. The pressure loss at the exit from the flow pipe into the test section can be ignored since it is offset by the addition of a velocity head at the surface pressure gage (see paragraph 2.4.3).

2.4 Test Program - The general sequence of operations for using the single packer technique as the borehole progresses is listed below. Changes in the sequence applying to the double packer test are noted.

(a) Step 1 - Drill the desired length of test section, ℓ, (usually 5 or 10 ft (1.5 or 3 m)) and remove the drill equipment. Where the double packer method is to be used, the borehole is drilled to any desired depth.

(b) Step 2 - Study the core to determine the location, number, and characteristics of fissures intersecting the proposed test interval. If only equivalent permeability is to be computed based on test section length, ℓ, fissure information is not needed. However, such information, when correlated with measured permeability, is helpful in understanding the influence of various fissures or fissure sets on the overall permeability of the mass.

(c) Step 3 - Insert the flow pipe and packer, and seal the packer against the borehole wall. To ensure the best possible seal, additional inflation (or tightening) of packers should be accomplished under each test pressure. When tightening packers, a significant and lasting increase in test pressure accompanied by a decrease in flow rate is an indication that the seal has been improved.

(d) Step 4 - Conduct tests using a series of test pressure.

(e) Step 5 - Remove the packer and flow pipe, and begin drilling as in Step 1. In the double packer test, packers are moved to a new test zone; removal of test equipment will be required to alter the test section length as necessary.

2.4.1 The actual pressure testing is conducted in Step 4. The recommended test procedures are:

(a) Inject water into the borehole and establish a constant pressure.

(b) Take readings of pressure and flow rate over a 3- to 5-min period to ensure that steady-state conditions have been attained. If volume of flow rather than volume flow rate is measured, the average volume flow rate should be checked at 30-sec to 1-min intervals and compared with the overall average volume flow rate for the 3- to 5-min period. At the end of the test, conduct a pressure drop test. This is done simply by shutting off the pump and recording the drop in pressure with time.

(c) Increase or decrease the flow rate (and pressure) and conduct the next test.

2.4.2 The test program should be designed to check for turbulent flow and the effects of fissure widening. This requires that in a selected number of test sections, a series of tests be conducted at different pressures. A minimum of three tests, each at an increased pressure, are required to detect the nonlinear flow rate versus pressure relationship characteristic of turbulent flow. However, more tests should be conducted as necessary to completely describe any nonlinear behavior. Typical flow rate versus pressure curves are shown in Figs. 2-6. Consistency of results should also be checked by repeating the tests in the same sequence. A significant increase in permeability under the lower pressures would indicate the possibility of permanent fissure expansion.

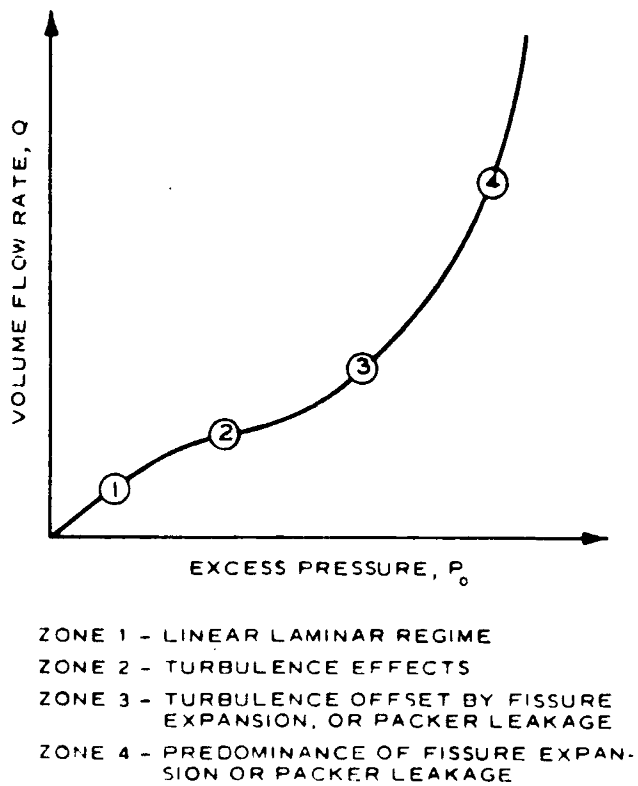
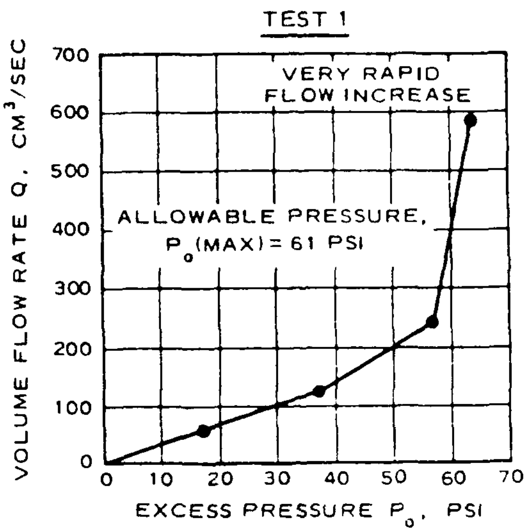
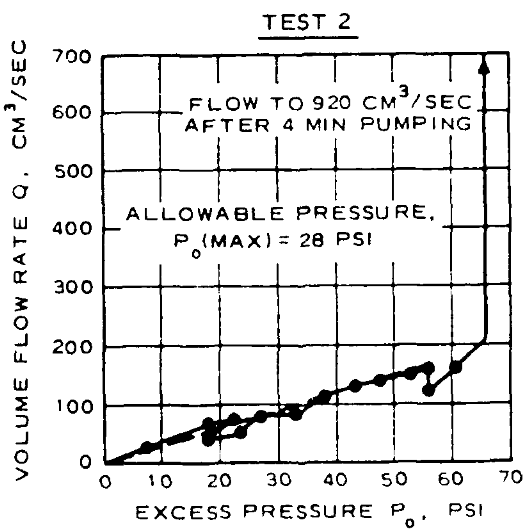


Fig. 2. Typical result of water pressure tests conducted at a series of increasing pressures (after Louis and Maini, 1970; Zeigler, 1975).



a. TEST CONDUCTED IN A VERTICAL BOREHOLE BETWEEN DEPTHS OF 97 AND 102 FT; GROUND-WATER TABLE AT A DEPTH OF 10 FT



b. TEST CONDUCTED IN A VERTICAL BOREHOLE BETWEEN DEPTHS OF 40 AND 45 FT, GROUND-WATER TABLE AT A DEPTH OF 10 FT

Fig. 3. Results of pressure tests in horizontally bedded sedimentary rock (after Morgenstern and Vaughan, 1963; Zeigler, 1975).

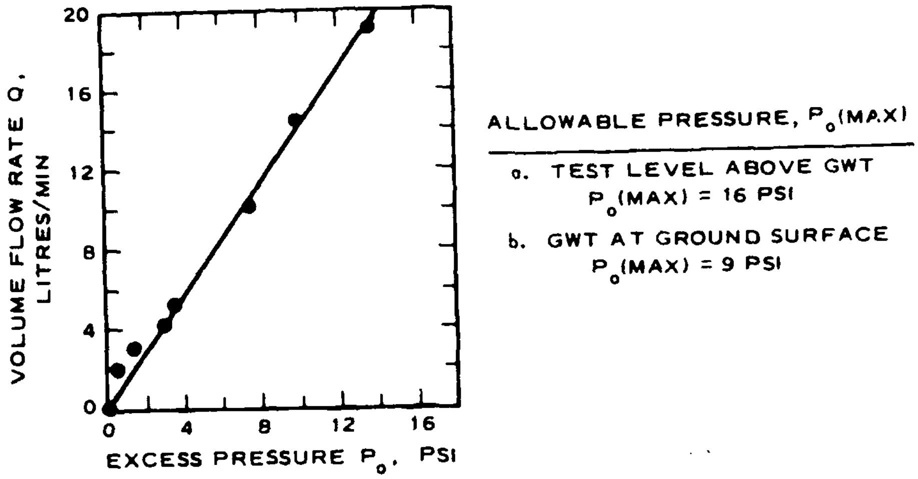


Fig. 4. Pressure test in a vertical borehole between depths of 13.3 and 18.8 ft (after Maini, 1971; Zeigler, 1975).

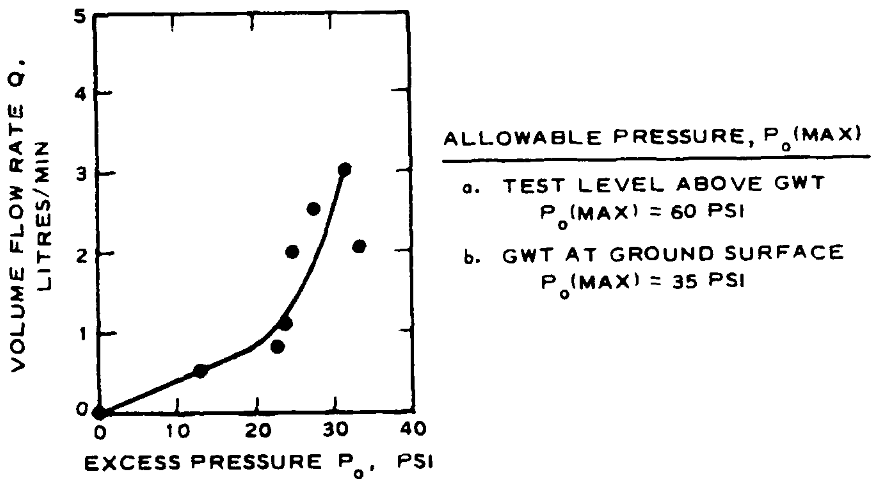
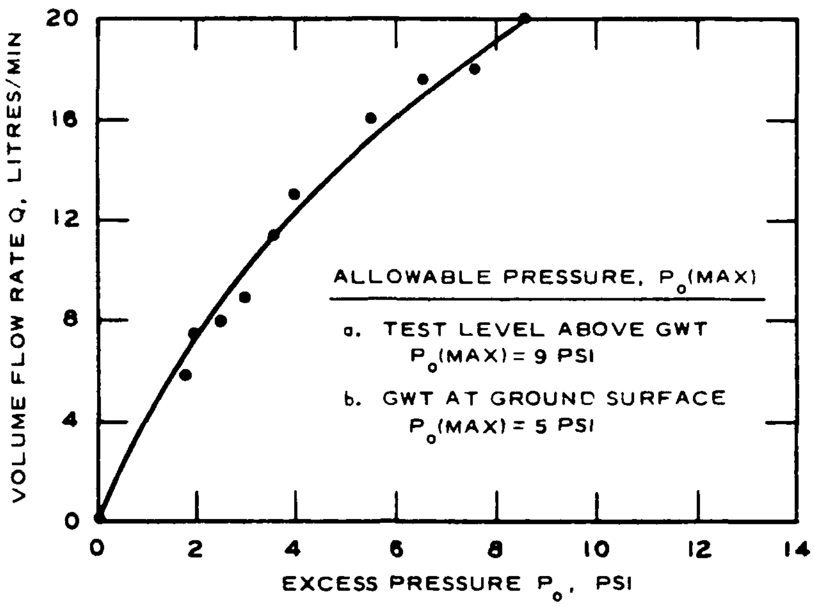
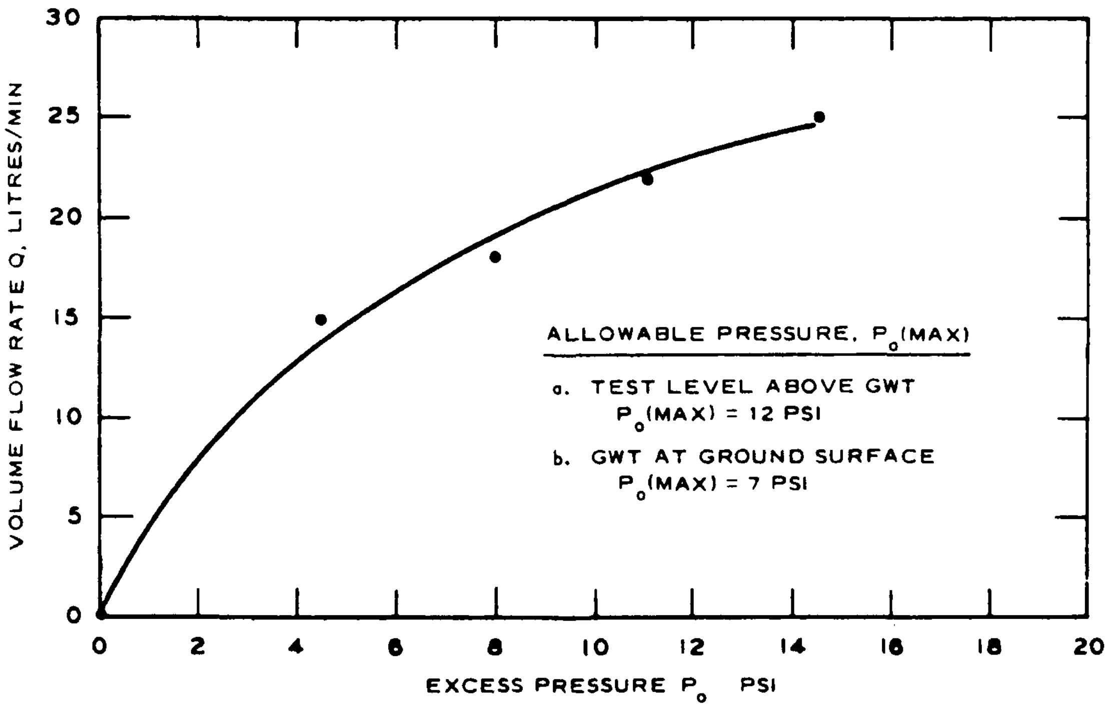


Fig. 5. Pressure test in a vertical borehole between depths of 58.3 and 63.8 ft (after Maini, 1971; Zeigler, 1975).



a. TEST CONDUCTED IN A VERTICAL BOREHOLE BETWEEN DEPTHS OF 6.0 AND 11.5 FT



b. TEST CONDUCTED IN A VERTICAL BOREHOLE BETWEEN DEPTHS OF 10.0 AND 15.5 FT

Fig. 6. Results of pressure tests (after Maini, 1971; Zeigler, 1975).

2.4.3 A typical test sequence using five separate pressures is as follows:

Test No.	Excess Pressure at Center of Test Section, P_o
1	1/5 P_o (MAX)
2	2/5 P_o (MAX)
3	3/5 P_o (MAX)
4	4/5 P_o (MAX)
5	P_o (MAX)
6	1/5 P_o (MAX)
7	3/5 P_o (MAX)
8	P_o (MAX)

The range of pressures over which tests should be conducted can be estimated by choosing P_o(MAX) to equal 1 psi/ft (22.62 kPa/m) of depth above the water table and 0.57 psi/ft (12.89 kPa/m) of depth below the water table. It is not intended that the computed P_o(MAX) be interpreted as a limit below which only laminar flow will occur; it should be used only as a guide in selecting a series of test pressures. Test results should be plotted as they are obtained to determine if further testing of the interval at other pressures is necessary to completely describe any nonlinear behavior.

2.5 Test Data Reduction - The quantities required for use in computing permeability parameters are:

(a) Length of the test section, ℓ (L).

(b) Radius of borehole, r_c (L).

(c) Number, n, and location of fissures intersecting the borehole test section.

(d) Elevation of groundwater table (L).

(e) Volume flow rate, Q (L^3/T).

(f) Excess pressure head at the center of the test section, H_o(L).

2.5.1 The test section length, ℓ, is simply the distance between the packer and borehole bottom (single packer test) or between the two packers (double packer test) as shown in Fig. 1. The radius of the borehole, r_o, is determined from the drill equipment. The number, n, and location of fissures intersecting the borehole test section are obtained from study of the core or a borehole camera survey. The fissure data are not needed if only equivalent permeability is computed based on the assumption that the tested medium is homogeneous and isotropic. The elevation of the groundwater table is determined before testing and assumed to remain constant during testing.

2.5.2 The volume flow rate, Q, will have been continuously recorded or determined by averaging the volume of flow over known time periods. The excess pressure head at the center of the test section, H_o, is a measure of pressure in height of water and is determined from

$$H_o = \frac{P_o}{\gamma_w} \tag{1}$$

where

P_o = excess pressure at center of test section (F/L^2)
γ_w = unit weight of water (F/L^3)

If total pressure in the test section is measured during the test with, for example, an electric transducer, the excess pressure head, H_o, is given by

$$H_o = \frac{P_t}{\gamma_w} - \frac{P_{t_i}}{\gamma_w} \tag{2}$$

where

P_t = total pressure at the center of the test section (F/L^2)
P_{t_i} = pretest (or natural) groundwater pressure at the center of the test section (F/L^2)

The pressure head (P_{t_i}/γ_w) will generally be equivalent to the height of the groundwater table above the center of the test section. In tests above the groundwater table, P_{t_i}/γ_w will be zero. If a natural groundwater pressure exists and is set equal to zero on the recording device, the excess pressure, P_o, and not total pressure will be recorded during testing and H_o would be determined from Equation 1.

2.5.3 The excess pressure head, H_o, can also be determined from gage pressure measured at the ground surface. The following relationship is derived by application of Bernoulli's equation

$$H_o = \frac{P_g}{\gamma_w} + \frac{v_g^2}{2g} + H(\text{gravity}) - h_t \tag{3}$$

where

P_g = pressure measured at the surface gage (F/L^2)

V_g = flow velocity at the surface gage (L/T)

g = acceleration due to gravity (L/T^2)

$H(\text{gravity})$ = excess pressure head due to the height of the water in the flow pipe (Fig. 1) (L)

h_t = sum of all the head losses between the surface gage and the test section (L)

By assuming the test section and surrounding medium to behave as a large reservoir, the head loss at the exit from the flow pipe to the test section can be approximated as $v_e^2/2g$, where v_e is the flow velocity at the exit point as noted by Vennard.[3.6] Equation 3 can be revised to

$$H_o = \frac{P_g}{\gamma_w} + \frac{v_g^2}{2g} + H(\text{gravity}) - h_L - \frac{v_e^2}{2g} \tag{4a}$$

where h_L = friction head loss plus minor losses due to pipe bends, constrictions, and enlargements (L). Pipe diameters at the surface and test section are usually equal, such that $v_g = v_e$. Also, the minor pressure losses due to pipe bends, constrictions, and enlargements can normally be ignored. Consequently, the pressure head, H_o, can be expressed as

$$H_o = \frac{P_g}{\gamma_w} + H(\text{gravity}) - h_f \tag{4b}$$

where h_f = friction head loss (L).

2.6 Equivalent Permeability - An equivalent permeability should be computed for each test section. Equivalent permeability is computed based on the assumption that the tested medium is homogeneous and isotropic. An equivalent permeability can be computed for laminar or turbulent flow, whichever is indicated by the test data. Radial flow will be assumed since the geometry of the test section (in particular, the high borehole length to diameter ratio) tends to dictate radial flow in a zone near the borehole which is most affected by the pressure test:

(a) Laminar flow governed by Darcy's law ($v = k_e i$, where v = discharge velocity (L/T), k_e = laminar equivalent permeability (L/T), and i = hydraulic gradient (L/C):

$$k_e = \frac{Q}{\ell H_o} \frac{1}{2\pi} \ln (R/r_o) \tag{5}$$

The radius of influence, R, can be estimated from $\ell/2$ to ℓ. To compute k_e, a value of volume flow rate, Q, and corresponding excess pressure head, H_o, are chosen from a straight-line approximation of a plot of Q versus H_o. The straight line must pass through the origin as shown in Fig. 4.

(b) Turbulent flow governed by the Missbach law ($v^m = k'_e i$, where k'_e = turbulent equivalent permeability $(L/T)^m$, and m = degree of nonlinearity):

$$k'_e = \frac{Q^m(R^{1-m} - r_o^{\ 1-m})}{(2\pi\ell)^m\ H_o(1 - m)} \tag{6}$$

The radius of influence, R, can be estimated from $\ell/2$ to ℓ. The degree of nonlinearity, m, is determined as the arithmetic slope of a straight-line approximation to a plot of log H_o versus log Q. The log-log plot may involve all or only a portion of the test data. The value of m should be between 1 and 2. To compute k'_e, values of flow rate, Q, and corresponding excess pressure head, H_o, are chosen from the approximated straight-line log-log plot.

2.6.1 In computing equivalent permeability of particular fissure systems, test section length, ℓ, should be replaced by the term nb_{avg} where n is the number of fissures intersecting the test section, and b_{avg} is the average spacing between fissures intersecting the test section. Substitution of nb_{avg} for ℓ is important where fissures are clustered over a small portion of the test interval. A fairly even distribution of fissures along the test length will normally yield ℓnb_{avg}.

2.7 Permeability of Individual Fissures - Laminar or turbulent permeabilities are estimated for individual fissures by assuming the test section to be intersected by a group of parallel and identical fissures. Each fissure is assumed to be an equivalent parallel plate. Flow is assumed to be radial and to occur only within the fissures. The material between fissures is assumed impermeable. The following equations are applicable:

(a) Laminar flow governed by Darcy's law ($v = k_j$ (where k_j = laminar fissure permeability (L/T))). The equivalent parallel plate aperture, e, is computed from Equation 7 below and used to compute the permeability of each fissure, k_j, from Equation 8:

$$e = \left[\frac{Q \ln (R/r_o)}{2\pi n H_o} \frac{12\mu_w}{\gamma_w} \right]^{1/3} \tag{7}$$

where μ_w = dynamic viscosity of water $(F\text{-}T/L^2)$

and

$$k_j = \frac{e^2 \gamma_w}{12\mu_w} \tag{8}$$

To compute e, corresponding values of Q and H_o are chosen from a straight-line approximation of Q versus H_o, which must pass through the origin as shown in Fig. 4. The radius of influence, R, can be estimated between $\ell/2$ and ℓ.

(b) Turbulent flow governed by Missbach's law ($v^m = k_j' i$, where k_j' = turbulent fissure permeability $(L/T)^m$):

$$k_j' = \frac{Q^m (R^{1-m} - r_o^{\,1-m})}{(2\pi n e)^m \, H_o (1 - m)} \tag{9}$$

To apply Equation 9, an equivalent parallel plate aperture, e, must first be estimated from the linear portion of the flow rate, Q, versus pressure, H_o, curve (i.e., zone 1, Fig. 2) as given by Equation 7. The degree of nonlinearity, m, is the slope of a straight-line approximation to a log-log plot of H_o versus Q. The log-log plot may involve all or only a portion of the test data. Corresponding values of Q and H_o can be chosen from the straight-line log-log plot for substitution in Equation 9. The radius of influence, R, can be chosen between $\ell/2$ and ℓ.

2.8 Directional Permeability - Equivalent permeabilities computed for fissure sets must be interrelated to obtain overall directional permeabilities which are needed in continuum seepage analyses. Directional permeabilities can be obtained by adding the equivalent permeabilities of fissure sets (computed via Equation 5 or 6) oriented in the same direction. This procedure is illustrated for an assumed laminar flow in the three cases shown in Fig. 7. In case (a), the zone tested contains one set of horizontal fissures (fissure set 1). The vertical borehole in case (a) will give a measure of the laminar equivalent permeability, k_{e_1}, of fissure set 1. Permeability in the vertical direction, $k_e(V)$, would be that of the intact rock since there are no vertical fissures. Permeabilities in a direction contained within the horizontal plane (such as $k_e(H1)$ and $k_e(H2)$ in Fig. 7) would be interpreted as k_{e_1}, since k_{e_1} is based on a radial flow.

2.8.1 In case (b) there are two intersecting fissure sets: the horizontal fissures (fissure set 1) and a series of vertical fissures (fissure set 2). The pressure test boreholes are oriented so that each intersects only one of the fissure sets. It is assumed that each test measures only the permeability of the intersected fissure set. In computing directional permeabilities both fissure sets 1 and 2 can transmit flow in the horizontal direction, H2; consequently, their equivalent permeabilities are summed ($k_e(H2) = k_{e_1} + k_{e_2}$). In the vertical direction, V, and horizontal direction, H1, the equivalent permeability of each fissure set is considered separately ($k_e(V) = k_{e_2}$, $k_e(H1) = k_{e_1}$).

2.8.2 In case (c) there are three intersecting fissure sets. Three boreholes are each oriented to intersect only one of the fissure sets. The directional permeabilities are each the sum of equivalent permeabilities corresponding to two fissure sets ($k_e(V) = k_{e_2} + k_{e_3}$; $k_e(H1) = k_{e_1}$; $k_e(H2) = k_e + k_{e_2}$).

2.8.3 When only vertical boreholes are tested, but structures such as in case (b) and case (c) of Fig. 7 are known to exist, the additional permeability added by the other fissure sets must be estimated. This

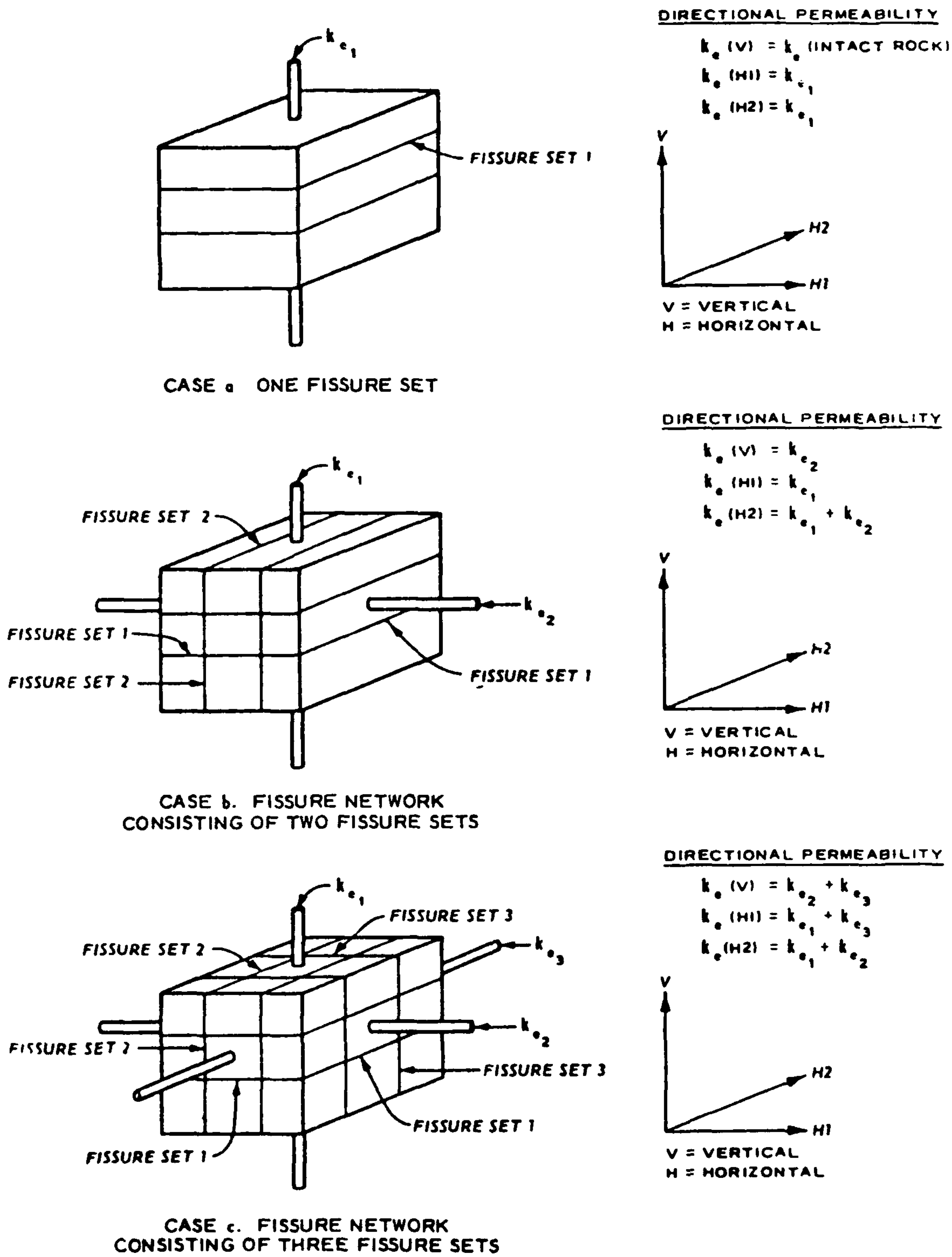


Fig. 7. Directional permeability from superposition of laminar equivalent permeabilities of fissure sets.

can be done based on the assumption that fissures not tested have the same equivalent parallel plate aperture as the tested fissures. Any difference in equivalent permeability between the fissure sets would be a function of the difference in fissure frequency. For example, in case (b) in Fig. 7 under conditions of laminar flow,

$$k_{e_2} = k_{e_1} \frac{b_{avg_1}}{b_{avg_2}} \tag{10}$$

where

b_{avg_1} = average fissure spacing of tested fissure set 1

b_{avg_2} = average fissure spacing of untested fissure set 2

2.8.4 The procedure of adding permeabilities of separate fissure sets relies heavily on the assumption that pressure tests reflect only the permeability of the fissure sets intersecting the test section. This assumption is based on the theoretical rapid loss in pressure away from the borehole. The assumption is likely to become less accurate as average fissure spacing within secondary fissure sets (i.e., fissures tending to parallel the borehole) is decreased. In complex fissure networks with fissure spacings less than 1 ft, it is recommended that the method of Snow,[3.4, 3.5] based on the assumption of a homogeneous anisotropic continuum, be used in computing directional permeabilities.

2.8.5 The problem of combining fissure set permeabilities is avoided by using discontinuum rather than continuum seepage analyses. In the discontinuum analyses, fissures can be oriented to correspond to the field geologic structure and assigned individual permeabilities and/or equivalent parallel plate openings as determined from pressure tests. However, a three-dimensional analysis such as that presented by Wittke et al.[3.7] would be required in many situations. In structures similar to case (c) in Fig. 7, a two-dimensional seepage analysis in any of the indicated directions would consider only two of the three fissure sets. For example, in direction H1, fissure set 3 would be ignored, although it may be a major contributor to seepage in direction H1.

3. References

3.1 Louis, C. and Maini, Y. N. T. (1970), "Determination of In Situ Hydraulic Parameters in Jointed Rock," Proceedings, Second Congress on Rock Mechanics, Belgrade, Vol 1.

3.2 Maini, Y. N. T. (1971), In Situ Hydraulic Parameters in Jointed Rock; Their Measurement and Interpretation, Ph.D. Dissertation, Imperial College, London.

3.3 Morgenstern, N. R. and Vaughn, P. R. (1963), "Some Observations on Allowable Grouting Pressures," Grouting and Drilling Mud in Engineering Practice, Butterworth and Company, London.

3.4 Snow, D. T. (1966), "Three-Hole Pressure Test for Anisotropic Foundation Permeability," Rock Mechanics and Engineering Geology, Vol IV, No. 4, pp 298-316.

3.5 Snow, D. T. (1969), "Anisotropic Permeability of Fractured Media," Water Resources Research, Vol 5, No. 6, Dec, pp 1273-1289.

3.6 Vennard, J. K. (1965), Elementary Fluid Mechanics, 4th ed., John Wiley and Sons, Inc., New York.

3.7 Wittke, W., Rissler, P., and Semprich, S. (1972), "Three-Dimensional Laminar and Turbulent Flow Through Fissured Rock According to Discontinuous and Continuous Models," Proceedings of the Symposium on Percolation Through Fissured Rock, Stuttgart, Germany.

3.8 Zeigler, T. W. (1976), "Determination of Rock Mass Permeability," Technical Report S-76-2, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi.

BIBLIOGRAPHY

Davis, S. N. and DeWiest, R. J. M. (1966), Hydrogeology, John Wiley and Sons, Inc., New York.

Hvorslev, M. J. (1951), "Time Lag and Soil Permeability in Groundwater Observations," Bulletin No. 36, Apr, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi.

Lynch, E. J. (1962), Formation Evaluation, Harper and Row, New York.

Muskat, M. (1946), "The Flow of Homogeneous Fluids Through Porous Media," J. W. Edwards, Inc., Ann Arbor, Michigan.

Corps of Engineers

U.S. Army

ROCK TESTING HANDBOOK

(Test Standards 1993)

Prepared by:
Geotechnical Laboratory
Rock Mechanics Branch
U.S. Army Engineer Waterways Experiment Station
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199

REPLY TO
ATTENTION OF

CEWES-GS-R (1110-1-1150a)

MEMORANDUM FOR SEE DISTRIBUTION

SUBJECT: Transmittal of Rock Testing Handbook - Test Standards 1993, Part I - Laboratory Test Methods

1. The subject Part I - Laboratory Test Methods (encl 1) is a compilation of standards and recommended rock testing methods and has been prepared for use in laboratory offices of the Corps of Engineers. Revision of Part I - Laboratory Test Methods was authorized and funded by the Office, Chief of Engineers, FY 93.

2. The subject Part I - Laboratory Test Methods, supersedes the previous Part I of the "Rock Testing Handbook (Standard and Recommended Methods) March 1990". The current Part I includes: two new methods RTH-107a and RTH 101a-93; small modifications to RTH-102, 104, 106, 108, 109, and 114; replacement of RTH-101, 107, 110, 112, 113, 115, and 205 with appropriate American Society for Testing and Materials standards; and no changes to RTH-105, 111, 201, 203, and 207.

3. Correspondence concerning the subject Part I - Laboratory Test Methods or any part of the Rock Testing Handbook should be addressed to Director, U.S. Army Engineer Waterways Experiment Station, ATTN: CEWES-GS-R, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

for

Encl

ROBERT W. WHALIN, PhD, PE
Director

DISTRIBUTION
For all Commanders, Divisions and Districts

PREFACE

This handbook is a compilation of standard and recommended rock testing methods and has been prepared for use in both the laboratory and the field.

Preparation of the handbook was authorized and funded by the Office, Chief of Engineers, U.S. Army. The cooperation of the American Society for Testing and Materials, the International Society for Rock Mechanics, and the U.S. Bureau of Reclamation in permitting the use of a number of their standards is appreciated.

Suggestions for revisions, corrections, and additions are welcomed. Correspondence concerning such matters should be addressed either to the Commander, U.S. Army Engineer Waterways Experiment Station (ATTN: CEWES-GS-R), 3909 Halls Ferry Road, Vicksburg, MS 39180-6199; or to the Office, Chief of Engineers, U.S. Army (ATTN: Engineering Division, Civil Works), Washington, DC 20314-1000.

In order that this handbook may be of greatest service, it is intended that it be kept up to date by the issuance of supplementary items and the revision of existing items whenever necessary. To facilitate such revisions, the handbook has been issued in loose-leaf form.

The handbook has been prepared in the Geotechnical Laboratory (GL) and Structures Laboratory (SL) of the U.S. Army Engineer Waterways Experiment Station (WES). It was compiled by Ms. M. Eileen Glynn and Mr. Willie E. McDonald under the direct supervision of Messrs. Jerry S. Huie, Chief, Rock Mechanics Branch, GL; and Kenneth L. Saucier, Chief, Concrete Technology Division, SL; and the general guidance of Dr. Don C. Banks, Chief, Soil and Rock Mechanics Division, GL. Dr. W. F. Marcuson III was Chief of GL; Mr. Bryant Mather was Chief of SL. Commander of WES during publication of this handbook was COL Bruce K. Howard, EN. Director was Dr. Robert W. Whalin.

CONTENTS
June 1991

CONTENTS
June 1991

CONTENTS
June 1991

CONTENTS
June 1991

PART I. LABORATORY TEST METHODS

A. Characterization Methods

Designation: D 653 - 90

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Terminology Relating to Soil, Rock, and Contained Fluids[1]

These definitions were prepared jointly by the American Society of Civil Engineers and the American Society for Testing and Materials.

INTRODUCTION

A number of the definitions include symbols and indicate the units of measurement. The symbols appear in italics immediately after the name of the term, followed by the unit in parentheses. No significance should be placed on the order in which the symbols are presented where two or more are given for an individual term. The applicable units are indicated by capital letters, as follows:

F—Force, such as pound-force, ton-force, newton

L—Length, such as inch, foot, centimetre

T—Time, such as second, minute

D—Dimensionless

Positive exponents designate multiples in the numerator. Negative exponents designate multiples in the denominator. Degrees of angle are indicated as "degrees."

Expressing the units either in SI or the inch-pound system has been purposely omitted in order to leave the choice of the system and specific unit to the engineer and the particular application, for example:

FL^{-2}—may be expressed in pounds-force per square inch, kilopascals, tons per square foot, etc.

LT^{-1}—may be expressed in feet per minute, centimetres per second, etc.

Where synonymous terms are cross-referenced, the definition is usually included with the earlier term alphabetically. Where this is not the case, the later term is the more significant.

Definitions marked with (ISRM) are taken directly from the publication in Ref 42 and are included for the convenience of the user.

For a list of ISRM symbols relating to soil and rock mechanics, refer to Appendix X1.

A list of references used in the preparation of these definitions appears at the end.

AASHTO compaction—see **compaction test**.

"A" Horizon—see **horizon**.

abrasion—a rubbing and wearing away. (ISRM)

abrasion—the mechanical wearing, grinding, scraping or rubbing away (or down) of rock surfaces by friction or impact, or both.

abrasive—any rock, mineral, or other substance that, owing to its superior hardness, toughness, consistency, or other properties, is suitable for grinding, cutting, polishing, scouring, or similar use.

abrasiveness—the property of a material to remove matter when scratching and grinding another material. (ISRM)

absorbed water—water held mechanically in a soil or rock mass and having physical properties not substantially different from ordinary water at the same temperature and pressure.

absorption—the assimilation of fluids into interstices.

absorption loss—that part of transmitted energy (mechanical) lost due to dissipation or conversion into other forms (heat, etc.).

accelerator—a material that increases the rate at which chemical reactions would otherwise occur.

activator—a material that causes a catalyst to begin its function.

active earth pressure—see **earth pressure**.

active state of plastic equilibrium—see **plastic equilibrium**.

additive—any material other than the basic components of a grout system.

adhesion—shearing resistance between soil and another material under zero externally applied pressure.

	Symbol	Unit
Unit Adhesion	c_a	FL^{-2}
Total Adhesion	C_a	F or FL^{-1}

adhesion—shearing resistance between two unlike materials under zero externally applied pressure.

admixture—a material other than water, aggregates, or cementitious material, used as a grout ingredient for cement-based grouts.

adsorbed water—water in a soil or rock mass attracted to the particle surfaces by physiochemical forces, having properties that may differ from those of pore water at the same temperature and pressure due to altered molecular ar-

[1] This terminology is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.93 on Terminology for Soil, Rock, and Contained Fluids.

Current edition approved Oct. 27, 1989, and Feb. 1, 1990. Published March 1990. Originally published as D 653 – 42 T. Last previous edition D 653 – 89.

This extensive list of definitions represents the joint efforts of Subcommittee D18.93 on Terminology for Soil, Rock, and Contained Fluids of ASTM Committee D-18 on Soil and Rock, and the Committee on Definitions and Standards of the Geotechnical Engineering Division of the American Society of Civil Engineers. These two groups function together as the Joint ASCE/ASTM Committee on Nomenclature in Soil and Rock Mechanics. This list incorporates some terms from ASTM Definitions D 1707, Terms Relating to Soil Dynamics, which were discontinued in 1967.

rangement; adsorbed water does not include water that is chemically combined within the clay minerals.

adsorption—the attachment of water molecules or ions to the surfaces of soil particles.

advancing slope grouting—a method of grouting by which the front of a mass of grout is caused to move horizontally by use of a suitable grout injection sequence.

aeolian deposits—wind-deposited material such as dune sands and loess deposits.

aggregate—*as a grouting material*, relatively inert granular mineral material, such as sand, gravel, slag, crushed stone, etc. "Fine aggregate" is material that will pass a No. 4 (6.4-mm) screen,

"Coarse aggregate" is material that will not pass a No. 4 (6.4-mm) screen. Aggregate is mixed with a cementing agent (such as Portland cement and water) to form a grout material.

agitator tank—a tank, usually vertical and with open top, with rotation paddles used to prevent segregation of grout after mixing.

air-space ratio, G_a (D)—ratio of: (*1*) volume of water that can be drained from a saturated soil or rock under the action of force of gravity, to (*2*) total volume of voids.

air-void ratio, G_v (D)—the ratio of: (*1*) the volume of air space, to (*2*) the total volume of voids in a soil or rock mass.

alkali aggregate reaction—a chemical reaction between Na_2O and K_2O in the cement and certain silicate minerals in the cement and certain silicate minerals in the aggregate, which causes expansion resulting in weakening and cracking of Portland cement grout. See **reactive aggregate**.

allowable bearing value (allowable soil pressure), q_a, p_a (FL^{-2})—the maximum pressure that can be permitted on foundation soil, giving consideration to all pertinent factors, with adequate safety against rupture of the soil mass or movement of the foundation of such magnitude that the structure is impaired.

allowable pile bearing load, Q_a, P_a (F)—the maximum load that can be permitted on a pile with adequate safety against movement of such magnitude that the structure is endangered.

alluvium—soil, the constituents of which have been transported in suspension by flowing water and subsequently deposited by sedimentation.

amplification factor—ratio of dynamic to static displacement.

amorphous peat—see **sapric peat**.

angle of external friction (angle of wall friction), δ (degrees)—angle between the abscissa and the tangent of the curve representing the relationship of shearing resistance to normal stress acting between soil and surface of another material.

angle of friction (angle of friction between solid bodies), ϕs (degrees)—angle whose tangent is the ratio between the maximum value of shear stress that resists slippage between two solid bodies at rest with respect to each other, and the normal stress across the contact surfaces.

angle of internal friction (angle of shear resistance), ϕ (degrees)—angle between the axis of normal stress and the tangent to the Mohr envelope at a point representing a given failure-stress condition for solid material.

angle of obliquity, α, β, ϕ, Ψ (degrees)—the angle between the direction of the resultant stress or force acting on a given plane and the normal to that plane.

angle of repose, α (degrees)—angle between the horizontal and the maximum slope that a soil assumes through natural processes. For dry granular soils the effect of the height of slope is negligible; for cohesive soils the effect of height of slope is so great that the angle of repose is meaningless.

angle of shear resistance—see **angle of internal friction**.

angle of wall friction—see **angle of external friction**.

angular aggregate—aggregate, the particles of which possess well-defined edges formed at the intersection of roughly planar faces.

anisotropic mass—a mass having different properties in different directions at any given point.

anisotropy—having different properties in different directions. (ISRM)

apparent cohesion—see **cohesion**.

aquiclude—a relatively impervious formation capable of absorbing water slowly but will not transmit it fast enough to furnish an appreciable supply for a well or spring.

aquifer—a water-bearing formation that provides a ground water reservoir.

aquitard—a confining bed that retards but does not prevent the flow of water to or from an adjacent aquifer; a leaky confining bed.

arching—the transfer of stress from a yielding part of a soil or rock mass to adjoining less-yielding or restrained parts of the mass.

area grouting—grouting a shallow zone in a particular area utilizing holes arranged in a pattern or grid.

Discussion—This type of grouting is sometimes referred to as blanket or consolidation grouting.

area of influence of a well, a (L^2)—area surrounding a well within which the piezometric surface has been lowered when pumping has produced the maximum steady rate of flow.

area ratio of a sampling spoon, sampler, or sampling tube, A_r (D)—the area ratio is an indication of the volume of soil displaced by the sampling spoon (tube), calculated as follows:

$$A_r = [(D_e^2 - D_i^2/D_i^2] \times 100$$

where:

D_e = maximum external diameter of the sampling spoon, and

D_i = minimum internal diameter of the sampling spoon at the cutting edge.

armor—the artificial surfacing of bed, banks, shore, or embankment to resist erosion or scour.

armor stone—(generally one ton to three tons in weight) stone resulting from blasting, cutting, or by other methods to obtain rock heavy enough to require handling two individual pieces by mechanical means.

ash content—the percentage by dry weight of material remaining after an oven dry organic soil or peat is burned by a prescribed method.

attenuation—reduction of amplitude with time or distance.

"B" horizon—see **horizon**.

backpack grouting—the filling with grout of the annular space between a permanent tunnel lining and the surrounding formation.

Discussion—Same as crown grouting and backfill grouting.

back-packing—any material (usually granular) that is used to fill the empty space between the lagging and the rock surface. (ISRM)

baffle—a pier, weir, sill, fence, wall, or mound built on the bed of a stream to parry, deflect, check, or regulate the flow or to float on the surface to dampen the wave action.

base—*in grouting*, main component in a grout system.

base course (base)—a layer of specified or selected material of planned thickness constructed on the subgrade or subbase for the purpose of serving one or more functions such as distributing load, providing drainage, minimizing frost action, etc.

base exchange—the physicochemical process whereby one species of ions adsorbed on soil particles is replaced by another species.

batch—*in grouting*, quantity of grout mixed at one time.

batch method—*in grouting*, a quantity of grout materials are mixed or catalyzed at one time prior to injection.

batch mixer—*in grouting*, a machine that mixes batches of grout, in contrast to a continuous mixer.

bearing capacity—see **ultimate bearing capacity.**

bearing capacity (of a pile), Q_p, P_p (F)—the load per pile required to produce a condition of failure.

bedding—applies to rocks resulting from consolidation of sediments and exhibiting surfaces of separation (bedding planes) between layers of the same or different materials, that is, shale, siltstone, sandstone, limestone, etc. (ISRM)

bedding—collective term signifying the existence of layers of beds. Planes or other surfaces dividing sedimentary rocks of the same or different lithology.

bedrock—the more or less continuous body of rock which underlies the overburden soils. (ISRM)

bedrock (ledge)—rock of relatively great thickness and extent in its native location.

bench—(*1*) the unexcavated rock having a nearly horizontal surface which remains after a top heading has been excavated, or (*2*) step in a slope; formed by a horizontal surface and a surface inclined at a steeper angle than that of the entire slope. (ISRM)

bending—process of deformation normal to the axis of an elongated structural member when a moment is applied normal to its long axis. (ISRM)

bentonitic clay—a clay with a high content of the mineral montmorillonite, usually characterized by high swelling on wetting.

berm—a shelf that breaks the continuity of a slope.

biaxial compression—compression caused by the application of normal stresses in two perpendicular directions. (ISRM)

biaxial state of stress—state of stress in which one of the three principal stresses is zero. (ISRM)

binder (soil binder)—portion of soil passing No. 40 (425-μm) U.S. standard sieve,

binder—anything that causes cohesion in loosely assembled substances, such as clay or cement.

bit—any device that may be attached to or is an integral part of a drill string and is used as a cutting tool to bore into or penetrate rock or other materials.

blaine fineness—the fineness of powdered materials, such as cement and pozzolans, expressed as surface area usually in square centimetres per gram.

blanket grouting—a method in which relatively closely spaced shallow holes are drilled and grouted on a grid pattern over an area, for the purpose of making the upper portions of the bedrock stronger and less pervious.

blastibility—index value of the resistance of a rock formation to blasting. (ISRM)

blasting cap (detonator, initiator)—a small tube containing a flashing mixture for firing explosives. (ISRM)

bleeding—*in grouting*, the autogeneous flow of mixing water within, or its emergence from, newly placed grout caused by the settlement of the solid materials within the mass.

bleeding rate—*in grouting*, the rate at which water is released from grout by bleeding.

blocking—wood blocks placed between the excavated surface of a tunnel or shaft and the main bracing system. (ISRM)

body force—a force such as gravity whose effect is distributed throughout a material body by direct action on each elementary part of the body independent of the others. (ISRM)

bog—a peat covered area with a high water table and a surface dominated by a carpet of mosses, chiefly sphagnum. It is generally nutrient poor and acidic. It may be treed or treeless.

bond strength—*in grouting*, resistance to separation of set grout from other materials with which it is in contact; a collective expression for all forces such as adhesion, friction, and longitudinal shear.

bottom charge—concentrated explosive charge at the bottom of a blast hole. (ISRM)

boulder clay—a geological term used to designate glacial drift that has not been subjected to the sorting action of water and therefore contains particles from boulders to clay sizes.

boulders—a rock fragment, usually rounded by weathering or abrasion, with an average dimension of 12 in. (305 mm) or more.

breakwater stone—(generally three tons to twenty tons in weight) stone resulting from blasting, cutting, or other means to obtain rock heavy enough to require handling individual pieces by mechanical means.

buckling—a bulge, bend, bow, kink, or wavy condition produced in sheets, plates, columns, or beams by compressive stresses.

bulb of pressure—see **pressure bulb.**

bulkhead—a steep or vertical structure supporting natural or artificial embankment.

bulking—the increase in volume of a material due to manipulation. Rock bulks upon being excavated; damp sand bulks if loosely deposited, as by dumping, because the apparent cohesion prevents movement of the soil particles to form a reduced volume.

buoyant unit weight (*submerged unit weight*)—see **unit weight.**

burden—*in an explosive blasting*, the distance between the charge and the free face of the material to be blasted.

burden—distance between charge and free surface in direction of throw. (ISRM)

"C" Horizon—see **horizon**.

California bearing ratio, *CBR* (D)—the ratio of: (*1*) the force per unit area required to penetrate a soil mass with a 3-in.^{2} (19-cm)^2 circular piston (approximately 2-in. (51-mm) diameter) at the rate of 0.05 in. (1.3 mm)/min, to (*2*) that required for corresponding penetration of a standard material. The ratio is usually determined at 0.1-in. (2.5-mm) penetration, although other penetrations are sometimes used. Original California procedures required determination of the ratio at 0.1-in. intervals to 0.5 in. (12.7 mm). Corps of Engineers' procedures require determination of the ratio at 0.1 in. and 0.2 in. (5.1 mm). Where the ratio at 0.2 in. is consistently higher than at 0.1 in., the ratio at 0.2 in. is used.

camouflet—the underground cavity created by a fully contained explosive. (ISRM)

capillary action (capillarity)—the rise or movement of water in the interstices of a soil or rock due to capillary forces.

capillary flow—see **capillary action**.

capillary fringe zone—the zone above the free water elevation in which water is held by capillary action.

capillary head, *h* (L)—the potential, expressed in head of water, that causes the water to flow by capillary action.

capillary migration—see **capillary action**.

capillary rise (height of capillary rise), h_c (L)—the height above a free water elevation to which water will rise by capillary action.

capillary water—water subject to the influence of capillary action.

catalyst—a material that causes chemical reactions to begin.

catalyst system—those materials that, in combination, cause chemical reactions to begin; catalyst systems normally consist of an initiator (catalyst) and an activator.

cation—an ion that moves, or would move toward a cathode; thus nearly always synonymous with positive ion.

cation exchange—see **base exchange**.

cavity—a natural underground opening that may be small or large.

cavity—underground opening created by a fully contained explosive. (ISRM)

cement factor—quantity of cement contained in a unit volume of concrete or grout, expressed as weight, or volume (specify which).

cement grout—a grout in which the primary cementing agent is Portland cement.

cementitious factor—quantity of cement and other cementitious materials contained in a unit volume of concrete or grout, expressed as weight or volume (specify which).

centrifuge moisture equivalent—see **moisture equivalent**.

chamber—a large room excavated underground, for example, for a powerhouse, pump station, or for storage. (ISRM)

chamber blasting (coyotehole blasting)—a method of quarry blasting in which large explosive charges are confined in small tunnel chambers inside the quarry face. (ISRM)

chemical grout—any grouting material characterized by being a true solution; no particles in suspension. See also **particulate grout**.

chemical grout system—any mixture of materials used for grouting purposes in which all elements of the system are true solutions (no particles in suspension).

chip—crushed angular rock fragment of a size smaller than a few centimetres. (ISRM)

chisel—the steel cutting tool used in percussion drilling. (ISRM)

circuit grouting—a grouting method by which grout is circulated through a pipe extending to the bottom of the hole and back up the hole via the annular space outside the pipe. Then the excess grout is diverted back over a screen to the agitator tank by means of a packing gland at the top of the hole. The method is used where holes tend to cave and sloughing material might otherwise clog openings to be grouted.

clay (clay soil)—fine-grained soil or the fine-grained portion of soil that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength when air-dry. The term has been used to designate the percentage finer than 0.002 mm (0.005 mm in some cases), but it is strongly recommended that this usage be discontinued, since there is ample evidence from an engineering standpoint that the properties described in the above definition are many times more important.

clay size—that portion of the soil finer than 0.002 mm (0.005 mm in some cases) (see also **clay**).

clay soil—see **clay**.

cleavage—*in crystallography*, the splitting, or tendency to split, along planes determined by the crystal structure. *In petrology*, a tendency to cleave or split along definite, parallel, closely spaced planes. It is a secondary structure, commonly confined to bedded rocks.

cleavage—the tendency to cleave or split along definite parallel planes, which may be highly inclined to the bedding. It is a secondary structure and is ordinarily accompanied by at least some recrystallization of the rock. (ISRM)

cleavage planes—the parallel surfaces along which a rock or mineral cleaves or separates; the planes of least cohesion, usually parallel to a certain face of the mineral or crystal.

cleft water—water that exists in or circulates along the geological discontinuities in a rock mass.

closure—the opening is reduced in dimension to the extent that it cannot be used for its intended purpose. (ISRM)

closure—*in grouting*, closure refers to achieving the desired reduction in grout take by splitting the hole spacing. If closure is being achieved, there will be a progressive decrease in grout take as primary, secondary, tertiary, and quanternary holes are grouted.

cobble (cobblestone)—a rock fragment, usually rounded or semirounded, with an average dimension between 3 and 12 in. (75 and 305 mm).

coefficient of absolute viscosity—see **coefficient of viscosity**.

coefficient of active earth pressure—see **coefficient of earth pressure**.

coefficient of compressibility (coefficient of compression), a_v (L^2F^{-1})—the secant slope, for a given pressure increment,

of the pressure-void ratio curve. Where a stress-strain curve is used, the slope of this curve is equal to $a_v/(1 + e)$.

coefficient of consolidation, c_v **(L^2T^{-1})**—a coefficient utilized in the theory of consolidation, containing the physical constants of a soil affecting its rate of volume change.

$$c_v = k(1 + e)/a_v\gamma_w$$

where:

k = coefficient of permeability, LT^{-1},
e = void ratio, D,
a_v = coefficient of compressibility, L^2F^{-1}, and
γ_w = unit weight of water, FL^{-3}.

Discussion—In the literature published prior to 1935, the coefficient of consolidation, usually designated c, was defined by the equation:

$$c = k/a_v\gamma_w(1 + e)$$

This original definition of the coefficient of consolidation may be found in some more recent papers and care should be taken to avoid confusion.

coefficient of earth pressure, K **(D)**—the principal stress ratio at a point in a soil mass.

coefficient of earth pressure, active, K_A (D)—the minimum ratio of: (*1*) the minor principal stress, to (*2*) the major principal stress. This is applicable where the soil has yielded sufficiently to develop a lower limiting value of the minor principal stress.

coefficient of earth pressure, at rest, K_O (D)—the ratio of: (*1*) the minor principal stress, to (*2*) the major principal stress. This is applicable where the soil mass is in its natural state without having been permitted to yield or without having been compressed.

coefficient of earth pressure, passive, K_P (D)—the maximum ratio of: (*1*) the major principal stress, to (*2*) the minor principal stress. This is applicable where the soil has been compressed sufficiently to develop an upper limiting value of the major principal stress.

coefficient of friction (coefficient of friction between solid bodies), f **(D)**—the ratio between the maximum value of shear stress that resists slippage between two solid bodies with respect to each other, and the normal stress across the contact surfaces. The tangent of the angle of friction is ϕs.

coefficient of friction, f—a constant proportionality factor, μ, relating normal stress and the corresponding critical shear stress at which sliding starts between two surfaces: $T = \mu \cdot \sigma$. (ISRM)

coefficient of internal friction, μ **(D)**—the tangent of the angle of internal friction (angle of shear resistance) (see internal friction).

coefficient of permeability (permeability), k **(LT^{-1})**—the rate of discharge of water under laminar flow conditions through a unit cross-sectional area of a porous medium under a unit hydraulic gradient and standard temperature conditions (usually 20°C).

coefficient of shear resistance—see coefficient of internal friction, μ (D).

coefficient of subgrade reaction (modulus of subgrade reaction), k, k_s **(FL^{-3})**—ratio of: (*1*) load per unit area of horizontal surface of a mass of soil, to (*2*) corresponding settlement of the surface. It is determined as the slope of the secant, drawn between the point corresponding to zero settlement and the point of 0.05-in. (1.3-mm) settlement, of a load-settlement curve obtained from a plate load test on a soil using a 30-in. (762-mm) or greater diameter loading plate. It is used in the design of concrete pavements by the Westergaard method.

coefficient of transmissibility—the rate of flow of water in gallons per day through a vertical strip of the aquifer 1 ft (0.3 m) wide, under a unit hydraulic gradient.

coefficient of uniformity, C_u **(D)**—the ratio D_{60}/D_{10}, where D_{60} is the particle diameter corresponding to 60 % finer on the cumulative particle-size distribution curve, and D_{10} is the particle diameter corresponding to 10 % finer on the cumulative particle-size distribution curve.

coefficient of viscosity (coefficient of absolute viscosity), η **(FTL^{-2})**—the shearing force per unit area required to maintain a unit difference in velocity between two parallel layers of a fluid a unit distance apart.

coefficient of volume compressibility (modulus of volume change), m_v **(L^2F^{-1})**—the compression of a soil layer per unit of original thickness due to a given unit increase in pressure. It is numerically equal to the coefficient of compressibility divided by one plus the original void ratio, or $a_v/(1 + e)$.

cohesion—shear resistance at zero normal stress (an equivalent term in rock mechanics is intrinsic shear strength). (ISRM)

cohesion, c **(FL^{-2})**—the portion of the shear strength of a soil indicated by the term c, in Coulomb's equation, $s = c + p \tan \phi$. See intrinsic shear strength.

apparent cohesion—cohesion in granular soils due to capillary forces.

cohesionless soil—a soil that when unconfined has little or no strength when air-dried and that has little or no cohesion when submerged.

cohesive soil—a soil that when unconfined has considerable strength when air-dried and that has significant cohesion when submerged.

collar—*in grouting,* the surface opening of a borehole.

colloidal grout—*in grouting,* a grout in which the dispersed solid particles remain in suspension (colloids).

colloidal mixer—*in grouting,* a mixer designed to produce colloidal grout.

colloidal particles—particles that are so small that the surface activity has an appreciable influence on the properties of the aggregate.

communication—*in grouting,* subsurface movement of grout from an injection hole to another hole or opening.

compaction—the densification of a soil by means of mechanical manipulation.

compaction curve (Proctor curve) (moisture-density curve)—the curve showing the relationship between the dry unit weight (density) and the water content of a soil for a given compactive effort.

compaction test (moisture-density test)—a laboratory compacting procedure whereby a soil at a known water content is placed in a specified manner into a mold of given dimensions, subjected to a compactive effort of controlled magnitude, and the resulting unit weight determined. The procedure is repeated for various water contents sufficient to establish a relation between water content and unit weight.

compressibility—property of a soil or rock pertaining to its susceptibility to decrease in volume when subjected to load.

compression curve—see pressure-void ratio curve.

compression index, C_c (D)—the slope of the linear portion of the pressure-void ratio curve on a semi-log plot.

compression wave (irrotational)—wave in which element of medium changes volume without rotation.

compressive strength (unconfined or uniaxial compressive strength), p_c, q_u, C_o (FL^{-2})—the load per unit area at which an unconfined cylindrical specimen of soil or rock will fail in a simple compression test. Commonly the failure load is the maximum that the specimen can withstand in the test.

compressive stress—normal stress tending to shorten the body in the direction in which it acts. (ISRM)

concentration factor, n (D)—a parameter used in modifying the Boussinesq equations to describe various distributions of vertical stress.

conjugate joints (faults)—two sets of joints (faults) that formed under the same stress conditions (usually shear pairs). (ISRM)

consistency—the relative ease with which a soil can be deformed.

consistency—*in grouting,* the relative mobility or ability of freshly mixed mortar or grout to flow; the usual measurements are slump for stiff mixtures and flow for more fluid grouts.

consistency index—see relative consistency.

consolidated-drained test (slow test)—a soil test in which essentially complete consolidation under the confining pressure is followed by additional axial (or shearing) stress applied in such a manner that even a fully saturated soil of low permeability can adapt itself completely (fully consolidate) to the changes in stress due to the additional axial (or shearing) stress.

consolidated-undrained test (consolidated quick test)—a soil test in which essentially complete consolidation under the vertical load (in a direct shear test) or under the confining pressure (in a triaxial test) is followed by a shear at constant water content.

consolidation—the gradual reduction in volume of a soil mass resulting from an increase in compressive stress.

initial consolidation (*initial compression*)—a comparatively sudden reduction in volume of a soil mass under an applied load due principally to expulsion and compression of gas in the soil voids preceding primary consolidation.

primary consolidation (*primary compression*) (*primary time effect*)—the reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to a squeezing out of water from the void spaces of the mass and accompanied by a transfer of the load from the soil water to the soil solids.

secondary consolidation (*secondary compression*) (*secondary time effect*)—the reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to the adjustment of the internal structure of the soil mass after most of the load has been transferred from the soil water to the soil solids.

consolidation curve—see consolidation time curve.

consolidation grouting—injection of a fluid grout, usually sand and Portland cement, into a compressible soil mass in order to displace it and form a lenticular grout structure for support.

Discussion—In rock, grouting is performed for the purpose of strengthening the rock mass by filling open fractures and thus eliminating a source of settlement.

consolidation ratio, U_z (D)—the ratio of: (*1*) the amount of consolidation at a given distance from a drainage surface and at a given time, to (*2*) the total amount of consolidation obtainable at that point under a given stress increment.

consolidation test—a test in which the specimen is laterally confined in a ring and is compressed between porous plates.

consolidation-time curve (time curve) (consolidation curve) (theoretical time curve)—a curve that shows the relation between: (*1*) the degree of consolidation, and (*2*) the elapsed time after the application of a given increment of load.

constitutive equation—force deformation function for a particular material. (ISRM)

contact grouting—see backpack grouting.

contact pressure, p (FL^{-2})—the unit of pressure that acts at the surface of contact between a structure and the underlying soil or rock mass.

continuous mixer—a mixer into which the ingredients of the mixture are fed without stopping, and from which the mixed product is discharged in a continuous stream.

contraction—linear strain associated with a decrease in length. (ISRM)

controlled blasting—includes all forms of blasting designed to preserve the integrity of the remaining rocks, that is, smooth blasting or pre-splitting. (ISRM)

controlled-strain test—a test in which the load is so applied that a controlled rate of strain results.

controlled-stress test—a test in which the stress to which a specimen is subjected is applied at a controlled rate.

convergence—generally refers to a shortening of the distance between the floor and roof of an opening, for example, in the bedded sedimentary rocks of the coal measures where the roof sags and the floor heaves. Can also apply to the convergence of the walls toward each other. (ISRM)

core—a cylindrical sample of hardened grout, concrete, rock, or grouted deposits, usually obtained by means of a core drill.

core drilling; diamond drilling—a rotary drilling technique, using diamonds in the cutting bit, that cuts out cylindrical rock samples. (ISRM)

core recovery—ratio of the length of core recovered to the length of hole drilled, usually expressed as a percentage.

cover—the perpendicular distance from any point in the roof of an underground opening to the ground surface. (ISRM)

cover—*in grouting,* the thickness of rock and soil material overlying the stage of the hole being grouted.

crack—a small fracture, that is, small with respect to the scale of the feature in which it occurs. (ISRM)

crater—excavation (generally of conical shape) generated by an explosive charge. (ISRM)

creep—slow movement of rock debris or soil usually imperceptible except to observations of long duration. Time-dependent strain or deformation, for example, continuing strain with sustained stress.

critical circle (critical surface)—the sliding surface assumed in a theoretical analysis of a soil mass for which the factor of safety is a minimum.

critical damping—the minimum viscous damping that will allow a displaced system to return to its initial position without oscillation.

critical density—the unit weight of a saturated granular material below which it will lose strength and above which it will gain strength when subjected to rapid deformation. The critical density of a given material is dependent on many factors.

critical frequency, f_c—frequency at which maximum or minimum amplitudes of excited waves occur.

critical height, H_c **(L)**—the maximum height at which a vertical or sloped bank of soil or rock will stand unsupported under a given set of conditions.

critical hydraulic gradient—see **hydraulic gradient**.

critical slope—the maximum angle with the horizontal at which a sloped bank of soil or rock of given height will stand unsupported.

critical surface—see **critical circle**.

critical void ratio—see **void ratio**.

crown—also roof or back, that is, the highest point of the cross section. *In tunnel linings*, the term is used to designate either the arched roof above spring lines or all of the lining except the floor or invert. (ISRM)

cryology—the study of the properties of snow, ice, and frozen ground.

cure—*in grouting*, the change in properties of a grout with time.

cure time—*in grouting*, the interval between combining all grout ingredients or the formation of a gel and substantial development of its potential properties.

curtain grouting—injection of grout into a sub-surface formation in such a way as to create a barrier of grouted material transverse to the direction of the anticipated water flow.

cuttings—small-sized rock fragments produced by a rock drill. (ISRM)

damping—reduction in the amplitude of vibration of a body or system due to dissipation of energy internally or by radiation. (ISRM)

damping ratio—*for a system with viscous damping*, the ratio of actual damping coefficient to the critical damping coefficient.

decay time—the interval of time required for a pulse to decay from its maximum value to some specified fraction of that value. (ISRM)

decomposition—*for peats and organic soils*, see **humification**.

decoupling—the ratio of the radius of the blasthole to the radius of the charge. In general, a reducing of the strain wave amplitude by increasing the spacing between charge and blasthole wall. (ISRM)

deflocculating agent (deflocculant) (dispersing agent)—an agent that prevents fine soil particles in suspension from coalescing to form flocs.

deformability—*in grouting*, a measure of the elasticity of the grout to distort in the interstitial spaces as the sediments move.

deformation—change in shape or size.

deformation—a change in the shape or size of a solid body. (ISRM)

deformation resolution (deformation sensitivity), R_d **(L)**—ratio of the smallest subdivision of the indicating scale of a deformation-measuring device to the sensitivity of the device.

degree-days—the difference between the average temperature each day and 32°F (0°C). In common usage degree-days are positive for daily average temperatures above 32°F and negative for those below 32°F (see **freezing index**).

degree of consolidation (percent consolidation), U **(D)**—the ratio, expressed as a percentage, of: (*1*) the amount of consolidation at a given time within a soil mass, to (*2*) the total amount of consolidation obtainable under a given stress condition.

degrees-of-freedom—the minimum number of independent coordinates required in a mechanical system to define completely the positions of all parts of the system at any instant of time. In general, it is equal to the number of independent displacements that are possible.

degree of saturation—see **percent saturation**.

degree of saturation—the extent or degree to which the voids in rock contain fluid (water, gas, or oil). Usually expressed in percent related to total void or pore space. (ISRM)

degree of sensitivity (*sensitivity ratio*)—see **remolding index**.

delay—time interval (fraction of a second) between detonation of explosive charges. (ISRM)

density—the mass per unit, ρ (ML^{-3}) kg/m^3.

density of dry soil or rock, ρ_d (ML^{-3}) kg/m^3—the mass of solid particles per the total volume of soil or rock.

density of saturated soil or rock, ρ_{sat} (ML^{-3}) kg/m^3—the total mass per total volume of completely saturated soil or rock.

density of soil or rock (*bulk density*), ρ (ML^{-3}) kg/m^3—the total mass (solids plus water) per total volume.

density of solid particles, ρ_s (ML^{-3}) kg/m^3—the mass per volume of solid particles.

density of submerged soil or rock, ρ_{sub} (ML^{-3}) kg/m^3—the difference between the density of saturated soil or rock, and the density of water.

density of water, ρ_w (ML^{-3}) kg/m^3—the mass per volume of water.

detonation—an extremely rapid and violent chemical reaction causing the production of a large volume of gas. (ISRM)

deviator stress, Δ, σ **(**FL^{-2}**)**—the difference between the major and minor principal stresses in a triaxial test.

deviator of stress (strain)—the stress (strain) tensor obtained by subtracting the mean of the normal stress (strain) components of a stress (strain) tensor from each normal stress (strain) component. (ISRM)

differential settlement—settlement that varies in rate or amount, or both, from place to place across a structure.

dilatancy—property of volume increase under loading. (ISRM)

dilatancy—the expansion of cohesionless soils when subject to shearing deformation.

direct shear test—a shear test in which soil or rock under an applied normal load is stressed to failure by moving one section of the sample or sample container (shear box) relative to the other section.

discharge velocity, v, q (LT^{-1})—rate of discharge of water through a porous medium per unit of total area perpendicular to the direction of flow.

discontinuity surface—any surface across which some property of a rock mass is discontinuous. This includes fracture surfaces, weakness planes, and bedding planes, but the term should not be restricted only to mechanical continuity. (ISRM)

dispersing agent—*in grouting*, an addition or admixture that promotes dispersion of particulate grout ingredients by reduction of interparticle attraction.

dispersing agent—see **deflocculating agent.**

dispersion—the phenomenon of varying speed of transmission of waves, depending on their frequency. (ISRM)

displacement—a change in position of a material point. (ISRM)

displacement grouting—injection of grout into a formation in such a manner as to move the formation; it may be controlled or uncontrolled. See also **penetration grouting.**

distortion—a change in shape of a solid body. (ISRM)

divergence loss—that part of transmitted energy lost due to spreading of wave rays in accordance with the geometry of the system.

double amplitude—total or peak to peak excursion.

drag bit—a noncoring or full-hole boring bit, which scrapes its way through relatively soft strata. (ISRM)

drain—a means for intercepting, conveying, and removing water.

drainage curtain—*in grouting*, a row of open holes drilled parallel to and downstream from the grout curtain of a dam for the purpose of reducing uplift pressures.

Discussion—Depth is ordinarily approximately one-third to one-half that of the grout curtain.

drainage gallery—*in grouting*, an opening or passageway from which grout holes or drainage curtain holes, or both, are drilled. See also **grout gallery.**

drawdown (L)—vertical distance the free water elevation is lowered or the pressure head is reduced due to the removal of free water.

drill—a machine or piece of equipment designed to penetrate earth or rock formations, or both.

drillability—index value of the resistance of a rock to drilling. (ISRM)

drill carriage; jumbo—a movable platform, stage, or frame that incorporates several rock drills and usually travels on the tunnel track; used for heavy drilling work in large tunnels. (ISRM)

drilling pattern—the number, position, depth, and angle of the blastholes forming the complete round in the face of a tunnel or sinking pit. (ISRM)

drill mud—*in grouting*, a dense fluid or slurry used in rotary drilling; to prevent caving of the bore hole walls, as a circulation medium to carry cuttings away from the bit and out of the hole, and to seal fractures or permeable formations, or both, preventing loss of circulation fluid.

Discussion—The most common drill mud is a water-bentonite mixture, however, many other materials may be added or substituted to increase density or decrease viscosity.

dry pack—a cement-sand mix with minimal water content used to fill small openings or repair imperfections in concrete.

dry unit weight (*dry density*)—see **unit weight.**

ductility—condition in which material can sustain permanent deformation without losing its ability to resist load. (ISRM)

dye tracer—*in grouting*, an additive whose primary purpose is to change the color of the grout or water.

earth—see **soil.**

earth pressure—the pressure or force exerted by soil on any boundary.

	Symbol	Unit
Pressure	p	FL^{-2}
Force	P	F or FL^{-1}

active earth pressure, P_A, p_A—the minimum value of earth pressure. This condition exists when a soil mass is permitted to yield sufficiently to cause its internal shearing resistance along a potential failure surface to be completely mobilized.

earth pressure at rest, P_o, p_o—the value of the earth pressure when the soil mass is in its natural state without having been permitted to yield or without having been compressed.

passive earth pressure, P_p, p_p—the maximum value of earth pressure. This condition exists when a soil mass is compressed sufficiently to cause its internal shearing resistance along a potential failure surface to be completely mobilized.

effect diameter (effective size), D_{10}, D_e (L)—particle diameter corresponding to 10 % finer on the grain-size curve.

effective drainage porosity—see **effective porosity.**

effective force, $\bar{F}$ (F)—the force transmitted through a soil or rock mass by intergranular pressures.

effective porosity (effective drainage porosity), n_e (D)—the ratio of: (*1*) the volume of the voids of a soil or rock mass that can be drained by gravity, to (*2*) the total volume of the mass.

effective pressure—see **stress.**

effective size—see **effective diameter.**

effective stress—see **stress.**

effective unit weight—see **unit weight.**

efflux time—time required for all grout to flow from a flow cone.

elasticity—property of material that returns to its original form or condition after the applied force is removed. (ISRM)

elastic limit—point on stress strain curve at which transition from elastic to inelastic behavior takes place. (ISRM)

elastic state of equilibrium—state of stress within a soil mass when the internal resistance of the mass is not fully mobilized.

elastic strain energy—potential energy stored in a strained solid and equal to the work done in deforming the solid from its unstrained state less any energy dissipated by inelastic deformation. (ISRM)

electric log—a record or log of a borehole obtained by lowering electrodes into the hole and measuring any of the various electrical properties of the rock formations or materials traversed.

electrokinetics—involves the application of an electric field to soil for the purpose of dewatering materials of very low permeability to enhance stability. The electric field produces negative pore pressures near a grout pipe that facilitates grout injection.

emulsifier—a substance that modifies the surface tension of colloidal droplets, keeping them from coalescing, and keeping them suspended.

emulsion—a system containing dispersed colloidal droplets.

endothermic—pertaining to a reaction that occurs with the adsorption of heat.

envelope grouting—grouting of rock surrounding a hydraulic pressure tunnel for purpose of consolidation, and primarily, reduction of permeability.

epoxy—a multicomponent resin grout that usually provides very high, tensile, compressive, and bond strengths.

equipotential line—see **piezometric line.**

equivalent diameter (equivalent size), D (L)—the diameter of a hypothetical sphere composed of material having the same specific gravity as that of the actual soil particle and of such size that it will settle in a given liquid at the same terminal velocity as the actual soil particle.

equivalent fluid—a hypothetical fluid having a unit weight such that it will produce a pressure against a lateral support presumed to be equivalent to that produced by the actual soil. This simplified approach is valid only when deformation conditions are such that the pressure increases linearly with depth and the wall friction is neglected.

excess hydrostatic pressure—see **hydrostatic pressure.**

exchange capacity—the capacity to exchange ions as measured by the quantity of exchangeable ions in a soil or rock.

excitation (stimulus)—an external force (or other input) applied to a system that causes the system to respond in some way.

exothermic—pertaining to a reaction that occurs with the evolution of heat.

expansive cement—a cement that tends to increase in volume after it is mixed with water.

extender—an additive whose primary purpose is to increase total grout volume.

extension—linear strain associated with an increase in length. (ISRM)

external force—a force that acts across external surface elements of a material body. (ISRM)

extrados—the exterior curved surface of an arch, as opposed to intrados, which is the interior curved surface of an arch. (ISRM)

fabric—*for rock or soil,* the spatial configuration of all textural and structural features as manifested by every recognizable material unit from crystal lattices to large scale features requiring field studies.

fabric—the orientation in space of the elements composing the rock substance. (ISRM)

face (heading)—the advanced end of a tunnel, drift, or excavation at which work is progressing. (ISRM)

facing—the outer layer of revetment.

failure (in rocks)—exceeding the maximum strength of the rock or exceeding the stress or strain requirement of a specific design. (ISRM)

failure by rupture—see **shear failure.**

failure criterion—specification of the mechanical condition under which solid materials fail by fracturing or by deforming beyond some specified limit. This specification may be in terms of the stresses, strains, rate-of-change of stresses, rate-of-change of strains, or some combination of these quantities, in the materials.

failure criterion—theoretically or empirically derived stress or strain relationship characterizing the occurrence of failure in the rock. (ISRM)

false set—*in grouting,* the rapid development of rigidity in a freshly mixed grout without the evolution of much heat.

Discussion—Such rigidity can be dispelled and plasticity regained by further mixing without the addition of water; premature stiffening, hesitation set, early stiffening, and rubber set are other much used terms referring to the same phenomenon.

fatigue—the process of progressive localized permanent structural change occurring in a material subjected to conditions that produce fluctuating stresses and strains at some point or points and that may culminate in cracks or complete fracture after a sufficient number of fluctuations.

fatigue—decrease of strength by repetitive loading. (ISRM)

fatigue limit—point on stress-strain curve below which no fatigue can be obtained regardless of number of loading cycles. (ISRM)

fault—a fracture or fracture zone along which there has been displacement of the two sides relative to one another parallel to the fracture (this displacement may be a few centimetres or many kilometres). (See also **joint fault set** and **joint fault system.**) (ISRM)

fault breccia—the assemblage of broken rock fragments frequently found along faults. The fragments may vary in size from inches to feet. (ISRM)

fault gouge—a clay-like material occurring between the walls of a fault as a result of the movement along the fault surfaces. (ISRM)

fiber—*for peats and organic soils,* a fragment or piece of plant tissue that retains a recognizable cellular structure and is large enough to be retained after wet sieving on a 100-mesh sieve (openings 0.15 mm).

fibric peat—peat in which the original plant fibers are slightly decomposed (greater than 67 % fibers).

fibrous peat—see **fibric peat.**

field moisture equivalent—see **moisture equivalent.**

fill—man-made deposits of natural soils or rock products and waste materials.

filling—generally, the material occupying the space between joint surfaces, faults, and other rock discontinuities. The filling material may be clay, gouge, various natural cementing agents, or alteration products of the adjacent rock. (ISRM)

filter bedding stone—(generally 6-in. minus material) stone placed under graded riprap stone or armor stone in a layer or combination of layers designed and installed in such a manner as to prevent the loss of underlying soil or finer bedding materials due to moving water.

filter (protective filter)—a layer or combination of layers of pervious materials designed and installed in such a manner as to provide drainage, yet prevent the movement of soil particles due to flowing water.

final set—*in grouting*, a degree of stiffening of a grout mixture greater than initial set, generally stated as an empirical value indicating the time in hours and minutes that is required for cement paste to stiffen sufficiently to resist the penetration of a weighted test needle.

fineness—a measure of particle-siz-

fineness modulus—an empirical factor obtained by adding the total percentages of an aggregate sample retained on each of a specified series of sieves, and dividing the sum by 100; in the United States, the U.S. Standard sieve sizes are: No. 100 (149 μm), No. 50 (297 μm), No. 30 (590 μm), No. 16 (1,190 μm), No. 8 (2,380 μm), and No. 4 (4,760 μm) and ⅜ in. (9.5 mm), ¾ in. (19 mm), 1½ in. (38 mm), 3 in. (76 mm), and 6 in. (150 mm).

fines—portion of a soil finer than a No. 200 (75-μm) U.S. standard sieve.

finite element—one of the regular geometrical shapes into which a figure is subdivided for the purpose of numerical stress analysis. (ISRM)

fishing tool—*in grouting*, a device used to retrieve drilling equipment lost or dropped in the hole.

fissure—a gapped fracture. (ISRM)

flash set—*in grouting*, the rapid development of rigidity in a freshly mixed grout, usually with the evolution of considerable heat; this rigidity cannot be dispelled nor can the plasticity be regained by further mixing without addition of water; also referred to as quick set or grab set.

floc—loose, open-structured mass formed in a suspension by the aggregation of minute particles.

flocculation—the process of forming flocs.

flocculent structure—see soil structure.

floor—bottom of near horizontal surface of an excavation, approximately parallel and opposite to the roof. (ISRM)

flow channel—the portion of a flow net bounded by two adjacent flow lines.

flow cone—*in grouting*, a device for measurement of grout consistency in which a predetermined volume of grout is permitted to escape through a precisely sized orifice, the time of efflux (flow factor) being used as the indication of consistency.

flow curve—the locus of points obtained from a standard liquid limit test and plotted on a graph representing water content as ordinate on an arithmetic scale and the number of blows as abscissa on a logarithmic scale.

flow failure—failure in which a soil mass moves over relatively long distances in a fluid-like manner.

flow index, F_w, I_f (D)—the slope of the flow curve obtained from a liquid limit test, expressed as the difference in water contents at 10 blows and at 100 blows.

flow line—the path that a particle of water follows in its course of seepage under laminar flow conditions.

flow net—a graphical representation of flow lines and equipotential (piezometric) lines used in the study of seepage phenomena.

flow slide—the failure of a sloped bank of soil in which the movement of the soil mass does not take place along a well-defined surface of sliding.

flow value, N_ϕ (degrees)—a quantity equal to tan [45 deg + (ϕ/2)].

fluidifier—*in grouting*, an admixture employed in grout to increase flowability without changing water content.

fly ash—the finely divided residue resulting from the combustion of ground or powdered coal and which is transported from the firebox through the boiler by flue gases.

fold—a bend in the strata or other planar structure within the rock mass. (ISRM)

foliation—the somewhat laminated structure resulting from segregation of different minerals into layers parallel to the schistosity. (ISRM)

footing—portion of the foundation of a structure that transmits loads directly to the soil.

footwall—the mass of rock beneath a discontinuity surface. (ISRM)

forced vibration (forced oscillation)—vibration that occurs if the response is imposed by the excitation. If the excitation is periodic and continuing, the oscillation is steady-state.

forepoling—driving forepoles (pointed boards or steel rods) ahead of the excavation, usually over the last set erected, to furnish temporary overhead protection while installing the next set. (ISRM)

foundation—lower part of a structure that transmits the load to the soil or rock.

foundation soil—upper part of the earth mass carrying the load of the structure.

fracture—the general term for any mechanical discontinuity in the rock: it therefore is the collective term for joints, faults, cracks, etc. (ISRM)

fracture—a break in the mechanical continuity of a body of rock caused by stress exceeding the strength of the rock. Includes joints and faults.

fracture frequency—the number of natural discontinuities in a rock or soil mass per unit length, measured along a core or as exposed in a planar section such as the wall of a tunnel.

fracture pattern—spatial arrangement of a group of fracture surfaces. (ISRM)

fracturing—*in grouting*, intrusion of grout fingers, sheets, and lenses along joints, planes of weakness, or between the strata of a formation at sufficient pressure to cause the strata to move away from the grout.

fragmentation—the breaking of rock in such a way that the bulk of the material is of a convenient size for handling. (ISRM)

free water (gravitational water) (ground water) (phreatic water)—water that is free to move through a soil or rock mass under the influence of gravity.

free water elevation (water table) (ground water surface) (free water surface) (ground water elevation)—elevations at which the pressure in the water is zero with respect to the atmospheric pressure.

freezing index, F (degree-days)—the number of degree-days between the highest and lowest points on the cumulative degree-days—time curve for one freezing season. It is used as a measure of the combined duration and magnitude of below-freezing temperature occurring during any given freezing season. The index determined for air temperatures at 4.5 ft (1.4 m) above the ground is commonly

designated as the air freezing index, while that determined for temperatures immediately below a surface is known as the surface freezing index.

free vibration—vibration that occurs in the absence of forced vibration.

frequency, f (T^{-1})—number of cycles occurring in unit time.

frost action—freezing and thawing of moisture in materials and the resultant effects on these materials and on structures of which they are a part or with which they are in contact.

frost boil—(*a*) softening of soil occurring during a thawing period due to the liberation of water form ice lenses or layers.

(*b*) the hole formed in flexible pavements by the extrusion of soft soil and melt waters under the action of wheel loads.

(*c*) breaking of a highway or airfield pavement under traffic and the ejection of subgrade soil in a soft and soupy condition caused by the melting of ice lenses formed by frost action.

frost heave—the raising of a surface due to the accumulation of ice in the underlying soil or rock.

fundamental frequency—lowest frequency of periodic variation.

gage length, L (L)—distance over which the deformation measurement is made.

gage protector—*in grouting,* a device used to transfer grout pressure to a gage without the grout coming in actual contact with the gage.

gage saver—see **gage protector.**

gel—*in grouting,* the condition where a liquid grout begins to exhibit measurable shear strength.

gel time—*in grouting,* the measured time interval between the mixing of a grout system and the formation of a gel.

general shear failure—see **shear failure.**

glacial till (till)—material deposited by glaciation, usually composed of a wide range of particle sizes, which has not been subjected to the sorting action of water.

gradation (grain-size distribution) (texture)—the proportions by mass of a soil or fragmented rock distributed in specified particle-size ranges.

grain-size analysis (mechanical analysis) (particle-size analysis)—the process of determining grain-size distribution.

gravel—rounded or semirounded particles of rock that will pass a 3-in. (76.2-mm) and be retained on a No. 4 (4.75-μm) U.S. standard sieve.

gravitational water—see **free water.**

gravity grouting—grouting under no applied pressure other than the height of fluid in the hole.

groin—bank or shore-protection structure in the form of a barrier placed oblique to the primary motion of water, designed to control movement of bed load.

ground arch—the theoretical stable rock arch that develops some distance back from the surface of the opening and supports the opening. (ISRM)

ground water—that part of the subsurface water that is in the saturated zone.

Discussion—Loosely, all subsurface water as distinct from surface water.

ground-water barrier—soil, rock, or artificial material which has a relatively low permeability and which occurs below the land surface where it impedes the movement of ground water and consequently causes a pronounced difference in the potentiometric level on opposite sides of the barrier.

ground-water basin—a ground-water system that has defined boundaries and may include more than one aquifer of permeable materials, which are capable of furnishing a significant water supply.

Discussion—A basin is normally considered to include the surface area and the permeable materials beneath it. The surface-water divide need not coincide with ground-water divide.

ground-water discharge—the water released from the zone of saturation; also the volume of water released.

ground-water divide—a ridge in the water table or other potentiometric surface from which ground water moves away in both directions normal to the ridge line.

ground-water elevation—see **free water elevation.**

ground-water flow—the movement of water in the zone of saturation.

ground-water level—the level below which the rock and subsoil, to unknown depths, are saturated. (ISRM)

ground-water, perched—see **perched ground-water.**

ground-water recharge—the process of water addition to the saturated zone; also the volume of water added by this process.

ground-water surface—see **free water elevation.**

grout—*in soil and rock grouting,* a material injected into a soil or rock formation to change the physical characteristics of the formation.

groutability—the ability of a formation to accept grout.

groutability ratio of granular formations—the ratio of the 15 % size of the formation particles to be grouted to the 85 % size of grout particles (suspension-type grout). This ratio should be greater than 24 if the grout is to successfully penetrate the formation.

groutable rock bolts—rock bolts with hollow cores or with tubes adapted to the periphery of the bolts and extending to the bottom of the bolts to facilitate filling the holes surrounding the bolts with grout.

grouted-aggregate concrete—concrete that is formed by injecting grout into previously placed coarse aggregate. See also **preplaced aggregate concrete.**

grout cap—a "cap" that is formed by placing concrete along the top of a grout curtain. A grout cap is often used in weak foundation rock to secure grout nipples, control leakage, and to form an impermeable barrier at the top of a grout curtain.

grout gallery—an opening or passageway within a dam utilized for grouting or drainage operations, or both.

grout header—a pipe assembly attached to a ground hole, and to which the grout lines are attached for injecting grout. Grout injector is monitored and controlled by means of valves and a pressure gate mounted on the header; sometimes called grout manifold.

grout mix—the proportions or amounts of the various materials used in the grout, expressed by weight or volume. (The words "by volume" or "by weight" should be used to specify the mix.)

grout nipple—*in grouting,* a short length of pipe, installed at the collar of the grout hole, through which drilling is done and to which the grout header is attached for the purpose of injecting grout.

grout slope—the natural slope of grout injected into preplaced-aggregate or other porous mass.

grout system—formulation of different materials used to form a grout.

grout take—the measured quantity of grout injected into a unit volume of formation, or a unit length of grout hole.

hanging wall—the mass of rock above a discontinuity surface. (ISRM)

hardener—*in grouting*, in a two component epoxy or resin, the chemical component that causes the base component to cure.

hardness—resistance of a material to indentation or scratching. (ISRM)

hardpan—a hard impervious layer, composed chiefly of clay, cemented by relatively insoluble materials, that does not become plastic when mixed with water and definitely limits the downward movement of water and roots.

head—pressure at a point in a liquid, expressed in terms of the vertical distance of the point below the surface of the liquid. (ISRM)

heat of hydration—heat evolved by chemical reactions with water, such as that evolved during the setting and hardening of Portland cement.

heave—upward movement of soil caused by expansion or displacement resulting from phenomena such as: moisture absorption, removal of overburden, driving of piles, frost action, and loading of an adjacent area.

height of capillary rise—see **capillary rise**.

hemic peat—peat in which the original plant fibers are moderately decomposed (between 33 and 67 % fibers).

heterogeneity—having different properties at different points. (ISRM)

homogeneity—having different properties at different points. (ISRM)

homogeneity—having the same properties at all points. (ISRM)

homogeneous mass—a mass that exhibits essentially the same physical properties at every point throughout the mass.

honeycomb structure—see **soil structure**.

horizon (soil horizon)—one of the layers of the soil profile, distinguished principally by its texture, color, structure, and chemical content.

"A" horizon—the uppermost layer of a soil profile from which inorganic colloids and other soluble materials have been leached. Usually contains remnants of organic life.

"B" horizon—the layer of a soil profile in which material leached from the overlying "A" horizon is accumulated.

"C" horizon—undisturbed parent material from which the overlying soil profile has been developed.

humic peat—see **sapric peat**.

humification—a process by which organic matter decomposes.

Discussion—The degree of humification for peats is indicated by the state of the fibers. In slightly decomposed material, most of the volume consists of fibers. In moderately decomposed material, the fibers may be preserved but may break down with disturbance, such as rubbing between the fingers. In highly decomposed materials, fibers will be virtually absent; see von Post humification scale.

humus—a brown or black material formed by the partial decomposition of vegetable or animal matter; the organic portion of soil.

hydration—formation of a compound by the combining of water with some other substance.

hydraulic conductivity—see **coefficient of permeability**.

hydraulic fracturing—the fracturing of an underground strata by pumping water or grout under a pressure in excess of the tensile strength and confining pressure; also called hydrofracturing.

hydraulic gradient, i, s (D)—the loss of hydraulic head per unit distance of flow, dh/dL.

critical hydraulic gradient, i_c (D)—hydraulic gradient at which the intergranular pressure in a mass of cohesionless soil is reduced to zero by the upward flow of water.

hydrostatic head—the fluid pressure of formation water produced by the height of water above a given point.

hydrostatic pressure, u_o (FL^{-2})—a state of stress in which all the principal stresses are equal (and there is no shear stress), as in a liquid at rest; the product of the unit weight of the liquid and the different in elevation between the given point and the free water elevation.

excess hydrostatic pressure (*hydrostatic excess pressure*), $\bar{u}$, u (FL^{-2})—the pressure that exists in pore water in excess of the hydrostatic pressure.

hydrostatic pressure—a state of stress in which all the principal stresses are equal (and there is no shear stress). (ISRM)

hygroscopic capacity (hygroscopic coefficient), w_e (D)—ratio of: (*1*) the weight of water absorbed by a dry soil or rock in a saturated atmosphere at a given temperature, to (*2*) the weight of the oven-dried soil or rock.

hygroscopic water content, w_H (D)—the water content of an air-dried soil or rock.

hysteresis—incomplete recovery of strain during unloading cycle due to energy consumption. (ISRM)

impedance, acoustic—the product of the density and sonic velocity of a material. The extent of wave energy transmission and reflection at the boundary of two media is determined by their acoustic impedances. (ISRM)

inelastic deformation—the portion of deformation under stress that is not annulled by removal of stress. (ISRM)

inert—not participating in any fashion in chemical reactions.

influence value, I (D)—the value of the portion of a mathematical expression that contains combinations of the independent variables arranged in dimensionless form.

inhibitor—a material that stops or slows a chemical reaction from occurring.

initial consolidation (*initial compression*)—see **consolidation**.

initial set—a degree of stiffening of a grout mixture generally stated as an empirical value indicating the time in hours and minutes that is required for a mixture to stiffen sufficiently to resist the penetration of a weighted test needle.

injectability—see **groutability**.

inorganic silt—see **silt**.

in situ—applied to a rock or soil when occurring in the situation in which it is naturally formed or deposited.

intergranular pressure—see **stress**.

intermediate principal plane—see **principal plane**.

intermediate principal stress—see **stress**.

internal friction (shear resistance), (FL^{-2})—the portion of the shearing strength of a soil or rock indicated by the terms $p \tan \phi$ in Coulomb's equation $s = c + p \tan \phi$. It is usually considered to be due to the interlocking of the soil or rock grains and the resistance to sliding between the grains.

interstitial—occurring between the grains or in the pores in rock or soil.

intrinsic shear strength, S_o (FL^{-2})—the shear strength of a rock indicated by Coulomb's equation when $p \tan \phi$ (shear resistance or internal friction) vanishes. Corresponds to cohesion, c, in soil mechanics.

invert—on the cross section, the lowest point of the underground excavation or the lowest section of the lining. (ISRM)

isochrome—a curve showing the distribution of the excess hydrostatic pressure at a given time during a process of consolidation.

isotropic mass—a mass having the same property (or properties) in all directions.

isotropic material—a material whose properties do not vary with direction.

isotropy—having the same properties in all directions. (ISRM)

jackhammer—an air driven percussion drill that imparts a rotary hammering motion to the bit and has a passageway to the bit for the injection of compressed air for cleaning the hole of cuttings.

Discussion—These two characteristics distinguish it from the pavement breaker which is similar in size and general appearance.

jack-leg—a portable percussion drill of the jack-hammer type, used in underground work; has a single pneumatically adjustable leg for support.

jet grouting—technique utilizing a special drill bit with horizontal and vertical high speed water jets to excavate alluvial soils and produce hard impervious columns by pumping grout through the horizontal nozzles that jets and mixes with foundation material as the drill bit is withdrawn.

jetty—an elongated artificial obstruction projecting into a body of water form a bank or shore to control shoaling and scour by deflection of the force of water currents and waves.

joint—a break of geological origin in the continuity of a body of rock occurring either singly, or more frequently in a set or system, but not attended by a visible movement parallel to the surface of discontinuity. (ISRM)

joint diagram—a diagram constructed by accurately plotting the strike and dip of joints to illustrate the geometrical relationship of the joints within a specified area of geologic investigation. (ISRM)

joint pattern—a group of joints that form a characteristic geometrical relationship, and which can vary considerably from one location to another within the same geologic formation. (ISRM)

joint (fault) set—a group of more or less parallel joints. (ISRM)

joint (fault) system—a system consisting of two or more joint sets or any group of joints with a characteristic pattern, that is, radiating, concentric, etc. (ISRM)

jumbo—a specially built mobile carrier used to provide a work platform for one or more tunneling operations, such as drilling and loading blast holes, setting tunnel supports, installing rock bolts, grouting, etc.

kaolin—a variety of clay containing a high percentage of kaolinite.

kaolinite—a common clay mineral having the general formula $Al_2(Si_2O_5)$ (OH_4); the primary constituent of kaolin.

karst—a geologic setting where cavities are developed in massive limestone beds by solution of flowing water. Caves and even underground river channels are produced into which surface runoff drains and often results in the land above being dry and relatively barren. (ISRM)

kelly—a heavy-wall tube or pipe, usually square or hexagonal in cross section, which works inside the matching center hole in the rotary table of a drill rig to impart rotary motion to the drill string.

lagging, *n—in mining or tunneling,* short lengths of timber, sheet steel, or concrete slabs used to secure the roof and sides of an opening behind the main timber or steel supports. The process of installation is also called lagging or lacing.

laminar flow (streamline flow) (viscous flow)—flow in which the head loss is proportional to the first power of the velocity.

landslide—the perceptible downward sliding or movement of a mass of earth or rock, or a mixture of both. (ISRM)

landslide (slide)—the failure of a sloped bank of soil or rock in which the movement of the mass takes place along a surface of sliding.

leaching—the removal in solution of the more soluble materials by percolating or moving waters. (ISRM)

leaching—the removal of soluble soil material and colloids by percolating water.

lime—specifically, calcium oxide (CaO_2); also loosely, a general term for the various chemical and physical forms of quicklime, hydrated lime, and hydraulic hydrated lime.

ledge—see bedrock.

linear (normal) strain—the change in length per unit of length in a given direction. (ISRM)

line of creep (path of percolation)—the path that water follows along the surface of contact between the foundation soil and the base of a dam or other structure.

line of seepage (seepage line) (phreatic line)—the upper free water surface of the zone of seepage.

linear expansion, L_e (D)—the increase in one dimension of a soil mass, expressed as a percentage of that dimension at the shrinkage limit, when the water content is increased from the shrinkage limit to any given water content.

linear shrinkage, L_s (D)—decrease in one dimension of a soil mass, expressed as a percentage of the original dimension, when the water content is reduced from a given value to the shrinkage limit.

lineation—the parallel orientation of structural features that are lines rather than planes; some examples are parallel orientation of the long dimensions of minerals; long axes of pebbles; striae on slickensides; and cleavage-bedding plane intersections. (ISRM)

liquefaction—the process of transforming any soil from a solid state to a liquid state, usually as a result of increased pore pressure and reduced shearing resistance.

liquefaction potential—the capability of a soil to liquefy or develop cyclic mobility.

liquefaction (spontaneous liquefaction)—the sudden large decrease of the shearing resistance of a cohesionless soil. It is caused by a collapse of the structure by shock or other type of strain and is associated with a sudden but temporary increase of the prefluid pressure. It involves a temporary transformation of the material into a fluid mass.

liquid, limit, LL, L_w, w_L (D)—(*a*) the water content corresponding to the arbitrary limit between the liquid and plastic states of consistency of a soil.

(*b*) the water content at which a pat of soil, cut by a groove of standard dimensions, will flow together for a distance of ½ in. (12.7 mm) under the impact of 25 blows in a standard liquid limit apparatus.

liquidity index (water-plasticity ratio) (relative water content), B, R_w, I_L (D)—the ratio, expressed as a percentage, of: (*1*) the natural water content of a soil minus its plastic limit, to (*2*) its plasticity index.

liquid-volume measurement—*in grouting,* measurement of grout on the basis of the total volume of solid and liquid constituents.

lithology—the description of rocks, especially sedimentary clastics and especially in hand specimens and in outcrops, on the basis of such characteristics as color, structures, mineralogy, and particle size.

loam—a mixture of sand, silt, or clay, or a combination of any of these, with organic matter (see **humus**).

Discussion—It is sometimes called topsoil in contrast to the subsoils that contain little or no organic matter.

local shear failure—see **shear failure.**

loess—a uniform aeolian deposit of silty material having an open structure and relatively high cohesion due to cementation of clay or calcareous material at grain contacts.

Discussion—A characteristic of loess deposits is that they can stand with nearly vertical slopes.

logarithmic decrement—the natural logarithm of the ratio of any two successive amplitudes of like sign, in the decay of a single-frequency oscillation.

longitudinal rod wave—see **compression wave.**

longitudinal wave, v_l (LT^{-1})—wave in which direction of displacement at each point of medium is normal to wave front, with propagation velocity, calculated as follows:

$$v_l = \sqrt{(E/\rho)[(1 - \nu)/(1 + \nu)(1 - 2\nu)]} = \sqrt{(\lambda + 2\mu)/\rho}$$

where:

E = Young's modulus,
ρ = mass density,
λ and μ = Lamé's constants, and
ν = Poisson's ratio.

long wave (quer wave), W (LT^{-1})—dispersive surface wave with one horizontal component, generally normal to the direction of propagation, which decreases in propagation velocity with increase in frequency.

lubricity—*in grouting,* the physico-chemical characteristic of a grout material flow through a soil or rock that is the inverse of the inherent friction of that material to the soil or rock; comparable to "wetness."

lugeon—a measure of permeability defined by a pump-in test or pressure test, where one Lugeon unit is a water take of 1 L/min per metre of hole at a pressure of 10 bars.

major principal plane—see **principal plane.**

major principal stress—see **stress.**

manifold—see **grout header.**

marl—calcareous clay, usually containing from 35 to 65 % calcium carbonate ($CaCO_3$).

marsh—a wetland characterized by grassy surface mats which are frequently interspersed with open water or by a closed canopy of grasses, sedges, or other herbacious plants.

mass unit weight—see **unit weight.**

mathematical model—the representation of a physical system by mathematical expressions from which the behavior of the system can be deduced with known accuracy. (ISRM)

matrix—*in grouting,* a material in which particles are embedded, that is, the cement paste in which the fine aggregate particles of a grout are embedded.

maximum amplitude (L, LT^{-1}, LT^{-2})—deviation from mean or zero point.

maximum density (*maximum unit weight*)—see **unit weight.**

mechanical analysis—see **grain-size analysis.**

mesic peat—see **hemic peat.**

metering pump—a mechanical arrangement that permits pumping of the various components of a grout system in any desired proportions or in fixed proportions. (*Syn.* proportioning pump, variable proportion pump.)

microseism—seismic pulses of short duration and low amplitude, often occurring previous to failure of a material or structure. (ISRM)

minor principal plane—see **principal plane.**

minor principal stress—see **stress.**

mixed-in-place pile—a soil-cement pile, formed in place by forcing a grout mixture through a hollow shaft into the ground where it is mixed with the in-place soil with an auger-like head attached to the hollow shaft.

mixer—a machine employed for blending the constituents of grout, mortar, or other mixtures.

mixing cycle—the time taken for the loading, mixing, and unloading cycle.

mixing speed—the rotation rate of a mixer drum or of the paddles in an open-top, pan, or trough mixer, when mixing a batch; expressed in revolutions per minute.

modifier—*in grouting,* an additive used to change the normal chemical reaction or final physical properties of a grout system.

modulus of deformation—see **modulus of elasticity.**

modulus of elasticity (modulus of deformation), E, M (FL^{-2})—the ratio of stress to strain for a material under given loading conditions; numerically equal to the slope of the tangent or the secant of a stress-strain curve. The use of the term **modulus of elasticity** is recommended for materials that deform in accordance with Hooke's law; the term **modulus of deformation** for materials that deform otherwise.

modulus of subgrade reaction—see **coefficient of subgrade reaction.**

modulus of volume change—see **coefficient of volume compressibility.**

Mohr circle—a graphical representation of the stresses acting on the various planes at a given point.

Mohr circle of stress (strain)—a graphical representation of the components of stress (strain) acting across the various planes at a given point, drawn with reference to axes of normal stress (strain) and shear stress (strain). (ISRM)

Mohr envelope—the envelope of a sequence of Mohr circles representing stress conditions at failure for a given material. (ISRM)

Mohr envelope (rupture envelope) (rupture line)—the envelope of a series of Mohr circles representing stress conditions at failure for a given material.

Discussion—According to Mohr's rupture hypothesis, a rupture envelope is the locus of points the coordinates of which represent the combinations of normal and shearing stresses that will cause a given material to fail.

moisture content—see **water content.**

moisture-density curve—see **compaction curve.**

moisture-density test—see **compaction test.**

moisture equivalent:

centrifuge moisture equivalent, W_e, CME (D)—the water content of a soil after it has been saturated with water and then subjected for 1 h to a force equal to 1000 times that of gravity.

field moisture equivalent, FME—the minimum water content expressed as a percentage of the weight of the oven-dried soil, at which a drop of water placed on a smoothed surface of the soil will not immediately be absorbed by the soil but will spread out over the surface and give it a shiny appearance.

montmorillonite—a group of clay minerals characterized by a weakly bonded sheet-like internal molecular structure; consisting of extremely finely divided hydrous aluminum or magnesium silicates that swell on wetting, shrink on drying, and are subject to ion exchange.

muck—stone, dirt, debris, or useless material; or an organic soil of very soft consistency.

mud—a mixture of soil and water in a fluid or weakly solid state.

mudjacking—see **slab jacking.**

multibench blasting—the blasting of several benches (steps) in quarries and open pits, either simultaneously or with small delays. (ISRM)

multiple-row blasting—the drilling, charging, and firing of several rows of vertical holes along a quarry or opencast face. (ISRM)

muskeg—level, practically treeless areas supporting dense growth consisting primarily of grasses. The surface of the soil is covered with a layer of partially decayed grass and grass roots which is usually wet and soft when not frozen.

mylonite—a microscopic breccia with flow structure formed in fault zones. (ISRM)

natural frequency—the frequency at which a body or system vibrates when unconstrained by external forces. (ISRM)

natural frequency (displacement resonance), f_n—frequency for which phase angle is 90° between the direction of the excited force (or torque) vector and the direction of the excited excursion vector.

neat cement grout—a mixture of hydraulic cement and water without any added aggregate or filler materials.

Discussion—This may or may not contain admixture.

neutral stress—see **stress.**

newtonian fluid—a true fluid that tends to exhibit constant viscosity at all rates of shear.

node—point, line, or surface of standing wave system at which the amplitude is zero.

normal force—a force directed normal to the surface element across which it acts. (ISRM)

normal stress—see **stress.**

normally consolidated soil deposit—a soil deposit that has never been subjected to an effective pressure greater than the existing overburden pressure.

no-slump grout—grout with a slump of 1 in. (25 mm) or less according to the standard slump test (Test Method C 143).[2] See also **slump** and **slump test.**

open cut—an excavation through rock or soil made through a hill or other topographic feature to facilitate the passage of a highway, railroad, or waterway along an alignment that varies in topographic relief. An open cut can be comprised of single slopes or multiple slopes, or multiple slopes and horizontal benches, or both. (ISRM)

optimum moisture content (optimum water content), *OMC*, w_o (D)—the water content at which a soil can be compacted to a maximum dry unit weight by a given compactive effort.

organic clay—a clay with a high organic content.

organic silt—a silt with a high organic content.

organic soil—soil with a high organic content.

Discussion—In general, organic soils are very compressible and have poor load-sustaining properties.

organic terrain—see **peatland.**

oscillation—the variation, usually with time, of the magnitude of a quantity with respect to a specified reference when the magnitude is alternately greater and smaller than the reference.

outcrop—the exposure of the bedrock at the surface of the ground. (ISRM)

overbreak—the quantity of rock that is excavated or breaks out beyond the perimeter specified as the finished excavated tunnel outline. (ISRM)

overburden—the loose soil, sand, silt, or clay that overlies bedrock. In some usages it refers to all material overlying the point of interest (tunnel crown), that is, the total cover of soil and rock overlying an underground excavation. (ISRM)

overburden load—the load on a horizontal surface underground due to the column of material located vertically above it. (ISRM)

overconsolidated soil deposit—a soil deposit that has been subjected to an effective pressure greater than the present overburden pressure.

overconsolidation ratio, OCR—the ratio of preconsolidation vertical stress to the current effective overburden stress.

packer—*in grouting*, a device inserted into a hole in which grout or water is to be injected which acts to prevent return of the grout or water around the injection pipe; usually an expandable device actuated mechanically, hydraulically, or pneumatically.

---

[2] *Annual Book of ASTM Standards*, Vol 04.02.

paddle mixer—a mixer consisting essentially of a trough within which mixing paddles revolve about the horizontal axis, or a pan within which mixing blades revolve about the vertical axis.

pan mixer—a mixer comprised of a horizontal pan or drum in which mixing is accomplished by means of the rotating pan of fixed or rotating paddles, or both; rotation is about a vertical axis.

parent material—material from which a soil has been derived.

particle-size analysis—see **grain-size analysis.**

particle-size distribution—see **gradation, grain-size distribution.**

particulate grout—any grouting material characterized by undissolved (insoluble) particles in the mix. See also **chemical grout.**

passive earth pressure—see **earth pressure.**

passive state of plastic equilibrium—see **plastic equilibrium.**

path percolation (line of creep)—the path that water follows along the surface of contact between the foundation soil or rock and the base of a dam or other structure.

pavement pumping—ejection of soil and water mixtures from joints, cracks, and edges of rigid pavements, under the action of traffic.

peak shear strength—maximum shear strength along a failure surface. (ISRM)

peat—a naturally occurring highly organic substance derived primarily from plant materials.

Discussion—Peat is distinguished from other organic soil materials by its lower ash content (less than 25 % ash by dry weight) and from other phytogenic material of higher rank (that is, lignite coal) by its lower calorific value on a water saturated basis.

peatland—areas having peat-forming vegetation on which peak has accumulated or is accumulating.

penetrability—a grout property descriptive of its ability to fill a porous mass; primarily a function of lubricity and viscosity.

penetration—depth of hole cut in rock by a drill bit. (ISRM)

penetration grouting—filling joints or fractures in rock or pore spaces in soil with a grout without disturbing the formation; this grouting method does not modify the solid formation structure. See also **displacement grouting.**

penetration resistance (standard penetration resistance) (Proctor penetration resistance), p_R, N (FL^{-2} or Blows L^{-1})—(*a*) number of blows of a hammer of specified weight falling a given distance required to produce a given penetration into soil of a pile, casing, or sampling tube.

(*b*) unit load required to maintain constant rate of penetration into soil of a probe or instrument.

(*c*) unit load required to produce a specified penetration into soil at a specified rate of a probe or instrument. For a Proctor needle, the specified penetration is 2½ in. (63.5 mm) and the rate is ½ in. (12.7 mm)/s.

penetration resistance curve (Proctor penetration curve)—the curve showing the relationship between: (*1*) the penetration resistance, and (*2*) the water content.

percent compaction—the ratio, expressed as a percentage, of: (*1*) dry unit weight of a soil, to (*2*) maximum unit weight obtained in a laboratory compaction test.

percent consolidation—see **degree of consolidation.**

percent fines—amount, expressed as a percentage by weight, of a material in aggregate finer than a given sieve, usually the No. 200 (74 μm) sieve.

percent saturation (degree of saturation), S,S_r (D)—the ratio, expressed as a percentage, of: (*1*) the volume of water in a given soil or rock mass, to (*2*) the total volume of intergranular space (voids).

perched ground water—confined ground water separated from an underlying body of ground water by an unsaturated zone.

perched water table—a water table usually of limited area maintained above the normal free water elevation by the presence of an intervening relatively impervious confining stratum.

perched water table—groundwater separated from an underlying body of groundwater by unsaturated soil or rock. Usually located at a higher elevation than the groundwater table. (ISRM)

percolation—the movement of gravitational water through soil (see **seepage**).

percolation—movement, under hydrostatic pressure of water through the smaller interstices of rock or soil, excluding movement through large openings such as caves and solution channels. (ISRM)

percussion drilling—a drilling technique that uses solid or hollow rods for cutting and crushing the rock by repeated blows. (ISRM)

percussion drilling—a drilling process in which a hole is advanced by using a series of impacts to the drill steel and attached bit; the bit is normally rotated during drilling. See **rotary drilling.**

period—time interval occupied by one cycle.

permafrost—perennially frozen soil.

permanent strain—the strain remaining in a solid with respect to its initial condition after the application and removal of stress greater than the yield stress (commonly also called "residual" strain). (ISRM)

permeability—see **coefficient of permeability.**

permeability—the capacity of a rock to conduct liquid or gas. It is measured as the proportionality constant, k, between flow velocity, v, and hydraulic gradient, I; $v = k \cdot I$. (ISRM)

permeation grouting—filling joints or fractures in rock or pore spaces in soil with a grout, without disturbing the formation.

pH, pH (D)—an index of the acidity or alkalinity of a soil in terms of the logarithm of the reciprocal of the hydrogen ion concentration.

phase difference—difference between phase angles of two waves of same frequency.

phase of periodic quantity—fractional part of period through which independent variable has advanced, measured from an arbitrary origin.

phreatic line—the trace of the phreatic surface in any selected plane of reference.

phreatic line—see **line of seepage.**

phreatic surface—see **free water elevation.**

phreatic water—see **free water.**

piezometer—an instrument for measuring pressure head.

piezometric line (equipotential line)—line along which water will rise to the same elevation in piezometric tubes.

piezometric surface—the surface at which water will stand in a series of piezometers.

piezometric surface—an imaginary surface that everywhere coincides with the static level of the water in the aquifer. (ISRM)

pile—relatively slender structural element which is driven, or otherwise introduced, into the soil, usually for the purpose of providing vertical or lateral support.

pillar—in-situ rock between two or more underground openings: crown pillars; barrier pillars; rib pillars; sill pillars; chain pillars; etc. (ISRM)

pilot drift (pioneer tunnel)—a drift or tunnel first excavated as a smaller section than the dimensions of the main tunnel. A pilot drift or tunnel is usually used to investigate rock conditions in advance of the main tunnel, to permit installation of bracing before the principal mass of rock is removed, or to serve as a drainage tunnel. (ISRM)

piping—the progressive removal of soil particles from a mass by percolating water, leading to the development of channels.

pit—an excavation in the surface of the earth from which ore is obtained as in large open pit mining or as an excavation made for test purposes, that is, a testpit. (ISRM)

plane of weakness—surface or narrow zone with a (shear or tensile) strength lower than that of the surrounding material. (ISRM)

plane stress (strain)—a state of stress (strain) in a solid body in which all stress (strain) components normal to a certain plane are zero. (ISRM)

plane wave—wave in which fronts are parallel to plane normal to direction of propagation.

plastic deformation—see **plastic flow**.

plastic equilibrium—state of stress within a soil or rock mass or a portion thereof, which has been deformed to such an extent that its ultimate shearing resistance is mobilized.

active state of plastic equilibrium—plastic equilibrium obtained by an expansion of a mass.

passive state of plastic equilibrium—plastic equilibrium obtained by a compression of a mass.

plastic flow (plastic deformation)—the deformation of a plastic material beyond the point of recovery, accompanied by continuing deformation with no further increase in stress.

plasticity—the property of a soil or rock which allows it to be deformed beyond the point of recovery without cracking or appreciable volume change.

plasticity—property of a material to continue to deform indefinitely while sustaining a constant stress. (ISRM)

plasticity index, I_p, PI, I_w (D)—numerical difference between the liquid limit and the plastic limit.

plasticizer—*in grouting*, a material that increases the plasticity of a grout, cement paste, or mortar.

plastic limit, w_p, PL, P_w (D)—(*a*) the water content corresponding to an arbitrary limit between the plastic and the semisolid states of consistency of a soil. (*b*) water content at which a soil will just begin to crumble when rolled into a thread approximately ⅛ in. (3.2 mm) in diameter.

plastic soil—a soil that exhibits plasticity.

plastic state (plastic range)—the range of consistency within which a soil or rock exhibits plastic properties.

Poisson's ratio, (v)—ratio between linear strain changes perpendicular to and in the direction of a given uniaxial stress change.

pore pressure (*pore water pressure*)—see **neutral stress** under **stress**.

pore water—water contained in the voids of the soil or rock.

porosity, n (D)—the ratio, usually expressed as a percentage, of: (*1*) the volume of voids of a given soil or rock mass, to (*2*) the total volume of the soil or rock mass.

porosity—the ratio of the aggregate volume of voids or interstices in a rock or soil to its total volume. (ISRM)

portal—the surface entrance to a tunnel. (ISRM)

positive displacement pump—a pump that will continue to build pressure until the power source is stalled if the pump outlet is blocked.

potential drop, Δh (L)—the difference in total head between two equipotential lines.

power spectral density—the limiting mean-square value (for example, of acceleration, velocity, displacement, stress, or other random variable) per unit bandwidth, that is the limit of the mean-square value in a given rectangular bandwidth divided by the bandwidth, as the bandwidth approaches zero.

pozzolan—a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties.

preconsolidation pressure (prestress), p_c (FL^{-2})—the greatest effective pressure to which a soil has been subjected.

preplaced aggregate concrete—concrete produced by placing coarse aggregate in a form and later injecting a portland cement-sand or resin grout to fill the interstices.

pressure, p (FL^{-2})—the load divided by the area over which it acts.

pressure bulb—the zone in a loaded soil or rock mass bounded by an arbitrarily selected isobar of stress.

pressure testing—a method of permeability testing with water or grout pumped downhole under pressure.

pressure-void ratio curve (compression curve)—a curve representing the relationship between effective pressure and void ratio of a soil as obtained from a consolidation test. The curve has a characteristic shape when plotted on semilog paper with pressure on the log scale. The various parts of the curve and extensions to the parts of the curve and extensions to the parts have been designated as recompression, compression, virgin compression, expansion, rebound, and other descriptive names by various authorities.

pressure washing—the cleaning of soil or rock surfaces accomplished by injection of water, air, or other liquids, under pressure.

primary consolidation (*primary compression*) (*primary time effect*)—see **consolidation**.

primary hole—*in grouting*, the first series of holes to be drilled and grouted, usually at the maximum allowable spacing.

primary lining—the lining first placed inside a tunnel or shaft, usually used to support the excavation. The primary lining may be of wood or steel sets with steel or wood lagging or rock bolts and shot-crete. (ISRM)

primary permeability—internal permeability of intack rock; intergranular permeability (not permeability due to fracturing).

primary porosity—the porosity that developed during the final stages of sedimentation or that was present within sedimentary particles at the time of deposition.

primary state of stress—the stress in a geological formation before it is disturbed by man-made works. (ISRM)

principal plane—each of three mutually perpendicular planes through a point in a soil mass on which the shearing stress is zero.

intermediate principal plane—the plane normal to the direction of the intermediate principal stress.

major principal plane—the plane normal to the direction of the major principal stress.

minor principal plane—the plane normal to the direction of the minor principal stress.

principal stress—see **stress.**

principal stress (strain)—the stress (strain) normal to one of three mutually perpendicular planes on which the shear stresses (strains) at a point in a body are zero. (ISRM)

Proctor compaction curve—see **compaction curve.**

Proctor penetration curve—see **penetration resistance curve.**

Proctor penetration resistance—see **penetration resistance.**

profile—see **soil profile.**

progressive failure—failure in which the ultimate shearing resistance is progressively mobilized along the failure surface.

progressive failure—formation and development of localized fractures which, after additional stress increase, eventually form a continuous rupture surface and thus lead to failure after steady deterioration of the rock. (ISRM)

proportioning pump—see **metering pump.**

proprietary—made and marketed by one having the exclusive right to manufacture and sell; privately owned and managed.

protective filter—see **filter.**

pumpability—*in grouting*, a measure of the properties of a particular grout mix to be pumped as controlled by the equipment being used, the formation being injected, and the engineering objective limitations.

pumping of pavement (pumping)—see **pavement pumping.**

pumping test—a field procedure used to determine in situ permeability or the ability of a formation to accept grout.

pure shear—a state of strain resulting from that stress condition most easily described by a Mohr circle centered at the origin. (ISRM)

quarry—an excavation in the surface of the earth from which stone is obtained for crushed rock or building stone. (ISRM)

Quer-wave (love wave), W—dispersive surface wave with one horizontal component, generally normal to the direction of propagation, which decreases in propagation velocity with increase in frequency.

quick condition (quicksand)—condition in which water is flowing upwards with sufficient velocity to reduce significantly the bearing capacity of the soil through a decrease in intergranular pressure.

quick test—see **unconsolidated undrained test.**

radius of influence of a well—distance from the center of the well to the closest point at which the piezometric surface is not lowered when pumping has produced the maximum steady rate of flow.

raise—upwardly constructed shaft; that is, an opening, like a shaft, made in the roof of one level to reach a level above. (ISRM)

range (of a deformation-measuring instrument)—the amount between the maximum and minimum quantity an instrument can measure without resetting. In some instances provision can be made for incremental extension of the range.

Rayleigh wave, v_R (LT^{-1})—dispersive surface wave in which element has retrograding elliptic orbit with one major vertical and one minor horizontal component both in plane of propagation velocity:

$$v_R = \alpha v_t \text{ with } 0.910 < \alpha < 0.995 \text{ for } 0.25 < \nu < 0.5$$

reactant—*in grouting*, a material that reacts chemically with the base component of grout system.

reactive aggregate—an aggregate containing siliceous material (usually in amorphous or crypto-crystalline state) which can react chemically with free alkali in the cement.

Discussion—The reaction can result in expansion of the hardened material, frequently to a damaging extent.

reflected (or refracted) wave—components of wave incident upon second medium and reflected into first medium (or refracted) into second medium.

reflection and refraction loss—that part of transmitted energy lost due to nonuniformity of mediums.

refusal—*in grouting*, when the rate of grout take is low, or zero, at a given pressure.

relative consistency, I_c, C_r (D)—ratio of: (*1*) the liquid limit minus the natural water content, to (*2*) the plasticity index.

relative density, D_d, I_D (D)—the ratio of (*1*) the difference between the void ratio of a cohesionless soil in the loosest state and any given void ratio, to (*2*) the difference between the void ratios in the loosest and in the densest states.

relative water content—see **liquidity index.**

remodeled soil—soil that has had its natural structure modified by manipulation.

remolding index, I_R (D)—the ratio of: (*1*) the modulus of deformation of a soil in the undisturbed state, to (*2*) the modulus of deformation of the soil in the remolded state.

remodeling sensitivity (sensitivity ratio), S_t (D)—the ratio of: (*1*) the unconfined compressive strength of an undisturbed specimen of soil, to (*2*) the unconfined compressive strength of a specimen of the same soil after remolding at unaltered water content.

residual soil—soil derived in place by weathering of the underlying material.

residual strain—the strain in a solid associated with a state of residual stress. (ISRM)

residual stress—stress remaining in a solid under zero external stress after some process that causes the dimensions of the various parts of the solid to be incompatible under zero stress, for example, (*1*) deformation under the action of external stress when some parts of the body suffer permanent strain; or (*2*) heating or cooling of a body in which the thermal expansion coefficient is not uniform throughout the body. (ISRM)

resin—*in grouting*, a material that usually constitutes the base of an organic grout system.

resin grout—a grout system composed of essentially resinous materials such as epoxys, polyesters, and urethanes.

Discussion—In Europe, this refers to any chemical grout system regardless of chemical origin.

resolution (of a deformation-measuring instrument)—the ratio of the smallest divisional increment of the indicating scale to the sensitivity of the instrument. Interpolation within the increment may be possible, but is not recommended in specifying resolution.

resonance—the reinforced vibration of a body exposed to the vibration, at about the frequency, of another body.

resonant frequency—a frequency at which resonance exists.

response—the motion (or other output) in a device or system resulting from an excitation (stimulus) under specified conditions.

retard—bank-protection structure designed to reduce the riparian velocity and induce silting or accretion.

retardation—delay in deformation. (ISRM)

retarder—a material that slows the rate at which chemical reactions would otherwise occur.

reverse circulation—a drilling system in which the circulating medium flows down through the annulus and up through the drill rod, that is, in the reverse of the normal direction of flow.

revetment—bank protection by armor, that is, by facing of a bank or embankment with erosion-resistant material.

riprap stone—(generally less than 1 ton in weight) specially selected and graded quarried stone placed to prevent erosion through wave action, tidal forces, or strong currents and thereby preserve the shape of a surface, slope, or underlying structure.

rise time (pulse rise time)—the interval of time required for the leading edge of a pulse to rise from some specified small fraction to some specified larger fraction of the maximum value.

rock—natural solid mineral matter occurring in large masses or fragments.

rock—any naturally formed aggregate of mineral matter occurring in large masses or fragments. (ISRM)

rock anchor—a steel rod or cable installed in a hole in rock; in principle the same as rock bolt, but generally used for rods longer than about four metres. (ISRM)

rock bolt—a steel rod placed in a hole drilled in rock used to tie the rock together. One end of the rod is firmly anchored in the hole by means of a mechanical device or grout, or both, and the threaded projecting end is equipped with a nut and plate that bears against the rock surface. The rod can be pretensioned. (ISRM)

rock burst—a sudden and violent expulsion of rock from its surroundings that occurs when a volume of rock is strained beyond the elastic limit and the accompanying failure is of such a nature that accumulated energy is released instantaneously.

rock burst—sudden explosive-like release of energy due to the failure of a brittle rock of high strength. (ISRM)

rock flour—see **silt**.

rock mass—rock as it occurs in situ, including its structural discontinuities. (ISRM)

rock mechanics—the application of the knowledge of the mechanical behavior of rock to engineering problems dealing with rock. Rock mechanics overlaps with structural geology, geophysics, and soil mechanics.

rock mechanics—theoretical and applied science of the mechanical behaviour of rock. (ISRM)

roof—top of excavation or underground opening, particularly applicable in bedded rocks where the top surface of the opening is flat rather than arched. (ISRM)

rotary drilling—a drilling process in which a hole is advanced by rotation of a drill bit under constant pressure without impact. See **percussion drilling**.

round—a set of holes drilled and charged in a tunnel or quarry that are fired instantaneously or with short-delay detonators. (ISRM)

running ground—*in tunneling*, a granular material that tends to flow or "run" into the excavation.

rupture—that stage in the development of a fracture where instability occurs. It is not recommended that the term rupture be used in rock mechanics as a synonym for fracture. (ISRM)

rupture envelope (*rupture line*)—see **Mohr envelope**.

sagging—usually occurs in sedimentary rock formations as a separation and downward bending of sedimentary beds in the roof of an underground opening. (ISRM)

sand—particles of rock that will pass the No. 4 (4.75-mm) sieve and be retained on the No. 200 (75-μm) U.S. standard sieve.

sand boil—the ejection of sand and water resulting from piping.

sand equivalent—a measure of the amount of silt or clay contamination in fine aggregate as determined by test (Test Method D 2419).[3]

sanded grout—grout in which sand is incorporated into the mixture.

sapric peat—peat in which the original plant fibers are highly decomposed (less than 33 % fibers).

saturated unit weight—see **unit weight**.

saturation curve—see **zero air voids curve**.

scattering loss—that part of transmitted energy lost due to roughness of reflecting surface.

schistosity—the variety of foliation that occurs in the coarser-grained metamorphic rocks and is generally the result of the parallel arrangement of platy and ellipsoidal mineral grains within the rock substance. (ISRM)

secant modulus—slope of the line connecting the origin and a given point on the stress-strain curve. (ISRM)

secondary consolidation (*secondary compression*) (*secondary time effect*)—see **consolidation**.

secondary hole—*in grouting*, the second series of holes to be drilled and grouted usually spaced midway between primary holes.

secondary lining—the second-placed, or permanent, structural lining of a tunnel, which may be of concrete, steel, or masonry. (ISRM)

[3] *Annual Book of ASTM Standards*, Vol 04.03.

secondary state of stress—the resulting state of stress in the rock around man-made excavations or structures. (ISRM)

sediment basin—a structure created by construction of a barrier or small dam-like structure across a waterway or by excavating a basin or a combination of both to trap or restrain sediment.

seep—a small area where water oozes from the soil or rock.

seepage—the infiltration or percolation of water through rock or soil to or from the surface. The term seepage is usually restricted to the very slow movement of ground water. (ISRM)

seepage (percolation)—the slow movement of gravitational water through the soil or rock.

seepage force—the frictional drag of water flowing through voids or interstices in rock, causing an increase in the intergranular pressure, that is, the hydraulic force per unit volume of rock or soil which results from the flow of water and which acts in the direction of flow. (ISRM)

seepage force, J (F)—the force transmitted to the soil or rock grains by seepage.

seepage line—see **line of seepage.**

seepage velocity, V_s, V_1 (LT^{-1})—the rate of discharge of seepage water through a porous medium per unit area of void space perpendicular to the direction of flow.

segregation—*in grouting*, the differential concentration of the components of mixed grout, resulting in nonuniform proportions in the mass.

seismic support—mass (heavy) supported on springs (weak) so that mass remains almost at rest when free end of springs is subjected to sinusoidal motion at operating frequency.

seismic velocity—the velocity of seismic waves in geological formations. (ISRM)

seismometer—instrument to pick up linear (vertical, horizontal) or rotational displacement, velocity, or acceleration.

self-stressing grout—expansive-cement grout in which the expansion induces compressive stress in grout if the expansion movement is restrained.

sensitivity—the effect of remolding on the consistency of a cohesive soil.

sensitivity (of an instrument)—the differential quotient dQ_0/dQ_1, where Q_0 is the scale reading and Q_1 is the quantity to be measured.

sensitivity (of a transducer)—the differential quotient dQ_0/dQ_1, where Q_0 is the output and Q_1 is the input.

series grouting—similar to stage grouting, except each successively deeper zone is grouted by means of a newly drilled hole, eliminating the need for washing grout out before drilling the hole deeper.

set—*in grouting*, the condition reached by a cement paste, or grout, when it has lost plasticity to an arbitrary degree, usually measured in terms of resistance to penetration or deformation; initial set refers to first stiffening and final set refers to an attainment of significant rigidity.

setting shrinkage—*in grouting*, a reduction in volume of grout prior to the final set of cement caused by bleeding, by the decrease in volume due to the chemical combination of water with cement, and by syneresis.

set time—(*1*) the hardening time of portland cement; or (*2*) the gel time for a chemical grout.

shaft—generally a vertical or near vertical excavation driven downward from the surface as access to tunnels, chambers, or other underground workings. (ISRM)

shaking test—a test used to indicate the presence of significant amounts of rock flour, silt, or very fine sand in a fine-grained soil. It consists of shaking a pat of wet soil, having a consistency of thick paste, in the palm of the hand; observing the surface for a glossy or livery appearance; then squeezing the pat; and observing if a rapid apparent drying and subsequent cracking of the soil occurs.

shear failure (failure by rupture)—failure in which movement caused by shearing stresses in a soil or rock mass is of sufficient magnitude to destroy or seriously endanger a structure.

general shear failure—failure in which the ultimate strength of the soil or rock is mobilized along the entire potential surface of sliding before the structure supported by the soil or rock is impaired by excessive movement.

local shear failure—failure in which the ultimate shearing strength of the soil or rock is mobilized only locally along the potential surface of sliding at the time the structure supported by the soil or rock is impaired by excessive movement.

shear force—a force directed parallel to the surface element across which it acts. (ISRM)

shear plane—a plane along which failure of material occurs by shearing. (ISRM)

shear resistance—see **internal friction.**

shear strain—the change in shape, expressed by the relative change of the right angles at the corner of what was in the undeformed state an infinitesimally small rectangle or cube. (ISRM)

shear strength, s, τ_f (FL^{-2})—the maximum resistance of a soil or rock to shearing stresses. See **peak shear strength.**

shear stress—stress directed parallel to the surface element across which it acts. (ISRM)

shear stress (*shearing stress*) (*tangential stress*)—see **stress.**

shear wave (rotational, equivoluminal)—wave in which medium changes shape without change of volume (shear-plane wave in isotropic medium is transverse wave).

shelf life—maximum time interval during which a material may be stored and remain in a usable condition; usually related to storage conditions.

shock pulse—a substantial disturbance characterized by a rise of acceleration from a constant value and decay of acceleration to the constant value in a short period of time.

shock wave—a wave of finite amplitude characterized by a shock front, a surface across which pressure, density, and internal energy rise almost discontinuously, and which travels with a speed greater than the normal speed of sound. (ISRM)

shotcrete—mortar or concrete conveyed through a hose and pneumatically projected at high velocity onto a surface. Can be applied by a "wet" or "dry" mix method. (ISRM)

shrinkage-compensating—*in grouting*, a characteristic of grout made using an expansive cement in which volume increase, if restrained, induces compressive stresses that are intended to offset the tendency of drying shrinkage to induce tensile stresses. See also **self-stressing grout.**

shrinkage index, *SI* (D)—the numerical difference between the plastic and shrinkage limits.

shrinkage limit, *SL*, w_s (D)—the maximum water content at which a reduction in water content will not cause a decrease in volume of the soil mass.

shrinkage ratio, *R* (D)—the ratio of: (*1*) a given volume change, expressed as a percentage of the dry volume, to (*2*) the corresponding change in water content above the shrinkage limit, expressed as a percentage of the weight of the oven-dried soil.

sieve analysis—determination of the proportions of particles lying within certain size ranges in a granular material by separation on sieves of different size openings.

silt (inorganic silt) (rock flour)—material passing the No. 200 (75-μm) U.S. standard sieve that is nonplastic or very slightly plastic and that exhibits little or no strength when air-dried.

silt size—that portion of the soil finer than 0.02 mm and coarser than 0.002 mm (0.05 mm and 0.005 mm in some cases).

simple shear—shear strain in which displacements all lie in one direction and are proportional to the normal distances of the displaced points from a given reference plane. The dilatation is zero. (ISRM)

single-grained structure—see **soil structure.**

size effect—influence of specimen size on its strength or other mechanical parameters. (ISRM)

skin friction, *f* (FL^{-2})—the frictional resistance developed between soil and an element of structure.

slabbing—the loosening and breaking away of relatively large flat pieces of rock from the excavated surface, either immediately after or some time after excavation. Often occurring as tensile breaks which can be recognized by the subconchoidal surfaces left on remaining rock surface. (ISRM)

slabjacking—*in grouting*, injection of grout under a concrete slab in order to raise it to a specified grade.

slaking—deterioration of rock on exposure to air or water.

slaking—the process of breaking up or sloughing when an indurated soil is immersed in water.

sleeved grout pipe—see **tube A manchette.**

sliding—relative displacement of two bodies along a surface, without loss of contact between the bodies. (ISRM)

slope—the excavated rock surface that is inclined to the vertical or horizontal, or both, as in an open-cut. (ISRM)

slow test—see **consolidated-drain test.**

slump—a measure of consistency of freshly mixed concrete or grout. See also **slump test.**

slump test—the procedure for measuring slump (Test Method C 143).[2]

slurry cutoff wall—a vertical barrier constructed by excavating a vertical slot under a bentonite slurry and backfilling it with materials of low permeability for the purpose of the containment of the lateral flow of water and other fluids.

slurry grout—a fluid mixture of solids such as cement, sand, or clays in water.

slurry trench—a trench that is kept filled with a bentonite slurry during the excavation process to stabilize the walls of the trench.

slush grouting—application of cement slurry to surface rock as a means of filling cracks and surface irregularities or to prevent slaking; it is also applied to riprap to form grouted riprap.

smooth (-wall) blasting—a method of accurate perimeter blasting that leaves the remaining rock practically undamaged. Narrowly spaced and lightly charged blastholes, sometimes alternating with empty dummy holes, located along the breakline and fired simultaneously as the last round of the excavation. (ISRM)

soil (earth)—sediments or other unconsolidated accumulations of solid particles produced by the physical and chemical disintegration of rocks, and which may or may not contain organic matter.

soil binder—see **binder.**

soil-forming factors—factors, such as parent material, climate, vegetation, topography, organisms, and time involved in the transformation of an original geologic deposit into a soil profile.

soil horizon—see **horizon.**

soil mechanics—the application of the laws and principles of mechanics and hydraulics to engineering problems dealing with soil as an engineering material.

soil physics—the organized body of knowledge concerned with the physical characteristics of soil and with the methods employed in their determinations.

soil profile (profile)—vertical section of a soil, showing the nature and sequence of the various layers, as developed by deposition or weathering, or both.

soil stabilization—chemical or mechanical treatment designed to increase or maintain the stability of a mass of soil or otherwise to improve its engineering properties.

soil structure—the arrangement and state of aggregation of soil particles in a soil mass.

flocculent structure—an arrangement composed of flocs of soil particles instead of individual soil particles.

honeycomb structure—an arrangement of soil particles having a comparatively loose, stable structure resembling a honeycomb.

single-grained structure—an arrangement composed of individual soil particles; characteristic structure of coarse-grained soils.

soil suspension—highly diffused mixture of soil and water.

soil texture—see **gradation.**

solution cavern—openings in rock masses formed by moving water carrying away soluble materials.

sounding well—*in grouting*, a vertical conduit in a mass of coarse aggregate for preplaced aggregate concrete which contains closely spaced openings to permit entrance of grout.

Discussion—The grout level is determined by means of a measuring line on a float within the sounding well.

spacing—the distance between adjacent blastholes in a direction parallel to the face. (ISRM)

spalling—(*1*) longitudinal splitting in uniaxial compression, or (*2*) breaking-off of plate-like pieces from a free rock surface. (ISRM)

specific gravity:

specific gravity of solids, G, G_s, S_s (D)—ratio of: (*1*) the weight in air of a given volume of solids at a stated temperature to (*2*) the weight in air of an equal volume of distilled water at a stated temperature.

apparent specific gravity, G_a, S_a (D)—ratio of: (*1*) the weight in air of a given volume of the impermeable portion of a permeable material (that is, the solid matter including its impermeable pores or voids) at a stated temperature to (*2*) the weight in air of an equal volume of distilled water at a stated temperature.

bulk specific gravity (*specific mass gravity*), G_m, S_m (D)—ratio of: (*1*) the weight in air of a given volume of a permeable material (including both permeable and impermeable voids normal to the material) at a stated temperature to (*2*) the weight in air of an equal volume of distilled water at a stated temperature.

specific surface (L^{-1})—the surface area per unit of volume of soil particles.

spherical wave—wave in which wave fronts are concentric spheres.

split spacing grouting—a grouting sequence in which initial (primary) grout holes are relatively widely spaced and subsequent grout holes are placed midway between previous grout holes to "split the spacing."; this process is continued until a specified hole spacing is achieved or a reduction in grout take to a specified value occurs, or both.

spring characteristics, c (FL^{-1})—ratio of increase in load to increase in deflection:

$$c = l/C$$

where:
C = compliance.

stability—the condition of a structure or a mass of material when it is able to support the applied stress for a long time without suffering any significant deformation or movement that is not reversed by the release of stress. (ISRM)

stability factor (stability number), N_s (D)—a pure number used in the analysis of the stability of a soil embankment, defined by the following equation:

$$N_s = H_c \gamma_e / c$$

where:
H_c = critical height of the sloped bank,
γ_e = effective unit of weight of the soil, and
c = cohesion of the soil

NOTE—Taylor's "stability number" is the reciprocal of Terzaghi's "stability factor."

stabilization—see **soil stabilization**.

stage—*in grouting*, the length of hole grouted at one time. See also **stage grouting**.

stage grouting—sequential grouting of a hole in separate steps or stages in lieu of grouting the entire length at once; holes may be grouted in ascending stages by using packers or in descending stages downward from the collar of the hole.

standard compaction—see **compaction test**.

standard penetration resistance—see **penetration resistance**.

standing wave—a wave produced by simultaneous transmission in opposite directions of two similar waves resulting in fixed points of zero amplitudes called nodes.

steady-state vibration—vibration in a system where the velocity of each particle is a continuing periodic quantity.

stemming—(*1*) the material (chippings, or sand and clay) used to fill a blasthole after the explosive charge has been inserted. Its purpose is to prevent the rapid escape of the explosion gases. (*2*) the act of pushing and tamping the material in the hole. (ISRM)

stick-slip—rapid fluctuations in shear force as one rock mass slides past another, characterized by a sudden slip between the rock masses, a period of no relative displacement between the two masses, a sudden slip, etc. The oscillations may be regular as in a direct shear test, or irregular as in a triaxial test.

sticky limit, T_w (D)—the lowest water content at which a soil will stick to a metal blade drawn across the surface of the soil mass.

stiffness—the ratio of change of force (or torque) to the corresponding change in translational (or rotational) deflection of an elastic element.

stiffness-force—displacement ratio. (ISRM)

stone—crushed or naturally angular particles of rock.

stop—*in grouting*, a packer setting at depth.

stop grouting—the grouting of a hole beginning at the lowest packer setting (stop) after the hole is drilled to total depth.

Discussion—Packers are placed at the top of the zone being grouted. Grouting proceeds from the bottom up. Also called upstage grouting.

strain, ϵ (D)—the change in length per unit of length in a given direction.

strain (linear or normal), ϵ (D)—the change in length per unit of length in a given direction.

strain ellipsoid—the representation of the strain in the form of an ellipsoid into which a sphere of unit radius deforms and whose axes are the principal axes of strain. (ISRM)

strain (stress) rate—rate of change of strain (stress) with time. (ISRM)

strain resolution (strain sensitivity), R_s (D)—the smallest subdivision of the indicating scale of a deformation-measuring device divided by the product of the sensitivity of the device and the gage length. The deformation resolution, R_d, divided by the gage length.

strain (stress) tensor—the second order tensor whose diagonal elements consist of the normal strain (stress) components with respect to a given set of coordinate axes and whose off-diagonal elements consist of the corresponding shear strain (stress) components. (ISRM)

streamline flow—see **laminar flow**.

strength—maximum stress which a material can resist without failing for any given type of loading. (ISRM)

stress, σ, p, f (FL^{-2})—the force per unit area acting within the soil mass.

effective stress (*effective pressure*) (*intergranular pressure*), $\bar{\sigma}$, f (FL^{-2})—the average normal force per unit area transmitted from grain to grain of a soil mass. It is the stress that is effective in mobilizing internal friction.

neutral stress (*pore pressure*) (*pore water pressure*), u, u_w (FL^{-2})—stress transmitted through the pore water (water filling the voids of the soil).

normal stress, σ, p (FL^{-2})—the stress component normal to a given plane.

principal stress, σ_1, σ_2, σ_3 (FL^{-2})—stresses acting normal to three mutually perpendicular planes intersecting at a point in a body, on which the shearing stress is zero.

major principal stress, σ_1 (FL^{-2})—the largest (with regard to sign) principal stress.

minor principal stress, σ_3 (FL^{-2})—the smallest (with regard to sign) principal stress.

intermediate principal stress, σ_2 (FL^{-2})—the principal stress whose value is neither the largest nor the smallest (with regard to sign) of the three.

shear stress (*shearing stress*) (*tangential stress*), τ, s FL^{-2})—the stress component tangential to a given plane.

total stress, σ, f (FL^{-2})—the total force per unit area acting within a mass of soil. It is the sum of the neutral and effective stresses.

stress ellipsoid—the representation of the state of stress in the form of an ellipsoid whose semi-axes are proportional to the magnitudes of the principal stresses and lie in the principal directions. The coordinates of a point *P* on this ellipse are proportional to the magnitudes of the respective components of the stress across the plane normal to the direction *OP*, where *O* is the center of the ellipsoid. (ISRM)

stress (strain) field—the ensemble of stress (strain) states defined at all points of an elastic solid. (ISRM)

stress relaxation—stress release due to creep. (ISRM)

strike—the direction or azimuth of a horizontal line in the plane of an inclined stratum, joint, fault, cleavage plane, or other planar feature within a rock mass. (ISRM)

structure—one of the larger features of a rock mass, like bedding, foliation, jointing, cleavage, or brecciation; also the sum total of such features as contrasted with texture. Also, in a broader sense, it refers to the structural features of an area such as anti-clines or synclines. (ISRM)

structure—see soil structure.

subbase—a layer used in a pavement system between the subgrade and base coarse, or between the subgrade and portland cement concrete pavement.

subgrade—the soil prepared and compacted to support a structure or a pavement system.

subgrade surface—the surface of the earth or rock prepared to support a structure or a pavement system.

submerged unit weight—see unit weight.

subsealing—*in grouting,* grouting under concrete slabs for the purpose of filling voids without raising the slabs.

subsidence—the downward displacement of the overburden (rock or soil, or both) lying above an underground excavation or adjoining a surface excavation. Also the sinking of a part of the earth's crust. (ISRM)

subsoil—(*a*) soil below a subgrade of fill. (*b*) that part of a soil profile occurring below the "A" horizon.

sulfate attack—*in grouting,* harmful or deleterious reactions between sulfates in soil or groundwater and the grout.

support—structure or structural feature built into an underground opening for maintaining its stability. (ISRM)

surface force—any force that acts across an internal or external surface element in a material body, not necessarily in a direction lying in the surface. (ISRM)

surface wave—a wave confined to a thin layer at the surface of a body. (ISRM)

suspension—a mixture of liquid and solid materials.

suspension agent—an additive that decreased the settlement rate of particles in liquid.

swamp—a forested or shrub covered wetland where standing or gently flowing water persists for long periods on the surface.

syneresis—*in grouting,* the exudation of liquid (generally water) from a set gel which is not stressed, due to the tightening of the grout material structure.

take—see grout take.

talus—rock fragments mixed with soil at the foot of a natural slope from which they have been separated.

tangential stress—see stress.

tangent modulus—slope of the tangent to the stress-strain curve at a given stress value (generally taken at a stress equal to half the compressive strength). (ISRM)

tensile strength (unconfined or uniaxial tensile strength), T_o (FL^{-2})—the load per unit area at which an unconfined cylindrical specimen will fail in a simple tension (pull) test.

tensile stress—normal stress tending to lengthen the body in the direction in which it acts. (ISRM)

tertiary hole—*in grouting,* the third series of holes to be drilled and grouted usually spaced midway between previously grouted primary and secondary holes.

texture—*of soil and rock,* geometrical aspects consisting of size, shape, arrangement, and crystallinity of the component particles and of the related characteristics of voids.

texture—the arrangement in space of the components of a rock body and of the boundaries between these components. (ISRM)

theoretical time curve—see consolidation time curve.

thermal spalling—the breaking of rock under stresses induced by extremely high temperature gradients. High-velocity jet flames are used for drilling blast holes with this effect. (ISRM)

thermo-osmosis—the process by which water is caused to flow in small openings of a soil mass due to differences in temperature within the mass.

thickness—the perpendicular distance between bounding surfaces such as bedding or foliation planes of a rock. (ISRM)

thixotropy—the property of a material that enables it to stiffen in a relatively short time on standing, but upon agitation or manipulation to change to a very soft consistency or to a fluid of high viscosity, the process being completely reversible.

throw—the projection of broken rock during blasting. (ISRM)

thrust—force applied to a drill in the direction of penetration. (ISRM)

tight—rock remaining within the minimum excavation lines after completion of a blasting record. (ISRM)

till—see glacial till.

time curve—see consolidation time curve.

time factor, T_v, T (D)—dimensionless factor, utilized in the theory of consolidation, containing the physical constants of a soil stratum influencing its time-rate of consolidation, expressed as follows:

$$T = k(1 + e)t/(a_v \gamma_w \cdot H^2) = (c_v \cdot t)/H^2$$

where:

k = coefficient of permeability (LT^{-1}),

e = void ratio (dimensionless),

t = elapsed time that the stratum has been consolidated (T),
a_v = coefficient of compressibility (L^2F^{-1}),
γ_w = unit weight of water (FL^{-3}),
H = thickness of stratum drained on one side only. If stratum is drained on both sides, its thickness equals $2H$ (L), and
c_v = coefficient of consolidation (L^2T^{-1}).

topsoil—surface soil, usually containing organic matter.

torsional shear test—a shear test in which a relatively thin test specimen of solid circular or annular cross-section, usually confined between rings, is subjected to an axial load and to shear in torsion. In-place torsion shear tests may be performed by pressing a dentated solid circular or annular plate against the soil and measuring its resistance to rotation under a given axial load.

total stress—see **stress**.

toughness index, I_T, T_w—the ratio of: (*1*) the plasticity index, to (*2*) the flow index.

traction, S_1, S_2, S_3 (FL^{-2})—applied stress.

transformed flow net—a flow net whose boundaries have been properly modified (transformed) so that a net consisting of curvilinear squares can be constructed to represent flow conditions in an anisotropic porous medium.

transported soil—soil transported from the place of its origin by wind, water, or ice.

transverse wave, v_t (LT^{-1})—wave in which direction of displacement of element of medium is parallel to wave front. The propagation velocity, v_t, is calculated as follows:

$$v_t = \sqrt{G/\rho} = \sqrt{\mu/\rho} = \sqrt{(E/\rho)[1/2(1+v)]}$$

where:
G = shear modulus,
ρ = mass density,
v = Poisson's ratio, and
E = Young's modulus.

transverse wave (shear wave)—a wave in which the displacement at each point of the medium is parallel to the wave front. (ISRM)

tremie—material placed under water through a tremie pipe in such a manner that it rests on the bottom without mixing with the water.

trench—usually a long, narrow, near vertical sided cut in rock or soil such as is made for utility lines. (ISRM)

triaxial compression—compression caused by the application of normal stresses in three perpendicular directions. (ISRM).

triaxial shear test (triaxial compression test)—a test in which a cylindrical specimen of soil or rock encased in an impervious membrane is subjected to a confining pressure and then loaded axially to failure.

triaxial state of stress—state of stress in which none of the three principal stresses is zero. (ISRM)

true solution—one in which the components are 100 % dissolved in the base solvent.

tube A manchette—*in grouting*, a grout pipe perforated with rings of small holes at intervals of about 12 in. (305 mm).

Discussion—Each ring of perforations is enclosed by a short rubber sleeve fitting tightly around the pipe so as to act as a one-way valve when used with an inner pipe containing two packer elements that isolate a stage for injection of grout.

tunnel—a man-made underground passage constructed without removing the overlying rock or soil. Generally nearly horizontal as opposed to a shaft, which is nearly vertical. (ISRM)

turbulent flow—that type of flow in which any water particle may move in any direction with respect to any other particle, and in which the head loss is approximately proportional to the second power of the velocity

ultimate bearing capacity, q_o, q_{ult} (FL^{-2})—the average load per unit of area required to produce failure by rupture of a supporting soil or rock mass.

unconfined compressive strength—the load per unit area at which an unconfined prismatic or cylindrical specimen of material will fail in a simple compression test without lateral support.

unconfined compressive strength—see **compressive strength**.

unconsolidated-undrained test (quick test)—a soil test in which the water content of the test specimen remains practically unchanged during the application of the confining pressure and the additional axial (or shearing) force.

undamped natural frequency—*of a mechanical system*, the frequency of free vibration resulting from only elastic and inertial forces of the system.

underconsolidated soil deposit—a deposit that is not fully consolidated under the existing overburden pressure.

undisturbed sample—a soil sample that has been obtained by methods in which every precaution has been taken to minimize disturbance to the sample.

uniaxial (unconfined) compression—compression caused by the application of normal stress in a single direction. (ISRM)

uniaxial state of stress—state of stress in which two of the three principal stresses are zero. (ISRM)

unit weight, γ (FL^{-3})—weight per unit volume (with this, and all subsequent unit-weight definitions, the use of the term weight means force).

dry unit weight (*unit dry weight*), γ_d, γ_e (FL^{-3})—the weight of soil or rock solids per unit of total volume of soil or rock mass.

effective unit weight, γ_e (FL^{-3})—that unit weight of a soil or rock which, when multiplied by the height of the overlying column of soil or rock, yields the effective pressure due to the weight of the overburden.

maximum unit weight, γ_{max} (FL^{-3})—the dry unit weight defined by the peak of a compaction curve.

saturated unit weight, γ_G, γ_{sat} (FL^{-3})—the wet unit weight of a soil mass when saturated.

submerged unit weight (*buoyant unit weight*), γ_m, γ', γ_{sub} (FL^{-3})—the weight of the solids in air minus the weight of water displaced by the solids per unit of volume of soil or rock mass; the saturated unit weight minus the unit weight of water.

unit weight of water, γ_w (FL^{-3})—the weight per unit volume of water; nominally equal to 62.4 lb/ft^3 or 1 g/cm^3.

wet unit weight (*mass unit weight*), γ_m, γ_{wet} (FL^{-3})—the weight (solids plus water) per unit of total volume of soil or rock mass, irrespective of the degree of saturation.

zero air voids unit weight, γ_z, γ_s (FL^{-3})—the weight of solids per unit volume of a saturated soil or rock mass.

unloading modulus—slope of the tangent to the unloading stress-strain curve at a given stress value. (ISRM)

uplift—the upward water pressure on a structure.

	Symbol	Unit
unit symbol	u	FL^{-2}
total symbol	U	F or FL^{-1}

uplift—the hydrostatic force of water exerted on or underneath a structure, tending to cause a displacement of the structure. (ISRM)

uplift—*in grouting*, vertical displacement of a formation due to grout injection.

vane shear test—an in-place shear test in which a rod with thin radial vanes at the end is forced into the soil and the resistance to rotation of the rod is determined.

varved clay—alternating thin layers of silt (or fine sand) and clay formed by variations in sedimentation during the various seasons of the year, often exhibiting contrasting colors when partially dried.

vent hole—*in grouting*, a hole drilled to allow the escape of air and water and also used to monitor the flow of grout.

vent pipe—*in grouting*, a small-diameter pipe used to permit the escape of air, water, or diluted grout from a formation.

vibrated beam wall (injection beam wall)—barrier formed by driving an H-beam in an overlapping pattern of prints and filling the print of the beam with cement-bentonite slurry or other materials as it is withdrawn.

vibration—an oscillation wherein the quantity is a parameter that defines the motion of a mechanical system (see **oscillation**).

virgin compression curve—see **compression curve**.

viscoelasticity—property of materials that strain under stress partly elastically and partly viscously, that is, whose strain is partly dependent on time and magnitude of stress. (ISRM)

viscosity—the internal fluid resistance of a substance which makes it resist a tendency to flow.

viscous damping—the dissipation of energy that occurs when a particle in a vibrating system is resisted by a force that has a magnitude proportional to the magnitude of the velocity of the particle and direction opposite to the direction of the particle.

viscous flow—see **laminar flow**.

void—space in a soil or rock mass not occupied by solid mineral matter. This space may be occupied by air, water, or other gaseous or liquid material.

void ratio, e (D)—the ratio of: (*1*) the volume of void space, to (*2*) the volume of solid particles in a given soil mass.

critical void ratio, e_c (D)—the void ratio corresponding to the critical density.

volumetric shrinkage (volumetric change), V_s (D)—the decrease in volume, expressed as a percentage of the soil mass when dried, of a soil mass when the water content is reduced from a given percentage to the shrinkage limit.

von Post humification scale—a scale describing various stages of decomposition of peat ranging from H1, which is completely undecomposed, to H10, which is completely decomposed.

wall friction, f' (FL^{-2})—frictional resistance mobilized between a wall and the soil or rock in contact with the wall.

washing—*in grouting*, the physical act of cleaning the sides of a hole by circulating water, water and air, acid washes, or chemical substances through drill rods or tremie pipe in an open hole.

water-cement ratio—the ratio of the weight of water to the weights of Portland cement in a cement grout or concrete mix. See also **grout mix**.

water content, w (D)—the ratio of the mass of water contained in the pore spaces of soil or rock material, to the solid mass of particles in that material, expressed as a percentage.

water gain—see **bleeding**.

water-holding capacity (D)—the smallest value to which the water content of a soil or rock can be reduced by gravity drainage.

water-plasticity ratio (relative water content) (liquidity index)—see **liquidity index**.

water table—see **free water elevation**.

wave—disturbance propagated in medium in such a manner that at any point in medium the amplitude is a function of time, while at any instant the displacement at point is function of position of point.

wave front—moving surface in a medium at which a propagated disturbance first occurs.

wave front—(*1*) a continuous surface over which the phase of a wave that progresses in three dimensions is constant, or (*2*) a continuous line along which the phase of a surface wave is constant. (ISRM)

wave length—normal distance between two wave fronts with periodic characteristics in which amplitudes have phase difference of one complete cycle.

weathering—the process of disintegration and decomposition as a consequence of exposure to the atmosphere, to chemical action, and to the action of frost, water, and heat. (ISRM)

wetland—land which has the water table at, near, or above the land surface, or which is saturated for long enough periods to promote hydrophylic vegetation and various kinds of biological activity which are adapted to the wet environment.

wetting agent—a substance capable of lowering the surface tension of liquids, facilitating the wetting of solid surfaces, and facilitating the penetration of liquids into the capillaries.

wet unit weight—see **unit weight**.

working pressure—the pressure adjudged best for any particular set of conditions encountered during grouting.

Discussion—Factors influencing the determination are size of voids to be filled, depth of zone to be grouted, lithology of area to be grouted, grout viscosity, and resistance of the formation to fracture.

yield—*in grouting*, the volume of freshly mixed grout produced from a known quantity of ingredients.

yielding arch—type of support of arch shape, the joints of which deform plastically beyond a certain critical load, that is, continue to deform without increasing their resistance. (ISRM)

yield stress—the stress beyond which the induced deformation is not fully annulled after complete destressing. (ISRM)

Young's modulus—the ratio of the increase in stress on a test specimen to the resulting increase in strain under constant transverse stress limited to materials having a linear stress-strain relationship over the range of loading. Also called elastic modulus.

zero air voids curve (saturation curve)—the curve showing the zero air voids unit weight as a function of water content.

zero air voids density (*zero air voids unit weight*)—see unit weight.

APPENDIX

(Nonmandatory Information)

X1. ISRM SYMBOLS RELATING TO SOIL AND ROCK MECHANICS

NOTE—These symbols may not correlate with the symbols appearing in the text.

X1.1 Space

Ω, ω	solid angle
l	length
b	width
h	height or depth
r	radius
A	area
V	volume
t	time
v	velocity
ω	angular velocity
g	gravitational acceleration

X1.2 Periodic and Related Phenomena

T	periodic time
f	frequency
ω	angular frequency
λ	wave length

X1.3 Statics and Dynamics

m	mass
ρ	density (mass density)
G_m	mass specific gravity
G_s	specific gravity of solids
G_w	specific gravity of water
F	force
T	tangential force
W	weight
γ	unit weight
γ_d	dry unit weight
γ_w	unit weight of water
γ'	buoyant unit weight
γ_s	unit of solids
T	torque
I	moment of inertia
W	work
W	energy

X1.4 Applied Mechanics

e	void ratio
n	porosity
w	water content
S_r	degree of saturation
p	pressure
u	pore water pressure
σ	normal stress
$\sigma_x, \sigma_y, \sigma_z$	stress components in rectangular coordinates
$\sigma_1, \sigma_2, \sigma_3$	principal stresses
S_1, S_2, S_3	applied stresses (and reactions)
σ_h	horizontal stress
σ_v	vertical stress
τ	shear stress
$\tau_{xy}, \tau_{yz}, \tau_{zx}$	shear stress components in rectangular coordinates
ϵ	strain
$\epsilon_x, \epsilon_y, \epsilon_z$	strain components in rectangular coordinates
$\gamma_{xy}, \gamma_{yz}, \gamma_{zx}$	shear strain components in rectangular coordinates
θ	volume strain
E	Young's modulus; modulus of elasticity $E = \sigma/\epsilon$
$\epsilon_1, \epsilon_2, \epsilon_3$	principal strains
G	shear modulus; modulus of rigidity $G = \tau/\gamma$
c	cohesion
ϕ_s	angle of friction between solid bodies
ϕ	angle of shear resistance (angle of internal friction)
h	hydraulic head
i	hydraulic gradient
j	seepage force per unit volume or seepage pressure per unit length
k	coefficient of permeability
η	viscosity
η_{pl}	plasticity (viscosity of Bingham body)
t_{re}	retardation time
t_{rel}	relaxation time
T_s	surface tension
q	quantity rate of flow; rate of discharge
Q	quantity of flow
FS	safety factor

X1.5 Heat

T	temperature
β	coefficient of volume expansion

X1.6 Electricity

I	electric current
Q	electric charge
C	capacitance
L	self-inductance
R	resistance
ρ	resistivity

REFERENCES

(1) Terzaghi, *Theoretical Soil Mechanics*, John Wiley & Sons, Inc., New York, NY (1943).

(2) Terzaghi and Peck, *Soil Mechanics in Engineering Practice*, John Wiley & Sons, Inc., New York, NY (1948).

(3) Taylor, D. W., *Fundamentals of Soil Mechanics*, John Wiley & Sons, Inc., New York, NY (1948).

(4) Krynine, D. P., *Soil Mechanics*, 2nd Edition, McGraw-Hill Book Co., Inc., New York, NY (1947).

(5) Plummer and Dore, *Soil Mechanics and Foundations*, Pitman Publishing Corp., New York, NY (1940).

(6) Tolman, C. F., *Ground Water*, McGraw-Hill Book Co., Inc., New York, NY (1937).

(7) Stewart Sharpe, C. F., *Land Slides and Related Phenomena*, Columbia University Press, New York, NY (1938).

(8) "Letter Symbols and Glossary for Hydraulics with Special Reference to Irrigation," Special Committee on Irrigation Hydraulics, *Manual of Engineering Practice*, Am. Soc. Civil Engrs., No. 11 (1935).

(9) "Soil Mechanics Nomenclature," Committee of the Soil Mechanics and Foundations Division on Glossary of Terms and

Definitions and on Soil Classification, *Manual of Engineering Practice*, Am. Soc. Civil Engrs., No. 22 (1941).
(10) "Pile Foundations and Pile Structures," Joint Committee on Bearing Value of Pile Foundations of the Waterways Division, Construction Division, and Soil Mechanics and Foundations Division, *Manual of Engineering Practice*, Am. Soc. Civil Engrs., No. 27 (1956).
(11) Webster's New International Dictionary of the English Language, unabridged, 2nd Edition, G. and C. Merriam Co., Springfield, MA (1941).
(12) Baver, L. D., *Soil Physics*, John Wiley & Sons, Inc., New York, NY (1940).
(13) Longwell, Knopf and Flint, "Physical Geology," *Textbook of Geology*, Part I, 2nd Edition, John Wiley & Sons, Inc., New York, NY (1939).
(14) Runner, D. G., *Geology for Civil Engineers*, Gillette Publishing Co., Chicago, IL (1939).
(15) Leggett, R. F., *Geology and Engineering*, McGraw-Hill Book Co., Inc., New York, NY (1939).
(16) Holmes, A., *The Nomenclature of Petrology*, Thomas Murby and Co., London, England (1920).
(17) Meinzer, O. E., "Outline of Ground Water Hydrology with Definitions," *U. S. Geological Survey Water Supply Paper 494* (1923).
(18) "Reports of the Committee on Sedimentation of the Division of Geology and Geography of the National Research Council," Washington, DC (1930–1938).
(19) Twenhofel, W. H., *A Treatise on Sedimentation*, 2nd Edition, Williams & Wilkins Co., Baltimore, MD (1932).
(20) Hogentogler, C. A., *Engineering Properties of Soils*, McGraw Hill Book Co., Inc., New York, NY (1937).
(21) *Special Procedures for Testing Soil and Rock for Engineering Purposes, ASTM STP 479*, ASTM, 1970.
(22) "Glossary of Terms and Definitions," Preliminary Report of Subcommittee G-3 on Nomenclature and Definitions of ASTM Committee D-18 on Soils for Engineering Purposes.
(23) Sowers and Sowers, *Introductory Soil Mechanics and Foundations*, The Macmillan Co., New York, NY (1951).
(24) Lambe, T. William, *Soil Testing for Engineers*, John Wiley & Sons, Inc., New York, NY (1951).
(25) Capper and Cassie, *The Mechanics of Engineering Soils*, McGraw-Hill Book Co., Inc., New York, NY (1949).
(26) Dunham, C. W., *Foundations of Structures*, McGraw-Hill Book Co., Inc., New York, NY (1950).
(27) Casagrande, A., "Notes on Soil Mechanics," Graduate School of Engineering, Harvard University (1938).
(28) Tschebotarioff, G. P., *Soil Mechanics, Foundations, and Earth Structures*, McGraw-Hill Book Co., Inc., New York, (1951).
(29) Rice, C. M., "Dictionary of Geological Terms," Edwards Bros., Inc., Ann Arbor, MI (1940).
(30) Creager, Justin and Hinds, *Engineering for Dams*, John Wiley & Sons, Inc., New York, NY (1945).
(31) Krumbein and Sloss, *Stratigraphy and Sedimentation*, W. H. Freeman and Co., San Francisco, CA (1951).
(32) Pettijohn, F. J. *Sedimentary Rocks*, Harper and Bros., New York, NY (1949).
(33) Reiche, Parry, *A Survey of Weathering Processes and Products*, University of New Mexico Press, Albuquerque, NM (1945).
(34) Garrels, R. M., *A Textbook of Geology*, Harper and Bros., New York, NY (1951).
(35) Ries and Watson, *Engineering Geology*, John Wiley & Sons, Inc., New York, NY (1936).
(36) Ross and Hendricks, *Minerals of the Montmorillonite Group*, U. S. Geological Survey Professional Paper 205-B (1945).
(37) Hartman, R. J., *Colloid Chemistry*, Houghton Mifflin Co., New York, NY (1947).
(38) "Frost Investigations," Corps of Engineers, Frost Effects Laboratory, Boston, MA, June 1951.
(39) "Standard Specifications for Highway Materials and Methods of Sampling and Testing," Parts I and II, adopted by the American Association of State Highway Officials (1950).
(40) Coates, D. G., "Rock Mechanics Principles," rev ed. Mines Br., Dept. Mines and Tech. Surv., Ottawa, Mines Br. Mon. 874 (1970).
(41) Gary, M., McAfee, R., Jr., and Wolf, C. L., (eds.), *Glossary of Geology*, American Geological Institute (1972).
(42) International Society for Rock Mechanics, Commission on Terminology, Symbols and Graphic Representation, Final Document on Terminology, English Version, 1972, and List of Symbols, 1970.
(43) Jaeger, J. C., and Cook, N. G. W., *Fundamentals of Rock Mechanics*, Methuen, London (1969).
(44) Nelson, A., and Nelson, K. D., *Dictionary of Applied Geology* (Mining and Civil Engineering), Philosophical Library, Inc., New York, NY (1967).
(45) Obert, L. A., and Duvall, W. I., *Rock Mechanics and the Design of Structures in Rock*, John Wiley & Sons, New York, NY (1967).
(46) SME Mining Engineering Handbook: Society Mining Engineers, Vol 2, New York, NY (1973).
(47) Thrush, R. P. (ed), et al., *A Dictionary of Mining, Mineral and Related Terms*, U. S. Bureau of Mines (1968).
(48) Lohman, W. W., and others, *Definitions of Selected Ground-Water Terms—Revisions and Conceptual Refinements*, U. S. Geological Survey Water-Supply Paper 1988, 21 pp., 1972.
(49) *Glossary of Soil Science Terms*, Madison, WI, Soil Science Society of America, 34 pp. (1975).
(50) *International Glossary of Hydrology*, Geneva, Switzerland, World Meterological Organization, WMO No. 385, 393 pp., (1974).

Designation: C 294 – 86 (Reapproved 1991)[ε1]

Standard Descriptive Nomenclature for Constituents of Natural Mineral Aggregates[1]

This standard is issued under the fixed designation C 294; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This standard has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

ε1 Note—Section 25, Keywords, was added editorially in April 1991.

1. Scope

1.1 This descriptive nomenclature provides brief descriptions of some of the more common, or more important, natural materials of which mineral aggregates are composed (Note 1). The descriptions provide a basis for understanding these terms as used to designate aggregate constituents. Many of the materials described frequently occur in particles that do not display all the characteristics given in the descriptions, and most of these rocks grade from varieties meeting one description to varieties meeting another with all intermediate stages being found.

Note 1—These descriptions characterize minerals and rocks as they occur in nature and do not include blast-furnace slag or lightweight aggregates that are prepared by the alteration of the structure and composition of natural material. Blast-furnace slag is defined in Definitions C 125. Information about lightweight aggregates is given in Specifications C 330, C 331, and C 332.

1.2 The accurate identification of rocks and minerals can, in many cases, be made only by a qualified geologist, mineralogist, or petrographer using the apparatus and procedures of these sciences. Reference to these descriptions may, however, serve to indicate or prevent gross errors in identification. Identification of the constituent materials in an aggregate may assist in recognizing its properties, but identification alone cannot provide a basis for predicting the behavior of aggregates in service. Mineral aggregates composed of any type or combination of types of rocks and minerals may perform well or poorly in service depending upon the exposure to which they are subjected, the physical and chemical properties of the matrix in which they are embedded, their physical condition at the time they are used, and other factors. Small amounts of minerals or rocks that may occur only as contaminants or accessories in the aggregate may decisively influence its quality.

2. Referenced Documents

2.1 *ASTM Standards:*

C 125 Terminology Relating to Concrete and Concrete Aggregates[2]
C 289 Test Method for Potential Reactivity of Aggregates (Chemical Method)[2]
C 330 Specification for Lightweight Aggregates for Structural Concrete[2]
C 331 Specification for Lightweight Aggregates for Concrete Masonry Units[2]
C 332 Specification for Lightweight Aggregates for Insulating Concrete[2]

3. Classes and Types

3.1 The materials found as constituents of natural mineral aggregates are rocks and minerals. Minerals are naturally occurring inorganic substances of more or less definite chemical composition and usually of a specific crystalline structure. Most rocks are composed of several minerals but some are composed of only one mineral. Certain examples of the rock quartzite are composed exclusively of the mineral quartz, and certain limestones are composed exclusively of the mineral calcite. Individual sand grains frequently are composed of particles of rock, but they may be composed of a single mineral, particularly in the finer sizes.

3.2 Rocks are classified according to origin into three major divisions: igneous, sedimentary, and metamorphic. These three major groups are subdivided into types according to mineral and chemical composition, texture, and internal structure. *Igneous rocks* form from molten rock matter either above or below the earth's surface. *Sedimentary rocks* form at the earth's surface by the accumulation and consolidation of the products of weathering and erosion of existing rocks. *Metamorphic rocks* form from pre-existing rocks by the action of heat, pressure, or shearing forces in the earth's crust. It is obvious that not only igneous but also sedimentary and metamorphic rocks may be weathered and eroded to form new sedimentary rocks. Similarly, metamorphic rocks may again be metamorphosed.

DESCRIPTIONS OF MINERALS

4. General

4.1 For the purpose of indicating significant relationships, the descriptions of minerals are presented in groups in the following sections.

---

[1] This descriptive nomenclature is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.02.06 on Petrography of Concrete and Concrete Aggregates.

Current edition approved Nov. 28, 1986. Published January 1987. Originally published as C 294 – 52. Last previous edition C 294 – 69 (1981)[ε1].

This standard has been extensively revised. The reader should compare this edition with the last previous edition for exact revisions.

[2] *Annual Book of ASTM Standards*, Vol 04.02.

5. Silica Minerals

5.1 **quartz**—a very common hard mineral composed of silica (SiO_2). It will scratch glass and is not scratched by a knife. When pure it is colorless with a glassy (vitreous) luster and a shell-like (conchoidal) fracture. It lacks a visible cleavage (the ability to break in definite directions along even planes) and, when present in massive rocks such as granite, it usually has no characteristic shape. It is resistant to weathering and is therefore an important constituent of many sand and gravel deposits and many sandstones. It is also abundant in many light-colored igneous and metamorphic rocks. Some strained or intensely fractured (granulated) quartz may be deleteriously reactive with alkalies in concrete.

5.2 **opal**—a hydrous form of silica ($SiO_2 \cdot nH_2O$) which occurs without characteristic external form or internal crystalline arrangement as determined by ordinary visible light methods. When X-ray diffraction methods are used, opal may show some evidences of internal crystalline arrangement. Opal has a variable water content, generally ranging from 3 to 9 %. The specific gravity and hardness are always less than those of quartz. The color is variable and the luster is resinous to glassy. It is usually found in sedimentary rocks, especially some cherts, and is the principal constituent of diatomite. It is also found as a secondary material filling cavities and fissures in igneous rocks and may occur as a coating on gravel and sand. The recognition of opal in aggregates is important because it reacts with the alkalies in portland-cement paste, or with the alkalies from other sources, such as aggregates containing zeolites, and ground water.

5.3 **chalcedony**—chalcedony has been considered both as a distinct mineral and a variety of quartz. It is frequently composed of a mixture of microscopic fibers of quartz with a large number of submicroscopic pores filled with water and air. The properties of chalcedony are intermediate between those of opal and quartz, from which it can sometimes be distinguished only by laboratory tests. It frequently occurs as a constituent of the rock chert and is reactive with the alkalies in portland-cement paste.

5.4 **tridymite and cristobalite**—crystalline forms of silica (SiO_2) sometimes found in volcanic rocks. They are metastable at ordinary temperatures and pressures. They are rare minerals in aggregates except in areas where volcanic rocks are abundant. A type of cristobalite is a common constituent of opal. Tridymite and cristobalite are reactive with the alkalies in portland-cement paste.

6. Feldspars

6.1 The minerals of the feldspar group are the most abundant rock-forming minerals in the crust of the earth. They are important constituents of all three major rock groups, igneous, sedimentary, and metamorphic. Since all feldspars have good cleavages in two directions, particles of feldspar usually show several smooth surfaces. Frequently, the smooth cleavage surfaces show fine parallel lines. All feldspars are slightly less hard than, and can be scratched by, quartz and will, when fresh, easily scratch a penny. The various members of the group are differentiated by chemical composition and crystallographic properties. The feldspars *orthoclase*, *sanidine*, and *microcline* are potassium aluminum silicates, and are frequently referred to as *potassium feldspars*. The *plagioclase* feldspars include those that are sodium aluminum silicates and calcium aluminum silicates, or both sodium and calcium aluminum silicates. This group, frequently referred to as the "soda-lime" group, includes a continuous series, of varying chemical composition and optical properties, from *albite*, the sodium aluminum feldspar, to *anorthite*, the calcium aluminum feldspar, with intermediate members of the series designated *oligoclase*, *andesine*, *labradorite*, and *bytownite*. Potassium feldspars and sodium-rich plagioclase feldspars occur typically in igneous rocks such as granites and rhyolites, whereas, plagioclase feldspars of higher calcium content are found in igneous rocks of lower silica content such as diorite, gabbro, andesite, and basalt.

7. Ferromagnesian Minerals

7.1 Many igneous and metamorphic rocks contain dark green to black minerals that are generally silicates of iron or magnesium, or of both. They include the minerals of the amphibole, pyroxene, and olivine groups. The most common amphibole mineral is hornblende; the most common pyroxene mineral is augite; and the most common olivine mineral is olivine. Dark mica, such as biotite and phlogopite, are also considered ferromagnesian minerals. The amphibole and pyroxene minerals are brown to green to black and generally occur as prismatic units. Olivine is usually olive green, glassy in appearance, and usually altered. Biotite has excellent cleavage and can be easily cleaved into thin flakes and plates. These minerals can be found as components of a variety of rocks, and in sands and gravels. Olivine is found only in dark igneous rocks where quartz is not present, and in sands and gravels close to the olivine source.

8. Micaceous Minerals

8.1 Micaceous minerals have perfect cleavage in one direction and can be easily split into thin flakes. The mica minerals of the muscovite group are colorless to light green; of the biotite group, dark brown to black or dark green; of the lepidolite group, white to pink and red or yellow; and of the chlorite group, shades of green. Another mica, phlogopite, is similar to biotite, commonly has a pearl-like luster and bronze color, and less commonly is brownish red, green, or yellow. The mica minerals are common and occur in igneous, sedimentary, and metamorphic rocks, and are common as minor to trace components in many sands and gravels. The muscovite, biotite, lepidolite, and phlogopite minerals cleave into flakes and plates that are elastic; the chlorite minerals, by comparison, form in elastic flakes and plates. Vermiculite (a mica-like mineral) forms by the alteration of other micas and is brown and has a bronze luster.

9. Clay Minerals

9.1 The term "clay" refers to natural material composed of particles in a specific size range, generally less than 2 μm (0.002 mm). Mineralogically, clay refers to a group of layered silicate minerals including the clay-micas (illites), the kaolin group, very finely divided chlorites, and the swelling clays—montmorillonites (smectites). Members of several groups, particularly micas, chlorites, and vermiculites, occur both in the clay-size range and in larger sizes. Some clays are made up of alternating layers of two or more clay groups. Random, regular, or both types of interlayering are known. If smectite is a significant constituent in such mixtures, then fairly large volume changes may occur with wetting and drying.

9.2 Clay minerals are hydrous aluminum, magnesium, and iron silicates that may contain calcium, magnesium, potassium, sodium and other exchangeable cations. They are formed by alteration and weathering of other silicates and volcanic glass. The clay minerals are major constituents of clays and shales. They are found disseminated in carbonate rocks as seams and pockets and in altered and weathered igneous and metamorphic rocks. Clays may also be found as matrix, void fillings, and cementing material in sandstones and other sedimentary rocks.

9.3 Most aggregate particles composed of, or containing large proportions of clay minerals are soft and, because of the large internal surface area of the constituents, they are porous. Some of these aggregates will disintegrate when wetted. Rocks in which the cementing matrix is principally clay, such as clay-bonded sandstones, and rocks in which montmorillonite is present as a continuous phase or matrix, such as some altered volcanics, may slake in water or may disintegrate in the concrete mixer. Rocks of this type are unsuitable for use as aggregates. Rocks having these properties less well developed will abrade considerably during mixing, releasing clay, and raising the water requirement of the concrete containing them. When such rocks are present in hardened concrete, the concrete will manifest greater volume change on wetting and drying than similar concrete containing non-swelling aggregate.

10. Zeolites

10.1 The zeolite minerals are a large group of hydrated aluminum silicates of the alkali and alkaline earth elements which are soft and usually white or light colored. They are formed as a secondary filling in cavities or fissures in igneous rocks, or within the rock itself as a product of hydrothermal alteration of original minerals, especially feldspars. Some zeolites, particularly *heulandite*, *natrolite*, and *laumontite*, reportedly produce deleterious effects in concrete, the first two having been reported to augment the alkali content in concrete by releasing alkalies through cation exchange and thus increasing alkali reactivity when certain siliceous aggregates are present. Laumontite and its partially dehydrated variety *leonhardite* are notable for their substantial volume change with wetting and drying. Both are found in rocks such as quartz diorites and some sandstones.

11. Carbonate Minerals

11.1 The most common carbonate mineral is *calcite* (calcium carbonate, $CaCO_3$). The mineral *dolomite* consists of calcium carbonate and magnesium carbonate ($CaCO_3 \cdot MgCO_3$ or $CaMg(CO_3)_2$) in equivalent molecular amounts, which are 54.27 and 45.73 by mass %, respectively. Both calcite and dolomite are relatively soft, the hardness of calcite being 3 and that of dolomite 3½ to 4 on the Mohs scale, and are readily scratched by a knife blade. They have rhombohedral cleavage, which results in their breaking into fragments with smooth parallelogram shaped sides. Calcite is soluble with vigorous effervescence in cold dilute hydrochloric acid; dolomite is soluble with slow effervescence in cold dilute hydrochloric acid and with vigorous effervescence if the acid or the sample is heated or if the sample is pulverized.

12. Sulfate Minerals

12.1 Carbonate rocks and shales may contain sulfates as impurities. The most abundant sulfate mineral is *gypsum* (hydrous calcium sulfate; $CaSO_4 \cdot 2H_2O$); *anhydrite* (anhydrous calcium sulfate, $CaSO_4$) is less common. Gypsum is usually white or colorless and characterized by a perfect cleavage along one plane and by its softness, representing hardness of 2 on the Mohs scale; it is readily scratched by the fingernail. Gypsum may form a whitish pulverulent or crystalline coating on sand and gravel. It is slightly soluble in water.

12.2 Anhydrite resembles dolomite in hand specimen but has three cleavages at right angles; it is less soluble in hydrochloric acid than dolomite, does not effervesce and is slightly soluble in water. Anhydrite is harder than gypsum. Gypsum and anhydrite occurring in aggregates offer risks of sulfate attack in concrete and mortar.

13. Iron Sulfide Minerals

13.1 The sulfides of iron, *pyrite*, *marcasite*, and *pyrrhotite* are frequently found in natural aggregates. Pyrite is found in igneous, sedimentary, and metamorphic rocks; marcasite is much less common and is found mainly in sedimentary rocks; pyrrhotite is less common but may be found in many types of igneous and metamorphic rocks. Pyrite is brass yellow, and pyrrhotite bronze brown, and both have a metallic luster. Marcasite is also metallic but lighter in color and finely divided iron sulfides are soot black. Pyrite is often found in cubic crystals. Marcasite readily oxidizes with the liberation of sulfuric acid and formation of iron oxides, hydroxides, and, to a much smaller extent, sulfates; pyrite and pyrrhotite do so less readily. Marcasite and certain forms of pyrite and pyrrhotite are reactive in mortar and concrete, producing a brown stain accompanied by a volume increase that has been reported as one source of popouts in concrete. Reactive forms of iron sulfides may be recognized by immersion in saturated lime water (calcium hydroxide solution); upon exposure to air the reactive varieties produce a brown coating within a few minutes.

14. Iron Oxide Minerals, Anhydrous and Hydrous

14.1 There are two common iron oxide minerals: (*1*) Black, magnetic: *magnetite* (Fe_3O_4), and (*2*) red or reddish when powdered: *hematite* (Fe_2O_3); and one common hydrous oxide mineral, brown or yellowish: *goethite* ($FeO(OH)$). Another common iron-bearing mineral is black, weakly magnetic, *ilmenite* ($FeTiO_3$). Magnetite and ilmenite are important accessory minerals in many dark igneous rocks and are common detrital minerals in sediments. Hematite is frequently found as an accessory mineral in reddish rocks. Limonite, the brown weathering product of iron-bearing minerals, is a field name for a variety of hydrous iron oxide minerals including goethite; it frequently contains adsorbed water, and various impurities such as colloidal or crystalline silica, clay minerals, and organic matter. The presence of substantial amounts of soft iron-oxide minerals in concrete aggregate can color concrete various shades of yellow or brown. Very minor amounts of iron minerals color many rocks, such as ferruginous sandstones, shales, clay-ironstones, and granites. Ma... :, ilmenite, and hematite ores are used as heavy aggregates.

DESCRIPTIONS OF IGNEOUS ROCKS

15. General

15.1 Igneous rocks are those formed by cooling from a molten rock mass (magma). They may be divided into two classes: (*1*) plutonic, or intrusive, that have cooled slowly within the earth; and (*2*) volcanic, or extrusive, that formed from quickly cooled lavas. Plutonic rocks have grain sizes greater than approximately 1 mm, and are classified as coarse- or medium-grained. Volcanic rocks have grain sizes less than approximately 1 mm, and are classified as fine-grained. Volcanic rocks frequently contain glass. Both plutonic and volcanic rocks may consist of porphyries, that are characterized by the presence of large mineral grains in a fine-grained or glassy groundmass. This is the result of sharp changes in rate of cooling or other physico-chemical conditions during solidification of the melt.

15.2 Igneous rocks are usually classified and named on the basis of their texture, internal structure, and their mineral composition which in turn depends to a large extent on their chemical composition. Rocks in the plutonic class generally have chemical equivalents in the volcanic class.

16. Plutonic Rocks

16.1 **granite**—granite is a medium- to coarse-grained, light-colored rock characterized by the presence of potassium feldspar with lesser amounts of plagioclase feldspars and quartz. The characteristic potassium feldspars are orthoclase or microcline, or both; the common plagioclase feldspars are albite and oligoclase. Feldspars are more abundant than quartz. Dark-colored mica (biotite) is usually present, and light-colored mica (muscovite) is frequently present. Other dark-colored ferromagnesian minerals, especially hornblende, may be present in amounts less than those of the light-colored constituents. *Quartz-monzonite* and *granodiorite* are rocks similar to granite, but they contain more plagioclase feldspar than potassium feldspar.

16.2 **syenite**—syenite is a medium- to coarse-grained, light-colored rock composed essentially of alkali feldspars, namely microcline, orthoclase, or albite. Quartz is generally absent. Dark ferromagnesian minerals such as hornblende, biotite, or pyroxene are usually present.

16.3 **diorite**—diorite is a medium- to coarse-grained rock composed essentially of plagioclase feldspar and one or more ferromagnesian minerals such as hornblende, biotite, or pyroxene. The plagioclase is intermediate in composition, usually of the variety andesine, and is more abundant than the ferromagnesian minerals. Diorite usually is darker in color than granite or syenite and lighter than gabbro. If quartz is present, the rock is called *quartz diorite*.

16.4 **gabbro**—gabbro is a medium- to coarse-grained, dark-colored rock consisting essentially of ferromagnesian minerals and plagioclase feldspar. The ferromagnesian minerals may be pyroxenes, amphiboles, or both. The plagioclase is one of the calcium-rich varieties, namely labradorite, bytownite, or anorthite. Ferromagnesian minerals are usually more abundant than feldspar. *Diabase* (in European usage *dolerite*) is a rock of similar composition to gabbro and basalt but is intermediate in mode of origin, usually occurring in smaller intrusions than gabbro, and having a medium to fine-grained texture. The terms "trap" or "trap rock" are collective terms for dark-colored, medium- to fine-grained igneous rocks especially diabase and basalt.

16.5 **peridotite**—peridotite is composed of olivine and pyroxene. Rocks composed almost entirely of pyroxene are known as *pyroxenites*, and those composed of olivine as *dunites*. Rocks of these types are relatively rare but their metamorphosed equivalent, serpentinite, is more common.

16.6 **pegmatite**—extremely coarse-grained varieties of igneous rocks are known as pegmatites. These are *usually* light-colored and are most frequently equivalent to granite or syenite in mineral composition.

17. Fine-Grained and Glassy Extrusive Igneous Rocks

17.1 **volcanic rock**—volcanic or extrusive rocks are the fine-grained equivalents of the coarse-and-medium-grained plutonic rocks described in Section 16. Equivalent types have similar chemical compositions and may contain the same minerals. Volcanic rocks commonly are so fine-grained that the individual mineral grains usually are not visible to the naked eye. Porphyritic textures are common, and the rocks may be partially or wholly glassy or non-crystalline. The glassy portion of a partially glassy rock usually has a higher silica content than the crystalline portion. Some volcanic or extrusive rocks may not be distinguishable in texture and structure from plutonic or intrusive rocks that originated at shallow depth.

17.2 **felsite**—light-colored, very fine-grained igneous rocks are collectively known as felsites. The felsite group includes *rhyolite, dacite, andesite*, and *trachyte*, which are the equivalents of granite, quartz diorite, diorite, and syenite, respectively. These rocks are usually light colored but they may be gray, green, dark red, or black. When they are dark they may incorrectly be classed as "trap" (see 16.4). When they are microcrystalline or contain natural glass, rhyolites, dacites, and andesites are reactive with the alkalies in portland-cement concrete.

17.3 **basalt**—fine-grained extrusive equivalent of gabbro and diabase. When basalt contains natural glass, the glass is generally lower in silica content than that of the lighter-colored extrusive rocks and hence is not deleteriously reactive with the alkalies in portland-cement paste; however, exceptions have been noted in the literature with respect to the alkali reactivity of basaltic glasses.

17.4 **volcanic glass**—igneous rocks composed wholly of glass are named on the basis of their texture and internal structure. A dense dark natural glass of high silica content is called *obsidian*, while lighter colored finely vesicular glassy froth filled with elongated, tubular bubbles is called *pumice*. Dark-colored coarsely vesicular types containing more or less spherical bubbles are called *scoria*. Pumices are usually silica-rich (corresponding to rhyolites or dacites), whereas scorias usually are more basic (corresponding to basalts). A high-silica glassy lava with an onion-like structure and a pearly luster, containing 2 to 5 % water, is called *perlite*. When heated quickly to the softening temperature, perlite puffs to become an artificial pumice. Glass with up to 10 % water and with a dull resinous luster is called *pitchstone*. Glassy rocks, particularly the more siliceous ones, are reactive with the alkalies in portland-cement paste.

DESCRIPTIONS OF SEDIMENTARY ROCKS

18. General

18.1 Sedimentary rocks are stratified rocks laid down for the most part under water, although wind and glacial action occasionally are important. Sediments may be composed of particles of preexisting rocks derived by mechanical agencies or they may be of chemical or organic origin. The sediments are usually indurated by cementation or compaction during geologic time, although the degree of consolidation may vary widely.

18.2 *Gravel, sand, silt,* and *clay* form the group of unconsolidated sediments. Although the distinction between these four members is made on the basis of their particle size, a general trend in the composition occurs. Gravel and, to a lesser degree, coarse sands usually consist of rock fragments; fine sands and silt consist predominantly of mineral grains; and clay exclusively of mineral grains, largely of the group of clay minerals. All types of rocks and minerals may be represented in unconsolidated sediments.

19. Conglomerates, Sandstones, and Quartzites

19.1 These rocks consist of particles of *sand* or *gravel*, or both, with or without interstitial and cementing material. If the particles include a considerable proportion of gravel, the rock is a *conglomerate*. If the particles are in the sand sizes, that is, less than 2 mm but more than 0.06 mm in major diameter, the rock is a *sandstone* or a *quartzite*. If the rock breaks around the sand grains, it is a sandstone; if the grains are largely quartz and the rock breaks through the grains, it is quartzite. Conglomerates, and sandstones are sedimentary rocks but quartzites may be sedimentary (*orthoquartzites*) or metamorphic (*metaquartzites*). The cementing or interstitial materials of sandstones may be quartz, opal, calcite, dolomite, clay, iron oxides, or other materials. These may influence the quality of a sandstone as concrete aggregate. If the nature of the cementing material is known, the rock name may include a reference to it, such as *opal-bonded sandstone* or *ferruginous conglomerate*.

19.2 **graywackes and subgraywackes**—gray to greenish gray sandstones containing angular quartz and feldspar grains, and sand-sized rock fragments in an abundant matrix resembling claystone, shale, argillite, or slate. Graywackes grade into subgraywackes, the most common sandstones of the geologic column.

19.3 arkose—coarse-grained sandstone derived from granite, containing conspicuous amounts of feldspar.

20. Claystones, Shales, Argillites, and Siltstones

20.1 These very fine-grained rocks are composed of, or derived by erosion of sedimentary *silts* and *clays*, or of any type of rock that contained clay. When relatively soft and massive, they are known as *claystones*, or *siltstones*, depending on the size of the majority of the particles of which they are composed. Siltstones consist predominantly of silt-sized particles (0.0625 to 0.002 mm in diameter) and are intermediate rocks between claystones and sandstones. When the claystones are harder and platy or fissile, they are known as *shales*. Claystones and shales may be gray, black, reddish, or green and may contain some carbonate minerals (calcareous shales). A massive, firmly indurated fine-grained argillaceous rock consisting of quartz, feldspar, and micaceous minerals is known as *argillite*. Argillites do not slake in water as some shales do. As an aid in distinguishing these fine-grained sediments from fine-grained, foliated metamorphic rocks such as slates and phyllites, it may be noted that the cleavage surfaces of shales are generally dull and earthy while those of slates are more lustrous. Phyllite has a glossier luster resembling a silky sheen.

20.2 Aggregates containing abundant shale are detrimental to concrete because they can produce high shrinkage, but not all shales are harmful. Some argillites are alkali-silica reactive.

20.3 Although aggregates which are volumetrically unstable in wetting and drying are not confined to any class of rock, they do share some common characteristics. If there is a matrix or continuous phase, it is usually physically weak and consists of material of high specific surface, frequently including clay. However, no general relation has been demonstrated between clay content or type of clay and large volume change upon wetting and drying. Volumetrically unstable aggregates do not have mineral grains of high modulus interlocked in a continuous rigid structure capable of resisting volume change.

20.4 Aggregates having high elastic modulus and low volume change from the wet to the dry condition contribute to the volume stability of concrete by restraining the volume change of the cement paste. In a relatively few cases, aggregates have been demonstrated to contribute to unsatisfactory performance of concrete because they have relatively large volume change from the wet to the dry condition combined with relatively low modulus of elasticity. On drying, such aggregates shrink away from the surrounding cement paste and consequently fail to restrain its volume change with change in moisture content.

21. Carbonate Rocks

21.1 *Limestones* are the most widespread of carbonate rocks. They range from pure limestones consisting of the mineral calcite to pure *dolomites* (dolostones) consisting of the mineral dolomite. Usually they contain both minerals in various proportions. If 50 to 90 % is the mineral dolomite, the rock is called *calcitic dolomite*. Magnesium limestone is sometimes applied to dolomitic limestones and calcitic dolomites but it is ambiguous and its use should be avoided. Most carbonate rocks contain some noncarbonate impurities such as quartz, chert, clay minerals, organic matter, gypsum, and sulfides. Carbonate rocks containing 10 to 50 % sand are *arenaceous* (or *sandy*) *limestones* (or *dolomites*); those containing 10 to 50 % clay are *argillaceous* (or *clayey* or *shaly*) *limestones* (or *dolomites*). *Marl* is a clayey limestone which is fine-grained and commonly soft. Chalk is fine-textured, very soft, porous, and somewhat friable limestone, composed chiefly of particles of microorganisms. Micrite is very fine-textured chemically precipitated carbonate or a mechanical ooze of carbonate particles, usually 0.001 to 0.003 mm in size. The term "limerock" is not recommended.

21.2 The reaction of the dolomite in certain carbonate rocks with alkalies in portland cement paste has been found to be associated with deleterious expansion of concrete containing such rocks as coarse aggregate. Carbonate rocks capable of such reaction possess a characteristic texture and composition. The characteristic microscopic texture is that in which relatively large crystals of dolomite (rhombs) are scattered in a finer-grained matrix of micritic calcite and clay. The characteristic composition is that in which the carbonate portion consists of substantial amounts of both dolomite and calcite, and the acid-insoluble residue contains a significant amount of clay. Except in certain areas, such rocks are of relatively infrequent occurrence and seldom make up a significant proportion of the material present in a deposit of rock being considered for use in making aggregate for concrete.

22. Chert

22.1 **chert**—the general term for a group of variously colored, very fine-grained (aphanitic), siliceous rocks composed of microcrystalline or cryptocrystalline quartz, chalcedony, or opal, either singly or in combinations of varying proportions. Identification of the form or forms of silica requires careful determination of optical properties, absolute specific gravity, loss on ignition, or a combination of these characteristics. Dense cherts are very tough, with a waxy to greasy luster, and are usually gray, brown, white, or red, and less frequently, green, black or blue. Porous varieties are usually lighter in color, frequently off-white, or stained yellowish, brownish, or reddish, firm to very weak, and grade to tripoli. Ferruginous, dense, red, and in some cases, dense, yellow, brown, or green chert is sometimes called *jasper*. Dense black or gray chert is sometimes called *flint*. A very dense, even textured, light gray to white chert, composed mostly of microcrystalline to cryptocrystalline quartz, is called *novaculite*. Chert is hard (scratches glass, but is not scratched by a knife blade) and has a conchoidal (shell-like) fracture in the dense varieties, and a more splintery fracture in the porous varieties. Chert occurs most frequently as nodules, lenses, or interstitial material, in limestone and dolomite formations, as extensively bedded deposits, and as components of sand and gravel. Most cherts have been found to be alkali-silica reactive to some degree when tested with high-alkali cement, or in the quick chemical test (Test Method C 289). However, the degree of the alkali-silica reactivity and whether a given chert will produce a deleterious degree of expansion in concrete are complex functions of several factors. Among them are: the mineralogic composition and internal structure of the chert; the amount of the chert as a proportion of the aggregates; the particle-size distribution; the alkali content of the cement; and the cement content of the concrete. In the absence of information to the contrary, all chert should be regarded as potentially alkali-silica reactive if combined with high-alkali cement. However, opaline cherts may produce deleterious expansion of mortar or concrete when present in very small proportions (less than 5 % by mass of the aggregate). Cherts that are porous may be susceptible to freezing and thawing deterioration in concrete and may cause popouts or cracking of the concrete surface above the chert particle.

DESCRIPTIONS OF METAMORPHIC ROCKS

23. General

23.1 Metamorphic rocks form from igneous, sedimentary, or pre-existing metamorphic rocks in response to changes in chemical and physical conditions occurring within the earth's crust after formation of the original rock. The changes may be textural, structural, or mineralogic and may be accompanied by changes in chemical composition. The rocks are dense and may be massive but are more frequently foliated (laminated or layered) and tend to break into platy particles. Rocks formed from argillaceous rocks by dynamic metamorphism usually split easily along one plane independent of original bedding; this feature is designated "platy cleavage." The mineral composition is very variable depending in part on the degree of metamorphism and in part on the composition of the original rock.

23.2 Most of the metamorphic rocks may derive either from igneous or sedimentary rocks but a few, such as marbles and slates, originate only from sediments.

23.3 Certain phyllites, slates, and metaquartzites containing low-temperature silica and silicate minerals or highly strained quartz may be deleteriously reactive when used with cements of high alkali contents.

24. Metamorphic Rocks

24.1 **marble**—a recrystallized medium- to coarse-grained carbonate rock composed of calcite or dolomite, or calcite and dolomite. The original impurities are present in the form of new minerals, such as micas, amphiboles, pyroxenes, and graphite.

24.2 **metaquartzite**—a granular rock consisting essentially of recrystallized quartz. Its strength and resistance to weathering derive from the interlocking of the quartz grains.

24.3 **slate**—a fine-grained metamorphic rock that is distinctly laminated and tends to split into thin parallel layers. The mineral composition usually cannot be determined with the unaided eye.

24.4 **phyllite**—a fine-grained thinly layered rock. Minerals, such as micas and chlorite, are noticeable and impart a silky sheen to the surface of schistosity. Phyllites are intermediate between slates and schists in grain size and mineral composition. They derive from argillaceous sedimentary rocks or fine-grained extrusive igneous rocks, such as felsites.

24.5 **schist**—a highly layered rock tending to split into nearly parallel planes (schistose) in which the grain is coarse enough to permit identification of the principal minerals. Schists are subdivided into varieties on the basis of the most prominent mineral present in addition to quartz or to quartz and feldspars; for instance, *mica schist*. *Greenschist* is a green schistose rock whose color is due to abundance of one or more of the green minerals, chlorite or amphibole, and is commonly derived from altered volcanic rock.

24.6 **amphibolite**—a medium- to coarse-grained dark-colored rock composed mainly of hornblende and plagioclase feldspar. Its schistosity, which is due to parallel alignment of hornblende grains, is commonly less obvious than in typical schists.

24.7 **hornfels**—equigranular, massive, and usually tough rock produced by complete recrystallization of sedimentary, igneous, or metamorphic rocks through thermal metamorphism sometimes with the addition of components of molten rock. Their mineral compositions vary widely.

24.8 **gneiss**—one of the most common metamorphic rocks, usually formed from igneous or sedimentary rocks by a higher degree of metamorphism than the schists. It is characterized by a layered or foliated structure resulting from approximately parallel lenses and bands of platy minerals, usually micas, or prisms, usually amphiboles, and of granular minerals, usually quartz and feldspars. All intermediate varieties between gneiss and schist, and between gneiss and granite are often found in the same areas in which well-defined gneisses occur.

24.9 **serpentinite**—a relatively soft, light to dark green to almost black rock formed usually from silica-poor igneous rocks, such as pyroxenites, peridotites, and dunites. It may contain some of the original pyroxene or olivine but is largely composed of softer hydrous ferromagnesian minerals of the serpentine group. Very soft talc-like material is often present in serpentinite.

25. Keywords

25.1 aggregates; carbonates; clays; concrete; feldspars; ferromagnesian minerals; igneous rocks; iron oxides; iron sulfides; metamorphic rocks; micas; minerals; nomenclature; rocks; sedimentary rocks; silica; sulfates; zeolites

RTH 102-93

RECOMMENDED PRACTICE FOR PETROGRAPHIC EXAMINATION OF ROCK CORES

1. Scope

1.1 This recommended practice outlines procedures for the petrographic examination of rock cores whose engineering properties can be determined by selected tests. The specific procedures employed in the petrographic examination of any sample will depend to a large extent on the purpose of the examination* and the nature of the sample. Complete petrographic examination may require use of such procedures as light microscopy, X-ray diffraction analysis, differential thermal analysis, infrared spectroscopy, or others; in some instances, such procedures are more rapid and more definitive than are microscopical methods. Petrographic examinations are made for the following purposes:

(a) To determine the physical and chemical properties of a material, by petrographic methods, that have a bearing on the quality of the material for its intended use.

(b) To describe and classify the constituents of the sample. (Note 1)

(c) To determine the relative amounts of the constituents of the sample, which is essential for proper evaluation of the sample, when the constituents differ significantly in properties that have a bearing on the quality of the material for its intended use.

NOTE 1--It is recommended that the rock and mineral names in "Descriptive Nomenclature of Constituents of Natural Mineral Aggregates" (ASTM Designation: C 294) be used insofar as they are appropriate in reports prepared according to this recommended practice.

---

*The practices described herein are applicable to the examination of rock cores from rock used as foundation or other similar purposes. However, if the cores are from rock proposed to be quarried for use as concrete aggregate or erosion control, reference should be made to ASTM C 295 or D 4992, respectively.

1.2 Detection of structural features and identification of the constituents of a sample are usually necessary steps toward recognition of the properties that may be expected to influence the behavior of the material in its intended use. However, the value of any petrographic examination will depend to a large extent on the representativeness of the samples examined, the completeness and accuracy of the information provided to the petrographer concerning the source and proposed use of the material, and the petrographer's ability to correlate these data with the findings of the examination.

1.3 This recommended practice does not attempt to outline the techniques of petrographic work since it is assumed that the method will be used by persons who are qualified by education and/or experience to employ such techniques for the recognition of the characteristic properties of rocks and minerals and to describe and classify the constituents of a sample. It is intended to outline the extent to which such techniques should be used, the selection of properties that should be looked for, and the manner in which such techniques may best be employed in the examination. These objectives will have been attained if engineers responsible for the application of the results of petrographic examinations have reasonable assurance that such results, wherever and whenever obtained, may confidently be compared.

2. Sampling and Examination Procedure

2.1 The purpose in specifying a sampling procedure is to ensure the selection of an adequate group of test specimens from each mechanically different rock type that forms an essential part of the core or cores that will be tested. The sampling procedure should also be guided by the project objectives and directed at obtaining the properties of the rock that eventually will comprise the roof, side walls, foundation, or other specific parts of the project structure. Samples for petrographic examination should be taken by or under the direct supervision of a geologist familiar with the requirements of the project. The exact location from which the sample was taken, the geology of the site, and other pertinent data should be submitted with the sample. The amount of material actually studied in the petrographic examination will be determined by the nature of the material to be examined. Areas to be studied should be sampled by means of cores drilled through the entire depth required for project investigation. Drilling of such cores should be in a direction that is essentially normal to the dominant structural features of the rock. Massive material may be sampled by "NX"(2-1/8-in.-diam) (54-mm-diam) cores. Thinly bedded or complex material should be represented by cores not less than 4 in. (100 mm) in diameter, preferably 6 in. (150 mm). There should be an adequate number of cores to cover the limits of the rock mass under consideration.

2.2 The following is considered a preferable but not a mandatory procedure. A petrographer should inspect all of the rock core before any tests are made. Each core should be logged to show footage of core recovered, core loss, and location; location and spacing of fractures and parting planes; lithologic type or types; alternation of types; physical conditions and variations in conditions; toughness, hardness, coherence; obvious porosity; grain size, texture, variations in grain size and texture; type or types of breakage. If the surface of the core being examined is wetted, it is usually easier to recognize significant features and changes in lithology. Most of the information usually required can be obtained by careful visual examination, scratch and acid tests, and hitting the core with a hammer. A preliminary analysis of the test results may indicate that the results from one or another of the subdivisions are not significantly different and the groups may be combined. On the other hand, the analysis may disclose significant differences within a given group of specimens and a further subdivision may be required. Most rock is anisotropic and, if the core stock and sample procedure permit, a group of specimens should be obtained from the three mutually perpendicular directions. Usually these directions are oriented with respect to some petrographic property of the rocks such as bedding, schistosity, cleavage, or fabric. In bedded rock the greatest difference in properties occurs in specimens taken perpendicular and parallel to the bedding, and generally this type of rock is sampled only in these two directions. The petrographic examination may disclose mineral components that are soluble or that expand or soften in water, as for example, bentonites or other clays. The intent of this inspection is to provide a basis for the selection of samples for engineering tests. This basis will be rock types, amounts of rock types, differences within a rock type, etc. At this point, the petrographer, in conjunction with the project leader, should select the sections of core(s) that will be subjected to engineering tests. The detailed petrographic examination will usually be made on unused portions of some or all of the test pieces so that the petrographic data can be matched to the engineering data. This matching should mean that, in addition to petrographic characterization, the petrographic data should serve as a basis for understanding the physical test data within and between sample groups.

3. Apparatus and Supplies

3.1 The apparatus and supplies listed in the following subparagraphs (a) and (b) comprise a recommended selection which will permit the use of all of the procedures described in this recommended practice. All specific items have been used in connection with the performance of petrographic examinations by the procedures described herein; it is not, however, intended to imply that other items cannot be substituted to serve similar functions. Whenever possible the selection of particular apparatus and supplies should be left to the judgment of the petrographer who is to perform the work so that items obtained will be those with which he has the greatest experience and familiarity. The minimum equipment regarded as essential to the making of petrographic examinations are those items, or equivalent apparatus or supplies that will serve the same purpose, that are indicated by asterisks in the lists in subparagraphs (a) and (b).

(a) Apparatus and Supplies for Preparation of Specimens:

(1) Rock-Cutting Saw,* preferably with a 20-in. (508-mm) diameter blade or larger.

(2) Horizontal Grinding Wheel,* preferably 16 in. (400 mm) in diameter.

(3) Polishing Wheel, preferably 8 to 12 in. (200 to 300 mm) in diameter.

(4) Abrasives:* silicon carbide grit Nos. 100, 220, 320, 600, and 800; optical finishing alumina.

(5) Geologist's pick or hammer.

(6) Microscope Slides,* clear, noncorrosive, 25 by 45 mm in size.

(7) Canada Balsam,* neutral in xylene or other material to cement cover slips.

(8) Xylene.*

(9) Mounting Medium,* suitable for mounting rock slices for thin sections.

(10) Laboratory Oven.*

(11) Plate-Glass Squares,* about 12 in. (300 mm) on an edge for thin-section grinding.

(12) Micro Cover Glasses,* No. 1 noncorrosive, square, 12 to 12 mm, 25 mm, etc.

(13) Plattner mortar.

(b) Apparatus and Supplies for Examination of Specimens:

(1) Polarizing Microscope* with mechanical stage; low-, medium-, and high-power objectives, and objective centering devices; eyepieces of various powers; full- and quarter-wave compensators; quartz wedge.

(2) Microscope Lamps* (preferably including a sodium arc lamp).

(3) Stereoscopic Microscope* with objectives and oculars to give final magnifications from about 7X to about 140X.

(4) Magnet,* preferably Alnico, or an electromagnet.

(5) Needleholder and Points.*

(6) Dropping Bottles, 60-ml capacity.

(7) Forceps, smooth straight-pointed.

(8) Lens Paper.*

(9) Immersion Media,* $n = 1.410$ to $n = 1.785$ in steps of 0.005. (Note 2)

(10) Counter.

(11) Photomicrographic Camera and accessories. (Note 3)

(12) X-ray diffractometer.

(13) Differential thermal analysis system.

(14) Infrared absorption spectrometer.

NOTE 2--It is necessary that facilities be available to the petrographer to check the index of refraction of the immersion media. If accurate identification of materials is to be attempted, as for example the differentiation of quartz and chalcedony or the differentiation of basic from intermediate volcanic glass, the indices of refraction of the media need to be known with precision. Media will not be stable for very long periods of time and are subject to considerable variation due to temperature change. In laboratories not provided with close temperature control, it is often necessary to recalibrate immersion media several times during the course of a single day when accurate identifications are required. The equipment needed for checking immersion media consists of an Abbe Refractometer. The refractometer should be equipped with compensating prisms to read indices for sodium light from white light, or it should be used with a sodium arc lamp.

NOTE 3--It is believed that a laboratory that undertakes any considerable amount of petrographic work should be provided with facilities to make photomicrographic records of such features as cannot adequately be described in words. Photomicrographs can be taken using standard microscope lamps for illumination; however, it is recommended that whenever possible a zirconium arc lamp be provided for this purpose. For illustrations of typical apparatus, reference may be made to the paper by Mather and Mather.[1]

4. Report

4.1 First and foremost the report should be clear and useful to the engineer for whom it is intended. It should identify samples, give their source as appropriate, describe test procedures and equipment used as appropriate, describe the samples, and list the petrographic findings. Tabulations of data and photographs should be included as needed. Results that may bear on the engineering test data and the potential performance of the material should be clearly stated and their significance should be emphasized. It may also be appropriate to mention past performance records of the same or similar mate-

---

[1]This recommended practice is modified from the "Method of Petrographic Examination of Aggregates for Concrete," by Katharine Mather and Bryant Mather. Proceedings, American Society for Testing Materials, ASTM, Vol 50, 1950, pp 1288-1312.

rials if such information is available. In general, the report should be an objective statement. If any opinion is presented it should be clearly indicated to be an opinion. Finally, the petrographic report should make recommendations if and as appropriate.

5. References

5.1 ASTM Standards

C 294 (RTH 116) Descriptive Nomenclature for Constituents of Natural Mineral Aggregate

C 295 Guide for Petrographic Examination of Aggregates for Concrete

D 4992 Practice for Evaluation of Rock to be Used for Erosion Control

PREPARATION OF TEST SPECIMENS

1. Scope

1.1 In order to obtain valid results from tests on brittle materials, careful and precise specimen preparation is required. This method outlines preparatory procedures recommended for normal rock mechanics test progress.

2. Collection and Storage

2.1 Test material is normally collected from the field in the form of drilled cores. Field sampling procedures should be rational and systematic, and the material should be marked to indicate its original position and orientation relative to identifiable boundaries of the parent rock mass. Ideally, samples should be moistureproofed immediately after collection either by waxing, spraying, or packing in polyethylene bags or sheet. (Example: For moistureproofing by waxing, the following procedure can be used for core that will not fail apart in handling.

(a) Wrap core in a clear thin polyethylene such as GLAD WRAP or SARAN WRAP,

(b) Wrap in cheese cloth,

(c) Coat wrapped core with a lukewarm wax mixture to an approximate 1/4-in. (6.4-mm) thickness. The wax should consist of a 1 to 1 mixture of paraffin and microcrystalline wax, such as Sacony Vacuum Mobile Wax No. 2300 and 2305, Gulf Oil Corporation Petrowax A, and Humble Oil Company Microvan No. 1650.

Cores that could easily be broken by handling should be prepared using the soil sampling technique described on pages 4-20 and 4-21 of EM 1110-2-1907, 31 March 1972[9.1]). They should be transported as a fragile material and protected from excessive changes in humidity and temperature. The identification markings of all samples should be verified immediately upon their receipt at the laboratory, and an inventory of the samples received should be maintained. Samples should be examined and tested as soon as possible after receipt; however, it is often necessary to store samples for several days or even weeks to complete a large testing program. Every care must be taken to protect stored samples against damage. Core logs of samples should be available.

3. Avoidance of Contamination

3.1 The deformation and fracture properties of rock may be influenced by air, water, and other fluids in contact with their internal (crack and pore) surfaces. If these internal surfaces are contaminated by oils or other substances, their properties may be altered appreciably and give misleading test results. Of course, a cutting fluid is required with many types of specimen preparation equipment. Clean water is the preferred fluid. Even so, one must be cognizant of the effect of moisture on the test specimens. While it may be impossible to exactly duplicate the in situ conditions even if they were known, a concerted effort should be made to simulate the environment from which the samples came. Generally, there are three conditions to be considered:

(a) Hard, dense rock and low porosity will not normally be affected by moisture. This type of material is normally allowed to air-dry prior to testing to bring all samples to an equilibrium condition. Drying at temperatures above 120 °F (49 °C) is not recommended as excessive heat may cause an irreversible change in rock properties.

(b) Some shales and rocks containing clay will disintegrate if allowed to dry. Usually the disintegration of diamond drill cores can be prevented by wrapping the cores as they are drilled in a moistureproof material such as aluminum foil or chlorinated rubber, or sealing them in moistureproof containers.

(c) Mud shales and rock containing bentonites (e.g., tuff) may soften if the moisture content is too high. Most of the softer rocks can be cored or cut using compressed air to clear cuttings and to cool the bit or saw.

It is imperative to determine very early in the test program the moisture sensitivity of all types of material to be tested and to take steps to accommodate the requirements throughout the test life of the selected specimens.

4. Selection

4.1 Under the most favorable circumstances, a laboratory determination of the engineering properties of a small specimen gives an approximate guide to the behavior of an extensive, nonhomogeneous geological formation under the complex system of induced stresses. No other aspect of laboratory rock testing is as important as the selection of test specimens to best represent those features of a foundation which influence the analysis or design of a project. Closest teamwork of the laboratory personnel and the project engineer/geologist must be continued throughout the testing program since, as quantitative data become available, changes in the initial allocation of samples or the securing of additional samples may be necessary. Second in importance only to the selection of the most representative undisturbed material is the preparation and handling of the test specimens to preserve in every way possible the natural structure of the material. Indifferent handling of undisturbed rock can result in erroneous test data.

5. Coring

5.1 Virtually all laboratory coring is done with thin-wall diamond rotary bits, which may be detachable or integral to the core barrel. The usual size range for laboratory core drills is from 6-in.-diam (152.4-mm) down to 1-in. (25.4-mm) outside diameter. Typical sample diameters for uniaxial testing are 2.125 in. (54 mm). Drilling machines range from small quarry drills to modified machine shop drill presses. Almost any kind can be adapted for rock work by fitting a water swivel, but a heavy, rigid machine is desirable in order to assure consistent production of high quality core. The work block must be clamped tightly to a strong base or table so as to prevent any tilting, oscillation, or other shifting. To avoid unnecessary unclamping and rearrangement of the work block, it is desirable to have provision for traversing the drill head or the work block. Traversing devices must lock securely to eliminate any play between drill and work. The drill travel should be sufficient to permit continuous runs of at least 10 to 12 in. (254 to 304.8 mm), without need for stopping the machine. Optimum drilling speeds vary with bit size and rock type, and to some extent with condition of the bit and the characteristics of the machine. The general trend is that drill speed increases as drill diameter decreases; also, higher drill speeds are sometimes used on softer rocks. The broad range of drill speeds lies mainly between 200 and 2,000 rpm. No hard-and-fast rules can be given, but an experienced operator can easily choose a suitable speed by trial. Some core drills are hand-fed but it is desirable to have some provision for automatic feed. The ideal feed arrangement is a constant-force hydraulic feed which can be set for each bit size and rock type, but such machines are quite rare. Constant-force feed can be improvised by means of a weight and pulley arrangement. On adapted metal-working drill presses, the automatic feed rate for a given drill size and rock type can be determined; however, since there is a danger of damaging the machine or the core barrel if too high a feed rate is used, an electrical overload breaker should be provided.

6. Sawing

6.1 For heavy sawing, a slabbing saw is adequate for most purposes. For exact sawing, a precision cutoff machine, with a diamond abrasive wheel about 10 in. (254 mm) in diameter, and a table with two-way screw traversing and provision for rotation are recommended. The speed of the wheel is usually fixed, but the feed rate of the wheel through the work can be controlled. Clean water, either direct from house supply or recirculated through a settling tank, is the standard cutting and cooling fluid. For crosscutting, core should be clamped in a vee-block slotted to permit passage of the wheel. By supporting the core on both sides of the cut, the problem of spalling and lip formation at the end of the cut is largely avoided. Saw cuts should be relatively smooth and perpendicular to the co.e axis in order to minimize the grinding or lapping needed to produce end conditions required for the various tests.

7. End Preparation

7.1 Due to the rather large degree of flatness required on bearing surfaces for many tests, end grinding or lapping is required. Conventional surface grinders provided the most practical means of preparing flat surfaces, especially on core samples with diameters greater than approximately 2 in. (50.8 mm). Procedures are essentially comparable to metal working. Quite often a special jig is constructed to hold one or more specimens in the grinding operation. The lathe can also be used for end-grinding cylindrical samples. A sample is held directly in the chuck, rotated at 200 to 300 rpm, and the grinding wheel, its axis inclined some 15 deg (0.26 radian) to the sample axis, is passed across end of the sample with rotating at 6,000 to 8,000 rpm. The "bite" ranges from about 0.003 in. (0.0762-mm) maximum to less than 0.001 in. (0.0254 mm) for finishing, and the grinding wheel is passed across the sample at about 0.5 in. (12.7 mm) per minute. For core diameters of 2-1/8 in. (54 mm) or less, a lap can be used for grinding flat end surfaces on specimens, although producing a sufficiently flat surface by this method is an art. To end-grind on the lap, a cylindrical specimen is placed in a steel-carrying tube which is machined to accept core with a clearance of about 0.002 in. (0.0508 mm). At the lower end of this tube is a steel collar which rests on the lapping wheel. The method requires use of grinding compounds and, hence, is not recommended where other methods are available.

8. Specimen Check

8.1 In general terms, test specimens should be straight, their diameter should be constant, and the ends should be flat, parallel, and normal to the long axis. Sample dimensions should be checked during machining with a micrometer or vernier caliper; final dimensions are normally measured with a micrometer and reported to the nearest 0.01 in. (0.254 mm). Tolerances are best checked on a comparator fitted with a dial micrometer reading to 0.0001 in. (0.00254 mm). There is a technique for revealing the roughness and planes qualitatively. Impressions are made by sandwiching a sheet of carbon paper and a sheet of white paper between the sample end and a smooth surface. The upper end of the sample is given a light blow with a rubber or plastic hammer, and an imprint is formed on the white paper. Areas where no impressions are made indicate dished or uneven surfaces. The importance of proper specimen preparation cannot be overemphasized. Specimens should not be tested which do not meet the dimensional tolerances specified in the respective test methods.

9. References

9.1 Department of the Army, Office, Chief of Engineers, "Soil Sampling," EM 1110-2-1907, Washington, D.C., 1972.

STATISTICAL CONSIDERATIONS

1. Scope

1.1 The purpose of this recommended practice is to outline some general statistical concepts which may be applied to small sample sizes typical of rock test data. It is not the intent to deal with accuracy or precision considerations, but rather to assess the results from the assumed point that the test has been conducted as specified in the respective test methods.

2. Variation and Sample Size

2.1 Most physical tests involve tabulation of a series of readings, with computation of an average said to be representative of the whole. The question arises as to how representative this average is as the measure of the characteristic under test. Three important factors introduce uncertainties in the result:

(a) Instrument and procedural errors.

(b) Variations in the sample being tested.

(c) Variations between the sample and the other samples that might have been drawn from the same source.

If a number of identical specimens were available for tests, or if the tests were nondestructive and could be repeated a number of times on the same specimens, determination of the procedural and instrument errors would be comparatively simple, because in such a test the sample variation would be zero. Periodic tests on this specimen or group of specimens could be used to check the performance of the test procedure and equipment. However, as most of the tests used in determining the mechanical properties of rock are destructive, the instrument and sample variations cannot be separated. Nevertheless, we may apply some elementary statistical concepts and still have confidence in the test results. Of course, the more test data available the more reliable the results. Due to the expense of testing occasionally the shortage of test specimens, rock test data almost always require treatment as groups of small samples. As a general rule at least 10 tests are recommended for any one condition of each individual test with an absolute minimum of 5.

3. Measures of Central Tendency and Deviation

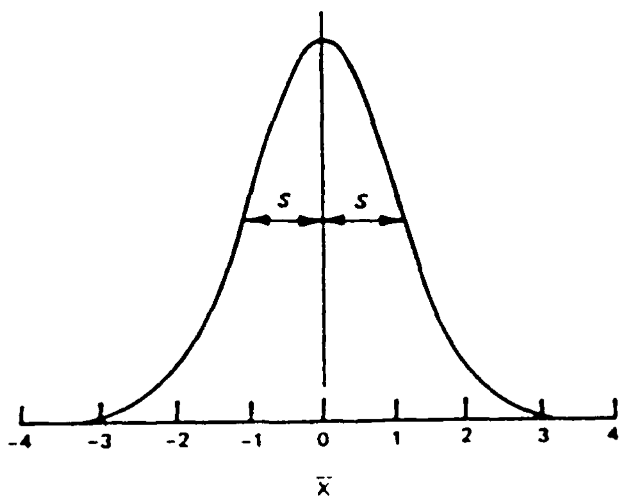
3.1 The most commonly used measure of the central tendency of a sample of n specimens is the arithmetic mean or average, $\bar{x}$. The standard deviation, s , is a measure of the sample variability. It is calculated as follows:

$$s = \sqrt{\frac{\sum (x_i - \bar{x})^2}{n-1}}$$

where x_i's are the individual observations. Most calculators have a built-in program that calculates standard deviation. Some have two programs, one that calculates s based on n-1 degrees of freedom and one that calculates s based on n degrees of freedom. The one that uses n-1 degrees of freedom is correct for most applications.

4. The Normal Distribution

4.1 A series of tests for any one property will, of course, have some variation in the individual determinations. The distribution of these bits of data quite often follows a pattern of normal distributions, i.e., a large portion of the data bits will be closely grouped to the mean on either side with progressively fewer bits distributed farther from the mean. The normal distribution is usually portrayed graphically as shown below.



Methods are available to check the normality of the distribution and to deal with those which are not normal (skewed).[7.1-7.3] Use of many statistical techniques requires the assumption that the data be normally distributed, but mild cases of skewness do not normally cause severe errors and can be ignored. A more important assumption is that the samples be taken at random from the population, as discussed in paragraph 5.

5. Sampling

5.1 It is extremely important that samples from a parent population be taken at random. Probably the most frequent cause of incorrect conclusions being drawn about a population from a sample is non-random sampling. Sometimes it is impossible to draw random samples because of physical or cost limitations on a project. When this happens, interpretations of results should be made with caution.

5.2 A study of sampling distributions of statistics for small samples (n < 30) is called small sampling theory. Statistical tables are available for use with the proper relationship to develop confidence in test data.[7.1] For small samples the confidence limits for a mean are given by:

$$\bar{x} \pm t_c \frac{s}{\sqrt{n-1}}$$

$\bar{x}$, n , and s are determined as given in paragraph 3.1. t_c is from and depends on the level of confidence desired and the sample size. Values of t_c for 90, 95, and 99 percent confidence limits are given below for n from 3 to 12.

		t_c, %		
n	DF*	90	95	99
3	2	2.92	4.30	9.92
4	3	2.35	3.18	5.84
5	4	2.13	2.78	4.60
6	5	2.02	2.57	4.03
7	6	1.94	2.45	3.71
8	7	1.90	2.36	3.50
9	8	1.86	2.31	3.36
10	9	1.83	2.26	3.25
11	10	1.81	2.23	3.17
12	11	1.80	2.20	3.11

* Degrees of freedom

Using this calculation and given an estimate of the standard deviation of a population, one can estimate the number of samples needed to get a desired level of confidence on the mean. See paragraph 7.2 for an example.

6. Dealing with Outliers

6.1 An outlying observation or "outlier" is one that appears to deviate markedly from other members of the sample in which it occurs. Outliers may be merely unusual examples of the population extremes or they may actually be samples taken accidentally from another population. In the former case, the observations should not be deleted from the sample, whereas in the latter case they should be deleted. It is often difficult to know which applies. When some assignable cause is known, for example when one rock specimen is allowed to dry out prior to strength testing and all others are wet, then the dry specimen is reasonably considered to belong to a different population and should be discarded. In the absence of an assignable cause, outliers should not be discarded without first examining the observation in light of an objective procedure. This is particularly critical with small samples. For example, it is quite common when samples of 3 are taken that two observations fall quite close together and the third to be somewhat away. Without substantial experience or an objective method, intuition often misleads one into discarding the outlier. Many test methods provide guidance on handling outliers. ASTM E 178[7.5] provides a general technique for handling outliers.

7. Examples

7.1 For the purposes of illustrating a confidence limit calculation, assume there is a normal distribution of compressive strength data[7.4] for a particular rock type yielding the following individual specimen strengths, taken at random: 18,000; 18,700; 19,200; 19,600; 20,000; 20,100; 20,500; 20,800; 21,100; and 22,000 psi. Find the 95 percent confidence limits for the mean strength.

By computation:

$$n = 10$$

$$\bar{x} = 20{,}000 \text{ psi}$$

$$s = 1183$$

$$95\% \text{ confidence limits} = \bar{x} \pm t_{95} \, (s/\sqrt{n-1}); \text{ thus}$$

$$\bar{x} \pm 2.26(1183/\sqrt{10-1}) =$$

$$20{,}000 \pm 2.26 \, (420) = 20{,}000 \pm 891 \text{ psi}$$

Thus, we can be 95 percent confident that the true mean lies between 19,109 and 20,891.

7.2 As an example of the use of confidence limit calculation to estimate sample sizes, consider the population described in paragraph 7.1. Suppose the objective of a sample was to estimate the mean strength of that population with a confidence limit on that mean of ±1,000 psi. Then confidence limit could be calculated for a range of sample sizes, as follows:

Sample Size	95% Confidence Interval
4	1636 psi
5	1238 psi
6	1024 psi
7	891 psi

By inspection, a sample size of about 6 should be suitable to achieve the desired confidence limit.

8. References

8.1 Spiegel, M. R., Theory and Problems of Statistics, Schaun's Outline Series, McGraw-Hill Company, New York, 1961.

8.2 Volk, William, Applied Statistics for Engineers, McGraw-Hill Company, New York, 1958.

8.3 Obert, Leonard and Duvall, W. I., Rock Mechanics and the Design of Structures in Rock, John Wiley and Sons, Inc., New York, 1967.

8.4 "Engineering Geology; Special Issue, Uniaxial Testing in Rock Mechanics Laboratories," Elsevier Publishing Company, Amsterdam, Vol 4, No. 3, July 1970.

8.5 ASTM E 178. "Standard Practice for Dealing with Outlying Observations," Annual Book of ASTM Standards, Vol 14.02, ASTM, Philadelphia, PA.

METHOD FOR DETERMINATION OF REBOUND NUMBER OF ROCK

1. Scope

1.1 This method provides instructions for the determination of a rebound number of rock using a spring-driven steel hammer (Fig. 1).

1 Impact plunger
3 Housing compl.
4 Rider with guide rod
5 Scale (starting with serial No. 230 printed on window No. 19)
6 Pushbutton compl.
7 Hammer guide bar
8 Disk
9 Cap
10 Two-part ring
11 Rear cover
12 Compression spring
13 Pawl
14 Hammer mass
15 Retaining spring
16 Impact spring
17 Guide sleeve
18 Felt washer
19 Plexiglass window
20 Trip screw
21 Lock nut
22 Pin
23 Pawl spring

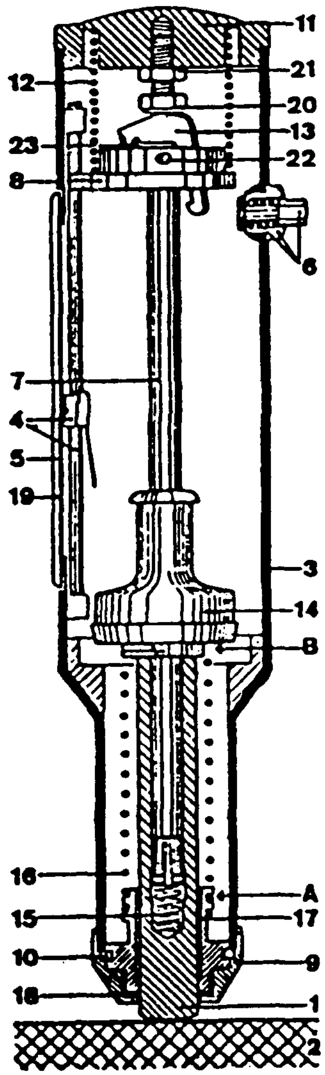


Fig. 1. Spring-driven steel hammer.

2. Significance

2.1 The rebound number determined by this method may be used to assess the uniformity of rock in situ, or on cored samples to indicate hardness characteristics of the rock.

3. Apparatus

3.1 Rebound Hammer - The rebound hammer consists of a spring-loaded steel hammer which when released strikes a steel plunger which is in contact with the test surface. The spring-loaded hammer must travel with a fixed and reproducible velocity. The rebound distance of the steel hammer from the steel plunger is measured by means of a linear scale attached to the frame of the instrument (Note 1).

NOTE 1—Several types and sizes of rebound hammers are commercially available. Hammers with an energy impact of 0.075 m-kg (0.542 ft-lb) have been found satisfactory for rock testing.

3.2 Abrasive Stone - An abrasive stone consisting of medium grain texture silicon carbide or equivalent material shall be provided.

4. Test Area

4.1 Selection of Test Surface - Surfaces to be tested shall be at least 2 in. (50 mm) thick and fixed within a stratum. Specimens should be rigidly supported. Some companies market a "rock cradle" for this purpose. Areas exhibiting scaling, rough texture, or high porosity should be avoided. Dry rocks give higher rebound numbers than wet.

4.2 Preparation of Test Surface - Heavily textured soft surfaces or surfaces with loose particles shall be ground smooth with the abrasive stone described in Section 3.2. Smooth surfaces shall be tested without grinding. The effects of drying and carbonation can be minimized by thoroughly wetting the surfaces for 24 hours prior to testing.

4.3 Factors Affecting Test Results - Other factors related to test circumstances may affect the results of the test:

(a) Rock at 32°F (0°C) or less may exhibit very high rebound values. Rock should be tested only after it has thawed.

(b) The temperature of the rebound hammer itself may affect the rebound number (Note 2).

NOTE 2—Rebound hammers at 0°F (-18°C) may produce rebound numbers reduced by as much as 2 or 3 units.

(c) For readings to be compared, the direction of impact—horizontal, downward, upward, etc.—must be the same.

(d) Different hammers of the same nominal design may give rebound numbers differing by 1 to 3 units, and therefore, to be compared, tests should be made with a single hammer. If more than one hammer is to be used, a sufficient number of tests must be made on typical rock surfaces to determine the magnitude of the differences to be expected. (Note 3)

NOTE 3—Rebound hammers require periodic servicing and verification annually for hammers in heavy use, biennially for hammers in less frequent use, and whenever there is reason to question their proper operation. Metal anvils are available for verification and are recommended. However, verification on an anvil will not guarantee that different hammers will yield the same results at other points on the rebound scale. Some users compare several hammers on surfaces encompassing the usual range of rebound values encountered in the field.

5. Test Procedure

5.1 The instrument shall be firmly held in a position which allows the plunger to strike perpendicular to the surface tested. The pressure on the plunger shall be gradually increased until the hammer impacts. After impact, the rebound number should be recorded to two significant figures. Ten readings shall be taken from each test area with no two impact tests being closer together than 1 in. (25.4 mm). Examine the impression made on the surface after impact and disregard the reading if the impact crushes or breaks through the surface.

6. Calculation

6.1 Readings differing from the average of 10 readings by more than 7 units are to be discarded and the average of the remaining readings determined. If more than 2 readings differ from the average by 7 units, the entire set of readings should be discarded.

7. Precision

7.1 The single-specimen-operator-machine-day precision is 2.5 units (1S) as defined in ASTM Recommended Practice E 177, "Use of the Terms Precision and Accuracy as Applied to Measurement of a Property of a Material."

8. Use and Interpretation of Rebound Hammer Results

8.1 Optimally, rebound numbers may be correlated with core testing information. There is a relationship between rebound number and strength and deformation and the relationship is normally provided by the rebound hammer manufacturer.

9. Report

9.1 The report should include the following information for each test area:

9.1.1 Rock identification.

9.1.2 Location of rock stratum.

9.1.3 Description and composition of rock if known.

9.1.4 Average rebound number for each test area or specimen.

9.1.5 Approximate angular direction of rebound hammer impact, with horizontal being considered 0, vertically upward being +90 deg (1.57 radians), and vertically downward being -90 deg (1.57 radians).

9.1.6 Hammer type and serial number.

RTH 106-93

METHOD FOR DETERMINATION OF THE WATER CONTENT OF A ROCK SAMPLE

1. Scope

1.1 This test method covers the determination of the percentage of evaporable water in the pores of a rock sample as a percentage of the ovendry sample mass.

2. Apparatus

(a) An oven capable of maintaining a temperature of 110 ± 5 °C for a period of at least 24 hr.

(b) A sample container of noncorrodible material, including an airtight lid.

(c) A desiccator to hold sample containers during cooling.

(d) A balance of adequate capacity, capable of weighing to an accuracy of 0.01 percent of the sample mass.

3. Procedure

(a) The container and lid is cleaned, dried, and its mass determined.

(b) A representative sample is selected, preferably comprising at least ten rock lumps each having a mass of at least 50 g to give a total sample mass of at least 500 g. For in situ water content determination, sampling, storage, and handling precautions should retain water content to within 1 percent of its in situ value.

(c) The sample is placed in the container, the lid replaced, and the mass of the sample plus container is determined.

(d) The lid is removed and the sample dried to constant mass. Constant mass is achieved when the mass loss is less than 0.1 percent of the sample mass in 4 hr of drying.

(e) The lid is replaced and the sample allowed to cool in the desiccator for 30 minutes. The mass of sample plus container is determined.

4. Calculation

$$\text{Water content } w = \frac{\text{pore water weight}}{\text{grain weight}} \cdot 100\% = \frac{Y-Z}{Z-X} \cdot 100\%$$

4.1 Calculate the water content of the rock sample as follows:

$$W = \frac{Y-Z}{Z-X}\,(100)$$

where

W - water content, percent

Y - original sample mass, g

Z - dried sample mass, g, and

X - sample container and lid mass, g

5. Reporting of Results

5.1 The water content should be reported to the nearest 0.1 percent stating whether this corresponds to in situ water content, in which case precautions taken to retain water during sampling and storage should be specified.

Designation: C 127 – 88

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate[1]

This standard is issued under the fixed designation C 127; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This test method has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

1. Scope

1.1 This test method covers the determination of specific gravity and absorption of coarse aggregate. The specific gravity may be expressed as bulk specific gravity, bulk specific gravity (SSD) (saturated-surface-dry), or apparent specific gravity. The bulk specific gravity (SSD) and absorption are based on aggregate after 24 h soaking in water. This test method is not intended to be used with lightweight aggregates.

1.2 The values stated in SI units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 29 Test Method for Unit Weight and Voids in Aggregate[2]

C 125 Definitions of Terms Relating to Concrete and Concrete Aggregates[2]

C 128 Test Method for Specific Gravity and Absorption of Fine Aggregate[2]

C 136 Method for Sieve Analysis of Fine and Coarse Aggregates[2]

C 566 Test Method for Total Moisture Content of Aggregate by Drying[3]

C 670 Practice for Preparing Precision Statements for Test Methods for Construction Materials[2]

C 702 Practice for Reducing Field Samples of Aggregate to Testing Size[3]

D 75 Practice for Sampling Aggregates[2]

D 448 Classification for Sizes of Aggregate for Road and Bridge Construction[2]

E 11 Specification for Wire-Cloth Sieves for Testing Purposes[4]

E 12 Definitions of Terms Relating to Density and Specific Gravity of Solids, Liquids, and Gases[5]

2.2 *AASHTO Standard:*

AASHTO No. T 85 Specific Gravity and Absorption of Coarse Aggregate[6]

3. Terminology

3.1 *Definitions:*

3.1.1 *absorption*—the increase in the weight of aggregate due to water in the pores of the material, but not including water adhering to the outside surface of the particles, expressed as a percentage of the dry weight. The aggregate is considered "dry" when it has been maintained at a temperature of 110 ± 5°C for sufficient time to remove all uncombined water.

3.1.2 *specific gravity*—the ratio of the mass (or weight in air) of a unit volume of a material to the mass of the same volume of water at stated temperatures. Values are dimensionless.

3.1.2.1 *apparent specific gravity*—the ratio of the weight in air of a unit volume of the impermeable portion of aggregate at a stated temperature to the weight in air of an equal volume of gas-free distilled water at a stated temperature.

3.1.2.2 *bulk specific gravity*—the ratio of the weight in air of a unit volume of aggregate (including the permeable and impermeable voids in the particles, but not including the voids between particles) at a stated temperature to the weight in air of an equal volume of gas-free distilled water at a stated temperature.

3.1.2.3 *bulk specific gravity (SSD)*—the ratio of the weight in air of a unit volume of aggregate, including the weight of water within the voids filled to the extent achieved by submerging in water for approximately 24 h (but not including the voids between particles) at a stated temperature, compared to the weight in air of an equal volume of gas-free distilled water at a stated temperature.

NOTE 1—The terminology for specific gravity is based on terms in Definitions E 12, and that for absorption is based on that term in Terminology C 125.

4. Summary of Test Method

4.1 A sample of aggregate is immersed in water for approximately 24 h to essentially fill the pores. It is then removed from the water, the water dried from the surface of

[1] This test method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.05 on Methods of Testing and Specifications for Physical Characteristics of Concrete Aggregates.

Current edition approved Oct. 31, 1988. Published December 1988. Originally published as C 127 – 36 T. Last previous edition C 127 – 84.

[2] *Annual Book of ASTM Standards*, Vols 04.02 and 04.03.

[3] *Annual Book of ASTM Standards*, Vol 04.02.

[4] *Annual Book of ASTM Standards*, Vol 14.02.

[5] *Annual Book of ASTM Standards*, Vols 04.02, 15.05.

[6] Available from American Association of State Highway and Transportation Officials, 444 North Capitol St. N.W., Suite 225, Washington, DC 20001.

the particles, and weighed. Subsequently the sample is weighed while submerged in water. Finally the sample is oven-dried and weighed a third time. Using the weights thus obtained and formulas in this test method, it is possible to calculate three types of specific gravity and absorption.

5. Significance and Use

5.1 Bulk specific gravity is the characteristic generally used for calculation of the volume occupied by the aggregate in various mixtures containing aggregate, including portland cement concrete, bituminous concrete, and other mixtures that are proportioned or analyzed on an absolute volume basis. Bulk specific gravity is also used in the computation of voids in aggregate in Test Method C 29. Bulk specific gravity (SSD) is used if the aggregate is wet, that is, if its absorption has been satisfied. Conversely, the bulk specific gravity (oven-dry) is used for computations when the aggregate is dry or assumed to be dry.

5.2 Apparent specific gravity pertains to the relative density of the solid material making up the constituent particles not including the pore space within the particles which is accessible to water.

5.3 Absorption values are used to calculate the change in the weight of an aggregate due to water absorbed in the pore spaces within the constituent particles, compared to the dry condition, when it is deemed that the aggregate has been in contact with water long enough to satisfy most of the absorption potential. The laboratory standard for absorption is that obtained after submerging dry aggregate for approximately 24 h in water. Aggregates mined from below the water table may have a higher absorption, when used, if not allowed to dry. Conversely, some aggregates when used may contain an amount of absorbed moisture less than the 24-h soaked condition. For an aggregate that has been in contact with water and that has free moisture on the particle surfaces, the percentage of free moisture can be determined by deducting the absorption from the total moisture content determined by Test Method C 566.

5.4 The general procedures described in this test method are suitable for determining the absorption of aggregates that have had conditioning other than the 24-h soak, such as boiling water or vacuum saturation. The values obtained for absorption by other test methods will be different than the values obtained by the prescribed 24-h soak, as will the bulk specific gravity (SSD).

5.5 The pores in lightweight aggregates may or may not become essentially filled with water after immersion for 24 h. In fact, many such aggregates can remain immersed in water for several days without satisfying most of the aggregates' absorption potential. Therefore, this test method is not intended for use with lightweight aggregate.

6. Apparatus

6.1 *Balance*—A weighing device that is sensitive, readable, and accurate to 0.05 % of the sample weight at any point within the range used for this test, or 0.5 g, whichever is greater. The balance shall be equipped with suitable apparatus for suspending the sample container in water from the center of the weighing platform or pan of the weighing device.

6.2 *Sample Container*—A wire basket of 3.35 mm (No. 6) or finer mesh, or a bucket of approximately equal breadth and height, with a capacity of 4 to 7 L for 37.5-mm (1½-in.) nominal maximum size aggregate or smaller, and a larger container as needed for testing larger maximum size aggregate. The container shall be constructed so as to prevent trapping air when the container is submerged.

6.3 *Water Tank*—A watertight tank into which the sample container may be placed while suspended below the balance.

6.4 *Sieves*—A 4.75-mm (No. 4) sieve or other sizes as needed (see 7.2, 7.3, and 7.4), conforming to Specification E 11.

7. Sampling

7.1 Sample the aggregate in accordance with Practice D 75.

7.2 Thoroughly mix the sample of aggregate and reduce it to the approximate quantity needed using the applicable procedures in Methods C 702. Reject all material passing a 4.75-mm (No. 4) sieve by dry sieving and thoroughly washing to remove dust or other coatings from the surface. If the coarse aggregate contains a substantial quantity of material finer than the 4.75-mm sieve (such as for Size No. 8 and 9 aggregates in Classification D 448), use the 2.36-mm (No. 8) sieve in place of the 4.75-mm sieve. Alternatively, separate the material finer than the 4.75-mm sieve and test the finer material according to Test Method C 128.

7.3 The minimum weight of test sample to be used is given below. In many instances it may be desirable to test a coarse aggregate in several separate size fractions; and if the sample contains more than 15 % retained on the 37.5-mm (1½-in.) sieve, test the material larger than 37.5 mm in one or more size fractions separately from the smaller size fractions. When an aggregate is tested in separate size fractions, the minimum weight of test sample for each fraction shall be the difference between the weights prescribed for the maximum and minimum sizes of the fraction.

Nominal Maximum Size, mm (in.)	Minimum Weight of Test Sample, kg (lb)
12.5 (½) or less	2 (4.4)
19.0 (¾)	3 (6.6)
25.0 (1)	4 (8.8)
37.5 (1½)	5 (11)
50 (2)	8 (18)
63 (2½)	12 (26)
75 (3)	18 (40)
90 (3½)	25 (55)
100 (4)	40 (88)
112 (4½)	50 (110)
125 (5)	75 (165)
150 (6)	125 (276)

7.4 If the sample is tested in two or more size fractions, determine the grading of the sample in accordance with Method C 136, including the sieves used for separating the size fractions for the determinations in this method. In calculating the percentage of material in each size fraction, ignore the quantity of material finer than the 4.75-mm (No. 4) sieve (or 2.36-mm (No. 8) sieve when that sieve is used in accordance with 7.2).

8. Procedure

8.1 Dry the test sample to constant weight at a temperature of 110 ± 5°C (230 ± 9°F), cool in air at room temperature for 1 to 3 h for test samples of 37.5-mm (1½-in.) nominal maximum size, or longer for larger sizes until the aggregate has cooled to a temperature that is comfortable to handle (approximately 50°C). Subsequently immerse the aggregate in water at room temperature for a period of 24 ± 4 h.

NOTE 2—When testing coarse aggregate of large nominal maximum size requiring large test samples, it may be more convenient to perform the test on two or more subsamples, and the values obtained combined for the computations described in Section 9.

8.2 Where the absorption and specific gravity values are to be used in proportioning concrete mixtures in which the aggregates will be in their naturally moist condition, the requirement for initial drying to constant weight may be eliminated, and, if the surfaces of the particles in the sample have been kept continuously wet until test, the 24-h soaking may also be eliminated.

NOTE 3—Values for absorption and bulk specific gravity (SSD) may be significantly higher for aggregate not oven dried before soaking than for the same aggregate treated in accordance with 8.1. This is especially true of particles larger than 75 mm (3 in.) since the water may not be able to penetrate the pores to the center of the particle in the prescribed soaking period.

8.3 Remove the test sample from the water and roll it in a large absorbent cloth until all visible films of water are removed. Wipe the larger particles individually. A moving stream of air may be used to assist in the drying operation. Take care to avoid evaporation of water from aggregate pores during the operation of surface-drying. Weigh the test sample in the saturated surface-dry condition. Record this and all subsequent weights to the nearest 0.5 g or 0.05 % of the sample weight, whichever is greater.

8.4 After weighing, immediately place the saturated-surface-dry test sample in the sample container and determine its weight in water at 23 ± 1.7°C (73.4 ± 3°F), having a density of 997 ± 2 kg/m³. Take care to remove all entrapped air before weighing by shaking the container while immersed.

NOTE 4—The container should be immersed to a depth sufficient to cover it and the test sample during weighing. Wire suspending the container should be of the smallest practical size to minimize any possible effects of a variable immersed length.

8.5 Dry the test sample to constant weight at a temperature of 110 ± 5°C (230 ± 9°F), cool in air at room temperature 1 to 3 h, or until the aggregate has cooled to a temperature that is comfortable to handle (approximately 50°C), and weigh.

9. Calculations

9.1 *Specific Gravity:*

9.1.1 *Bulk Specific Gravity*—Calculate the bulk specific gravity, 23/23°C (73.4/73.4°F), as follows:

$$\text{Bulk sp gr} = A/(B - C)$$

where:

A = weight of oven-dry test sample in air, g,
B = weight of saturated-surface-dry test sample in air, g, and
C = weight of saturated test sample in water, g.

9.1.2 *Bulk Specific Gravity (Saturated-Surface-Dry)*—Calculate the bulk specific gravity, 23/23°C (73.4/73.4°F), on the basis of weight of saturated-surface-dry aggregate as follows:

$$\text{Bulk sp gr (saturated-surface-dry)} = B/(B - C)$$

9.1.3 *Apparent Specific Gravity*—Calculate the apparent specific gravity, 23/23°C (73.4/73.4°F), as follows:

$$\text{Apparent sp gr} = A/(A - C)$$

9.2 *Average Specific Gravity Values*—When the sample is tested in separate size fractions the average value for bulk specific gravity, bulk specific gravity (SSD), or apparent specific gravity can be computed as the weighted average of the values as computed in accordance with 9.1 using the following equation:

$$G = \frac{1}{\dfrac{P_1}{100\,G_1} + \dfrac{P_2}{100\,G_2} + \ldots \dfrac{P_n}{100\,G_n}} \text{ (see Appendix X1)}$$

where:

G = average specific gravity. All forms of expression of specific gravity can be averaged in this manner.
$G_1, G_2 \ldots G_n$ = appropriate specific gravity values for each size fraction depending on the type of specific gravity being averaged.
$P_1, P_2, \ldots P_n$ = weight percentages of each size fraction present in the original sample.

NOTE 5—Some users of this test method may wish to express the results in terms of density. Density may be determined by multiplying the bulk specific gravity, bulk specific gravity (SSD), or apparent specific gravity by the weight of water (997.5 kg/m³ or 0.9975 Mg/m³ or 62.27 lb/ft³ at 23°C). Some authorities recommend using the density of water at 4°C (1000 kg/m³ or 1.000 Mg/m³ or 62.43 lb/ft³) as being sufficiently accurate. Results should be expressed to three significant figures. The density terminology corresponding to bulk specific gravity, bulk specific gravity (SSD), and apparent specific gravity has not been standardized

9.3 *Absorption*—Calculate the percentage of absorption, as follows:

$$\text{Absorption, \%} = [(B - A)/A] \times 100$$

9.4 *Average Absorption Value*—When the sample is tested in separate size fractions, the average absorption value is the average of the values as computed in 9.3, weighted in proportion to the weight percentages of the size fractions in the original sample as follows:

$$A = (P_1A_1/100) + (P_2A_2/100) + \ldots (P_nA_n/100)$$

where:

A = average absorption, %,
$A_1, A_2 \ldots A_n$ = absorption percentages for each size fraction, and
$P_1, P_2, \ldots P_n$ = weight percentages of each size fraction present in the original sample.

10. Report

10.1 Report specific gravity results to the nearest 0.01, and indicate the type of specific gravity, whether bulk, bulk (saturated-surface-dry), or apparent.

10.2 Report the absorption result to the nearest 0.1 %.

10.3 If the specific gravity and absorption values were determined without first drying the aggregate, as permitted in 8.2, it shall be noted in the report.

11. Precision and Bias

11.1 The estimates of precision of this test method listed in Table 1 are based on results from the AASHTO Materials Reference Laboratory Reference Sample Program, with testing conducted by this test method and AASHTO Method T 85. The significant difference between the methods is that Test Method C 127 requires a saturation period of 24 ± 4 h, while Method T 85 requires a saturation period of 15 h minimum. This difference has been found to have an insignificant effect on the precision indices. The data are based on the analyses of more than 100 paired test results from 40 to 100 laboratories.

11.1 *Bias*—Since there is no accepted reference material for determining the bias for the procedure in this test method, no statement on bias is being made.

TABLE 1 Precision

	Standard Deviation (1S)[A]	Acceptable Range of Two Results (D2S)[A]
Single-Operator Precision:		
Bulk specific gravity (dry)	0.009	0.025
Bulk specific gravity (SSD)	0.007	0.020
Apparent specific gravity	0.007	0.020
Absorption[B], %	0.088	0.25
Multilaboratory Precision:		
Bulk specific gravity (dry)	0.013	0.038
Bulk specific gravity (SSD)	0.011	0.032
Apparent specific gravity	0.011	0.032
Absorption[B], %	0.145	0.41

[A] These numbers represent, respectively, the (1S) and (D2S) limits as described in Practice C 670. The precision estimates were obtained from the analysis of combined AASHTO Materials Reference Laboratory reference sample data from laboratories using 15 h minimum saturation times and other laboratories using 24 ± 4 h saturation times. Testing was performed on normal-weight aggregates, and started with aggregates in the oven-dry condition.

[B] Precision estimates are based on aggregates with absorptions of less than 2 %.

APPENDIXES

(Nonmandatory Information)

X1. DEVELOPMENT OF EQUATIONS

X1.1 The derivation of the equation is apparent from the following simplified cases using two solids. Solid 1 has a weight W_1 in grams and a volume V_1 in millilitres; its specific gravity (G_1) is therefore W_1/V_1. Solid 2 has a weight W_2 and volume V_2, and $G_2 = W_2/V_2$. If the two solids are considered together, the specific gravity of the combination is the total weight in grams divided by the total volume in millilitres:

$$G = (W_1 + W_2)/(V_1 + V_2)$$

Manipulation of this equation yields the following:

$$G = \frac{1}{\dfrac{V_1 + V_2}{W_1 + W_2}} = \frac{1}{\dfrac{V_1}{V_1 + W_2} + \dfrac{V_2}{W_1}}$$

$$G = \frac{1}{\dfrac{W_1}{W_1 + W_2}\left(\dfrac{V_1}{W_1}\right) + \dfrac{W_2}{W_1 + W_2}\left(\dfrac{V_2}{W_2}\right)}$$

However, the weight fractions of the two solids are:

$$W_1/(W_1 + W_2) = P_1/100 \text{ and } W_2/(W_1 + W_2) = P_2/100$$

and,

$$1/G_1 = V_1/W_1 \text{ and } 1/G_2 = V_2/W_2$$

Therefore,

$$G = 1/[(P_1/100)(1/G_1) + (P_2/100)(1/G_2)]$$

An example of the computation is given in Table X1.1.

TABLE X1.1 Example of Calculation of Average Values of Specific Gravity and Absorption for a Coarse Aggregate Tested in Separate Sizes

Size Fraction, mm (in.)	% in Original Sample	Sample Weight Used in Test, g	Bulk Specific Gravity (SSD)	Absorption, %
4.75 to 12.5 (No. 4 to ½)	44	2213.0	2.72	0.4
12.5 to 37.5 (½ to 1½)	35	5462.5	2.56	2.5
37.5 to 63 (1½ to 2½)	21	12593.0	2.54	3.0

Average Specific Gravity (SSD)

$$G_{SSD} = \frac{1}{\dfrac{0.44}{2.72} + \dfrac{0.35}{2.56} + \dfrac{0.21}{2.54}} = 2.62$$

Average Absorption

$$A = (0.44)(0.4) + (0.35)(2.5) + (0.21)(3.0) = 1.7\ \%$$

X2. INTERRELATIONSHIPS BETWEEN SPECIFIC GRAVITIES AND ABSORPTION AS DEFINED IN TEST METHODS C 127 AND C 128

X2.1 Let:

S_d = bulk specific gravity (dry basis),
S_s = bulk specific gravity (SSD basis),
S_a = apparent specific gravity, and
A = absorption in %.

X2.2 Then,

$$S_s = (1 + A/100)S_d \tag{1}$$

$$S_a = \frac{1}{\dfrac{1}{S_d} - \dfrac{A}{100}} = \frac{S_d}{1 - \dfrac{AS_d}{100}} \tag{2}$$

$$S_a = \frac{1}{\dfrac{1 + A/100}{S_s} - \dfrac{A}{100}} = \frac{S_s}{1 - \left[\dfrac{A}{100}(S_s - 1)\right]} \tag{2a}$$

$$A = \left(\frac{S_s}{S_d} - 1\right)100 \tag{3}$$

$$A = \left(\frac{S_a - S_s}{S_a(S_s - 1)}\right)100 \tag{4}$$

The American Society for Testing and Materials takes no position respecting the validity of any patent rights asserted in connection with any item mentioned in this standard. Users of this standard are expressly advised that determination of the validity of any such patent rights, and the risk of infringement of such rights, are entirely their own responsibility.

This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 1916 Race St., Philadelphia, PA 19103.

Designation: C 128 – 88

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition.

Standard Test Method for Specific Gravity and Absorption of Fine Aggregate[1]

This standard is issued under the fixed designation C 128; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This standard has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

1. Scope

1.1 This test method covers the determination of bulk and apparent specific gravity, 23/23°C (73.4/73.4°F), and absorption of fine aggregate.

1.2 This test method determines (after 24 h in water) the bulk specific gravity and the apparent specific gravity as defined in Definitions E 12, the bulk specific gravity on the basis of weight of saturated surface-dry aggregate, and the absorption as defined in Definitions C 125.

NOTE 1—The subcommittee is considering revising Test Methods C 127 and C 128 to use the term "density" instead of "specific gravity" for coarse and fine aggregate, respectively.

1.3 The values stated in SI units are to be regarded as the standard.

1.4 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 29/C 29M Test Method for Unit Weight and Voids in Aggregate[2]

C 70 Test Method for Surface Moisture in Fine Aggregate[2]

C 125 Terminology Relating to Concrete and Concrete Aggregates[2]

C 127 Test Method for Specific Gravity and Absorption of Coarse Aggregate[2]

C 188 Test Method for Density of Hydraulic Cement[3]

C 566 Test Method for Total Moisture Content of Aggregate by Drying[2]

C 670 Practice for Preparing Precision and Bias Statements for Test Methods for Construction Materials[2,3,4]

C 702 Practice for Reducing Field Samples of Aggregate to Testing Size[2]

D 75 Practice for Sampling Aggregates[2,4]

E 12 Definitions of Terms Relating to Density and Specific Gravity of Solids, Liquids, and Gases[2,5]

E 380 Metric Practice[6]

2.2 *AASHTO Standard:*

AASHTO No. T 84 Specific Gravity and Absorption of Fine Aggregates[7]

3. Significance and Use

3.1 Bulk specific gravity is the characteristic generally used for calculation of the volume occupied by the aggregate in various mixtures containing aggregate including portland cement concrete, bituminous concrete, and other mixtures that are proportioned or analyzed on an absolute volume basis. Bulk specific gravity is also used in the computation of voids in aggregate in Test Method C 29 and the determination of moisture in aggregate by displacement in water in Test Method C 70. Bulk specific gravity determined on the saturated surface-dry basis is used if the aggregate is wet, that is, if its absorption has been satisfied. Conversely, the bulk specific gravity determined on the oven-dry basis is used for computations when the aggregate is dry or assumed to be dry.

3.2 Apparent specific gravity pertains to the relative density of the solid material making up the constituent particles not including the pore space within the particles that is accessible to water. This value is not widely used in construction aggregate technology.

3.3 Absorption values are used to calculate the change in the weight of an aggregate due to water absorbed in the pore spaces within the constituent particles, compared to the dry condition, when it is deemed that the aggregate has been in contact with water long enough to satisfy most of the absorption potential. The laboratory standard for absorption is that obtained after submerging dry aggregate for approximately 24 h in water. Aggregates mined from below the water table may have a higher absorption when used, if not allowed to dry. Conversely, some aggregates when used may contain an amount of absorbed moisture less than the 24 h-soaked condition. For an aggregate that has been in contact with water and that has free moisture on the particle surfaces, the percentage of free moisture can be determined by deducting the absorption from the total moisture content determined by Test Method C 566 by drying.

[1] This test method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.05 on Methods of Testing and Specifications for Physical Characteristics of Concrete Aggregates.

Current edition approved Nov. 25, 1988. Published January 1989. Originally published as C 128 – 36. Last previous edition C 128 – 84.

[2] *Annual Book of ASTM Standards*, Vol 04.02.

[3] *Annual Book of ASTM Standards*, Vol 04.01.

[4] *Annual Book of ASTM Standards*, Vol 04.03.

[5] *Annual Book of ASTM Standards*, Vol 15.05.

[6] *Annual Book of ASTM Standards*, Vol 14.02. Excerpts in all volumes.

[7] Available from American Association of State Highway and Transportation Officials, 444 North Capitol St. N.W., Suite 225, Washington, DC 20001.

4. Apparatus

4.1 *Balance*—A balance or scale having a capacity of 1 kg or more, sensitive to 0.1 g or less, and accurate within 0.1 % of the test load at any point within the range of use for this test. Within any 100-g range of test load, a difference between readings shall be accurate within 0.1 g.

4.2 *Pycnometer*—A flask or other suitable container into which the fine aggregate test sample can be readily introduced and in which the volume content can be reproduced within ±0.1 cm^3. The volume of the container filled to mark shall be at least 50 % greater than the space required to accommodate the test sample. A volumetric flask of 500 cm^3 capacity or a fruit jar fitted with a pycnometer top is satisfactory for a 500-g test sample of most fine aggregates. A Le Chatelier flask as described in Test Method C 188 is satisfactory for an approximately 55-g test sample.

4.3 *Mold*—A metal mold in the form of a frustum of a cone with dimensions as follows: 40 ± 3 mm inside diameter at the top, 90 ± 3 mm inside diameter at the bottom, and 75 ± 3 mm in height, with the metal having a minimum thickness of 0.8 mm.

4.4 *Tamper*—A metal tamper weighing 340 ± 15 g and having a flat circular tamping face 25 ± 3 mm in diameter

5. Sampling

5.1 Sampling shall be accomplished in general accordance with Practice D 75.

6. Preparation of Test Specimen

6.1 Obtain approximately 1 kg of the fine aggregate from the sample using the applicable procedures described in Methods C 702.

6.1.1 Dry it in a suitable pan or vessel to constant weight at a temperature of 110 ± 5°C (230 ± 9°F). Allow it to cool to comfortable handling temperature, cover with water, either by immersion or by the addition of at least 6 % moisture to the fine aggregate, and permit to stand for 24 ± 4 h.

6.1.2 As an alternative to 6.1.1, where the absorption and specific gravity values are to be used in proportioning concrete mixtures with aggregates used in their naturally moist condition, the requirement for initial drying to constant weight may be eliminated and, if the surfaces of the particles have been kept wet, the 24-h soaking may also be eliminated.

NOTE 2—Values for absorption and for specific gravity in the saturated surface-dry condition may be significantly higher for aggregate not oven dried before soaking than for the same aggregate treated in accordance with 6.1.1.

6.2 Decant excess water with care to avoid loss of fines, spread the sample on a flat nonabsorbent surface exposed to a gently moving current of warm air, and stir frequently to secure homogeneous drying desired, mechanical aids such as tumbling or stirring may be employed to assist in achieving the saturated surface-dry condition. Continue this operation until the test specimen approaches a free-flowing condition. Follow the procedure in 6.2.1 to determine whether or not surface moisture is present on the constituent fine aggregate particles. It is intended that the first trial of the cone test will be made with some surface water in the specimen. Continue drying with constant stirring and test at frequent intervals until the test indicates that the specimen has reached a surface-dry condition. If the first trial of the surface moisture test indicates that moisture is not present on the surface, it has been dried past the saturated surface-dry condition. In this case thoroughly mix a few millilitres of water with the fine aggregate and permit the specimen to stand in a covered container for 30 min. Then resume the process of drying and testing at frequent intervals for the onset of the surface-dry condition.

6.2.1 *Cone Test for Surface Moisture*—Hold the mold firmly on a smooth nonabsorbent surface with the large diameter down. Place a portion of the partially dried fine aggregate loosely in the mold by filling it to overflowing and heaping additional material above the top of the mold by holding it with the cupped fingers of the hand holding the mold. Lightly tamp the fine aggregate into the mold with 25 light drops of the tamper. Each drop should start about 5 mm (0.2 in.) above the top surface of the fine aggregate. Permit the tamper to fall freely under gravitational attraction on each drop. Adjust the starting height to the new surface elevation after each drop and distribute the drops over the surface. Remove loose sand from the base and lift the mold vertically. If surface moisture is still present, the fine aggregate will retain the molded shape. When the fine aggregate slumps slightly it indicates that it has reached a surface-dry condition. Some angular fine aggregate or material with a high proportion of fines may not slump in the cone test upon reaching a surface-dry condition. This may be the case if fines become airborne upon dropping a handful of the sand from the cone test 100 to 150 mm onto a surface. For these materials, the saturated surface-dry condition should be considered as the point that one side of the fine aggregate slumps slightly upon removing the mold.

NOTE 3—The following criteria have also been used on materials that do not readily slump:

(1) *Provisional Cone Test*—Fill the cone mold as described in 6.2.1 except only use 10 drops of the tamper. Add more fine aggregate and use 10 drops of the tamper again. Then add material two more times using 3 and 2 drops of the tamper, respectively. Level off the material even with the top of the mold, remove loose material from the base; and lift the mold vertically.

(2) *Provisional Surface Test*—If airborne fines are noted when the fine aggregate is such that it will not slump when it is at a moisture condition, add more moisture to the sand, and at the onset of the surface-dry condition, with the hand lightly pat approximately 100 g of the material on a flat, dry, clean, dark or dull nonabsorbent surface such as a sheet of rubber, a worn oxidized, galvanized, or steel surface, or a black-painted metal surface. After 1 to 3 s remove the fine aggregate. If noticeable moisture shows on the test surface for more than 1 to 2 s then surface moisture is considered to be present on the fine aggregate.

(3) Colorimetric procedures described by Kandhal and Lee, Highway Research Record No. 307, p. 44.

(4) For reaching the saturated surface-dry condition on a single size material that slumps when wet, hard-finish paper towels can be used to surface dry the material until the point is just reached where the paper towel does not appear to be picking up moisture from the surfaces of the fine aggregate particles.

7. Procedure

7.1 Make and record all weight determinations to 0.1 g.

7.2 Partially fill the pycnometer with water. Immediately introduce into the pycnometer 500 ± 10 g of saturated surface-dry fine aggregate prepared as described in Section 6, and fill with additional water to approximately 90 % of capacity. Roll, invert, and agitate the pycnometer to eliminate all air bubbles. Adjust its temperature to 23 ± 1.7°C (73.4 ± 3°F), if necessary by immersion in circulating water, and bring the water level in the pycnometer to its calibrated capacity. Determine the total weight of the pycnometer, specimen, and water.

NOTE 4—It normally takes about 15 to 20 min to eliminate air bubbles.

7.2.1 *Alternative to Weighing in 7.2*—The quantity of added water necessary to fill the pycnometer at the required temperature may be determined volumetrically using a buret accurate to 0.15 mL. Compute the total weight of the pycnometer, specimen, and water as follows:

$$C = 0.9975\ V_a + S + W$$

where:
C = weight of pycnometer with specimen and water to calibration mark, g,
V_a = volume of water added to pycnometer, mL,
S = weight of the saturated surface-dry specimen, and
W = weight of the empty pycnometer, g.

7.2.2 *Alternative to the Procedure in 7.2*—Use a Le Chatelier flask initially filled with water to a point on the stem between the 0 and the 1-mL mark. Record this initial reading with the flask and contents within the temperature range of 23 ± 1.7°C (73.4 ± 3°F). Add 55 ± 5 g of fine aggregate in the saturated surface-dry condition (or other weight as necessary to result in raising the water level to some point on the upper series of gradation). After all fine aggregate has been introduced, place the stopper in the flask and roll the flask in an inclined position, or gently whirl it in a horizontal circle so as to dislodge all entrapped air, continuing until no further bubbles rise to the surface. Take a final reading with the flask and contents within 1°C (1.8°F) of the original temperature.

7.3 Remove the fine aggregate from the pycnometer, dry to constant weight at a temperature of 110 ± 5°C (230 ± 9°F), cool in air at room temperature for 1 ± ½ h, and weigh.

7.3.1 If the Le Chatelier flask method is used, a separate sample portion is needed for the determination of absorption. Weigh a separate 500 ± 10-g portion of the saturated surface-dry fine aggregate, dry to constant weight, and reweigh.

7.4 Determine the weight of the pycnometer filled to its calibration capacity with water at 23 ± 1.7°C (73.4 ± 3°F).

7.4.1 *Alternative to Weighing in 7.4*—The quantity of water necessary to fill the empty pycnometer at the required temperature may be determined volumetrically using a buret accurate to 0.15 mL. Calculate the weight of the pycnometer filled with water as follows:

$$B = 0.9975\ V + W$$

where:
B = weight of flask filled with water, g,
V = volume of flask, mL, and
W = weight of the flask empty, g.

8. Bulk Specific Gravity

8.1 Calculate the bulk specific gravity, 23/23°C (73.4/73.4°F), as defined in Definitions E 12, as follows:

$$\text{Bulk sp gr} = A/(B + S - C)$$

where:
A = weight of oven-dry specimen in air, g,
B = weight of pycnometer filled with water, g,
S = weight of the saturated surface-dry specimen, and
C = weight of pycnometer with specimen and water to calibration mark, g.

8.1.1 If the Le Chatelier flask method was used, calculate the bulk specific gravity, 23/23°C, as follows:

$$\text{Bulk sp gr} = \frac{S_1\,[1 - ((S - A)/A)]}{0.9975\,(R_2 - R_1)}$$

where:
S_1 = weight of saturated surface-dry specimen used in Le Chatelier flask, g,
R_1 = initial reading of water level in Le Chatelier flask, and
R_2 = final reading of water level in Le Chatelier flask.

9. Bulk Specific Gravity (Saturated Surface-Dry Basis)

9.1 Calculate the bulk specific gravity, 23/23°C (73.4/73.4°F), on the basis of weight of saturated surface-dry aggregate as follows:

$$\text{Bulk sp gr (saturated surface-dry basis)} = S/(B + S - C)$$

9.1.1 If the Le Chatelier flask method was used, calculate the bulk specific gravity, 23/23°C, on the basis of saturated surface-dry aggregate as follows:

$$\text{Bulk sp gr (saturated surface-dry basis)} = \frac{S_1}{0.9975\,(R_2 - R_1)}$$

10. Apparent Specific Gravity

10.1 Calculate the apparent specific gravity, 23/23°C (73.4/73.4°F), as defined in Definitions E 12, as follows:

$$\text{Apparent sp gr} = A/(B + A - C)$$

11. Absorption

11.1 Calculate the percentage of absorption, as defined in Definitions C 125, as follows:

$$\text{Absorption, \%} = [(S - A)/A] \times 100$$

12. Report

12.1 Report specific gravity results to the nearest 0.01 and absorption to the nearest 0.1 %. The Appendix gives mathematical interrelationships among the three types of specific gravities and absorption. These may be useful in checking the consistency of reported data or calculating a value that was not reported by using other reported data.

12.2 If the fine aggregate was tested in a naturally moist condition other than the oven dried and 24 h-soaked condition, report the source of the sample and the procedures used to prevent drying prior to testing.

13. Precision and Bias

13.1 *Precision*—The estimates of precision of this test method (listed in Table 1) are based on results from the AASHTO Materials Reference Laboratory Reference Sample Program, with testing conducted by this test method and AASHTO Method T 84. The significant difference between the methods is that Test Method C 128 requires a saturation period of 24 ± 4 h, and Method T 84 requires a saturation period of 15 to 19 h. This difference has been

TABLE 1 Precision

	Standard Deviation (1S)[A]	Acceptable Range of Two Results (D2S)[A]
Single-Operator Precision:		
Bulk specific gravity (dry)	0.011	0.032
Bulk specific gravity (SSD)	0.0095	0.027
Apparent specific gravity	0.0095	0.027
Absorption[B], %	0.11	0.31
Multilaboratory Precision:		
Bulk specific gravity (dry)	0.023	0.066
Bulk specific gravity (SSD)	0.020	0.056
Apparent specific gravity	0.020	0.056
Absorption[B], %	0.23	0.66

[A] These numbers represent, respectively, the (1S) and (D2S) limits as described in Practice C 670. The precision estimates were obtained from the analysis of combined AASHTO Materials Research Laboratory reference sample data from laboratories using 15 to 19 h saturation times and other laboratories using 24 ± 4 h saturation time. Testing was performed on normal weight aggregates, and started with aggregates in the oven-dry condition.

[B] Precision estimates are based on aggregates with absorptions of less than 1 % and may differ for manufactured fine aggregates and fine aggregates having absorption values greater than 1 %.

found to have an insignificant effect on the precision indices. The data are based on the analyses of more than 100 paired test results from 40 to 100 laboratories.

13.2 *Bias*—Since there is no accepted reference material suitable for determining the bias for this test method, no statement on bias is being made.

APPENDIX

(Nonmandatory Information)

X1. INTERRELATIONSHIPS BETWEEN SPECIFIC GRAVITIES AND ABSORPTION AS DEFINED IN TEST METHODS C 127 AND C 128

X1.1 Let:

S_d = bulk specific gravity (dry-basis),
S_s = bulk specific gravity (SSD-basis),
S_a = apparent specific gravity, and
A = absorption in %.

Then:

$$S_s = (1 + A/100)S_d \quad (1)$$

$$S_a = \frac{1}{\frac{1}{S_d} - \frac{A}{100}} = \frac{S_d}{1 - \frac{AS_d}{100}} \quad (2)$$

$$\text{or } S_a = \frac{1}{\frac{1 + A/100}{S_s} - \frac{A}{100}} = \frac{S_s}{1 - \frac{A}{100}(S_s - 1)} \quad (2a)$$

$$A = \left(\frac{S_s}{S_d} - 1\right) 100 \quad (3)$$

$$A = \left(\frac{S_a - S_s}{S_a(S_s - 1)}\right) 100 \quad (4)$$

METHOD OF DETERMINING DENSITY OF SOLIDS

1. Scope and Definition

1 This method covers procedures for determining the density of solids. The density of solids is the ratio of the mass in air of a given volume of crushed solids to the total volume of solids.

2. Apparatus

1 The apparatus shall consist of the following:

(a) Volumetric flask, 500-mL capacity.

(b) Vacuum pump or aspirator connected to vacuum line.

(c) Oven of the forced draft type, automatically controlled to maintain a uniform temperature of 110 ± 5 °C throughout the oven.

(d) Balance, sensitive and accurate to 0.01 g, capacity 500 g or more.

(e) Thermometer, range 0 to 50 °C, graduated in 0.1 °C.

(f) Evaporating dish.

(g) Water dish.

(h) Sieves, U.S. Standard 4.75-mm (No. 4) and 600-μm (No. 30) conforming to ASTM Designation E 11, "Specifications for Wire-Cloth Sieves for Testing Purposes."

(i) Sample splitter suitable for splitting material passing 4.75-mm (No. 4) and 600-μm (No. 30) sieves.

3. Calibration of Volumetric Flask

1 The volumetric flask shall be calibrated for the mass of the flask and water at various temperatures. The flask and water are calibrated by direct determination of mass at the range of temperatures likely to be encountered in the laboratory. The calibration procedure is as follows.

2 Fill the flask with de-aired, distilled, and demineralized water to slightly below the calibration mark and place in a water bath which is at a temperature between 30 and 35 °C. Allow the flask to remain in the bath until the water in the flask reaches the temperature of the water bath. This may take several hours. Remove the flask from the water bath and adjust the water

level in the flask so that the bottom of meniscus is even with the calibration mark on the neck of the flask. Thoroughly dry the outside of the flask and remove any water adhering to the inside of the neck above the graduation, then determine the mass of the flask and water to the nearest 0.01 g. Immediately after determination of mass, shake the flask gently and determine the temperature of the water to the nearest 0.1 °C by immersing a thermometer to the middepth of the flask. Repeat the procedure outlined above at approximately the same temperature, than make two more determinations, one at room temperature and the other at approximately 5 °C less than room temperature. Draw a calibration curve showing the relation between temperature and corresponding values of mass of the flask plus water. Prepare a calibration curve for each flask used for density determination and maintain the curves as a permanent record. A typical calibration curve is shown in Fig. 1.

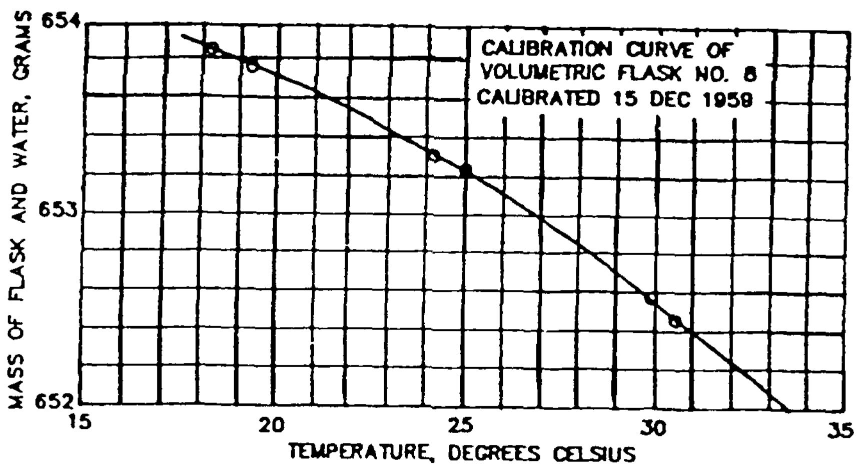


Fig. 1. Typical calibration curve of volumetric flask.

4. Sample

1 Crush the sample until it all passes a 4.75-mm (No. 4) sieve. With a sample splitter, separate out 120 to 150 g of representative crushed material. Pulverize this material to pass a 600-μm (No. 30) sieve. Oven-dry the crushed material to constant mass, determine the mass of the material to the nearest 0.01 g, and record the mass.

5. Procedure

1 After determination of mass, transfer the crushed material to a volumetric flask, taking care not to lose any material during this operation. To reduce possible error due to loss of material of known mass, the sample may have its mass determined after transfer to the flask. Fill the flask approximately half full with de-aired, distilled water. Shake the mixture well and allow it to stand overnight.

2 Then connect the flask to the vacuum line and apply a vacuum of approximately 99.99 Pa (750 mm of mercury) for approximately 4 to 6 h, agitating the flask at intervals during the evacuation process. Again, allow the flask to stand overnight. Finally, fill the flask with de-aired, distilled water to about 3/4 in. (19 mm) below the 500-mL graduation and again apply a vacuum to the flask until the suspension is de-aired, slowly and carefully remove the stopper from the flask, and observe the lowering of the water surface in the neck. If the water surface is lowered less than 1/8 in. (3.2 mm), the suspension can be considered sufficiently de-aired. Fill the flask until the bottom of the meniscus is coincident with the calibration line of the neck of the flask. Thoroughly dry the outside of the flask and remove the moisture on the inside of the neck by wiping with a paper towel. Determine the mass of the flask and contents to the nearest 0.01 g. Immediately after determination of mass, stir the suspension to assure uniform temperature, and determine the temperature of the suspension to the nearest 0.1 °C by immersing a thermometer to the middepth of the flask. Record the mass and temperature.

3 Compute the density of the solid, P_s, from the following formula:

$$P_s = \frac{W_s \ \gamma_w}{W_s + W_{fw} - W_{fws}}$$

where

W_s = the ovendry mass of the crushed rock sample, g

γ_w = density of water at test temperature, g/cc

W_{fw} = mass of flask plus water at test temperature, g (from calibration curve, Fig. 1)

W_{fws} = mass of flask plus water plus solids at test temperature, g

6. Report

1 The report shall include the following:

(a) The density of the solid.

(b) Ovendry mass of test sample.

(c) Water temperature during test.

RTH 109-93

METHOD OF DETERMINING EFFECTIVE (AS RECEIVED) AND DRY UNIT WEIGHTS AND TOTAL POROSITY OF ROCK CORES

1. Scope and Definition

1.1 This method covers the procedure for determining the effective unit weight, dry unit weight, and porosity of rock cores as defined in RTH 101. (This method covers determination of "total" porosity of a rock sample. Porosity calculated from the bulk volume and grain volume using the pulverization method is termed "total" since the pore volume obtained includes that of all closed pores. Other techniques give "effective" porosity since they measure the volume of interconnected pores only.)

2. Apparatus

2.1 The apparatus shall consist of the following:

(a) Balance having a capacity of 5 kg or more and sensitive and accurate to 0.5 g or 0.05 percent of the sample mass. Balances with capacities less than 5-kg shall be sensitive and accurate to 0.1 g.

(b) Wire basket of 4.75-mm (No. 4) mesh, diameter at least 50.8 mm (2 in.) greater than that of the core to be tested, walls at least one-half the height of the cylinder, and bail clearing the top the core by at least 25.4 mm (1 in.) at all points.

(c) Watertight container in which the wire basket may be suspended with a constant-level overflow spout at such a height that the wire basket, when suspended below the spout, will be at least 25.4 mm (1 in.) from the bottom of the container.

(d) Suspending apparatus suitable for suspending the wire basket in the container from the center of the balance platform or pan so that the basket will hang completely below the overflow spout and not be less than 25.4 mm (1 in.) from the bottom of the container.

(e) Thermometer, range 0 to 50 °C, graduated to 0.1 °C.

(f) Caliper or suitable measuring device capable of measuring lengths and diameters of test cores to the nearest 0.1 mm.

(g) Oven of the forced draft type, automatically controlled to maintain a uniform temperature 110 ± 5 °C throughout.

3. Sample

3.1 Select representative samples from the population and identify each sample. Individual sample mass should not exceed 5 kg.

4. Effective Unit Weight (As Received)

4.1 The test procedure for determining the effective unit weight of rock cores shall consist of the following steps:

(a) Determine the mass of the core (as received) to the nearest gram (0.1 g for 76.2-mm (3-in.) and smaller cores) (W_a) and the temperature in the working area near the core surface.

(b) Determine the bulk volume of the core in cubic centimetres by one of the following two methods:

(1) Determine the average length and diameter of the core from measurements of each of these dimensions at evenly spaced intervals covering the surface of the specimen. These measurements should be made to the nearest 0.1 mm. Calculate the volume using the formula $V = \frac{\pi}{4} d^2 L$, where V = volume, d = diameter of the core, and L = length of the core. (Note 1)

NOTE 1--If this method is used, the specimen should be sawed and machined to conform closely to the shape of a right cylinder or prism prior to determining its mass as in 4.1(a) above.

(2) Coat the surface of the core with wax or other suitable coating until it is watertight, making sure that the coating material does not measurably penetrate the pores of the core. Determine the mass of the specimen, after coating, to the nearest gram (or 0.1 g). The density of the coating material shall be determined. The volume of the coating on the core shall be determined by dividing the mass of coating by the density of the coating. Determine the volume of the coated core in cubic centimetres by liquid displacement. Subtract the volume of the coating material from the volume of the coated core to obtain the volume of the core (V) in cubic centimetres.

(c) Calculate the effective unit weight of the core from the following formula:

$$\gamma_e = \frac{W_a}{V}$$

where

γ_e = effective unit weight of the core, as received

W_a = mass of the core, in grams, as received

V = volume of the core, in cubic centrimetres

5. <u>Dry Unit Weight</u>

5.1 The test procedure for determining the dry unit weight of rock cores shall consist of the following steps:

(a) If a coating was utilized to waterproof the specimen as in 4.1(b)(2), remove it and, if applicable, brush to remove dust or elements of the coating. Then determine the mass of the core. (Note 2)

NOTE 2--If there is no mass loss in stripping or loss or gain in moisture, this mass should equal W_a.

(b) Crush the sample until it all passes a No. 4 sieve, taking care not to lose any material.

(c) Oven-dry the crushed material to constant mass W_b (constant mass is achieved when the mass loss is less than 0.1 percent of the sample mass during any 4-hour drying period), cool to room temperature, then record the mass and room temperature in the area of the test on the data sheet.

(d) Calculate the dry unit weight of the core from the following formula:

$$\gamma_d = \frac{W_b}{V}$$

where

γ_d = dry unit weight of the core

W_b = mass of the crushed, dried core, in grams

V = volume of the core, in cubic centrimetres

6. <u>Porosity</u>

6.1 The total porosity, n , may be determined from the dry unit weight and the gram unit weight of a sample. Determine the density of the solids, P_s , according to RTH 108.

Determine the total porosity by the expression:

$$n = \frac{(1 - W_b)}{P_s V} 100$$

where

n = total porosity, %

W_b = mass of crushed, dried core, g

V = volume of the core, cc

P_s = density of the solids, g/cc

(as determined by RTH - 108)

Designation: D 2845 – 90$^{\epsilon 1}$

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St. Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards. Copyright ASTM
If not listed in the current combined index, will appear in the next edition.

Standard Test Method for Laboratory Determination of Pulse Velocities and Ultrasonic Elastic Constants of Rock[1]

This standard is issued under the fixed designation D 2845; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

ε1 NOTE—Section 10 was changed editorially in December 1991.

1. Scope

1.1 This test method describes equipment and procedures for laboratory measurements of the pulse velocities of compression waves and shear waves in rock (1)[2] (Note 1) and the determination of ultrasonic elastic constants (Note 2) of an isotropic rock or one exhibiting slight anisotropy.

NOTE 1—The compression wave velocity as defined here is the dilatational wave velocity. It is the propagation velocity of a longitudinal wave in a medium which is effectively infinite in lateral extent. It should not be confused with the bar or rod velocity.

NOTE 2—The elastic constants determined by this test method are termed ultrasonic since the pulse frequencies used are above the audible range. The terms sonic and dynamic are sometimes applied to these constants but do not describe them precisely (2). It is possible that the ultrasonic elastic constants may differ from those determined by other dynamic methods.

1.2 This test method is valid for wave velocity measurements in both anisotropic and isotropic rocks although the velocities obtained in grossly anisotropic rocks may be influenced by such factors as direction, travel distance, and diameter of transducers.

1.3 The ultrasonic elastic constants are calculated from the measured wave velocities and the bulk density. The limiting degree of anisotropy for which calculations of elastic constants are allowed and procedures for determining the degree of anisotrophy are specified.

1.4 The values stated in U.S. customary units are to be regarded as the standard. The metric equivalents of U.S. customary units may be approximate.

2. Summary of Test Method

2.1 Details of essential procedures for the determination of the ultrasonic velocity, measured in terms of travel time and distance, of compression and shear waves in rock specimens include requirements of instrumentation, suggested types of transducers, methods of preparation, and effects of specimen geometry and grain size. Elastic constants may be calculated for isotropic or slightly anisotropic rocks, while anisotropy is reported in terms of the variation of wave velocity with direction in the rock.

3. Significance and Use

3.1 The primary advantages of ultrasonic testing are that it yields compression and shear wave velocities, and ultrasonic values for the elastic constants of intact homogeneous isotropic rock specimens (3). Elastic constants are not to be calculated for rocks having pronounced anisotropy by procedures described in this test method. The values of elastic constants often do not agree with those determined by static laboratory methods or the *in situ* methods. Measured wave velocities likewise may not agree with seismic velocities, but offer good approximations. The ultrasonic evaluation of rock properties is useful for preliminary prediction of static properties. The test method is useful for evaluating the effects of uniaxial stress and water saturation on pulse velocity. These properties are in turn useful in engineering design.

3.2 The test method as described herein is not adequate for measurement of stress-wave attenuation. Also, while pulse velocities can be employed to determine the elastic constants of materials having a high degree of anisotropy, these procedures are not treated herein.

4. Apparatus

4.1 *General*—The testing apparatus (Fig. 1) should have impedance matched electronic components and shielded leads to ensure efficient energy transfer. To prevent damage to the apparatus allowable voltage inputs should not be exceeded.

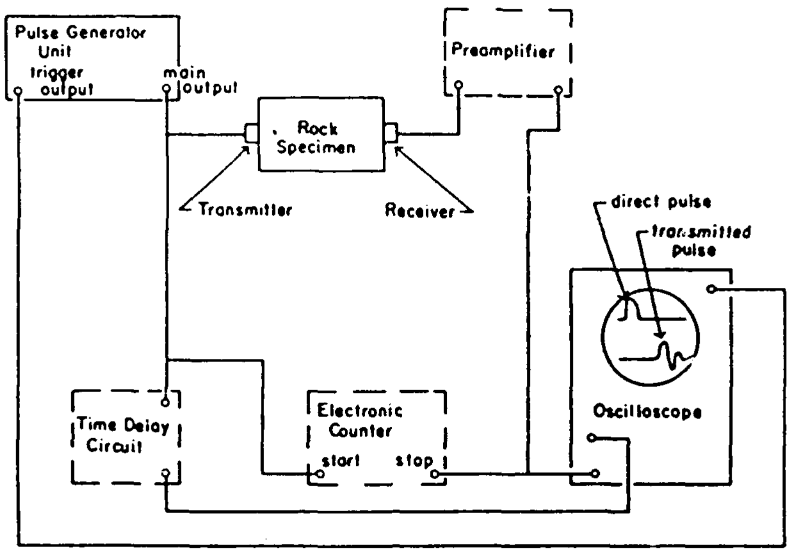
4.2 *Pulse Generator Unit*—This unit shall consist of an electronic pulse generator and external voltage or power amplifiers if needed. A voltage output in the form of either rectangular pulse or a gated sine wave is satisfactory. The generator shall have a voltage output with a maximum value after amplification of at least 50 V into a 50-Ω impedance load. A variable pulse width, with a range of 1 to 10 μs is desirable. The pulse repetition rate may be fixed at 60 repetitions per second or less although a range of 20 to 100 repetitions per second is recommended. The pulse generator shall also have a trigger-pulse output to trigger the oscilloscope. There shall be a variable delay of the main-pulse output with respect to the trigger-pulse output, with a minimum range of 0 to 20 μs.

4.3 *Transducers*—The transducers shall consist of a transmitter which converts electrical pulses into mechanical pulses and a receiver which converts mechanical pulses into electrical pulses. Environmental conditions such as ambient temperature, moisture, humidity, and impact should be

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved June 29, 1990. Published October 1990. Originally published as D 2845 – 69. Last previous edition D 2845 – 83.

[2] The boldface numbers in parentheses refer to the list of references at the end of this test method.



NOTE—Components shown by dashed lines are optional, depending on method of travel-time measurement and voltage sensitivity of oscilloscope.

FIG. 1 Schematic Diagram of Typical Apparatus

considered in selecting the transducer element. Piezoelectric elements are usually recommended, but magnetostrictive elements may be suitable. Thickness-expander piezoelectric elements generate and sense predominately compression-wave energy; thickness-shear piezoelectric elements are preferred for shear-wave measurements. Commonly-used piezoelectric materials include ceramics such as lead-zirconate-titanate for either compression or shear, and crystals such as a-c cut quartz for shear. To reduce scattering and poorly defined first arrivals at the receiver, the transmitter shall be designed to generate wavelengths at least three times the average grain size of the rock.

NOTE 3—Wavelength is the wave velocity in the rock specimen divided by the resonance frequency of the transducer. Commonly used frequencies range from 75 kHz to 3 MHz.

4.3.1 In laboratory testing, it may be convenient to use unhoused transducer elements. But if the output voltage of the receiver is low, the element should be housed in metal (grounded) to reduce stray electromagnetic pickup. If protection from mechanical damage is necessary, the transmitter as well as the receiver may be housed in metal. This also allows special backings for the transducer element to alter its sensitivity or reduce ringing (4). The basic features of a housed element are illustrated in Fig. 2. Energy transmission between the transducer element and test specimen can be improved by (*1*) machining or lapping the surfaces of the face plates to make them smooth, flat, and parallel, (*2*) making the face plate from a metal such as magnesium whose characteristic impedance is close to that of common rock types, (*3*) making the face plate as thin as practicable, and (*4*) coupling the transducer element to the face plate by a thin layer of an electrically conductive adhesive, an epoxy type being suggested.

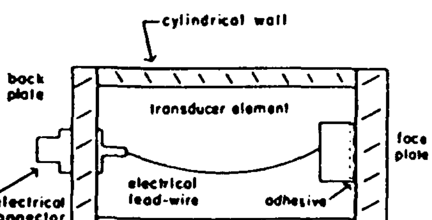


FIG. 2 Basic Features of a Housed Transmitter or Receiver

4.3.2 Pulse velocities may also be determined for specimens subjected to uniaxial states of stress. The transducer housings in this case will also serve as loading platens and should be designed with thick face plates to assure uniform loading over the ends of the specimen (5).

NOTE 4—The state of stress in many rock types has a marked effect on the wave velocities. Rocks *in situ* are usually in a stressed state and therefore tests under stress have practical significance.

4.4 *Preamplifier*—A voltage preamplifier is required if the voltage output of the receiving transducer is relatively low or if the display and timing units are relatively insensitive. To preserve fast rise times, the frequency response of the preamplifier shall drop no more that 2 dB over a frequency range from 5 kHz to 4 × the resonance frequency of the receiver. The internal noise and gain must also be considered in selecting a preamplifier. Oscilloscopes having a vertical-signal output can be used to amplify the signal for an electronic counter.

4.5 *Display and Timing Unit*—The voltage pulse applied to the transmitting transducer and the voltage output from the receiving transducer shall be displayed on a cathode-ray oscilloscope for visual observation of the waveforms. The oscilloscope shall have an essentially flat response between a frequency of 5 kHz and 4 × the resonance frequency of the transducers. It shall have dual beams or dual traces so that the two waveforms may be displayed simultaneously and their amplitudes separately controlled. The oscilloscope shall be triggered by a triggering pulse from the pulse generator. The timing unit shall be capable of measuring intervals between 2 μs and 5 ms to an accuracy of 1 part in 100. Two alternative classes of timing units are suggested, the respective positions of each being shown as dotted outlines in the block diagram in Fig. 1: (*1*) an electronic counter with provisions for time interval measurements, or (*2*) a time-delay circuit such as a continuously variable-delay generator, or a delayed-sweep feature on the oscilloscope. The travel-time measuring circuit shall be calibrated periodically with respect to its accuracy and linearity over the range of the instrument. The calibration shall be checked against signals transmitted by the National Institute of Standards and Technology radio station WWV, or against a crystal controlled time-mark or frequency generator which can be referenced back to the signals from WWV periodically. It is recommended that the calibration of the time measuring circuit be checked at least once a month and after any severe impact which the instrument may receive.

5. Test Specimens

5.1 *Preparation*--Exercise care in core drilling, handling, sawing, grinding, and lapping the test specimen to minimize the mechanical damage caused by stress and heat. It is recommended that liquids other than water be prevented from contacting the specimen, except when necessary as a coupling medium between specimen and transducer during the test. The surface area under each transducer shall be sufficiently plane that a feeler gage 0.001 in. (0.025 mm) thick will not pass under a straightedge placed on the surface. The two opposite surfaces on which the transducers will be placed shall be parallel to within 0.005 in./in. (0.1 mm/20 mm) of lateral dimension (Fig. 3). If the pulse velocity measurements are to be made along a diameter of a core, the above tolerance then refers to the parallelism of the lines of contact between the transducers and curved surface of the rock core. Moisture content of the test specimen can affect the measured pulse velocities (see 6.2). Pulse velocities may be determined on the velocity test specimen for rocks in the oven-dry state (0 % saturation), in a saturated condition (100 % saturation), or in any intermediate state. If the pulse velocities are to be determined with the rock in the same moisture condition as received or as exists underground, care must be exercised during the preparation procedure so that the moisture content does not change. In this case it is suggested that both the sample and test specimen be stored in moisture-proof bags or coated with wax and that dry surface-preparation procedures be employed. If results are desired for specimens in the oven-dried condition, the oven temperature shall not exceed 150°F (66°C). The specimen shall remain submerged in water up to the time of testing when results are desired for the saturated state.

5.2 *Limitation on Dimensions*—It is recommended that the ratio of the pulse-travel distance to the minimum lateral dimension not exceed 5. Reliable pulse velocities may not be measurable for high values of this ratio. The travel distance of the pulse through the rock shall be at least 10 × the average grain size so that an accurate average propagation velocity may be determined. The grain size of the rock sample, the natural resonance frequency of the transducers, and the minimum lateral dimension of the specimen are interrelated factors which affect test results. The wavelength corresponding to the dominant frequency of the pulse train in the rock is approximately related to the natural resonance frequency of the transducer and the pulse-propagation velocity, (compression or shear) as follows:

$$\Lambda \approx V/f, \quad (1)$$

where:

Λ = dominant wavelength of pulse train, in. (or m),
V = pulse propagation velocity (compression or shear), in./s (or m/s), and
f = natural resonance frequency of transducers, Hz.

The minimum lateral dimension of the test specimen shall be at least 5 × the wavelength of the compression wave so that the true dilational wave velocity is measured (Note 5), that is,

$$D \geqslant 5\Lambda, \quad (2)$$

where:

D = minimum lateral dimension of test specimen, in. (or m).

The wavelength shall be at least 3 × the average grain size (See 4.3) so that

$$\Lambda \geqslant 3d, \quad (3)$$

where:

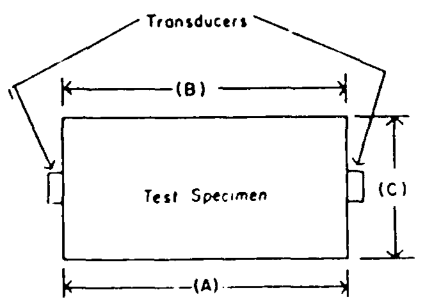
d = average grain size, in. (or m).

Equations 1, 2, and 3 can be combined to yield the relationship for compression waves as follows:

$$D \geq 5(V_p/f) \geq 15\, d, \quad (4)$$

where:

V_p = pulse propagation velocity (compression), in./s (or m/s).



NOTE—(A) must be within 0.1 mm of (B) for each 20 mm of width (C).

FIG. 3 Specification for Parallelism

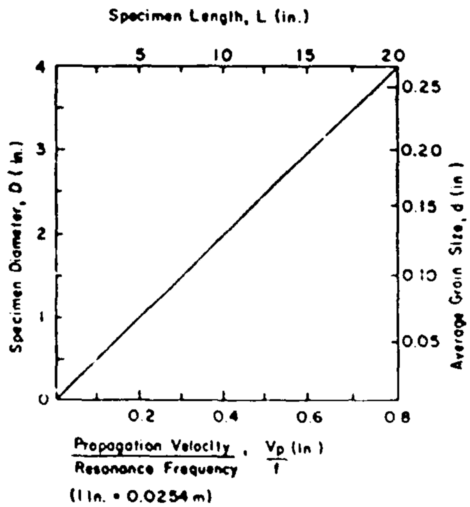


FIG. 4 Graph Showing Allowable Values of Specimen Diameter and Grain Size Versus the Ratio of Propagation Velocity to Resonance Frequency

Since V_p and d are inherent properties of the material, f and D shall be selected to satisfy Eq 4 (Fig. 4) for each test specimen. For any particular value of V_p/f the permissible values of specimen diameter D lie above the diagonal line in Fig. 4, while the permissible values of grain size d lie below the diagonal line. For a particular diameter, the permissible values for specimen length L lie to the left of the diagonal line.

NOTE 5—Silaeva and Shamina (6) found the limiting ratio of diameter to wavelength to be about 2 for metal rods. Data obtained by Cannady (3) on rock indicate the limiting ratio is at least 8 for a specimen length-to-diameter ratio of about 8.

6. Procedure

6.1 *Determination of Travel Distance and Density*—Mark off the positions of the transducers on the specimen so that the line connecting the centers of the transducer contact areas is not inclined more than 2° (approximately 0.1 in. in 3 in. (1 mm in 30 mm)) with a line perpendicular to either surface. Then measure the pulse-travel distance from center to center of the transducer contact area to within 0.1 %. The density of the test specimen is required in the calculation of the ultrasonic elastic constants (see 7.2). Determine the density of the test specimen from measurements of its mass and its volume calculated from the average external dimensions. Determine the mass and average dimensions within 0.1 %. Calculate the density as follows:

$$\rho = m/V$$

where:

ρ = density, lb sec^2/in.^{4} (or kg/m^3),
m = mass of test specimen, lb sec^2/386.4 in. (or kg), and
V = volume of test specimen, in^3 (or m^3).

6.2 *Moisture Condition*—The moisture condition of the sample shall be noted and reported as 8.1.3.

6.3 *Determination of Pulse-Travel Time:*

6.3.1 Increase the voltage output of the pulse generator, the gain of the amplifier, and the sensitivity of the oscilloscope and counter to an optimum level, giving a steeper pulse front to permit more accurate time measurements. The optimum level is just below that at which electromagnetic noise reaches an intolerable magnitude or triggers the counter at its lowest triggering sensitivity. The noise level shall not be greater than one tenth of the amplitude of the first peak of the signal from the receiver. Measure the travel time to a precision and accuracy of 1 part in 100 for compression waves and 1 part in 50 for shear waves by (*1*) using the delaying circuits in conjunction with the oscilloscope (see 6.1.1) or (*2*) setting the counter to its highest usable precision, (see 6.3.2).

6.3.1.1 The oscilloscope is used with the time-delay circuit to display both the direct pulse and the first arrival of the transmitted pulse, and to measure the travel time. Characteristically, the first arrival displayed on the oscilloscope consists of a curved transition from the horizontal zero-voltage trace followed by a steep, more or less linear, trace. Select the first break in a consistent manner for both the test measurement and the zero-time determination. Select it either at the beginning of the curved transition region or at the zero-voltage intercept of the straight line portion of the first arrival.

6.3.1.2 The counter is triggered to start by the direct pulse applied to the transmitter and is triggered to stop by the first arrival of the pulse reaching the receiver. Because a voltage change is needed to trigger the counter, it can not accurately detect the first break of a pulse. To make the most accurate time interval measurements possible, increase the counter's triggering sensitivity to an optimum without causing spurious triggering by extraneous electrical noise.

6.3.2 Determine the zero time of the circuit including both transducers and the travel-time measuring device and apply the correction to the measured travel times. This factor will remain constant for a given rock and stress level if the circuit characteristics do not change. Determine the zero time accordingly to detect any changes. Determine it by (*1*) placing the transducers in direct contact with each other and measuring the delay time directly, or (*2*) measuring the apparent travel time of some uniform material (such as steel) as a function of length, and then using the zero-length intercept of the line through the data points as the correction factor.

6.3.3 Since the first transmitted arrival is that of the compression wave, its detection is relatively easy. The shear-wave arrival, however, may be obscured by vibrations due to ringing of the transducers and reflections of the compression wave. The amplitude of the shear wave relative to the compression wave may be increased and its arrival time determined more accurately by means of thickness shear-transducer elements. This type of element generates some compressional energy, so that both waves may be detected. Energy transmission between the specimen and each transducer may be improved by using a thin layer of a coupling medium such as phenyl salicylate, high-vacuum grease, or resin, and by pressing the transducer against the specimen with a small seating force.

6.3.4 For specimens subjected to uniaxial stress fields, first arrivals of compression waves are usually well defined. However, the accurate determination of shear-wave first arrivals for specimens under stress is complicated by mode conversions at the interfaces on either side of the face plate and at the free boundary of the specimen (4). Shear-wave arrivals are therefore difficult to determine and experience is required for accurate readings.

6.4 *Ultrasonic Elastic Constants*—The rock must be isotropic or possess only a slight degree of anisotropy if the ultrasonic elastic constants are to be calculated (Section 7). In order to estimate the degree of anisotropy of the rock, measure the compression-wave velocity in three orthogonal directions, and in a fourth direction oriented at 45° from any one of the former three directions if required as a check. Make these measurements with the same geometry, that is, all between parallel flat surfaces or all across diameters. The equations in 7.2 for an isotropic medium shall not be applied if any of the three compression-wave velocities varies by more than 2 % from their average value. The error in E and G (see 7.2) due to both anisotropy and experimental error will then normally not exceed 6 %. The maximum possible error in μ, λ, and K depends markedly upon the relative values of V_p and V_s as well as upon testing errors and anisotropy. In common rock types the respective percent of errors for μ, λ, and K may be large as or even higher than 24, 36, and 6. For greater anisotropy, the possible percent of error in the elastic constants would be still greater.

7. Calculation

7.1 Calculate the propagation velocities of the compression and shear waves, V_p and V_s respectively, as follows:

$$V_p = L_p/T_p$$
$$V_s = L_s/T_s$$

where:
V = pulse-propagation velocity, in./s (or m/s),
L = pulse-travel distance, in. (or m),
T = effective pulse-travel time (measured time minus zero time correction), s,
and subscripts $_p$ and $_s$ denote the compression wave and shear wave, respectively.

7.2 If the degree of velocity anisotropy is 2 % or less, as specified in 6.4, calculate the ultrasonic elastic constants as follows:

$$E = [\rho V_s^2(3V_p^2 - 4V_s^2)]/(V_p^2 - V_s^2)$$

where:
E = Young's modulus of elasticity, psi (or Pa), and
ρ = density, lb/in.^{3} (or kg/m^3);

$$G = \rho V_s^2$$

where:
G = modulus of rigidity or shear modulus, psi (or Pa);

$$\mu = (V_p^2 - 2V_s^2)/[2(V_p^2 - V_s^2)]$$

where:
μ = Poisson's ratio;

$$\lambda = \rho(V_p^2 - 2V_s^2)$$

where:
λ = Lamè's constant, psi (or Pa); and

$$K = \rho(3V_p^2 - 4V_s^2)/3$$

where:
K = bulk modulus, psi (or Pa).

8. Report

8.1 The report shall include the following:

8.1.1 Identification of the test specimen including rock type and location,

8.1.2 Density of test specimen,

8.1.3 General indication of moisture condition of sample at time of test such as as-received, saturated, laboratory air dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation.

8.1.4 Degree of anisotropy expressed as the maximum percent deviation of compression-pulse velocity from the average velocity determined from measurements in three directions,

8.1.5 Stress level of specimens,

8.1.6 Calculated pulse velocities for compression and shear waves with direction of measurement,

8.1.7 Calculated ultrasonic elastic constants (if desired and if degree of anisotropy is not greater than specified limit),

8.1.8 Coupling medium between transducers and specimen, and

8.1.9 Other data such as physical properties, composition, petrography, etc., if determined.

9. Precision and Bias

9.1 *Precision*—Due to the nature of rock materials tested by this test method, it is either not feasible or too costly at this time to produce multiple specimens which have uniform physical properties. Any variation observed in the data is just as likely to be due to specimen variation as to operator or laboratory testing variation. Subcommittee D18.12 welcomes proposals that would allow for development of a valid precision statement.

9.2 *Bias*—There is no accepted reference value for this test method; therefore, bias cannot be determined.

10. Keywords

10.1 compression testing; isotrophy; ultrasonic testing; velocity-pulse

REFERENCES

(1) Simmons, Gene, "Ultrasonics in Geology," *Proceedings, Inst. Electrical and Electronic Engineers*, Vol 53, No. 10, 1965, pp. 1337–1345.

(2) Whitehurst, E. A., *Evaluation of Concrete Properties from Sonic Tests*, Am. Concrete Inst., Detroit, Mich., and the Iowa State Univ. Press, Ames, Iowa, 1966, pp. 1–2.

(3) Cannaday, F. X., "Modulus of Elasticity of a Rock Determined by Four Different Methods," *Report of Investigations U.S. Bureau of Mines 6533*, 1964.

(4) Thill, R. E., McWilliams, J. R., and Bur, T. R., "An Acoustical Bench for an Ultrasonic Pulse System," *Report of Investigations U.S. Bureau of Mines 7164*, 1968.

(5) Gregory, A. R., "Shear Wave Velocity Measurements of Sedimentary Rock Samples under Compression," *Rock Mechanics*, Pergamon Press, New York, N.Y., 1963, pp. 439–471.

(6) Silaeva, O. I., and Shamina, O. G., "The Distribution of Elastic Pulses in Cylindrical Specimens," *USSR Academy of Sciences (Izvestiya), Geophysics Series*, 1958, pp. 32–43, (English ed., Vol 1, No. 1, 1958, pp. 17–24).

Designation: D 2938 - 86 RTH 111-89

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa. 19103

Standard Test Method for UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMENS[1]

This standard is issued under the fixed designation D 2938; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This test method covers the determination of the unconfined compressive strength of intact cylindrical rock specimens.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

D 4543 Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances[2]

E 4 Practices for Load Verification of Testing Machines[3]

E 122 Recommended Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[4]

3. Apparatus

3.1 *Loading Device*, to apply and measure axial load on the specimens, of sufficient capacity to apply load at a rate conforming to the requirements set forth in 5.3. It shall be verified at suitable time intervals in accordance with the procedures given in Practices E 4, and comply with the requirements prescribed therein.

3.2 *Bearing Surfaces*—The testing machine shall be equipped with two steel bearing blocks having a Rockwell hardness of not less than HRC 58 (Note 1). One of the blocks shall be spherically seated and the other a plain rigid block. The bearing faces shall not depart from a plane by more than 0.0005 in. (0.013 mm) when the blocks are new and shall be maintained within a permissible variation of 0.001 in. (0.025 mm). The diameter of the spherically seated bearing face shall be at least as large as that of the test specimen but shall not exceed twice the diameter of the test specimen. The center of the sphere for the spherically seated block shall coincide with the bearing face of the specimen. The movable portion of the bearing block shall be held closely in the spherical seat, but the design shall be such that the bearing face can be rotated and tilted through small angles in any direction.

NOTE 1—False platens, with plane bearing faces conforming to the requirements of this method may be used. These shall consist of disks about ½ to ¾ in. (12.7 to 19.05 mm) thick, oil-hardened to more than HRC 58, and surface ground. With abrasive rocks, these platens tend to roughen after a number of specimens have been tested, and hence need to be resurfaced from time to time.

4. Test Specimens

4.1 Prepare test specimens in accordance with Practice D 4543.

4.2 The moisture condition of the specimen

---

[1] This method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Oct. 31, 1986. Published December 1986. Originally published as D 2938 – 71. Last previous edition D 2938 – 79.

[2] *Annual Book of ASTM Standards*, Vol 4.08.

[3] *Annual Book of ASTM Standards*, Vols 03.01, 04.01, 07.01, and 08.03.

[4] *Annual Book of ASTM Standards*, Vol 14.02.

at time of test can have a significant effect upon the indicated strength of the rock. Good practice generally dictates that laboratory tests be made upon specimens representative of field conditions. Thus it follows that the field moisture condition of the specimen should be preserved until time of test. On the other hand, there may be reasons for testing specimens at other moisture contents including zero. In any case, tailor the moisture content of the test specimen to the problem at hand and report it in accordance with 7.1.7.

5. Procedure

5.1 Check the ability of the spherical seat to rotate freely in its socket before each test.

5.2 Wipe clean the bearing faces of the upper and lower bearing blocks and of the test specimen and place the test specimen on the lower bearing block. As the load is gradually brought to bear on the specimen, adjust the movable portion of the spherically seated block so that uniform seating is obtained.

5.3 Many rock types fail in a violent manner when loaded to failure in compression. A protective shield should be placed around the test specimen to prevent injury from flying rock fragments.

5.4 Apply the load continuously and without shock to produce an approximately constant rate of load or deformation such that failure will occur within 5 to 15 min of loading.

NOTE 2—Results of tests by several investigators have shown that strain rates within this range will provide strength values that are reasonably free of rapid loading effects and reproducible within acceptable tolerances.

6. Calculation

6.1 Calculate the unconfined compressive strength of the specimen by dividing the maximum load carried by the specimen during the test by the cross-sectional area calculated and express the result to the nearest 10 psi (68.9 kPa).

7. Report

7.1 The report should include the following:

7.1.1 Source of sample including project name and location, and, if known, storage environment. The location is frequently specified in terms of the borehole number and depth of specimen from collar of hole.

7.1.2 Physical description of sample including rock type; location and orientation of apparent weakness planes, bedding planes, and schisotosity; large inclusions or inhomogeneities, if any.

7.1.3 Date of sampling and testing.

7.1.4 Specimen diameter and length, conformance with dimensional requirements (see Note 4).

7.1.5 Rate of loading or deformation rate.

7.1.6 General indication of moisture condition of sample at time of test such as as-received, saturated, laboratory air dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation.

7.1.7 Unconfined compressive strength for each specimen as calculated average unconfined compressive strength of all specimens tested, standard deviation or coefficient of variation.

7.1.8 Type and location of failure. A sketch of the fractured sample is recommended.

7.1.9 Other available physical data.

NOTE 3—If only cores with a length-to-diameter ratio (L/D) of less than 2 are available, these may be used for the test and this fact noted in the report. However, a length-to-diameter ratio (L/D) as close to 2 as possible should be used. This apparent compressive strength may then be corrected in accordance with the following equation:

$$C = C_a/(0.88 + (0.24b/h))$$

where:

C = computed compressive strength of an equivalent L/D = 2 specimen,
C_a = measured compressive strength of the specimen tested,
b = test core diameter, and
h = test core height.

NOTE 4—The number of specimens tested may depend upon availability of specimens, but normally a minimum of ten is preferred. The number of specimens tested should be indicated. The statistical basis of relating the required number of specimens to the variability of measurements is given in Recommended Practice E 122.

8. Precision and Bias

8.1 The variability of rock and resultant in-

ability to determine a true reference value prevent development of a meaningful statement of bias. Data are being evaluated to determine the precision of this test method. In addition, the subcommittee is seeking pertinent data from users of the method.

Designation: D 2936 – 84 (Reapproved 1989)$^{\epsilon2}$

Standard Test Method for Direct Tensile Strength of Intact Rock Core Specimens[1]

This standard is issued under the fixed designation D 2936; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

$^{\epsilon1}$ Note—Section 9 was changed editorially in July 1989.
$^{\epsilon2}$ Note—Section 10 was added editorially in December 1991.

1. Scope

1.1 This test method covers the determination of the direct tensile strength of intact cylindrical rock specimens.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

E 4 Practices for Load Verification of Testing Machines[2]

E 122 Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[3]

3. Significance and Use

3.1 Rock is much weaker in tension than in compression. Thus, in determining the failure condition for a rock structure, many investigators employ the tensile strength of the component rock as the failure strength for the structure. Direct tensile stressing of rock is the most basic test for determining the tensile strength of rock.

4. Apparatus

4.1 *Loading Device,* to apply and measure axial load on the specimen, of sufficient capacity to apply the load at a rate conforming to the requirements of 6.2. The device shall be verified at suitable time intervals in accordance with the procedures given in Practices E 4 and shall comply with the requirements prescribed therein.

4.2 *Caps*—Cylindrical metal caps that, when cemented to the specimen ends, provide a means through which the direct tensile load can be applied. The diameter of the metal caps shall not be less than that of the test specimen, nor shall it exceed the test specimen diameter by more than 0.0625 in. (1.6 mm). Caps shall have a thickness of at least 1¼ in. (32 mm). Caps shall be provided with a suitable linkage system for load transfer from the loading device to the test specimen. The linkage system shall be so designed that the load will be transmitted through the axis of the test specimen without the application of bending or torsional stresses. The length of the linkages at each end shall be at least two times the diameter of the metal end caps. One such system is shown in Fig. 1.

Note 1—Roller or link chain of suitable capacity has been found to perform quite well in this application. Because roller chain flexes in one plane only, the upper and lower segments should be positioned at right angles to each other to effectively reduce bending in the specimen. Ball-and-socket, cable, or similar arrangements have been found to be generally unsuitable as their tendency for bending and twisting makes the assembly unable to transmit a purely direct tensile stress to the test specimen.

5. Test Specimens

5.1 Test specimens shall be right circular cylinders within the following tolerances:

5.1.1 The sides of the specimen shall be generally smooth and free of abrupt irregularities with all the elements straight to within 0.020 in. (0.50 mm) over the full length of the specimen. The deviation from straightness of the elements shall be determined by either Method A or Method B as follows:

5.1.1.1 *Method A*—Roll the cylindrical specimen on a smooth flat surface and measure the height of the maximum gap between the specimen and the flat surface with a feeler gage. If the maximum gap exceeds 0.020 in. (0.50 mm), the specimen does not meet the required tolerance for straightness of the elements. The flat test surface on which the specimen is rolled shall not depart from a plane by more than 0.0005 in. (15 μm).

5.1.1.2 *Method B:*

5.1.1.2.1 Place the cylindrical surface of the specimen on a V-block that is laid flat on a surface. The smoothness of the surface shall not depart from a plane by more than 0.0005 in. (15 μm).

5.1.1.2.2 Place a dial indicator in contact with the top of the specimen as shown in Fig. 2, and observe the dial reading as the specimen is moved from one end of the V-block to the other along a straight line.

5.1.1.2.3 Record the maximum and minimum readings on the dial gage and calculate the difference, Δ_0. Repeat the same operations by rotating the specimen for every 90°, and obtain the differences, Δ_{90}, Δ_{180}, and Δ_{270}. The maximum value of these four differences shall be less than 0.020 in. (0.50 mm).

5.1.2 Cut the ends of the specimen parallel to each other,

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Sept. 28, 1984. Published November 1984. Originally published as D 2936 – 71. Last previous edition D 2936 – 71.

[2] *Annual Book of ASTM Standards,* Vols 03.01, 04.02, and 08.03.

[3] *Annual Book of ASTM Standards,* Vol 14.02.

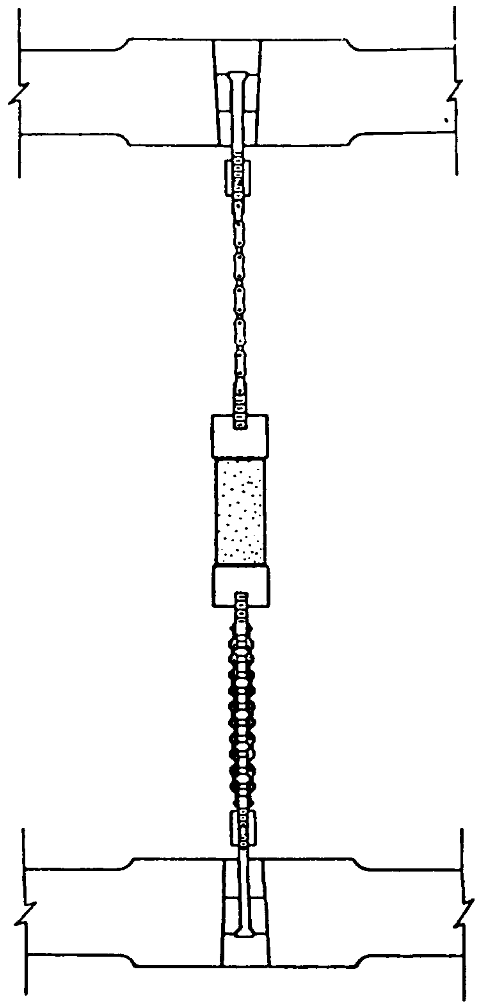


FIG. 1 Direct Tensile-Strength Test Assembly

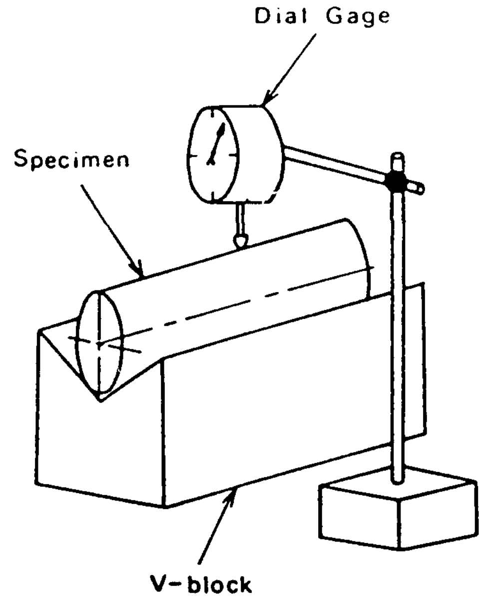


FIG. 2 Assembly for Determining the Straightness of the Cylindrical Surface

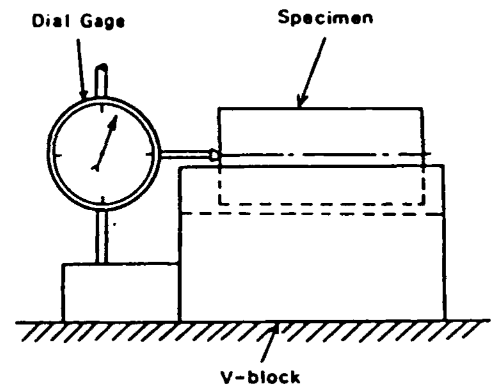


FIG. 3 Assembly for Determining the Perpendicularity of End Surfaces to the Specimen Axis

generally smooth (Note 2), and at right angles to the longitudinal axis. The ends shall not depart from perpendicularity to the axis of the specimen by more than 0.25°, approximately 0.01 in. (0.3 mm) in 2 in. (50.0 mm). The perpendicularity of the end surfaces to the longitudinal axis shall be determined by the similar setup as for the cylindrical surface (Fig. 3), except that the dial gage is mounted near the end of the V-block. Move the mounting pad horizontally so that the dial gage runs across the end surface of the specimen along a diametral direction. Take care to ensure that one end of the mounting pad maintains intimate contact with the end surface of the V-block during moving. Record the dial gage readings and calculate the difference between the maximum and the minimum values, Δ_1. Rotate the specimen 90° and repeat the same operations and calculate the difference, Δ_2. Turn the specimen around and repeat the same measurement procedures for the other end surface and obtain the difference values Δ'_1 and Δ'_2. The perpendicularity will be considered to have been met when:

$$\frac{\Delta_I}{D} \text{ and } \frac{\Delta'_I}{D} \leq 0.005$$

where:
I = 1 or 2, and
D = diameter.

The smoothness of the end surfaces can be determined by taking dial gage readings for every ⅛ in. (3.2 mm) during the perpendicularity measurements. The closeness of the readings are expected to provide a smooth curve of the end surface along the specific diametral plane. The smoothness requirement is met when the slope along any part of the curve is less than 0.25°.

NOTE 2—In this test method, the condition of the specimen ends with regard to the degree of flatness and smoothness is not as critical as it is, for example, in compression tests where good bearing is a prerequisite. In direct-tension tests it is more important that the ends be parallel to each other and perpendicular to the longitudinal axis of the specimen in order to facilitate the application of a direct tensile load. End surfaces, such as result from sawing with a diamond cut-off wheel, are entirely adequate. Grinding, lapping, or polishing beyond this point serves no useful purpose, and in fact, may adversely affect the adhesion of the cementing medium.

5.1.3 The specimen shall have a length-to-diameter ratio (L/D) of 2.0 to 2.5 (Note 3) and a diameter of not less than

NX wireline core size, approximately 1⅞ in. (48 mm) (Note 4).

NOTE 3—In direct-tension tests, the specimen should be free to select and fail on the weakest plane within its length. This degree of freedom becomes less as the specimen length diminishes. When cores of shorter than standard length must be tested, make suitable notation of this fact in the test report.

NOTE 4—It is desirable that the diameter of rock tension specimens be at least ten times greater than the diameter of the largest mineral grain. It is considered that the specified minimum specimen diameter of approximately 1⅞ in. (48 mm) will satisfy this criterion in the majority of cases. It may be necessary in some instances to test specimens that do not comply with this criterion. In this case, and particularly when cores of diameter smaller than the specified minimum must be tested because of the unavailability of larger size specimens (as is often the case in the mining industry) make suitable notation of these facts in the test report, and mention the grain size.

5.1.4 Determine the diameter of the test specimen to the nearest 0.01 in. (0.25 mm) by averaging two diameters measured at right angles to each other at about midlength of the specimen. Use this average diameter for calculating the cross-sectional area. Determine the length of the test specimen to the nearest 0.01 in. by averaging two height measurements along the diameter.

5.1.5 The moisture condition of the specimen at the time of test can have a significant effect upon the indicated strength of the rock. Good practice generally dictates that laboratory tests be made upon specimens representative of field conditions or conditions expected under the operating environment. However, consider also that there may be reasons for testing specimens at other moisture contents, or with none. In any case, relate the moisture content of the test specimen to the problem at hand and report the content in accordance with 8.1.6.

6. Procedure

6.1 Cement the metal caps to the test specimen to ensure alignment of the cap axes with the longitudinal axis of the specimen (Note 5). The thickness of the cement layer should not exceed 1/16 in. (1.6 mm) at each end. The cement layer must be of uniform thickness to ensure parallelism between the top surfaces of the metal caps attached to both ends of the specimens. This should be checked before the cement is hardened (Note 5) by measuring the length of the specimen and end-cap assembly at three locations 120° apart and near the edge. The maximum difference between these measurements should be less than 0.005 in. (0.13 mm) for each 1.0 in. (25.0 mm) of specimen diameter. After the cement has hardened sufficiently to exceed the tensile strength of the rock, place the test specimen in the testing machine, making certain that the load transfer system is properly aligned.

NOTE 5—In cementing the metal caps to the test specimens, use jigs and fixtures of suitable design to hold the caps and specimens in proper alignment until the cement has hardened. The chucking arrangement of a machine lathe or drill press is also suitable. The cement used should be one that sets at room temperature. Epoxy resin formulations of rather stiff consistency and similar to those used as a patching and filling compound in automobile body repair work have been found to be a suitable cementing medium.

6.2 Apply the tensile load continuously and without shock to failure. Apply the load or deformation at an approximately constant rate such that failure will occur in not less than 5 nor more than 15 min. Note and record the maximum load carried by the specimen during the test.

NOTE 6—In this test arrangement failure often occurs near one of the capped ends. Discard the results for those tests in which failure occurs either partly or wholly within the cementing medium.

7. Calculation

7.1 Calculate the tensile strength of the rock by dividing the maximum load carried by the specimen during test by the cross-sectional area computed in accordance with 5.3; express the result to the nearest 5 psi (35.0 kPa).

8. Report

8.1 The report shall include the following:

8.1.1 Source of sample including project name and location, and, if known, storage environment (the location is frequently specified in terms of the borehole number and depth of specimen from collar of hole),

8.1.2 Physical description of sample including: rock type, location and orientation of apparent planes, bedding planes, and schisotosity; and large inclusions or inhomogeneities, if any,

8.1.3 Date of sampling and testing,

8.1.4 Specimen length and diameter, also conformance with dimensional requirements as stated in Notes 3 and 4,

8.1.5 Rate of loading or deformation rate,

8.1.6 General indication of moisture condition of sample at time of test, such as as-received, saturated, laboratory air dry, or oven dry (It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation),

8.1.7 Direct tensile strength for each specimen as calculated, average direct tensile strength of all specimens, standard deviation or coefficient of variation,

8.1.8 Type and location of failure (A sketch of the fractured sample is recommended), and

8.1.9 Other available physical data.

NOTE 7—The number of specimens tested may depend upon the availability of specimens, but normally a minimum of ten is preferred. The number of specimens tested should be indicated. The statistical basis for relating the required number of specimens to the variability of measurements is given in Recommended Practice E 122.

9. Precision and Bias

9.1 *Precision*—Due to the nature of rock materials tested by this test method, it is, at this time, either not feasible or too costly to produce multiple specimens which have uniform physical properties. Therefore, since specimens which would yield the same test results cannot be tested, Subcommittee D18.12 cannot determine the variation between tests since any variation observed is just as likely to be due to specimen variation as to operator or laboratory testing variation. Subcommittee D18.12 welcomes proposals to resolve this problem that would allow for development of a valid precision statement.

9.2 *Bias*—There is no accepted reference value for this test method; therefore, bias cannot be determined.

10. Keywords

10.1 leading tests; rock; tension (tensile) properties/tests

The American Society for Testing and Materials takes no position respecting the validity of any patent rights asserted in connection with any item mentioned in this standard. Users of this standard are expressly advised that determination of the validity of any such patent rights, and the risk of infringement of such rights, are entirely their own responsibility.

This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 1916 Race St., Philadelphia, PA 19103.

Designation: D 3967 – 92 (Reapproved 1992)[ε1]

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St. Philadelphia, Pa 19103

Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens[1]

This standard is issued under the fixed designation D 3967; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

[ε1] NOTE—Section 10 was corrected editorially November 1992.

1. Scope

1.1 This test method covers testing apparatus, specimen preparation, and testing procedures for determining the splitting tensile strength of rock by diametral line compression of a disk.

NOTE 1—The tensile strength of rock determined by tests other than the straight pull test is designated as the "indirect" tensile strength and, specifically, the value obtained in Section 8 of this test is termed the "splitting" tensile strength.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard does not purport to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Document

2.1 *ASTM Standard:*

E 4 Practices for Load Verification of Testing Machines[2]

3. Significance and Use

3.1 By definition the tensile strength is obtained by the direct uniaxial tensile test. But the tensile test is difficult and expensive for routine application. The splitting tensile test appears to offer a desirable alternative, because it is much simpler and inexpensive. Furthermore, engineers involved in rock mechanics design usually deal with complicated stress fields, including various combinations of compressive and tensile stress fields. Under such conditions, the tensile strength should be obtained with the presence of compressive stresses to be representative of the field conditions. The splitting tensile strength test is one of the simplest tests in which such stress fields occur. Since it is widely used in practice, a uniform test method is needed for data to be comparable. A uniform test is also needed to insure positively that the disk specimens break diametrally due to tensile pulling along the loading diameter.

4. Apparatus

4.1 *Loading Device*, to apply and measure axial load on the specimen, of sufficient capacity to apply the load at a rate conforming to the requirements in 7.3. It shall be verified at suitable time intervals in accordance with Practices E 4 and shall comply with the requirements prescribed therein.

4.2 *Bearing Surfaces*—The testing machine shall be equipped with two steel bearing blocks having a Rockwell hardness of not less than 58 HRC (Note 2). One of the blocks shall be spherically seated and the other a plain rigid block. The bearing faces shall not depart from a plane by more than 0.0005 in. (0.0127 mm) when the blocks are new and shall be maintained within a permissible variation of 0.001 in. (0.025 mm). The diameter of the spherically seated bearing face shall be at least as large as that of the test specimen but shall not exceed twice the diameter of the test specimen. The movable portion of the bearing block shall be held closely in the spherical seat, but the design shall be such that the bearing face can be rotated and tilted through small angles in any direction.

NOTE 2—False platens, with plane bearing faces conforming to the requirements of this standard, may be used. These shall consist of disks about ½ to ¾ in. (12.7 to 19.05 mm) thick, oil hardened to more than 58 HRC, and surface ground. With abrasive rocks these platens tend to roughen after a number of specimens have been tested, and hence need to be resurfaced from time to time.

4.2.1 During testing the specimen can be placed in direct contact with the machine bearing plates (or false platens, if used) (Fig. 1). Otherwise, curved supplementary bearing plates or bearing strips should be placed between the specimen and the machine bearing plates to reduce high stress concentration.

4.2.2 Curved supplementary bearing plates with the same specifications as described in 4.2 may be used to reduce the contact stresses. The radius of curvature of the supplementary bearing plates shall be so designed that their arc of contact with the specimen will in no case exceed 15° or that the width of contact is less than $D/6$, where D is the diameter of the specimen.

NOTE 3—Since the equation used in 8.1 for splitting tensile strength is derived based on a line load, the applied load shall be confined to a very narrow strip if the splitting tensile strength test is to be valid. But a line load creates extremely high contact stresses which cause premature cracking. A wider contact strip can reduce the problem significantly. Investigations show that an arc of contact smaller than 15° causes no more than 2 % of error in principal tensile stress while reducing the incidence of premature cracking greatly.

4.3 *Bearing Strips* (0.01 D thick cardboard cushion, where D is the specimen diameter, or up to 0.25 in. thick plywood cushion are recommended to place between the machine bearing surfaces (or supplementary bearing plates; if used)

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved May 15, 1992. Published July 1992. Originally published as D 3967 – 81. Last previous edition D 3967 – 86[ε1].

[2] *Annual Book of ASTM Standards*, Vols 03.01, 04.02, and 08.03.

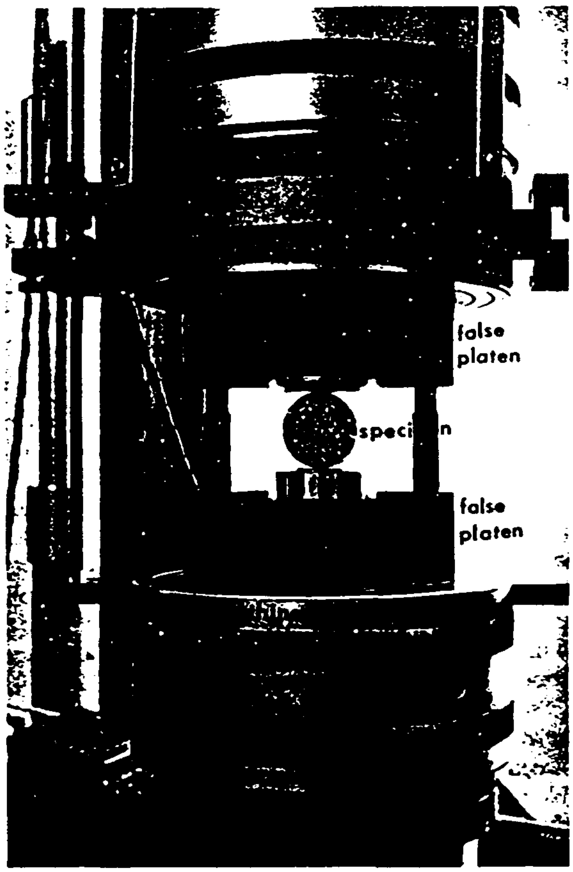


FIG. 1 One of the Proposed Testing Setup for Splitting Tensile Strength

and the specimen to reduce high stress concentration.

NOTE 4—Experiences have indicated that test results using the curved supplementary bearing plates and bearing strips, as specified in 4.2.2 and 4.3, respectively, do not significantly differ from each other, but there may be some consistent difference from the results of tests in which direct contact between the specimen and the machine platen is used.

5. Sampling

5.1 The specimen shall be selected from the core to represent a true average of the type of rock under consideration. This can be achieved by visual observations of mineral constituents, grain sizes and shape, partings, and defects such as pores and fissures.

6. Test Specimens

6.1 *Dimensions*—The test specimen shall be a circular disk with a thickness-to-diameter ratio (L/D) between 0.5 and 0.75. The diameter of the specimen shall be at least 10 times greater than the largest mineral grain constituent. A diameter of 1-$^{15}/_{16}$ in. (wireline core) will generally satisfy this criterion.

NOTE 5—When cores smaller than the specified minimum must be tested because of the unavailability of material, notation of the fact shall be made in the test report.

NOTE 6—If the specimen shows apparent anisotropic features such as bedding or schistosity, care shall be exercised in preparing the specimen so that the orientation of the loading diameter relative to anisotropic features can be determined precisely. Notation of this orientation shall be made in the test report.

6.2 *Number of specimens*—At least ten specimens shall be tested to obtain a meaningful average value. If the reproducibility of the test results is good (coefficient of variation less than 5 %), a smaller number of specimens is acceptable.

6.3 The circumferential surface of the specimen shall be smooth and straight to 0.020 in. (0.50 mm).

6.4 Cut the ends of the specimen parallel to each other and at right angles to the longitudinal axis. The ends of the specimen shall not deviate from perpendicular to the core axis by more than 0.5°. This requirement can be generally met by cutting the specimen with a precision diamond saw.

6.5 Determine the diameter of the specimen to the nearest 0.01 in. (0.25 mm) by taking the average of at least three measurements, one of which shall be along the loading diameter.

6.6 Determine the thickness of the specimen to the nearest 0.01 in. (0.25 mm) by taking the average of at least three measurements, one of which shall be at the center of the disk.

6.7 The moisture conditions of the specimen at the time of test can have a significant effect upon the indicated strength of the rock. The field moisture condition for the specimen shall be preserved until the time of test. On the other hand, there may be reasons for testing specimens at other moisture contents, including zero, and preconditioning of specimen when moisture control is needed. In any case, tailor the moisture content of the test specimen to the problem at hand and report it in accordance with 9.1.6.

7. Procedure

7.1 *Marking*—The desired vertical orientation of the specimen shall be indicated by marking a diametral line on each end of the specimen. These lines shall be used in centering the specimen in the testing machine to ensure proper orientation, and they are also used as the reference lines for thickness and diameter measurements.

NOTE 7—If the specimen is anisotropic, take care to ensure that the marked lines in each specimen refer to the same orientation.

7.2 *Positioning*—Position the test specimen to ensure that the diametral plane of the two lines marked on the ends of the specimen lines up with the center of thrust of the spherically seated bearing surface to within 0.05 in. (0.013 mm).

NOTE 8—A good line loading can often be attained by rotating the specimen about its axis until there is no light visible between the specimen and the loading platens. Back lighting helps in making this observation.

7.3 *Loading*—Apply a continuously increasing compressive load to produce an approximately constant rate of loading or deformation such that failure will occur within 1 to 10 min of loading, which should fall between 500 and 3000 psi/min of loading rate, depending on the rock type.

NOTE 9—Results of tests by several investigators indicate that rates of loading at this range are reasonably free from rapid loading effects.

8. Calculation

8.1 The splitting tensile strength of the specimen shall be calculated as follows:

$$\sigma_t = 2P/LD$$

and the result shall be expressed to the appropriate number of significant figures (usually 3), where:

σ_t = splitting tensile strength, psi or Pa,
P = maximum applied load indicated by the testing machine, lbf (or N),
L = thickness of the specimen, in. (or m), and
D = diameter of the specimen, in. (or m).

9. Report

9.1 The report shall include as much of the following as possible:

9.1.1 Sources of the specimen including project name and location, and if known, storage environment. The location is frequently specified in terms of the borehole number and depth of specimen from collar of hole.

9.1.2 Physical description of the specimen including rock type; location and orientation of apparent weakness planes, bedding planes, and schistosity; large inclusions or inhomogeneities, if any.

9.1.3 Dates of sampling and testing.

9.1.4 Specimen diameter and length, conformance with dimensional requirements, direction of loading if anisotropy exists. Type of contact between the specimen and the loading platens.

9.1.5 Rate of loading or deformation rate.

9.1.6 General indication of moisture condition of the specimen at time of test such as as-received, saturated, laboratory air dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation.

9.1.7 Splitting tensile strength of each specimen as calculated, average splitting tensile strength of all specimens, standard deviation or coefficient of variation.

9.1.8 Type and location of failure. A sketch of the fractured specimen is recommended.

10. Precision and Bias

10.1 Data are being evaluated via an interlaboratory test program for rock properties to determine the precision of this test method. There is no accepted reference value of rock for this test method, therefore, bias cannot be determined.

11. Keywords

11.1 indirect tensile strength; rock; splitting tensile strength

PROPOSED METHOD OF TEST FOR GAS PERMEABILITY OF ROCK CORE SAMPLES

1. Scope

1.1 This method describes procedures for determining gas permeabilities of rock core samples. Test procedures are described for both large-diameter cores (NX size--5.4 cm and larger) and small-diameter cores which include plugs taken from larger diameter cores. Samples are normally right cylinders although cube samples may be used with a suitably modified sample holder.

1.2 The permeability measurement will be made according to standard procedures with dry air as the permeant. The value obtained with this fluid may then be corrected to a corresponding value for a nonreacting liquid using a standard table of Klinkenberg corrections.[1] The use of such corrections should be specifically noted in reporting results. Determination of permeability of rock specimens may be made with other gases or with a liquid as the permeant. However, measurements of liquid permeability are difficult to standardize because of potential interaction between rock constituents and the liquid.

1.3 Permeability measurements made parallel to the bedding planes of sedimentary rocks shall be reported as horizontal permeability while those measured perpendicular to the bedding shall be reported as vertical permeability. Structural features other than bedding may be used to describe the direction of flow during measurement. These shall be specifically noted in reporting results.

1.4 Samples of hard, consolidated rock may be cut to shape and tested without artificial support. Friable, soft, shaly, or otherwise weak rock may require additional support to resist deformation or alteration during testing. Deformation or alteration will affect test results. Such samples shall be supported by mounting in a suitable potting material (i.e. cement slurry) exercising care not to alter the surfaces through which fluid flow will occur.

---

[1] "Recommended Practice for Determining Permeability of Porous Media," 3rd ed., American Petroleum Institute RP 27 (1952).

Testing procedures are the same for both unsupported and artificially supported samples.

1.5 The conventional direction of permeability measurement for small cylindrical samples is parallel to the axis of the cylinder. Large-diameter core samples are conventionally measured by one of two methods. In one method, referred to as the "linear permeability measurement," gas flow is either across the core perpendicular to a plane formed by a diameter of the core and the vertical axis or parallel to the core axis as with the small-diameter samples. When flow is across the sample, screens are used over diametrically opposite quadrants of the core circumference to uniformly distribute the gas flow. The second method is referred as a "radial permeability measurement" in which gas flows from the outside surface of the core radially through the core to a small-diameter hole drilled concentric with the axis of the core sample.

1.6 The permeability test is applicable to a wide range of rock types with a correspondingly wide range of permeabilities. With proper selection of test equipment components, permeabilities as low as 0.01 millidarcys and as high as 10 darcys can be measured accurately.

2. Summary of Method

2.1 The method for small samples consists of (a) placing the prepared sample in a Hassler- or Fancher-type core holder, Figs. 1 and 2, (b) applying the air pressure required to seal the sleeve around the core for the Hassler-type holder or loading the rubber compression ring for the Fancher holder, and (c) initiating dry gas flow through the sample. Because of the sensitivity of permeability to minor changes in lithology, no prescribed number of samples can be recommended to define the permeability of a given rock stratum. Reproducibility of approximately ±2 percent for samples of 0.1 millidarcy or greater permeability should be obtained for a given sample.

2.2 The method used to measure permeability for large-diameter cylindrical samples with vertical gas flow is the same as that described in 2.1.

2.3 The method for large-diameter samples with horizontal flow consists of (a) positioning appropriate-size screens diametrically opposite each other, (b) attaching the screens to the sample by light rubber bands, (c) placing the

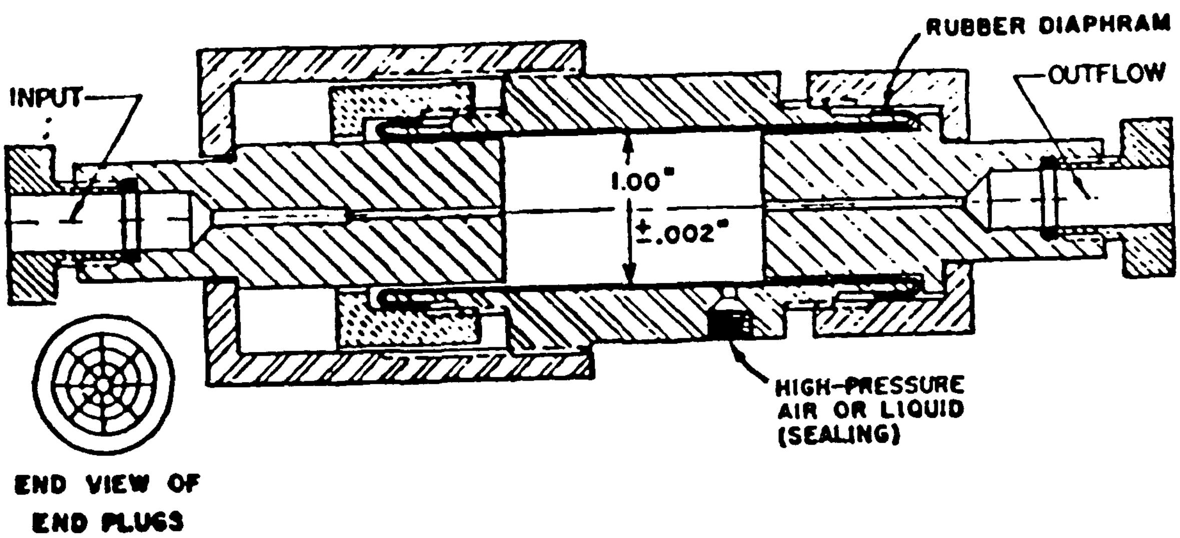


Fig. 1. Hassler-type permeability cell.

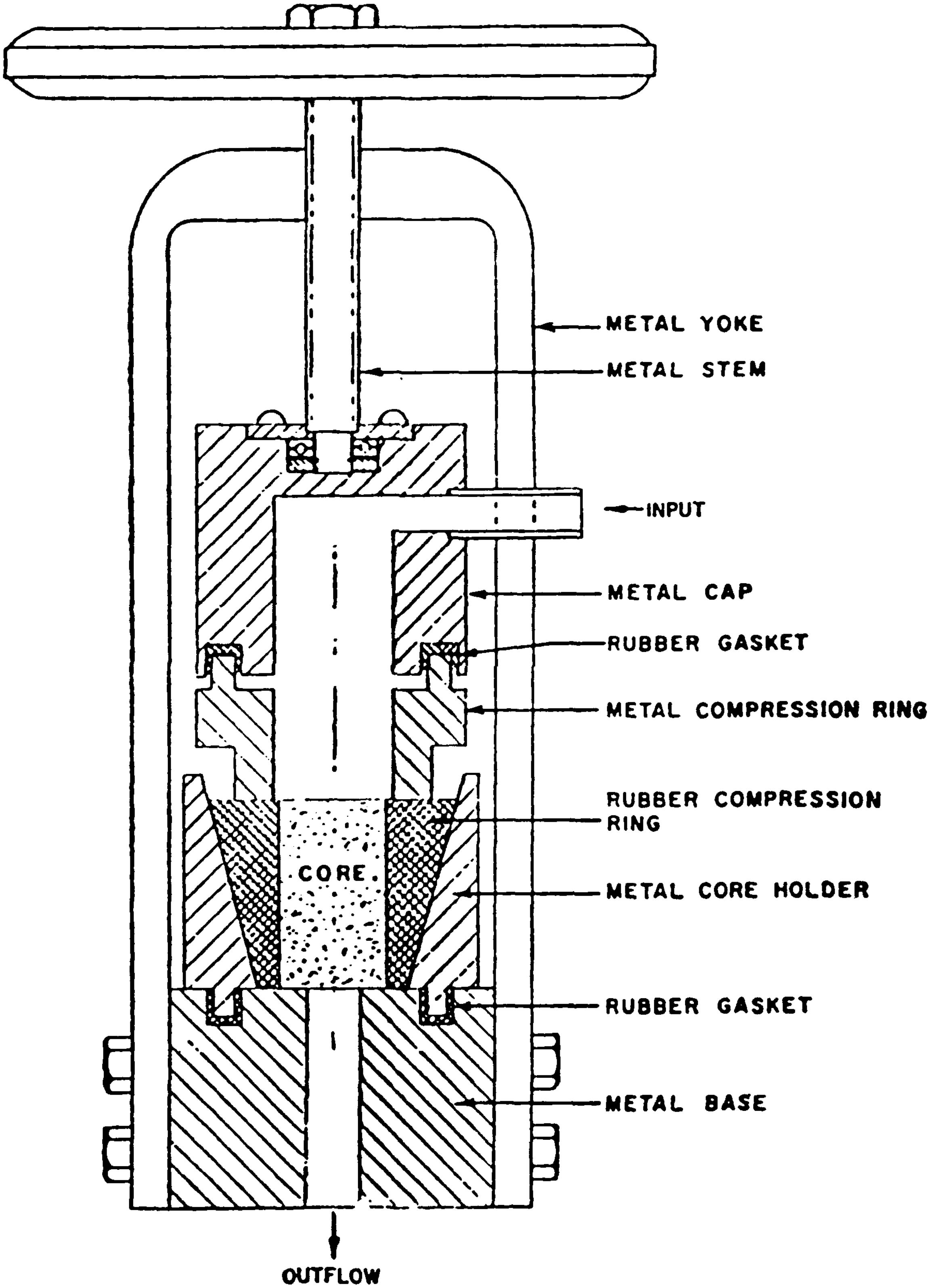


Fig. 2. Fancher-type core holder.

prepared sample in a Hassler-type or compression (ram) core holder, Figs. 3 and 4, (d) compressing the rubber gaskets which seal the ends of the samples in the Hassler-type holder (the ends of the samples used in the compression holder are presealed with plastic which is overlapped by the compression halves), (e) applying the air pressure (Hassler-type) or hydraulic force (compression) necessary to seal the sides of the sample except the area covered by the screens, and (f) initiating dry gas flow through the sample. Permeability is normally measured in two directions across the core: one is in the direction of apparent maximum permeability and the other is perpendicular to the first.

2.4 The method for large-diameter specimens with radial flow consists of (a) positioning the prepared sample in the radial flow core holder, Fig. 5, (b) raising the core against the closed lid by means of a piston, and (c) initiating dry gas flow through the sample.

3. Apparatus

3.1 Components - The apparatus used in dry air permeability testing consists of the following major components:

(a) A source of dry air

(b) Pressure regulator

(c) Inlet-pressure measuring device

(d) Core holder

(e) Outlet-pressure measuring device

(f) A dry air flow-rate metering device

3.2 Source of Air - The source of air for permeability measurements can be either the normal laboratory air supply or cylinders. Provisions should be made to filter particulate matter, absorb oil vapor, and remove water vapor. These devices should be periodically checked to insure proper operation.

3.3 Pressure Regulator - A suitable pressure regulator should be provided for the source of dry air. This regulator should apply air at a constant pressure and should be capable of doing so over a range of pressures between 1 and 80 cm of mercury (Note 1) which will produce the desired flow rate

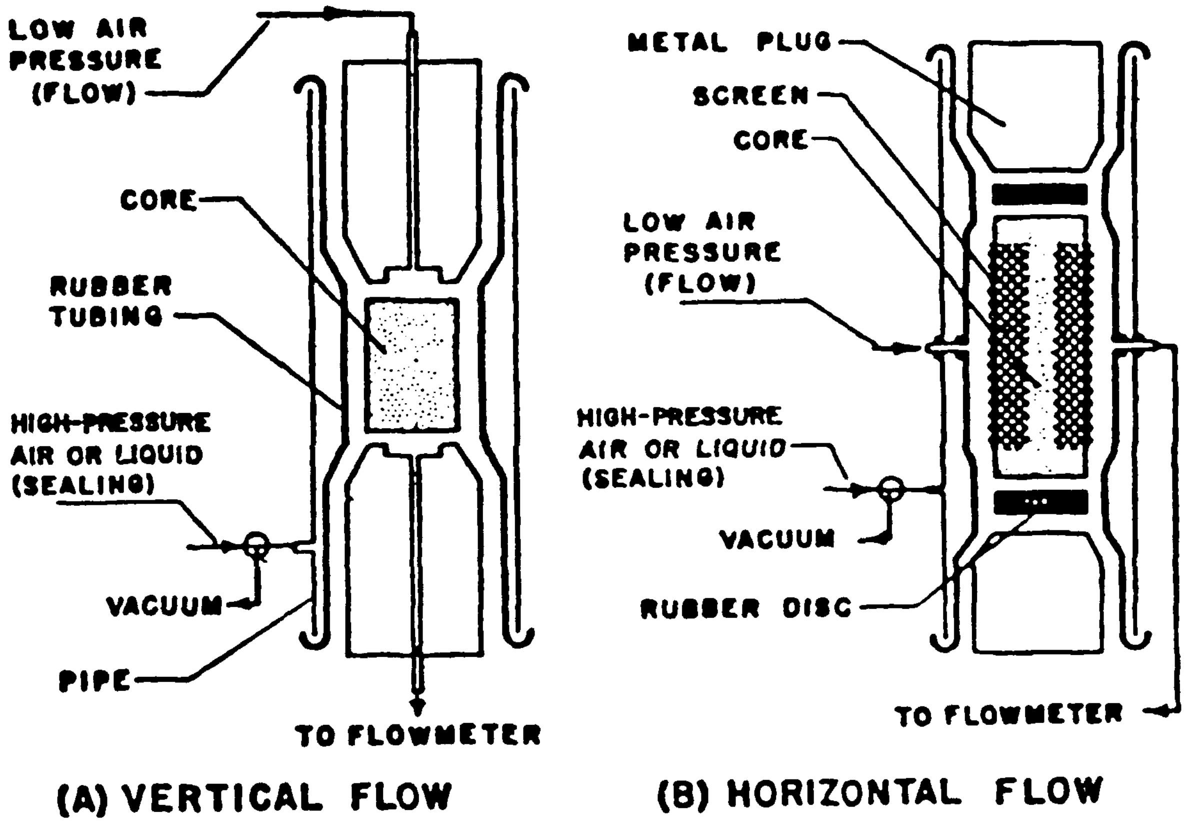


Fig. 3. Hassler-type permeameter.

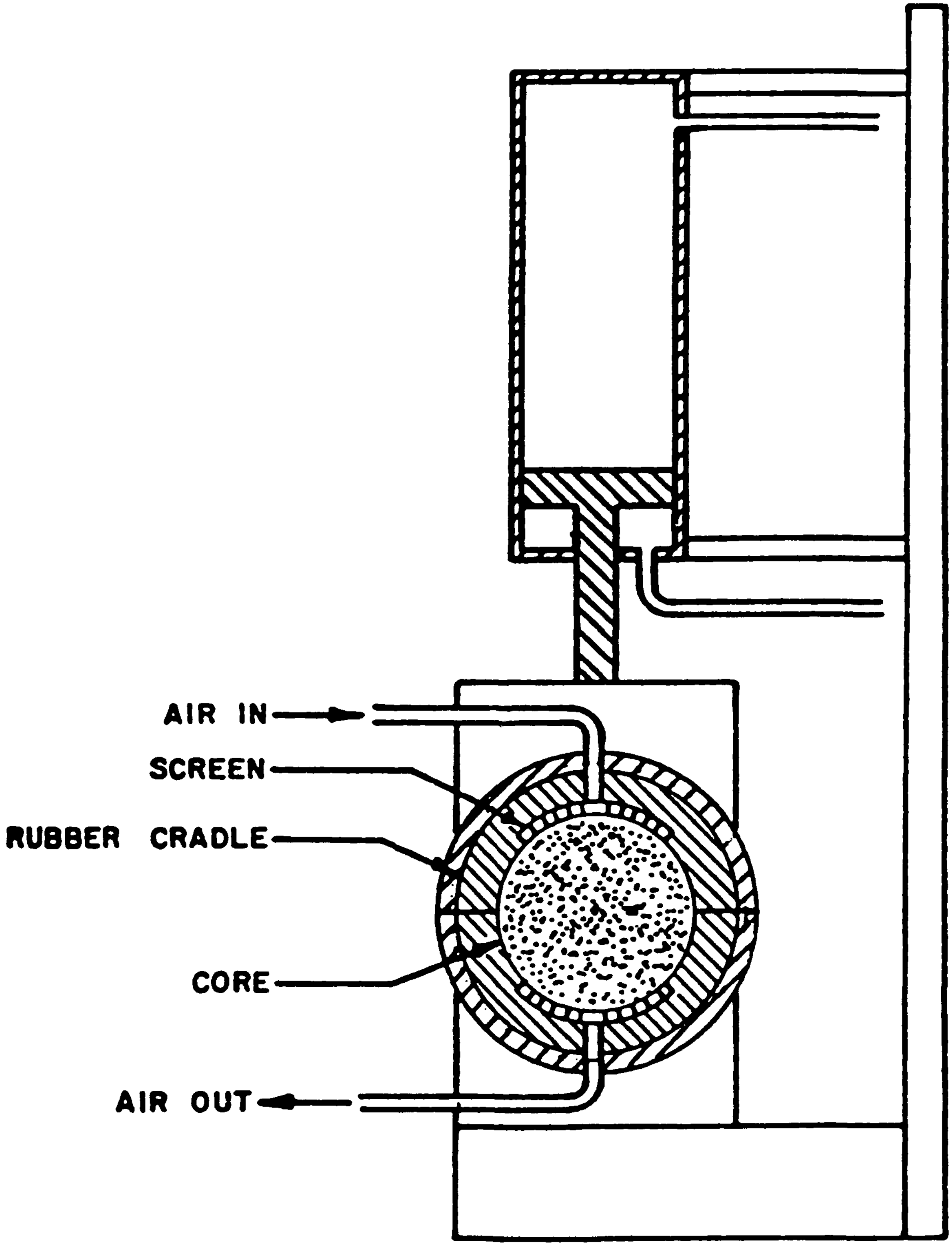


Fig. 4. Compression (RAM) permeameter.

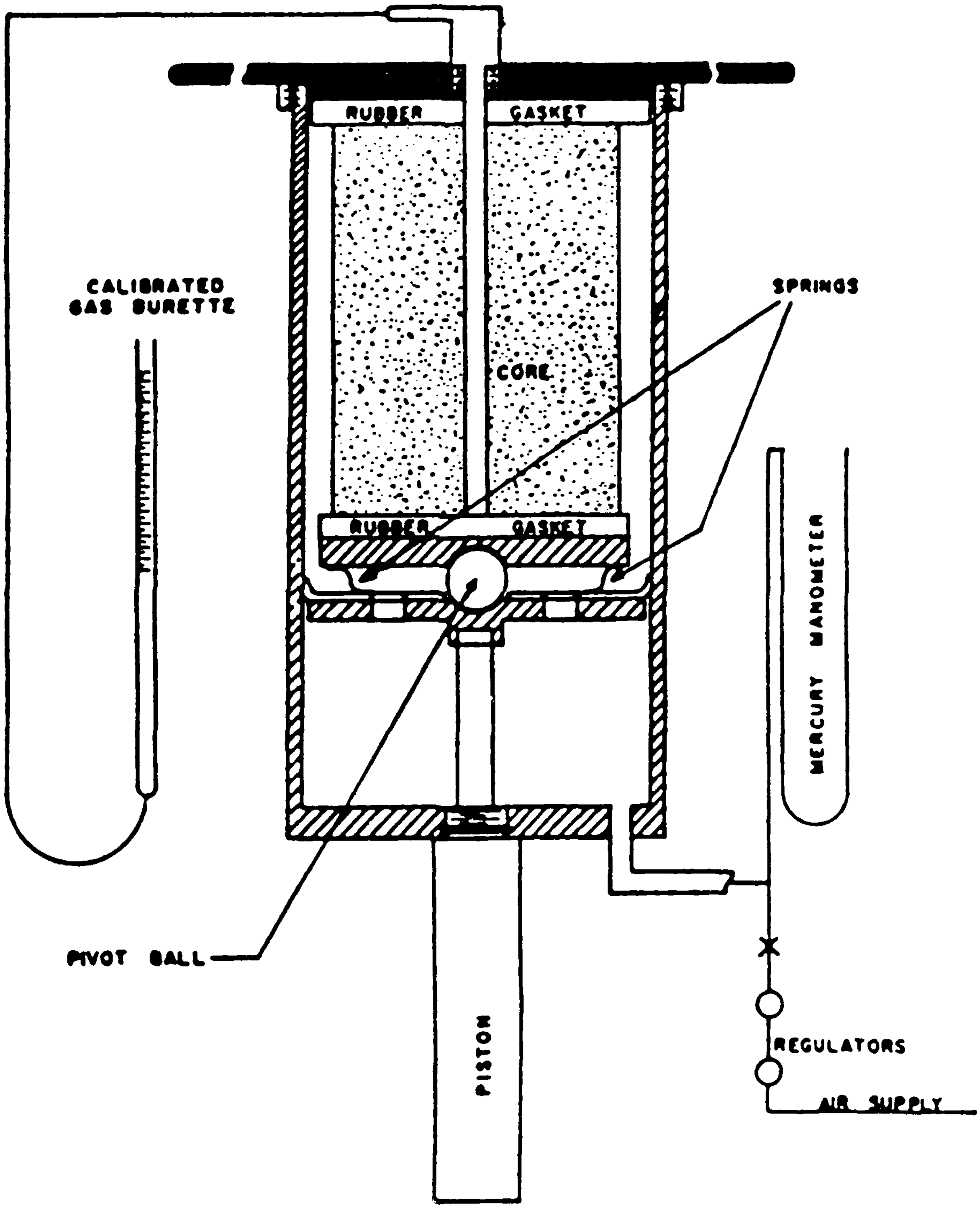


Fig. 5. Full-diameter radial permeameter.

(Note 1) of approximately 1 cu cm per second. Regulators of the pneumatic type are suitable for this purpose.

NOTE 1--The pressures required to produce the desired flow rate (1 cu cm per second) depend on the permeability and dimensions of the rock sample. Since the dimensions and permeability range of samples are not specified, the range of pressures over which pressure control is required cannot be specified. Equipment in common use operates over the pressure range of 1 to 80 cm of mercury. This pressure range is capable of producing laminar flow. This is the flow region required for permeability measurements. It is usually observed at flow rates up to 1 cu cm per second.

3.4 Pressure Measuring Devices - Inlet- and outlet-pressure measuring devices are manometers. These are water-, oil-, or mercury-filled and of a convenient length, usually 80 cm or less, with selection of length and fluid depending upon pressures to be measured. Manometers may be used in parallel to obtain the necessary accuracy over a range of pressures. Manometers are either open to the atmosphere or connected across the core. Connected across the core, they measure "differential" pressure. Where the pressure is in excess of 80 cm of mercury, a bourbon-type gage may be employed. This type of gage is normally only used in measuring extremely low permeabilities where high inlet pressures are required.

3.5 Core Holders

3.5.1 Three types of core measurements are commonly made:

(a) Axial flow in both small- and large-diameter cores

(b) Diametric flow in large-diameter cores

(c) Radial flow in large-diameter cores

3.5.2 Except for radial flow, more than one type of core holder may be used for permeability determinations. Irrespective of direction of gas flow or type of core holder used, the core holder must be such that when pressure is applied to one end of the system, all flow is through the sample. Care must be taken that no fluid bypasses the sample either through an imperfect seal between the core holder and sample or between the sample and the supporting material if the sample is mounted. Holders which accommodate several cores for simultaneous or sequential testing using matched pairs of inlet-outlet valves must be designed so that no fluid can leak between samples.

3.5.3 Selection of the type of core measurement is based on both the size of the sample available and the desired representativeness of the permeability value. For uniform, homogeneous rock small-diameter cores taken parallel and perpendicular to the bedding should provide representative permeability values. Measuring permeability of large-diameter cores is recommended for rock which is nonhomogeneous, vugular, fractured, or laminated. The radial method provides more representative data than the horizontal flow method since the entire diameter surface area of the core rather than a concentrated section of the diameter surface area of the core is involved in flow.

3.6 Screens to Distribute Gas Flow - When flow is across the core parallel to a diameter, screens should be of a wire diameter and mesh size such that a uniform gas distribution is obtained. Care should be exercised in positioning the screens prior to placing the sample in the holder to ensure that they maintain their proper position while the core is being seated.

3.7 Air Flow-Rate Metering Device- Three types of dry air flow-rate metering devices may be used:

(a) Calibrated orifices (a capillary tube which is calibrated for the conditions of testing, so that the pressure drop across the orifice is small compared to the core)

(b) Soap bubble in a calibrated burette

(c) Water-displacement meters

The calibrated orifice is the most commonly used type of flow metering device. Timing the movement of a soap bubble in a burette is also frequently used. Differential pressures across the core are adjusted to minimize turbulence in the flow of gas through the sample. Flow rates of 1 cu cm per second or less are used.

3.8 Sample Preparation Equipment - Sample preparation equipment shall include the following:

3.8.1 Diamond coring equipment for taking small-diameter cylinders from larger samples.

3.8.2 Diamond saw for trimming ends of samples. Ends should be sufficiently flat and parallel for leakage seating of rubber end seals.

3.8.3 Drill press and diamond drill for drilling axial holes for radial flow measurement samples. The holes should be concentric with the axis of the cylindrical sample. Care should be exercised in drilling to prevent cracking of the sample.

3.8.4 Sample cleaning to remove original fluids from the cores, external coatings such as drilling muds, and, in the case of highly saline original fluid, deposited salts. Where the original fluids are hydrocarbons, cleaning may be accomplished by solvent extraction, gas-driven solvent extraction, distillation-extraction, or other suitable method.[2] Drilling muds may be removed from the surface with water washing. If the interstitial water is very saline, several thorough freshwater washing should remove deposited salts.

3.8.5 Drying oven that can be maintained at 110 ± 5 °C.

3.8.6 Micrometer or vernier caliper for measuring length and diameter of test specimen. Micrometer or caliper should be direct reading to 1/50 mm.

3.8.7 Equipment as required for mounting samples of weak rock in optical pitch or suitable potting plastic.

3.8.8 Miscellaneous equipment such as timing devices, magnifying lenses, etc., used in preparing the sample and measuring pressures and gas flow rates.

4. Calibration

4.1 The permeameter should be calibrated regularly by means of capillary tubes of various known permeabilities or with standard plugs.

4.2 Orifices used to measure gas flow rates should be calibrated by allowing air to flow from the orifice to a burette containing a soap bubble.

4.3 Micrometers or vernier calipers used to measure sample dimensions should be checked against length standards.

5. Sample Preparation

5.1 There are no standard sample sizes. Small-diameter samples are commonly 1.9 to 3.8 cm in diameter. Large-diameter cores are arbitrarily 5.4 cm

---

[2] Darcy, H., The Public Fountain of the Village of Dijon, Paris: Victor Dalmont, 1856 (French text).

in diameter (equivalent to NX core) and larger. Cores as large as 15.2 cm in diameter are commonly tested. Core holders are designed for a specific size or sizes of cores. The core holders are designed for a specific size or sizes of cores. The core holder may be modified to accommodate a smaller sized sample by placing the core in a rubber sleeve whose inner diameter is that of the core and whose outer diameter is that for which the permeameter was designed. Thus a 1.9-cm-diam sample can be tested in a permeameter designed for 2.54-cm samples. Sample length is also not standardized. Sample lengths vary from 2.54 cm for small-diameter samples to 60.96 cm for large-diameter cores. Core holding devices are designed to accept different length samples. Where a sleeve is used to adapt a core, differences in sleeve and sample length may be compensated for by means of spacer rings placed on top of the sample.

5.2 Ratio of sample length to diameter also is not standardized. A ratio of 1:1 is recommended as a minimum for small-diameter samples. For large-diameter samples, a minimum L/D ratio of 1:2 is recommended.

5.3 The diameter of the test specimen is measured to the nearest 0.1 mm by averaging two diameters measured at right angles to each other at about midlength of the specimen. This average diameter is used for calculating the cross-sectional area. The length of the test specimen is determined to the nearest 0.1 mm by averaging three length measurements taken at third points around the circumference.

5.4 If the sample requires artificial support, the mounting material and method of mounting should be such that penetration into the sample is minimized, the sample does not extend beyond the surface of the mounting material, and mounting material does not cover any surface perpendicular to the direction of flow. Specific details of the mounting procedures, as well as mounting materials, are given in reference 1.

5.5 In cleaning and mounting the sample prior to testing, care should be used to prevent any alteration of the minerals comprising the rock sample which may produce changes in permeability. Samples containing clays or other hydratable minerals are especially susceptible.

5.6 Before measuring permeability any sample which is suspected of being cracked should be tested by coating with a liquid while air is passing through the sample. A row of bubbles on the downstream surface will serve to indicate the presence of a crack parallel to the direction of flow. Such samples should either be discarded as nonrepresentative or, if retained, note should be made of the crack in reporting permeability values.

6. Procedure

6.1 The prepared sample is mounted in the specified core holder as follows:

6.1.1 Hassler-type Holder - Retract rubber diaphragm or sleeve, Fig. 1, by applying a vacuum to the space between the diaphragm and the body of the core holder. (Note 2)

NOTE 2--A vacuum source of 5 cm of mercury is sufficient to retract the diaphragm.

Remove one end of the core holder and insert the cylindrical sample. Reinsert the end of the core holder, applying sufficient force to seat the sample. Remove the vacuum from the space between the body and the diaphragm, thus allowing the diaphragm to constrict around the sample. (Note 3)

NOTE 3--Fine-grained, smoothly formed samples can be effectively sealed at 690 N/m^2 pressure. Coarse-grained samples will require from 1035 to 1370 N/m^2 sealing pressure.

The pressure between the diaphragm and core holder body is maintained during the permeability measurement.

6.1.2 Fancher-type Holder - Select a rubber stopper drilled to the diameter and length of the sample. Carefully push the sample plug into the tapered stopper base until it is flush with the small end surface. Remove all loose sand grains. Place the stopper containing the test sample inside the tapered material core holder, Fig. 2. Turn the ram hand wheel to move the upper ram plug down to seal tightly against the top of the holder. Compress the tapered rubber stopper with the ram to effect a seal around the sample perimeter.

6.1.3 Compression (Ram) Holder - Place the sample in the lower half of the rubber cradle, Fig. 4, making certain that the screen is centered over the air outlet and the ends of the sample are overlapped by the rubber cradle. Lower the upper compression half by applying air or hydraulic pressure above the piston. After seating the two halves, apply sufficient pressure to compress the rubber cradle with the ram to effect the seal around the sample perimeter. (See Note 3).

6.1.4 Radial Flow Holder - Place the core on a (2.54-cm) solid rubber gasket which is attached to the lower floating plate, Fig. 5. Raise the core by means of the piston against the closed lid with the center hole of the core matching that of the upper gaskets. After the core contacts the upper gasket, increase the piston pressure slightly to adjust the lower floating plate if the core ends are not parallel. Sufficient piston pressure is then applied to effect a seal of the upper and lower core surfaces. (Note 4)

NOTE 4--With gas flowing through the core at a constant rate, the piston pressure is increased. Decreased flow rate with increasing piston pressure indicates a leak at the rubber gasket. Repeat the test until no change in the flow rate is noted.

6.2 Connect the source of dry gas and inlet pressure measuring devices to the inlet fitting on the core holder. Connect the outlet pressure measuring devices and flow rate metering device to the outlet fitting on the core holder.

6.3 Initiate dry gas flow through the sample. Control the flow rate to minimize turbulence. Measure the flow rate through the sample intermittently for a period of 3 to 10 minutes until the flow becomes constant. Record the constant flow rate and pressures.

7. Calculations

7.1 The calculation of permeability is based on the empirical expression of Darcy known as Darcy's law.[2] The coefficient, k, of proportionality is the permeability.

7.2 The unit of the permeability coefficient, k, is the darcy. For convenience the subunit millidarcy may be used where 1 millidarcy equals 0.001 darcy. A porous medium has a permeability of one darcy when a single-phase fluid of one centipoise viscosity that completely fills the voids of the medium will flow through it under "conditions of viscous flow" at a rate of 1 cu cm per second per square centimetre of cross-sectional area under a pressure gradient of one atmosphere per centimetre. "Conditions of viscous flow" mean that the rate of flow is sufficiently low to be directly proportional to the pressure gradient. The permeability coefficient so defined has the units of length squared or area.

7.3 Vertical Flow Parallel to Core Axis - Darcy's law in differential form for linear flow is

$$q = \frac{k}{\mu} \frac{dp}{dL}$$

where

- q – macroscopic velocity of flow, in centimetres per second
- k – permeability coefficient, in darcys
- μ – viscosity of the fluid that is flowing, in centipoises
- dp/dL – pressure gradient in the direction of flow, in atmospheres per centimetres

Relating the velocity of flow to the volume rate of flow through the cross-sectional area and performing the indicated integration produces the following working equation for permeability coefficient in millidarcys:

$$k = (2000\ Q_o O_o\ L\ \mu)/\left(P_i^{\ 2} - P_o^{\ 2}\right)$$

where

- Q_o = rate of flow of outlet air, in cubic centimetres per second
- P_o = outlet pressure, in atmospheres (absolute)
- P_i = inlet pressure, in atmospheres (absolute)
- L = length of sample, in centimetres
- A = cross-sectional area perpendicular to direction of flow, in square centimetres

Several methods of simplification exist for calculating the permeability coefficient. Two such methods are given below:

7.3.1 Method 1 - For this method inlet pressure and pressure drop across the rate measuring orifice during the test are selected so that the outlet pressure is essentially one atmosphere. The working equation then reduces to

$$k = Q\ CL/A$$

where

$C = (2000\ \mu)/(P_i^2 - 1)$

μ = viscosity of air under the conditions used to calibrate the orifice

C then is a constant for each fixed inlet pressure since the fact that the same air flows through both the core and the orifice means that any change in air viscosity resulting from temperature changes or water vapor will have no effect on the relative pressure readings.

7.3.2 Method 2 - For this method calibration charts or tables of permeance (Note 5) versus outlet pressure for given inlet pressures and orifices are prepared based on the following equation

$$k_c = \frac{L_c}{A_c}\left[\frac{k_{or}}{L_{or}/A_{or}} \cdot \frac{\Delta P_{or} Q_c}{\Delta P_c Q_{or}}\right]$$

where

k_c, k_{or} = permeability of the core and the equivalent permeability of the orifice, respectively, in millidarcys

L_c, L_{or} = length of core and orifice, respectively, in centimetres

A_c, A_{or} = cross-sectional area of core and orifice, respectively, in square centimetres

ΔP_c, ΔP_{or} = pressure drop across the core and orifice, respectively, in atmospheres

Q_c, Q_{or} = flow rate through the core and orifice, respectively, in cubic centimetres per second

NOTE 5--Permeance or apparent permeability is the proper term for flow capacity. The term as used is analogous to the term conductance for the flow of current through an electrolyte solution. Permeance and permeability, therefore, are related in the same way as conductance and conductivity.

This working equation can be simplified to

$$k_c = \frac{L_c}{A_c} \frac{Q_c}{\Delta P_c} \cdot L$$

where

L - orifice constant

L may be determined directly by use of a known permeability plug. When tables or nomographs are used to calculate the permeability coefficient from measured outlet pressure for given inlet pressures and orifices, the working equation reduces to

$$k_c = \frac{L_c}{A_c} k_c^{\theta}$$

where

k_c^{θ} - permeance of the core

The permeance as calculated when multiplied by the L/A ratio gives the core permeability coefficient, k_c.

7.4 Horizontal Flow Parallel to Core Diameter - The same differential form of Darcy's law and working equation as used for vertical flow are applied. The working equation is modified by a factor for shape to the following form:

$$k = (Q_m \mu / L \Delta P) (1000) (G)$$

where

k = permeability, in millidarcys

Q_m = volume rate of air flow at mean core pressure, in cubic centimetres per second

μ - viscosity of flowing fluid, in centipoises

L - length of sample, in centimetres

ΔP - pressure drop across the core, in atmospheres

G - shape factor,[3] Fig. 6

7.5 Radial Flow - The radial permeability is calculated directly from the integrated form of Darcy's law for radial flow. The equation in terms of permeability coefficient is

$$k = (\mu Q_a)(\ln d_e/d_w)(P_o)/(\pi h)(P_i^2 - P_o^2) \cdot 1000$$

where

k - permeability, in millidarcys

μ - viscosity of flowing fluid at test temperature, in centipoises

Q_a - measured flow rate at test temperature and pressure - P_o, in cubic centimetres per second

ln - logarithm to the base e

d_e - outside diameter of sample, in centimetres

d_w - inside diameter of inner hole, in centimetres

P_o - outlet pressure, in atmospheres (absolute)

h - height of sample, in centimetres

P_i - inlet pressure, in atmospheres (absolute)

As with vertical flow, if the outlet pressure is atmospheric and the orifices are calibrated over the range of inlet pressures, the working equation simplifies to

$$k = \mu Q_m (\ln d_e/d_w)/2\pi h\Delta \cdot 1000$$

---

[3]Collins, R. E., "Determination of the Transverse Permeabilities of Large Core Samples from Petroleum Reservoirs," Journal of Applied Physics 23, 681-84 (1952).

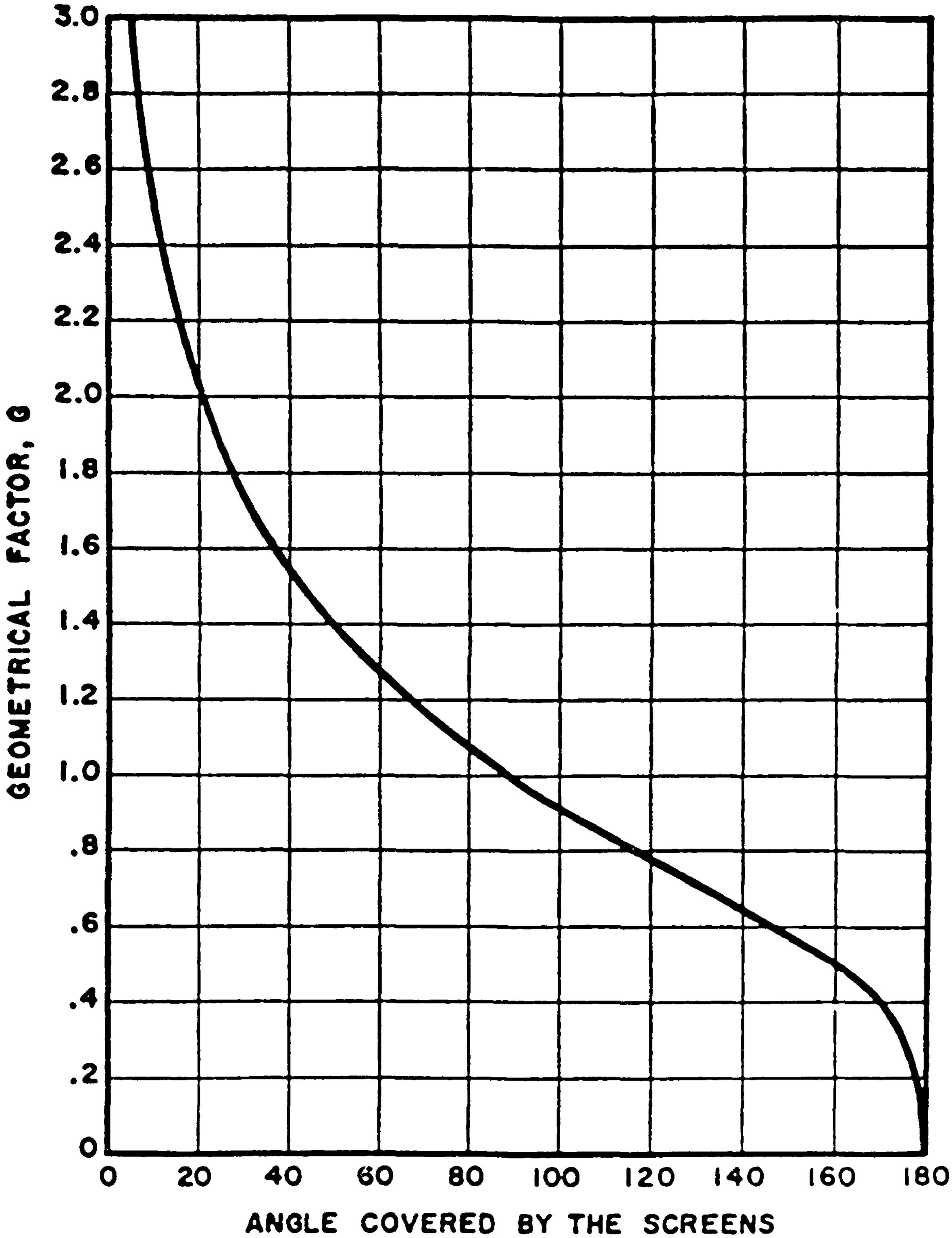


Fig. 6. Theoretical curve relating the geometric factor and the angular segment of the core covered by the screens (Collins, 1952).

where

Q_m = volume rate of air flow at mean core pressure, in cubic centimetres per second

ΔP = pressure drop across sample, in atmospheres (absolute)

8. Report

8.1 The report shall include the following:

8.1.1 A lithologic description of the rock tested.

8.1.2 Source of sample including depth and orientation, dates of sampling and testing, and storage environment.

8.1.3 Methods used for sample cleaning.

8.1.4 Methods used for sample support, capping, or other preparation such as sawing, grinding, or drilling.

8.1.5 Specimen length and diameter.

8.1.6 Type of core holder used.

8.1.7 Pressures used to seal core surfaces in core holder.

8.1.8 Flowing fluid used and direction flow.

8.1.9 Method used for calculating permeability coefficient.

8.1.10 Results of other physical tests, citing the method of determination for each.

8.1.11 Permeability corrections used.

8.2.12 Description of air source.

8.1.13 Calculation of permeability, with values defined for all variables.

RTH 115-93

Designation: C 535 – 89

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards. Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Test Method for Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine[1]

This standard is issued under the fixed designation C 535; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This method has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

1. Scope

1.1 This test method covers testing sizes of coarse aggregate larger than ¾ in. (19 mm) for resistance to degradation using the Los Angeles testing machine.

NOTE 1—A procedure for testing coarse aggregate smaller than 1½ in. (37.5 mm) is covered in Method C 131.

1.2 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 131 Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine[2]

C 136 Method for Sieve Analysis of Fine and Coarse Aggregates[2]

C 670 Practice for Preparing Precision Statements for Test Methods for Construction Materials[2]

C 702 Practice for Reducing Field Samples of Aggregate to Testing Size[3]

D 75 Practice for Sampling Aggregates[2]

E 11 Specification for Wire-Cloth Sieves for Testing Purposes[4]

3. Summary of Method

3.1 The Los Angeles test is a measure of degradation of mineral aggregates of standard gradings resulting from a combination of actions including abrasion or attrition, impact, and grinding in a rotating steel drum containing a specified number of steel spheres, the number depending upon the grading of the test sample. As the drum rotates, a shelf plate picks up the sample and the steel spheres, carrying them around until they are dropped to the opposite side of the drum, creating an impact-crushing effect. The contents then roll within the drum with an abrading and grinding action until the shelf plate impacts and the cycle is repeated. After the prescribed number of revolutions, the contents are removed from the drum and the aggregate portion is sieved to measure the degradation as percent loss.

4. Significance and Use

4.1 The Los Angeles test has been widely used as an indicator of the relative quality or competence of various sources of aggregate having similar mineral compositions. The results do not automatically permit valid comparisons to be made between sources distinctly different in origin, composition, or structure. Specification limits based on this test should be assigned with extreme care in consideration of available aggregate types and their performance history in specific end uses.

5. Apparatus

5.1 *Los Angeles Machine* conforming to the requirements of Test Method C 131.

5.1.1 The machine shall be so driven and so counterbalanced as to maintain a substantially uniform peripheral speed (Note 2). If an angle is used as the shelf, the direction of rotation shall be such that the charge is caught on the outside surface of the angle.

NOTE 2—Backlash or slip in the driving mechanism is very likely to furnish test results that are not duplicated by other Los Angeles machines producing constant peripheral speed.

5.2 *Sieves*, conforming to Specification E 11.

5.3 *Balance*—A balance or scale accurate within 0.1 % of test load over the range required for this test

5.4 *Charge*—The charge shall consist of 12 steel spheres averaging approximately 1$^{27}/_{32}$ in. (46.8 mm) in diameter, each weighing between 390 and 445 g, and having a total weight of 5000 ± 25 g.

NOTE 3—Steel ball bearings 1$^{13}/_{16}$ in. (46.038 mm) and 1⅞ in. (47.625 mm) in diameter, weighing approximately 400 and 440 g each, respectively, are readily available. Steel spheres 1$^{27}/_{32}$ in. (46.8 mm) in diameter weighing approximately 420 g may also be obtainable. The charge may consist of a mixture of these sizes.

6. Sampling

6.1 The field sample shall be obtained in accordance with Practice D 75 and reduced to test portion in accordance with Practice C 702.

---

[1] This test method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.05 on Methods of Testing and Specifications for Physical Characteristics of Concrete Aggregates.

Current edition approved April 28, 1989. Published June 1989. Originally published as C 535 – 64 T. Last previous edition C 535 – 81 (1987).

[2] *Annual Book of ASTM Standards*, Vols 04.02 and 04.03.

[3] *Annual Book of ASTM Standards*, Vol 04.02.

[4] *Annual Book of ASTM Standards*, Vol 14.02.

TABLE 1 Gradings of Test Samples

Sieve Size, mm (in.) (Square Openings)		Weights of Indicated Sizes, g		
		Grading[A]		
Passing	Retained on	1	2	3
75 (3)	63 (2½)	2 500 ± 50	...	...
63 (2½)	50 (2)	2 500 ± 50	...	...
50 (2)	37.5 (1½)	5 000 ± 50	5 000 ± 50	...
37.5 (1½)	25.0 (1)	...	5 000 ± 25	5 000 ± 25
25.0 (1)	19.0 (¾)	...	...	5 000 ± 25
	Total	10 000 ± 100	10 000 ± 75	10 000 ± 50

[A] Gradings 1, 2, and 3 correspond, respectively, in their size distribution to Gradings, E, F, and G in the superseded ASTM Method C 131 – 55, Test for Abrasion of Coarse Aggregate by Use of the Los Angeles Machine, which appears in the 1961 *Book of ASTM Standards*, Part 4.

7. Test Sample

7.1 The test sample shall be washed and oven-dried at 221 to 230°F (105 to 110°C) to substantially constant weight (Note 4), separated into individual size fractions, and recombined to the grading of Table 1 most nearly corresponding to the range of sizes in the aggregate as furnished for the work. The weight of the sample prior to test shall be recorded to the nearest 1 g.

NOTE 4—If the aggregate is essentially free of adherent coatings and dust, the requirement for washing before and after test may be waived. Elimination of washing after test will seldom reduce the measured loss by more than about 0.2 % of the original sample weight.

8. Procedure

8.1 Place the test sample and charge in the Los Angeles testing machine and rotate the machine at 30 to 33 r/min for 1000 revolutions. After the prescribed number of revolutions, discharge the material from the machine and make a preliminary separation of the sample on a sieve coarser than the 1.70-mm (No. 12). The finer portion shall then be sieved on a 1.70-mm sieve in a manner conforming to Method C 136. The material coarser than the 1.70-mm sieve shall be washed (Note 4), oven-dried at 221 to 230°F (105 to 110°C) to substantially constant weight, and weighed to the nearest 5 g (Note 5).

NOTE 5—Valuable information concerning the uniformity of the sample under test may be obtained by determining the loss after 200 revolutions. This loss should be determined without washing the material coarser than the 1.70-mm (No. 12) sieve. The ratio of the loss after 200 revolutions to the loss after 1000 revolutions should not greatly exceed 0.20 for material of uniform hardness. When this determination is made, take care to avoid losing any part of the sample; return the entire sample, including the dust of fracture, to the testing machine for the final 800 revolutions required to complete the test.

9. Calculation

9.1 Express the loss (difference between the original weight and the final weight of the test sample) as a percentage of the original weight of the test sample. Report this value as the percent loss.

NOTE 6—The percent loss determined by this method has no known consistent relationship to the percent loss for the same material when tested by Test Method C 131.

10. Precision

10.1 *Precision*—The precision of this test method has not been determined. It is expected to be comparable to that of Test Method C 131.

10.2 *Bias*—No statement is being made about the bias of this Test Method since there is no accepted reference material suitable for determining the bias of this procedure.

APPENDIX

(Nonmandatory Information)

X1. MAINTENANCE OF SHELF

X1.1 The shelf of the Los Angeles machine is subject to severe surface wear and impact. With use, the working surface of the shelf is peened by the balls and tends to develop a ridge of metal parallel to and about 1¼ in. (32 mm) from the junction of the shelf and the inner surface of the cylinder. If the shelf is made from a section of rolled angle, not only may this ridge develop but the shelf itself may be bent longitudinally or transversely from its proper position.

X1.2 The shelf should be inspected periodically to determine that it is not bent either lengthwise or from its normal radial position with respect to the cylinder. If either condition is found, the shelf should be repaired or replaced before further tests are made. The influence on the test result of the ridge developed by peening of the working face of the shelf is not known. However, for uniform test conditions, it is recommended that the ridge be ground off if its height exceeds 0.1 in. (2 mm).

PART I. LABORATORY TEST METHODS

B. Engineering Design Tests

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Test Method for Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression[1]

This standard is issued under the fixed designation D 3148; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

[ε1] NOTE—Figure 2 was corrected editorially in December 1987.

INTRODUCTION

The deformation and strength properties of rock cores measured in the laboratory usually do not accurately reflect large scale *in situ* properties because the latter are strongly influenced by joints, faults, inhomogeneities, weakness planes, and other factors. Therefore, laboratory values for intact specimens must be employed with proper judgment in engineering applications.

1. Scope

1.1 This test method covers the determination of elastic moduli of intact rock core specimens in uniaxial compression. Procedure A specifies the apparatus, instrumentation, and procedures for determining the axial stress - strain curve and Young's modulus, E, of cylindrical rock specimens loaded in uniaxial compression. Method B specifies the additional apparatus, instrumentation, and procedures which are necessary also to determine the lateral stress - strain curve and Poisson's ratio, ν.

NOTE 1—Some applications require the value of Young's modulus, but not Poisson's ratio. Thus, the decision to use Procedure A or Procedure A and B shall be determined by the engineer in charge of the project.

NOTE 2—This test method does not include the procedures necessary to obtain a stress - strain curve beyond the ultimate strength.

1.2 Test methods are not normally specified for rock moduli in tension because its low tensile strength does not permit sufficient data points to be obtained to be significant. However, the basic principles given here may be applied to tension testing.

1.3 The relation between the three elastic constants,

$$G = E/2(1 + \nu)$$

where:

G = modulus of rigidity,
E = Young's modulus, and
ν = Poisson's ratio,

and most elastic design equations are based on the assumption of isotropy. The engineering applicability of these equations is therefore decreased if the rock is anisotropic. When possible, it is desirable to conduct tests in the plane of foliation, bedding, etc., and at right angles to it to determine the degree of anisotropy. It is noted that equations developed for isotropic materials may give only approximate calculated results if the difference in elastic moduli in any two directions is greater than 10 % for a given stress level.

NOTE 3—Elastic moduli measured by sonic methods may often be employed as preliminary measures of anisotropy.

1.4 The values stated in inch-pound units are to be regarded as the standard.

1.5 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

D 2938 Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens[2]
D 4543 Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances[2]
E 4 Practices for Load Verification of Testing Machines[3]
E 122 Recommended Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[4]

3. Apparatus

3.1 *Loading Device* (Procedures A and B), for applying and measuring axial load to the specimen and of sufficient capacity to apply load at a rate in accordance with the requirements prescribed in 5.4. It shall be verified at suitable time intervals in accordance with the procedures given in Practices E 4, and comply with the requirements prescribed therein.

NOTE 4—The loading apparatus employed for this test is the same as that required for Test Method D 2938.

3.2 *Bearing Surfaces* (Procedures A and B)—The testing machine shall be equipped with two steel bearing blocks having a Rockwell hardness of not less than 58 HRC. One of the blocks shall be spherically seated and the other a plain

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Oct. 31, 1986. Published December 1986. Originally published as D 3148 – 72. Last previous edition D 3148 – 80.

[2] *Annual Book of ASTM Standards*, Vol 04.08.
[3] *Annual Book of ASTM Standards*, Vols 03.01, 04.02, 07.?? and 08.03.
[4] *Annual Book of ASTM Standards*, Vol 14.02.

rigid block. The bearing faces shall not depart from a plane by more than 0.0006 in. (15 μm) when the blocks are new and shall be maintained within a permissible variation of 0.001 in. (25 μm). The diameter of the spherically seated bearing face shall be at least as large as that of the test specimen but shall not exceed twice the diameter of the test specimen. The center of the sphere for the spherically seated block shall coincide approximately with the center of the bearing face of the specimen. The movable portion of the bearing block shall fit closely in the spherical seat, but the design shall be such that the bearing face can be rotated and tilted through small angles in any direction. Accomplish seating by rotating the movable bearing block while the specimen is held in contact with the fixed block.

NOTE 5—False platens with plane bearing faces conforming to the requirements of this method may be used. These shall consist of disks about 0.6 to 0.8 in. (15 to 20 mm) thick, oil-hardened preferably through the disks to more than 58 HRC and surface ground. With abrasive rocks these platens tend to roughen after a number of specimens have been tested, and hence need to be resurfaced from time to time.

3.3 *Axial Strain Determination* (Procedure A)—The axial deformations or strains may be determined from data obtained by electrical resistance strain gages, compressometers, optical devices, or other suitable means. The design of the measuring device shall be such that the average of at least two axial strain measurements can be determined for each increment of load. Measuring positions shall be equally spaced around the circumference of the specimen close to midheight. The gage length over which the axial strains are determined shall be at least 10 grain diameters in magnitude. The axial strains shall be determined with an accuracy of 2 % of the reading and a precision of 0.2 % of full-scale.

NOTE 6—Accuracy should be within 2 % of value of readings above 250 μm/m strain and within 5 μm/m strain for readings lower than 250 μm/m strain.

3.4 *Lateral Strain Determination* (Procedure B)—The lateral deformations or strains may be measured by any of the methods mentioned in 3.3. Either circumferential or diametric deformations (or strains) may be measured. At least two lateral deformation sensors shall be used. These shall be equally spaced around the circumference of the specimen close to midheight. The average deformation (or strain) from the two sensors shall be recorded at each load increment. The use of a single transducer that wraps all the way around the specimen to measure the total change in circumference is also permitted. The gage length and the accuracy and precision the lateral strain measurement system shall be the same those specified in 3.3 for the axial direction.

4. Test Specimens

4.1 Test specimens shall be prepared in accordance with Practice D 4543.

4.2 The moisture condition of the sample shall be noted and reported in 8.1.9.

NOTE 7—The moisture condition of the specimen at time of test can have a significant effect upon the strength of the rock and, hence, upon the shape of the deformation curves. Good practice generally dictates that laboratory tests be made upon specimens representative of field conditions. Thus, it follows that the field moisture condition of the specimen should be preserved until time of test. On the other hand, there may be reasons for testing specimens at other moisture contents from saturation to dry. In any case the moisture content of the test specimen should be tailored to the problem at hand. Excess moisture will affect the adhesion of strain gages, if used, and the accuracy of their performance.

5. Procedure (Procedures A and B)

5.1 Check the ability of the spherical seat to rotate freely in its socket before each test.

5.2 Wipe clean the bearing faces of the upper and lower bearing blocks and of the test specimen and place the test specimen with the strain-measuring device attached on the lower bearing block. Carefully align the axis of the specimen with the center of thrust of the spherically seated block. Make electrical connections or adjustments to the strain- or deformation-measuring device. As the load is gradually brought to bear on the specimen, adjust the movable portion of the spherically seated block so that uniform seating is obtained.

5.3 Many rock types fail in a violent manner when loaded to failure in compression. A protective shield should be placed around the test specimen to prevent injury from flying rock fragments.

5.4 Apply the load continuously and without shock to produce an approximately constant rate of load or deformation such that failure would occur within 5 to 15 min from initiation of loading, if carried to failure (Note 8). Record the load and the axial strain or deformation frequently at evenly spaced load intervals during Procedure A of Practice D 4543. Take at least ten readings over the load range to define the axial stress-strain curve. Also record the circumferential or diametric strains (or deformations) at the same increments of load for Procedure B of Practice D 4543. Continuous recording of data with strip chart or X-Y recorders is permitted as long as the precision and accuracy of the recording system meets the requirements in 3.3.

NOTE 8—Results of tests by several investigators have shown that strain rates within this range will provide strength values that are reasonably free from rapid loading effects and reproducible within acceptable tolerances.

6. Calculation (Procedure A)

6.1 The axial strain, ϵ_a, may be recorded directly from strain-indicating equipment, or may be calculated from deformation readings depending upon type of apparatus or instrumentation employed. Calculate the axial strain, ϵ_a, as follows:

$$\epsilon_a = \Delta l / l$$

where:

l = original undeformed axial gage length, in. (mm), and
Δl = change in measured axial length (negative for a decrease in length) in. (mm).

6.2 Calculate the compressive stress in the test specimen from the compressive load on the specimen and the initial computed cross-sectional area as follows:

$$\sigma = -P/A$$

where:

σ = stress, psi (MPa),
P = load, lbf (N), and
A = area, in.^{2} (mm^2).

NOTE 9—Tensile stresses and strains are used as being positive herein. A consistent application of a compression-positive sign convention may be employed if desired.

6.3 Plot the stress versus strain curve for the axial direction

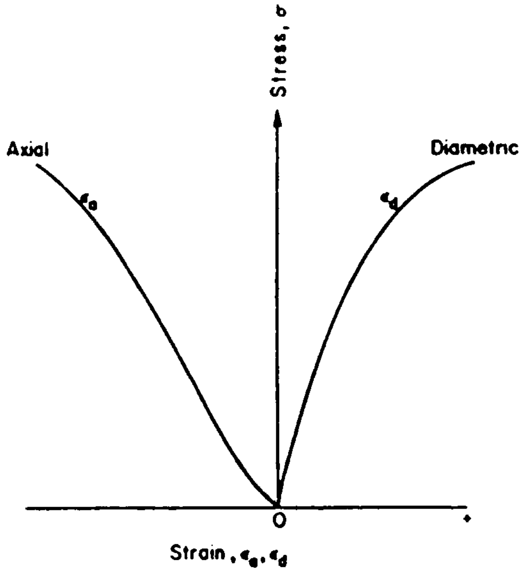


FIG. 1 Format for Graphical Presentation of Data

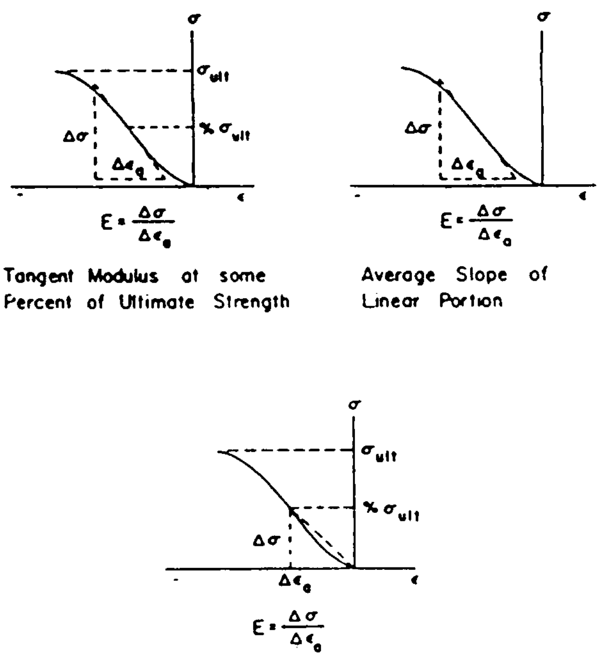


FIG. 2 Methods for Calculating Young's Modulus, E

(Fig. 1). The complete curve gives the best description of the deformation behavior of rocks having nonlinear stress-strain relationships at low and high stress levels.

6.4 The axial Young's modulus, E, may be calculated using any one of several methods employed in engineering practice. The most common methods, described in Fig. 2, are as follows:

6.4.1 Tangent modulus at a stress level which is some fixed percentage of the maximum strength.

6.4.2 Average slope of the more-or-less straight line portion of the stress-strain curve.

6.4.3 Secant modulus, usually from zero stress to some fixed percentage of maximum strength.

7. Calculation (Procedure B)

7.1 The circumferential or diametric strain, ϵ_d, may be recorded directly from strain-indicating equipment, or may be calculated from deformation readings depending upon the type of apparatus or instrumentation employed.

7.1.1 Calculate the diametric strain, ϵ_d, from the following equation:

$$\epsilon_d = \Delta d/d$$

where:

d = original undeformed diameter, in. (mm), and

Δd = change in diameter (positive for an increase in diameter), in. (mm).

NOTE 10—It should be noted that the circumferentially applied electrical resistance strain gages reflect diametric strain, the value necessary in computing Poisson's ratio, ν. Since $C = \pi d$; and $\Delta C = \pi \Delta d$, the circumferential strain, ϵ_c, is related to the diametric strain, ϵ_d, through the relation:

$$\epsilon_c = \Delta C/C = \pi\Delta d/\pi d = \Delta d/d$$

so that $\epsilon_c = \epsilon_d$ where C and d are the specimen circumference and diameter, respectively.

7.2 Plot the stresses calculated in 6.2 versus the corresponding diametric strains determined in 7.1.

7.3 The value of Poisson's ratio, ν, is greatly affected by nonlinearities at low stress levels in the axial and lateral stress-strain curves. It is suggested that Poisson's ratio be calculated from the equation:

$$\nu = -\text{slope of axial curve/slope of lateral curve}$$
$$= -E/\text{slope of lateral curve}$$

where the slope of the lateral curve is determined in the same manner as was done in 6.4 for Young's modulus, E.

NOTE 11—The denominator in the equation in 7.3 will have a negative value if the sign convention is applied properly. The negative sign in the equation thereby assures a positive value for ν.

8. Report

8.1 *Procedure A*—The report shall include the following:

8.1.1 Source of sample including project name and location. Often the location is specified in terms of the drill hole number and depth of specimen from collar of hole.

8.1.2 Date test is performed.

8.1.3 Specimen diameter and height, conformance with dimensional requirements.

8.1.4 Rate of loading or deformation rate.

8.1.5 Values of applied load, stress and axial strain as tabulated results or as recorded on a chart.

8.1.6 Plot of stress versus axial strain as shown in Fig. 1, if data are tabulated in 8.1.5. If data are recorded directly on a chart, the load and deformation axes may be scaled to give stress and strain without replotting the curve.

8.1.7 Young's modulus, E, method of determination as given in Fig. 2, and at what stress level or levels determined.

8.1.8 Physical description of sample including rock type such as sandstone, limestone, granite, etc.; location and orientation of apparent weakness planes, bedding planes, and schistosity; and large inclusions or inhomogeneities, if any.

8.1.9 General indication of moisture condition of sample

at time of test such as as-received, saturated, laboratory air-dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation.

8.2 *Procedure B*—Procedure B of the test method must always accompany Procedure A in order to determine Poisson's ratio. The report for Procedure B, which is in addition to that for Procedure A, shall include the following:

8.2.1 Values of applied load, stress, and diametric strain as tabulated results or as recorded on a chart.

8.2.2 Plot of stress versus diametric strain as shown in Fig. 1 if data is tabulated in 8.2.1. If data is recorded directly on a chart, the load and deformation axes may be scaled to give stress and strain without replotting the curve.

8.2.3 Poisson's ratio, ν, method of determination in 7.3, and at what stress level or levels determined.

9. Precision and Bias

9.1 The variability of rock and resultant inability to determine a true reference value prevent development of a meaningful statement of bias. Data are being evaluated to determine the precision of this test method. In addition, the subcommittee is seeking pertinent data from users of the method.

Designation: D 2664 – 86 RTH 202-89

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Test Method for TRIAXIAL COMPRESSIVE STRENGTH OF UNDRAINED ROCK CORE SPECIMENS WITHOUT PORE PRESSURE MEASUREMENTS[1]

This standard is issued under the fixed designation D 2664; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This test method covers the determination of the strength of cylindrical rock core specimens in an undrained state under triaxial compression loading. The test provides data useful in determining the strength and elastic properties of rock, namely: shear strengths at various lateral pressures, angle of internal friction, (angle of shearing resistance), cohesion intercept, and Young's modulus. It should be observed that this method makes no provision for pore pressure measurements. Thus the strength values determined are in terms of total stress, that is, not corrected for pore pressures.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

D 4543 Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances[2]

E 4 Practices for Load Verification of Testing Machines[3]

E 122 Recommended Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[4]

3. Significance and Use

3.1 Rock is known to behave as a function of the confining pressure. The triaxial compression test is commonly used to simulate the stress conditions under which most underground rock masses exist.

4. Apparatus

4.1 *Loading Device*—A suitable device for applying and measuring axial load to the specimen. It shall be of sufficient capacity to apply load at a rate conforming to the requirements specified in 7.2. It shall be verified at suitable time intervals in accordance with the procedures given in Practices E 4 and comply with the requirements prescribed in the method.

4.2 *Pressure-Maintaining Device*—A hydraulic pump, pressure intensifier, or other system of sufficient capacity to maintain constant the desired lateral pressure, σ_3.

NOTE 1—A pressure intensifier as described by Leonard Obert in U.S. Bureau of Mines Report of Investigations No. 6332, "An Inexpensive Triaxial Apparatus for Testing Mine Rock," has been found to fulfill the above requirements.

4.3 *Triaxial Compression Chamber*[5]—An apparatus in which the test specimen may be en-

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Oct. 31, 1986. Published December 1986. Originally published as D 2664 – 67. Last previous edition D 2664 – 80.

[2] *Annual Book of ASTM Standards*, Vol 04.08.

[3] *Annual Book of ASTM Standards*, Vols 03.01, 04.02, 07.01 and 08.03.

[4] *Annual Book of ASTM Standards*, Vol 14.02.

[5] Assembly and detail drawings of an apparatus that meets these requirements and which is designed to accommodate 2⅛-in. (53.975-mm) diameter specimens and operate at a lateral fluid pressure of 10 000 psi (689 MPa) are available from Headquarters. Request Adjunct No. 12-426640-00.

closed in an impermeable flexible membrane; placed between two hundred platens, one of which shall be spherically seated; subjected to a constant lateral fluid pressure; and then loaded axially to failure. The platens shall be made of tool steel hardened to a minimum of Rockwell 58 HRC, the bearing faces of which shall not depart from plane surfaces by more than 0.0005 in. (0.0127 mm) when the platens are new and which shall be maintained within a permissible variation of 0.001 in. (0.025 mm). In addition to the platens and membrane, the apparatus shall consist of a high-pressure cylinder with overflow valve, a base, suitable entry ports for filling the cylinder with hydraulic fluid and applying the lateral pressure, and hoses, gages, and valves as needed.

4.4 *Deformation and Strain-Measuring Devices*—High-grade dial micrometers or other measuring devices graduated to read in 0.0001-in. (0.0025-mm) units, and accurate within 0.0001 in. (0.0025 mm) in any 0.0010-in. (0.025-mm) range, and within 0.0002 in. (0.005 mm) in any 0.0100-in. (0.25-mm) range shall be provided for measuring axial deformation due to loading. These may consist of micrometer screws, dial micrometers, or linear variable differential transformers securely attached to the high pressure cylinder.

4.4.1 Electrical resistance strain gages applied directly to the rock specimen in the axial direction may also be used. In addition, the use of circumferentially applied strain gages will permit the observation of data necessary in the calculation of Poisson's ratio. In this case two axial (vertical) gages should be mounted on opposite sides of the specimen at mid-height and two circumferential (horizontal) gages similarly located around the circumference, but in the direction perpendicular to the axial gages.

4.5 *Flexible Membrane*—A flexible membrane of suitable material to exclude the confining fluid from the specimen, and that shall not significantly extrude into abrupt surface pores. It should be sufficiently long to extend well onto the platens and when slightly stretched be of the same diameter as the rock specimen.

NOTE 2—Neoprene rubber tubing of 1/16-in. (1.588-mm) wall thickness and of 40 to 60 Durometer hardness, Shore Type A or various sizes of bicycle inner tubing, have been found generally suitable for this purpose.

5. Sampling

5.1 The specimen shall be selected from the cores to represent a true average of the type of rock under consideration. This can be achieved by visual observations of mineral constituents, grain sizes and shape, partings and defects such as pores and fissures.

6. Test Specimens

6.1 *Preparation*—The test specimens shall be prepared in accordance with Practice D 4543.

6.2 Moisture condition of the specimen at the time of test can have a significant effect upon the indicated strength of the rock. Good practice generally dictates that laboratory tests be made upon specimens representative of field conditions. Thus it follows that the field moisture condition of the specimen should be preserved until the time of test. On the other hand, there may be reasons for testing specimens at other moisture contents, including zero. In any case the moisture content of the test specimen should be tailored to the problem at hand and reported in accordance with 9.1.6.

7. Procedure

7.1 Place the lower platen on the base. Wipe clean the bearing faces of the upper and lower platens and of the test specimen, and place the test specimen on the lower platen. Place the upper platen on the specimen and align properly. Fit the flexible membrane over the specimen and platen and install rubber or neoprene O-rings to seal the specimen from the confining fluid. Place the cylinder over the specimen, ensuring proper seal with the base, and connect the hydraulic pressure lines. Position the deformation measuring device and fill the chamber with hydraulic fluid. Apply a slight axial load, approximately 25 lbf (110 N), to the triaxial compression chamber by means of the loading device in order to properly seat the bearing parts of the apparatus. Take an initial reading on the deformation device. Slowly raise the lateral fluid pressure to the predetermined test level and at the same time apply sufficient axial load to prevent the deformation measuring device from deviating from the initial reading. When the predetermined test level of fluid pressure is reached note and record the axial load registered by the loading device. Consider this load to be the zero or starting load for the test.

7.2 Apply the axial load continuously and without shock until the load becomes constant, or reduces, or a predetermined amount of strain is achieved. Apply the load in such a manner as to produce a strain rate as constant as feasible throughout the test. Do not permit the strain rate at any given time to deviate by more than 10 % from that selected. The strain rate selected should be that which will produce failure of a similar test specimen in unconfined compression, in a test time of between 2 and 15 min. The selected strain rate for a given rock type shall be adhered to for all tests in a given series of investigation (Note 3). Maintain constant the predetermined confining pressure throughout the test and observe and record readings of deformation as required.

NOTE 3—Results of tests by other investigators have shown that strain rates within this range will provide strength values that are reasonably free from rapid loading effects and reproducible within acceptable tolerances.

7.3 To make sure that no testing fluid has penetrated into the specimen, the specimen membrane shall be carefully checked for fissures or punctures at the completion of each triaxial test. If in question, weigh the specimen before and after the test.

8. Calculations

8.1 Make the following calculations and graphical plots:

8.1.1 Construct a stress difference versus axial strain curve (Note 5). Stress difference is defined as the maximum principal axial stress, σ_1, minus the lateral pressure, σ_3. Indicate the value of the lateral pressure, σ_3, on the curve.

NOTE 4—If the specimen diameter is not the same as the piston diameter through the chamber, a correction must be applied to the measured load to account for differences in area between the specimen and the loading piston where it passes through the seals into the chamber.

NOTE 5—If the total deformation is recorded during the test, suitable calibration for apparatus deformation must be made. This may be accomplished by inserting into the apparatus a steel cylinder having known elastic properties and observing differences in deformation between the assembly and steel cylinder throughout the loading range. The apparatus deformation is then subtracted from the total deformation at each increment of load in order to arrive at specimen deformation from which the axial strain of the specimen is computed.

8.1.2 Construct the Mohr stress circles on an arithmetic plot with shear stresses as ordinates and normal stresses as abscissas. Make at least three triaxial compression tests, each at a different confining pressure, on the same material to define the envelope to the Mohr stress circles.

NOTE 6—Because of the heterogeneous nature of rock and the scatter in results often encountered, it is considered good practice to make at least three tests of essentially identical specimens at each confining pressure or single tests at nine different confining pressures covering the range investigated. Individual stress circles shall be plotted and considered in drawing the envelope.

8.1.3 Draw a "best-fit", smooth curve (the Mohr envelope) approximately tangent to the Mohr circles as in Fig. 1. The figure shall also include a brief note indicating whether a pronounced failure plane was or was not developed during the test and the inclination of this plane with reference to the plane of major principal stress.

NOTE 7—If the envelope is a straight line, the angle the line makes with the horizontal shall be reported as the angle of interval friction, ϕ (or the slope of the line as tan ϕ depending upon preference) and the intercept of this line at the vertical axis reported as the cohesion intercept, C. If the envelope is not a straight line, values of ϕ (or tan ϕ) should be determined by constructing a tangent to the Mohr circle for each confining stress at the point of contact with the envelope and the corresponding cohesion intercept noted.

9. Report

9.1 The report shall include as much of the following as possible:

9.1.1 Sources of the specimen including project name and location, and if known, storage environment. The location is frequently specified in terms of the borehole number and depth of specimen from collar of hole.

9.1.2 Physical description of the specimen including rock type; location and orientation of apparent weakness planes, bedding planes, and schistosity; large inclusions or inhomogeneities, if any.

9.1.3 Dates of sampling and testing.

9.1.4 Specimen diameter and length, conformance with dimensional requirements.

9.1.5 Rate of loading or deformation or strain rate.

9.1.6 General indication of moisture condition of the specimen at time of test such as: as-received, saturated, laboratory air-dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible

and reported as either water content or degree of saturation.

9.1.7 Type and location of failure. A sketch of the fractured specimen is recommended.

NOTE 8—If it is a ductile failure and $\sigma_1 - \sigma_3$ is still increasing when the test is terminated, the maximum strain at which $\sigma_1 - \sigma_3$ is obtained shall be clearly stated.

10. Precision and Bias

10.1 The variability of rock and resultant inability to determine a true reference value prevent development of a meaningful statement of bias. Data are being evaluated to determine the precision of this test method. In addition, the subcommittee is seeking pertinent data from users of the method.

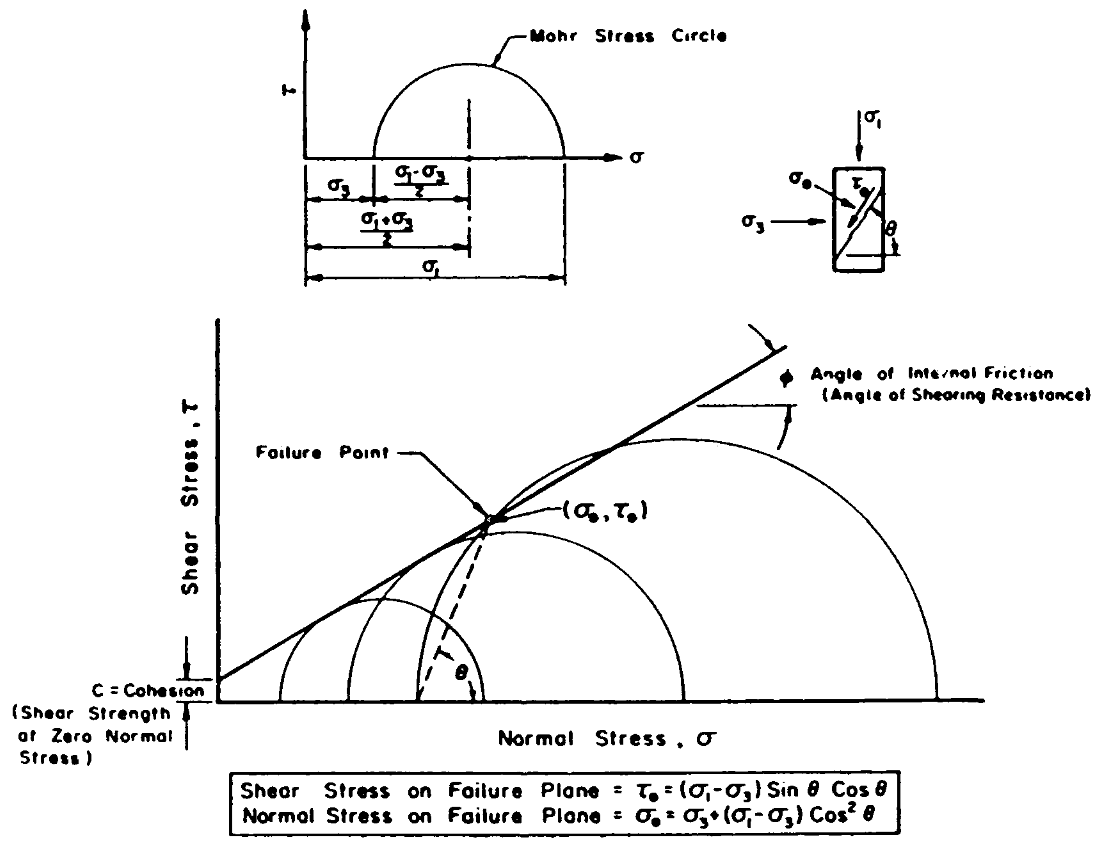


FIG. 1 Typical Mohr Stress Circles

RTH 203-80

METHOD OF TEST FOR DIRECT SHEAR STRENGTH OF ROCK CORE SPECIMENS

1. Scope

1.1 This method describes apparatus and procedures for determining the shear strength of a rock material in direct shear. The test can be made on rock core specimens from 2 to 6 in. (5 to 15 cm) in diameter. The test can be made on intact specimens to determine intact shear strength, on intact specimens with recognizable thin weak planes to determine the shearing resistance along these planes, on presawn shear surfaces to determine lower bound residual shear strengths, and on rock core to concrete bond specimens to determine the shearing resistance between the bond. The principle of the rock core direct shearing is illustrated schematically in Fig. 1.

1.2 A minimum of three test specimens of any rock type are subjected to different but constant normal stresses during the shearing process. For each type of intact rock, cohesion and an angle of internal friction are determined. For each type of rock with sawn failure surfaces, a lower bound residual angle of internal friction is determined.

1.3 The test is not suited to the development of exact stress-strain relationships within the test specimen because of the nonuniform distribution of shearing stresses and displacements. Care should be taken so that the testing conditions represent those being investigated. The results of these tests are used where field design requirements dictate unconsolidated, undrained parameters.

2. Apparatus

2.1 Test Specimen Saw - For cores of 3 to 6 in. (7.5 to 15 cm) in diameter, use a rock saw with 20-in.- (50-cm-) diam safety abrasive blade fitted for dry and for wet cutting. Alternatively for wet cutting, a diamond blade may be used. For cores 2 to 2-1/2 in. (5 to 6.25 cm) in diameter, a rock saw with 12-in.- (60-cm-) diam blade should be used.

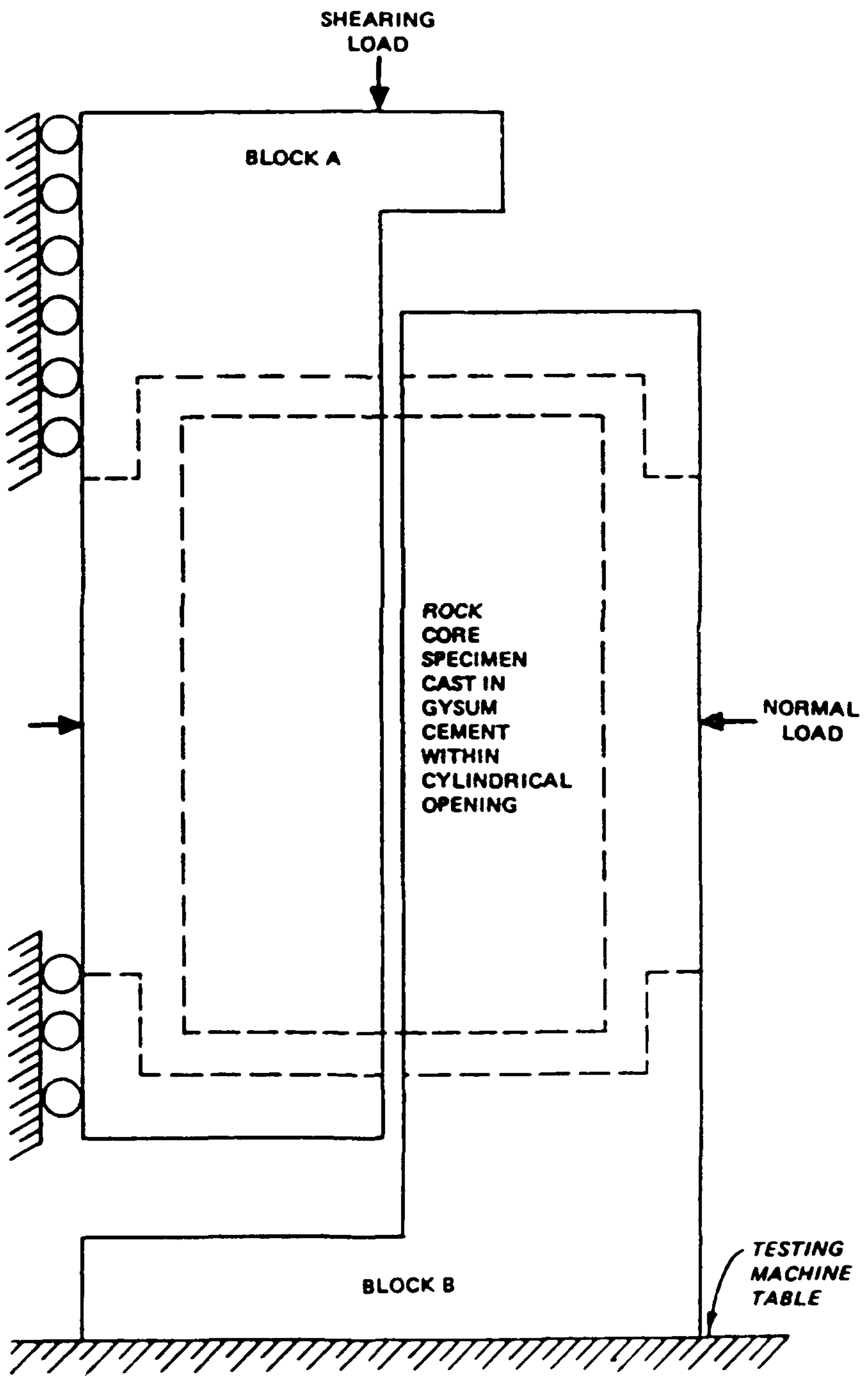


Fig 1. Schematic showing direct shear of rock core.

2.2 Shear Device - The shear device shall consist of a pair of shear boxes constructed so as to provide a means of applying a normal stress to the face of the specimen while applying a force to shear the specimen along a predetermined plane parallel to the vertical axis of the specimen. The device shall securely hold the specimen in such a way that torque cannot be applied to the specimen. The shear boxes that hold the specimen shall be sufficiently rigid to prevent their distortion during shearing. The various parts of the shear device shall be made of material not subject to corrosion by substances within the rock or moisture within the rock.

2.2.1 Shear boxes suitable for testing specimens from 3 to 6 in. (7.5 to 15 cm) in diameter should each have a recess 6-5/8 in. (16.6 cm) in diameter and 2-1/4 in. (5.67 cm) deep. Smaller shear boxes for 2- to 2-1/2-in.- (5- to 6.25-cm-) diam specimens should each have a recess 2-7/8 in. (7.36 cm) in diameter and 2 in. (5 cm) in depth. The shear boxes should be designed for a shear travel greater than 10 percent of the specimen shear plane length.

2.2.2 In both cases the two shear boxes assembled with the specimen shall be placed within a framework constructed so as to hold the boxes in proper position during testing. The framework shall include a pair of hardened stainless steel plates machined to accommodate roller bearings or ball bearings for minimizing friction of the moving shear box as indicated in Fig. 1. The roller plate device should ensure that resistance of the equipment to shear displacement is less than 1 percent of the maximum shear force applied in the test.

2.2.3 The shear device framework shall include capability of providing a submerging tank for tests in which maintaining specimen saturation is important to duplicate field conditions.

2.3 Loading Devices

2.3.1 Normal Force - The normal force device shall be capable of applying the specified force quickly without exceeding it and capable of maintaining it with an accuracy of ±2 percent for the duration of the

test. The device shall have a travel greater than the amount of dilation or compression to be expected.

2.3.2 Shear Force - The device for applying the shear force shall distribute the load uniformly along one-half face of the specimen with the resultant applied shear force acting in the plane of shearing. The required capabilities will depend upon whether a controlled-displacement test or a controlled-stress test is used. Controlled-displacement equipment shall be capable of shearing the specimen at a uniform rate of displacement with less than ±15 percent deviation and shall permit adjustment of the rate of displacement over a relatively wide range. Controlled-stress equipment shall be capable of applying the shear force in increments to the specimen in the same manner and to the same degree of accuracy as that described in 2.3.1.

2.4 Displacement Indicators - Equipment for measuring shear and normal displacements may consist of mechanical devices, such as dial gages or electric transducers. Displacement indicators shall have a sensitivity of at least 0.001 in. (0.025 mm). The shear displacement measuring system shall have a travel greater than 10 percent of the specimen shear plane length. Normal displacement systems shall have the capability of measuring both dilation and compression of the specimen. Resetting of gages during the test should if possible be avoided. If electric transducers or an automatic recording system is used, a recent calibration shall be included in the report.

2.5 Casting Compound - High-strength gypsum cement (such as hydrostone) or a capping compound (such as leadite) should be used to hold the test specimen in the recesses of the test device.

2.6 Spacer Plate - The spacer plate separating the shear boxes for development of the shear zone shall be 1/16 in. (1.6 mm) thick and constructed of a noncorrosive material.

3. Test Specimen

3.1 Intact Specimens

3.1.1 Test specimens shall be prepared by sawing rock cores into 3- to 4-in. (7.5- to 10-cm) lengths (Note 1). The diameter of each specimen shall be measured to the nearest 0.01 in. (0.025 mm) at several different positions along the length of the specimen axis. The average diameter shall be used to compute the cross-sectional area of the specimen. The volume of the specimen shall be determined by the volumetric or displacement method presented in EM 1110-2-1906, "Laboratory Soils Testing." The initial weight of the specimen shall be determined to the nearest 0.1 g for subsequent use in determining initial moisture content and density.

NOTE 1—Soft rock such as clay shales may be as short as 2-1/2 in. (6.25 cm) if material is scarce. Although helpful in the setup, ends of the test specimens need not be smooth, flat, nor square with the axis of the core. Generally, harder rocks are best cut in the wet; softer rocks are best cut in the dry, depending on fissility and reaction to pressure of cutting water.

3.1.2 A block of the shear box shall be set on a flat surface with the shear surface up. The inside of the recess shall be lightly coated with lubricant. A grout of the gypsum cement (hydrostone) and water shall be placed in the recess to approximately the one-third or midpoint. After approximately three minutes of setting, the specimen shall be set or pushed into the grout until the approximate midpoint (desired shear plane) of the specimen is opposite the top recess (Note 2). Excess grout shall be screeded off at the shear plane (Note 3).

NOTE 2—To prepare intact test specimens for testing along recognizable thin weak planes, orient the specimen so that the plane of weakness is parallel with the 1/16-in. (1.6-mm) shear gap provided by the spacer plate.

NOTE 3—Gypsum cement grout has only a few minutes pot life; hence a fresh mix will have to be prepared for each block. An alternative to gypsum cement grout for holding the test specimen in the recesses is capping compound, such as leadite. The procedure for preparing specimens with a capping compound is essentially the same as for gypsum cement. Capping compound has a shorter pot life after pouring than gypsum cement and must be heated to proper temperature and handled quickly and with great care. An overnight curing period is generally required. Capping compound is stronger in compression and shear than gypsum cement and is preferred for hard rock testing. Because capping compound must be placed hot, it should not be used to secure specimens subject to structural damage with loss of natural moisture or for tests in which it is desirable to maintain natural moisture.

3.1.3 A 1/16-in.- (1.6-mm-) thick spacer plate having a hole equal to the diameter of the specimen and split on a diameter from the front to the back of the block shall be coated with lubricant and placed on the block around the specimen. The spacer separates the two blocks of the shear box to prevent friction between the blocks during shearing. The recess of the remaining shear box block shall be lightly coated with lubricant and the block placed over the now protruding half of the specimen. The two blocks shall be aligned and temporarily clamped together with C clamps. The recess between the top block and specimen shall then be filled with the gypsum cement grout using appropriate tools to rod the grout thoroughly around the specimen. For soft rock such as clay shale, a 2-hour curing is usually sufficient before loading. For hard rocks, the grout must be allowed to cure overnight.

3.1.4 At the end of the curing period, the two halves of the spacer shall be pulled out and the C clamps removed. The specimens secured in the shear boxes are then ready for further assembly and shear testing.

3.2 <u>Presawn Shear Surfaces</u> - Test specimens shall be prepared the same as presented in paragraphs 3.1.1 to 3.1.4, except that the specimen shall be sawn in half near the center length before grouting the

specimen in the shear box blocks. The presawn shear surface shall be smooth and oriented in the shear box so as to be centered within the 1/16-in. (1.6-mm) shear gap provided by the spacer plate.

3.3 Concrete to Rock Core Bond

3.3.1 Test specimens shall be prepared by sawing rock cores into 1.5- to 2-in. (3.75- to 5-cm) length. The sawn specimen shall be tightly encased in the bottom of a 3- to 4-in.- (7.5- to 10-cm-) high mold (Note 4) with the smooth sawn surface (shear plane) facing upward and perpendicular to the axis of the mold. The remaining portion of the mold shall be filled with concrete, which is then consolidated and cured according to the procedures presented in CRD-C 10-73 (Note 5). The concrete mix design shall be compatible in consistency and strength with the anticipated field design mix and have a maximum aggregate no larger than 1/6 of the specimen diameter.

NOTE 4—Molds shall be made of steel, cast iron, or other nonabsorbent material, nonreactive with concrete containing portland or other hydraulic cements. Mold diameters shall conform to the dimensions of the rock core test specimen. Molds shall hold their dimensions and shape and be watertight under conditions adverse to use.

NOTE 5—"Handbook for Concrete and Cement," U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, published in quarterly supplements.

3.3.2 Procedures for measuring specimen weight, diameter, and volume are the same as presented in paragraph 3.1.1. Procedures for securing the test specimen in the shear box are the same as presented in paragraphs 3.1.2 to 3.1.4.

4. Procedure

4.1 Following the removal of the spacer plates and C clamps, transfer final assembly operations to the test shear and normal load area. Final assembly of the testing apparatus, to include orientation of the resultant normal and shear loads, will depend on the equipment utilized

in the testing. In general, the resultant of the normal load shall react through the axial center of the specimen, and the shear load shall react through the radial center of the specimen so as to pass through the shear plane. Position or activate, or both, the displacement indicators for measuring shear deformation and changes in specimen thickness.

4.2 Apply the selected normal force (normal stress " cross-section area) to the specimen as rapidly as practical (Note 6). Record and allow any initial elastic compression of the specimen to reach equilibrium. For those tests where applicable, as soon as possible after applying the initial normal force, fill the water reservoir to at least submerge the shear plane.

NOTE 6—The normal force used for each of the three or more specimens will depend upon the input information required for field analysis and/or design.

4.3 Shear the specimen.

4.3.1 After any elastic compression has reached equilibrium, apply the shearing force and shear the specimen. In a controlled-displacement test, the rate of displacement shall be less than 0.004 in./min (0.1 mm/min) until peak strength is reached. Approximately 10 sets of readings should be taken before reaching peak strength. If it is desired to determine the ultimate strength, the normal load shall be relieved and the specimen recentered. The normal load is then reapplied and the specimen sheared again. The rate of shear displacement to determine the ultimate strength shall be no greater than 0.01 in./min (0.25 mm/min). Readings should be taken at increments of from 0.02- to 0.2-in. (0.5- to 5-mm) shear displacement as required to adequately define the force-displacement curves.

4.3.2 In a controlled-stress test the rate of stress application should not exceed 5 psi/min (34.47 kPa/min) for soft rock (such as clay shale) and up to 100 psi (689.4 kPa) for the very hardest rock.

Concurrent time, shear load, and deformation readings shall be taken at convenient intervals (a minimum of 10 readings before reaching peak strength). After reaching peak strength, the ultimate strength may be determined as presented in paragraph 4.3.1.

5. Calculations

5.1 Calculate the following:

5.1.1 Initial cross-sectional area.

5.1.2 Initial water content.

5.1.3 Initial wet and dry unit weights.

5.1.4 Shear stress data.

5.1.5 Initial and final degrees of saturation, if desirable.

6. Report

6.1 The report shall include the following:

6.1.1 Description of type of shear device used in the test.

6.1.2 Identification and description of the sample.

6.1.3 Description of the shear surface.

6.1.4 Initial water content.

6.1.5 Initial wet and dry unit weights.

6.1.6 Initial and final degrees of saturation, if desirable.

6.1.7 All basic test data including normal stress, shear displacement, corresponding shear resistance values, and specimen thickness changes.

6.1.8 For each test specimen, a plot of shear and specimen thickness change versus shear displacement and a plot of composite maximum and ultimate shear stress versus normal stress.

6.1.9 Departures from the procedure outlined, such as special loading sequences or special wetting requirements.

STANDARD METHOD OF TEST FOR MULTISTAGE TRIAXIAL STRENGTH OF UNDRAINED ROCK CORE SPECIMENS WITHOUT PORE PRESSURE MEASUREMENTS

1. Scope

1.1 This method covers the determination of the strength of cylindrical rock specimens in an undrained state under multistage triaxial loading. The test provides data useful in delineating the strength of joints, seams, bedding planes, etc. This method makes no provision for pore pressure measurements. Thus, the strength values determined are in forms of total stress, i.e. not corrected for pore pressures.

2. Apparatus

2.1 The apparatus is identical with that used in RTH 202, "Triaxial Compressive Strength of Undrained Rock Core Specimens Without Pore Pressure Measurements."

3. Test Specimens

3.1 In the case of intact specimens that develop a well-defined shear failure plane, it is possible to continue testing beyond the first failure; that is, the confining pressure can be raised to a higher level and another peak stress recorded. This may be done immediately following the completion of a conventional triaxial test as conducted according to RTH 202.

3.2 Multistage testing can also be used with cores that are intact initially. The key factor in making these studies is the use of specimens with the failure plane preestablished to cause failure along an inherent weakness, such as seams, open joints, bedding planes, faults, schistosity bands, or laminations. These planes of weakness, when tested, should be oriented at 45 to 65 deg (0.79 to 1.14 radians) from the horizontal, which will normally produce a failure in the preoriented zone. When including these specific geologic features in an NX size core, the specimens can be drilled from 6-in. (15-cm) or larger cores by suitable orientation in the drilling apparatus. Care must be taken when coring these specimens to prevent breakage. However, if the core is

broken along a weakness plane, it may still be tested as an open joint. Broken cores can be taped together with plastic tape, only sufficiently to maintain the matching contact between the broken parts. The test specimens are then prepared in the manner described in Section 3 of RTH 202.

4. Procedure

4.1 The test procedure described in Section 4 of RTH 202 shall be used for the first stage of test. Subsequent stages should be achieved in like manner by applying progressively higher levels of lateral fluid pressure. This may be done as many times as desired, provided the total strain does not cause excessive misalignment of the steel platens and an eccentric loading. This procedure is referred to as multistage testing. Fig. 1 illustrates this loading sequence.

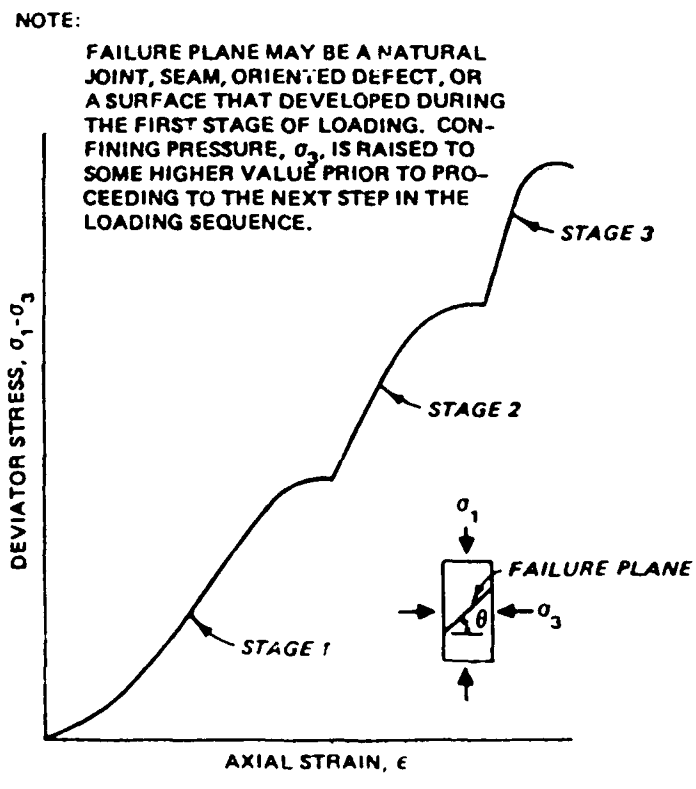


Fig. 1. Stress-strain curve for multistage triaxial test.

5. Calculations

5.1 The shear strength on the joint can be determined graphically by constructing a Mohr circle as shown in Fig. 2. Proof of this construction may be found in most soil mechanics texts.[1] For a multistage test on a given specimen, several Mohr's circles can be drawn and the same angle used to plot the stresses on the failure plane. A strength envelope for this condition, Fig. 3, which is the average of the results determined for each Mohr's circle, is considered to be the joint friction angle.

5.2 There are variations of this plotting technique that may also be employed. The multistage test described above produces a joint friction angle from a single specimen. For a strength envelope derived from tests of several intact specimens, plotting failure plane stresses would yield a higher limiting strength criterion than that of open joints. To report this type of data properly, all orientation data must be carefully and fully stated to ensure proper interpretation of results.

5.3 Various orientations of seams may also be tested to determine that which is most critical. Direct tension and unconfined compression tests may be included to completely define the strength envelope as shown in Fig. 4.

6. Report

6.1 In addition to the plots discussed in Section 5, the report should include the following:

6.1.1 Lithologic description of the rock, including the type of joint, seam, etc., tested.

6.1.2 Source of sample including depth and orientation, dates of sampling and testing, and storage environment.

6.1.3 Specimen diameter and height.

---

[1] Taylor, D., "Fundamentals of Soil Mechanics," John Wiley and Sons, Inc., p 317, or Spangler, M., "Soil Engineering," International Textbook Co., p 277.

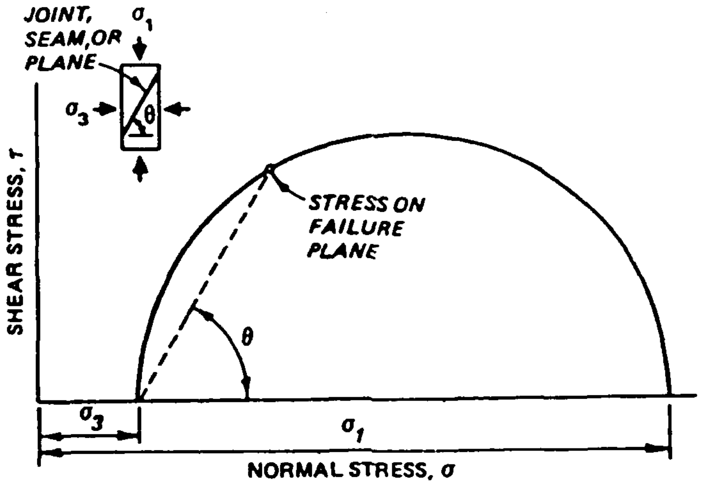


Fig. 2. Mohr circle showing method of construction for locating stresses in failure plane.

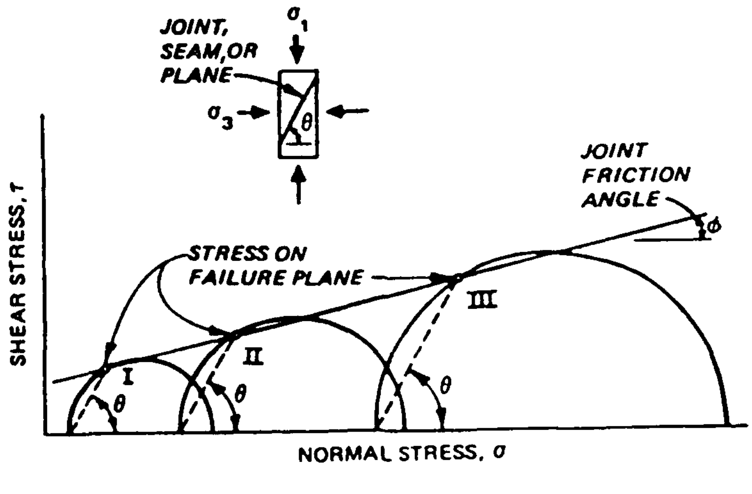


Fig. 3. Mohr diagram for locating stresses on failure plane in a multistage test.

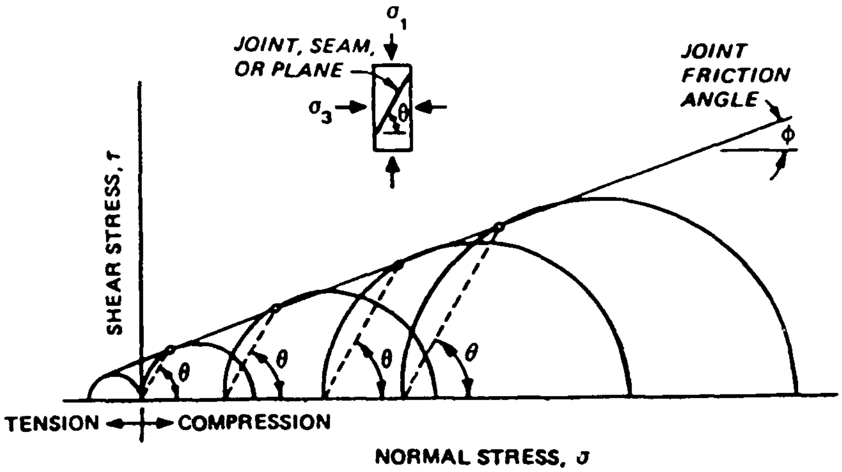


Fig. 4. Mohr diagram for locating stresses on failure plane, including direct tension and unconfined compression test data.

6.1.4 Moisture content and degree of saturation at time of test.

6.1.5 Other physical data, such as specific gravity, absorption, porosity, and permeability, citing the method of determination for each.

Designation: D 4405 – 84 (Reapproved 1989)$^{\epsilon 2}$

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St. Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards. Copyright ASTM
If not listed in the current combined index, will appear in the next edition.

Standard Test Method for Creep of Cylindrical Soft Rock Core Specimens in Uniaxial Compressions[1]

This standard is issued under the fixed designation D 4405; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

$^{\epsilon 1}$ NOTE—Section 9 was changed editorially in July 1989.
$^{\epsilon 2}$ NOTE—Section 10 was added editorially in December 1991.

1. Scope

1.1 This test method covers the determination of the creep behavior of intact cylindrical soft rock core specimens subjected to uniaxial compression up to a temperature of 572°F (300°C). Creep is the time-dependent strain or deformation under sustained axial stress. Soft rocks include such materials as salt and potash, which often exhibit very large strain at failure.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

E 4 Practices for Load Verification of Testing Machines[2]

E 122 Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[3]

3. Significance and Use

3.1 There are many underground structures that are created for permanent use. These structures are subjected to an approximately constant load. Creep tests provide quantitative parameters for stability analysis of these structures.

4. Apparatus

4.1 *Loading Device*—The loading device shall be capable of applying and maintaining the required load on the specimen, regardless of any changes in the dimensions of the specimen. The loading device consists of a reaction frame and a load generating component which may be a hydraulic actuator, spring, dead weight, or controlled screw-driven loading mechanism. Means shall be provided for measuring the load to within 2 % of the applied load. The applied load shall be maintained within ±2 % of the required test load for the duration of the testing. The stability of the loading device and the reaction frame as a function of time shall be evaluated prior to testing, and periodically during testing. To maintain the necessary stability of the loading device, the room temperature shall be maintained to within ±2°F (±1°C).

NOTE 1—By definition, creep is time-dependent deformation under constant axial stress. The loading device is specified to maintain constant axial load. The engineering stress (load divided by original cross-sectional area) will therefore be constant, while the true stress (load divided by actual cross-sectional area at the time) will decrease somewhat during the test as the cross-sectional area increases. Standard practice in creep testing is to maintain constant load (constant engineering stress) because of the experimental difficulties in controlling the load to maintain a constant true stress.

4.2 *Bearing Surfaces*—The testing machine shall be equipped with two steel bearing blocks having a hardness of not less than 58 HRC. One of the blocks shall be spherically seated and the other a plain rigid block. The bearing faces shall not depart from a plane by more than 0.0005 in. (0.013 mm) when the blocks are new and shall be maintained within a permissible variation of 0.001 in. (0.025 mm). The diameter of the spherically seated bearing face shall be at least as large as that of the test specimen but shall not exceed twice the diameter of the test specimen. The center of the sphere in the spherically seated block shall coincide with that of the bearing face of the specimen. The spherically seated block shall be properly lubricated to assure free movement. The movable portion of the bearing block shall be held closely in the spherical seat, but the design shall be such that the bearing face can be rotated and tilted through small angles in any direction.

4.2.1 *Room Temperature Bearing Blocks*—In a room temperature test, the specimen may be placed directly between the upper and lower machine platens, or false platens may be used between the specimen and the machine platens. False platens (steel spacer disks) shall possess the same material and flatness specifications as the machine platens. The diameter of the false platens shall be at least as great as the specimen, but not exceeding the specimen diameter by more than 0.050 in. (1.3 mm).

4.2.2 *Elevated Temperature Bearing Blocks*—The steel false platens shall be thermally insulated from the machine platens by insulating spacers in the load column or by cooling coils. These insulating spacers or cooling coils are necessary if the temperature enclosure directly surrounds the specimen, but not if the entire room is maintained at the elevated temperature.

---

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Sept. 28, 1984. Published November 1984.

[2] *Annual Book of ASTM Standards*, Vols 04.02, 07.01, and 08.03.

[3] *Annual Book of ASTM Standards*, Vol 14.02.

4.3 *Elevated Temperature Enclosure*—The elevated temperature tests can be conducted either in a temperature controlled room or in a small chamber directly surrounding the test specimen. Temperature shall be measured at three locations, with one sensor near the top, one at midheight, and one near the bottom of the specimen. The average specimen temperature based on the midheight sensor shall be maintained to within ±2 % of the required test temperature, measured in degrees Celcius. The maximum temperature difference between the midheight sensor and either end sensor shall not exceed 5 % of the test temperature in degrees Celcius.

4.4 *Temperature Measuring Devices*—The type of instrument chosen to monitor temperature depends primarily on the test apparatus and the maximum test temperature. Special limits-of-error thermocouples or platinum resistance thermometers (RTDs) are recommended. The temperature transducer shall be accurate to at least ±1.0°F (±0.5°C), with a resolution of 0.20°F (0.1°C).

4.5 *Strain/Deformation Measuring Devices:*

4.5.1 *Room Temperature:*

4.5.1.1 *Axial Strain Determination*—For these materials, the axial deformation or strain may not normally be determined by electrical strain gages, and some other forms of strain/deformation measuring devices must be utilized. Such devices include compressometers, optical devices, and the like. The design of the measuring device shall be such that the average of at least two axial strain measurements can be recorded either continuously or for each increment of time. Measuring positions shall be equally spaced around the circumference of the specimen close to midheight. The gage length over which the axial strains are determined shall be at least ten grain diameters in magnitude. The axial strains shall be determined with an accuracy of 2 % of the reading and a precision 0.2 % of full-scale over the duration of the test. Furthermore, for readings below 250 μin./in. (μm/m) strain, accuracy shall be within 5 μin./in. (μm/m) strain.

NOTE 2—The use of strain gage adhesives requiring cure temperatures above 150°F (65°C) is not permitted unless it is known that microfractures do not develop at the cure temperature.

4.5.1.2 *Lateral Strain Determination*—The lateral deformations or strains may be measured by any of the methods mentioned in 4.5.1.1. Either circumferential or diametrical deformations (or strains) may be measured. At least two lateral deformation sensors shall be used. These shall be equally spaced around the circumference of the specimen close to midheight. The average deformation (or strain) from the two sensors shall be recorded either continuously or for each increment of time. The use of a single transducer that wraps completely around the specimen to measure the total change in circumference may also be used. The gage length and the accuracy and precision of the lateral strain measurement system shall be the same as those specified in 4.5.1.1.

4.5.2 *Elevated Temperature*—Deformation measurements are normally made externally from the heating enclosure using suitable extensometers.

NOTE 3—An example of such a measurement arrangement is shown in Fig. 1.

5. Test Specimens

5.1 Test specimens shall be right circular cylinders within the tolerances specified herein.

5.1.1 The specimen shall have a length-to-diameter ratio (L/D) of 2.0 to 2.5 and a diameter of not less than NX wireline core size, approximately 1⅞ in. (48 mm).

NOTE 4—It is desirable that the diameter of rock compression specimens be at least ten times the diameter of the largest mineral grain.

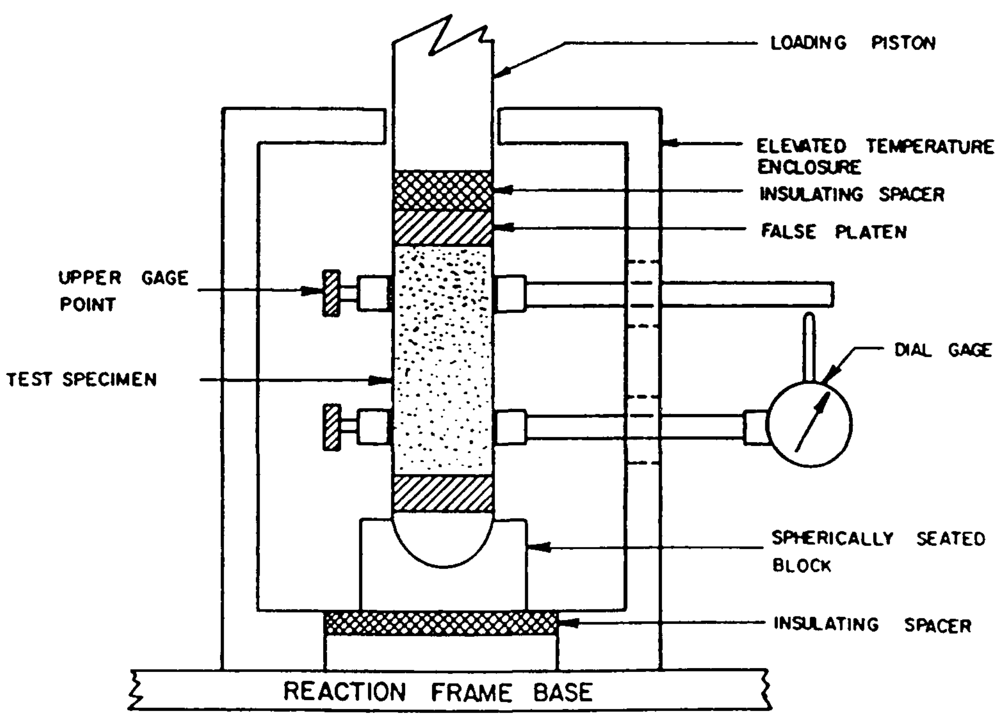


FIG. 1 Example of Technique for Monitoring Axial Specimen Deformation External to Elevated Temperature Enclosure

In soft rock, such as salt and potash, the average grain size is normally larger than it is for most hard rock. Research has shown that creep behavior of salt is highly dependent upon the specimen dimensions. Therefore, in order to obtain meaningful results, sufficiently large specimens should be used.

5.1.2 The sides of the specimen shall be generally smooth and free of abrupt irregularities with all the elements straight to within 0.020 in. (0.50 mm) over the full length of the specimen. The deviation from straightness of the elements shall be determined by either Method A or Method B.

5.1.2.1 *Method A*—Roll the cylindrical specimen on a smooth flat surface and measure the height of the maximum gap between the specimen and the flat surface with a feeler gage. If the maximum gap exceeds 0.020 in. (0.50 mm), the specimen does not meet the required tolerance for straightness of the elements. The flat test surface on which the specimen is rolled shall not depart from a plane by more than 0.0005 in. (13 µm).

5.1.2.2 *Method B:*

5.1.2.2.1 Place the cylindrical surface of the specimen on a V-block that is laid flat on a surface. The smoothness of the surface shall not depart from a plane by more than 0.0005 in. (13 µm).

5.1.2.2.2 Place a dial indicator in contact with the top of the specimen, as shown in Fig. 2, and observe the dial reading as the specimen is moved from one end of the V-block to the other along a straight line.

5.1.2.2.3 Record the maximum and minimum readings on the dial gage and calculate the difference, Δ_0. Repeat the same operations by rotating the specimen for every 90°, and obtain the differences, Δ_{90}, Δ_{180}, and Δ_{270}. The same maximum value of these four differences shall be less than 0.020 in. (0.50 mm).

5.1.3 The ends of the specimen shall be cut parallel to each other and at right angles to the longitudinal axis. They shall be surface ground or lapped flat to 0.001 in. (25 µm). Water shall not be used in any of the sample preparation procedures. The flatness tolerance shall be checked by a setup similar to that for the cylindrical surface (Fig. 3) except that the dial gage is mounted near the end of the V-block. Move the mounting pad horizontally so that the dial gage runs across the end surface of the specimen along a diametral direction. Take care to make sure that one end of the mounting pad maintains intimate contact with the end surface of the V-block during moving. Record the dial gage readings every ⅛ in. (3 mm) across the diameter.

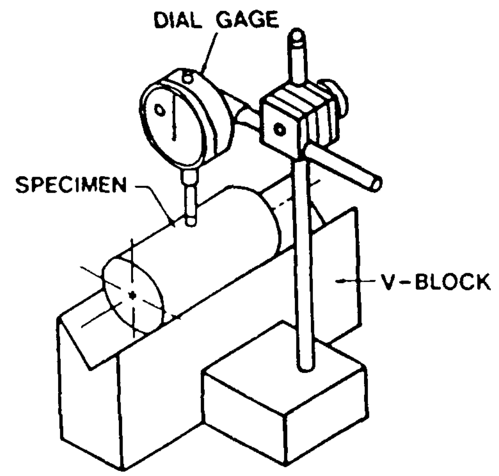


FIG. 2 Assembly for Determining the Straightness of the Cylindrical Surface

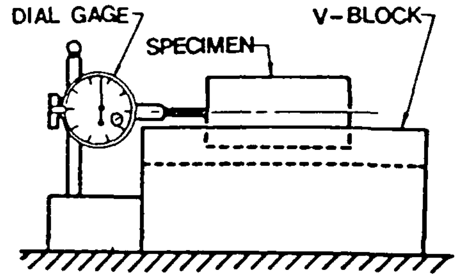


FIG. 3 Assembly for Determining the Flatness and Perpendicularity of End Surfaces to the Specimen Axis

5.1.3.1 Plot the readings and draw a smooth curve through the points to represent the surface profile along the specified diametral plane. The flatness tolerance is met when the smooth curve so determined does not depart from a visual best-fit straight line by more than 0.001 in. (25 µm).

5.1.3.2 Rotate the specimen 90° about its longitudinal axis and repeat the same operations and tolerance check for the new diametral plane. Turn the specimen end for end and repeat the same measurement procedures and tolerance checks for the other end surface.

5.1.4 The ends of the specimen shall not depart from perpendicularity to the axis of the specimen by more than 0.25°, which is a slope of approximately one part in 200. The tolerance shall be checked using the measurements taken in 5.1.3. Calculate the difference between the maximum and minimum readings on the dial gage along diameter 1. This difference is denoted as Δ_1. Calculate the corresponding difference for diameter 2, which is 90° from diameter 1. This difference for diameter 2 is Δ_2. Calculate the corresponding differences for the other end of the specimen, Δ'_1 and Δ'_2. The perpendicularity tolerance will be considered to have been met when:

$$\frac{\Delta_i}{D} \text{ and } \frac{\Delta'_i}{D} \leq 0.005$$

where:

i = 1 or 2, and
D = diameter.

5.1.5 The use of capping materials or end surface treatments other than the grinding and lapping specified herein is not permitted.

5.2 The diameter of the test specimen shall be determined to the nearest 0.01 in. (0.25 mm) by averaging two diameters measured at right angles to each other at about midheight of the specimen. This average diameter shall be used for calculating the cross-sectional area. The height of the test specimen shall be determined to the nearest 0.01 in. (0.25 mm) at the centers of the end faces.

5.3 The specimen should be prepared in a room with a relative humidity of less than 40 %. Prior to and following preparation, specimens shall be stored in an airtight container. During the creep test, the specimen shall be coated or jacketed with an impervious material.

6. Procedure

6.1 *Loading:*

6.1.1 *Room Temperature:*

6.1.1.1 Check the ability of the spherical seat to rotate freely in its socket before each test.

6.1.1.2 Wipe clean the bearing faces of the upper and lower bearing blocks, false platens, if used, and the ends of the test specimen. Place the test specimen with the attached strain-measuring device on the lower bearing block. Carefully align the axis of the specimen with the center of thrust of the spherically seated block. Make electrical connections or adjustments to the strain or deformation-measuring device.

6.1.1.3 Slowly load the specimen to a preload value of 25 psi (170 kPa). During this process, adjust the movable portion of the spherically seated block so that uniform seating is obtained. Zero the strain/deformation-measuring devices. Then rapidly raise the load, without shock to the required test load, within 20 s. Thereafter, the test load shall be held constant during the duration of the test.

6.1.2 *Elevated Temperature:*

6.1.2.1 Wipe clean the bearing faces of the upper and lower false platens and the ends of the test specimen. Jacket the specimen as described in 5.3. The jacket shall be such that after installation, it encloses the specimen tightly and extends over the false platens.

6.1.2.2 Place the specimen on the lower machine platen. Carefully align the axis of the specimen with the center of thrust of the lower platen of the test machine. Slowly load the specimen to a preload value of 25 psi (170 kPa). During this process, adjust the movable portion of the spherically seated block so that uniform seating is obtained. If the test temperature is below 350°F (175°C) and electrical resistance strain gages are used, make the necessary connections. Zero the strain/deformation-measuring devices.

6.1.2.3 Place the heating enclosure in position. Raise the temperature at a rate not exceeding 3.6°F (2°C)/min until it reaches the required temperature (Note 5). The test specimen shall be considered to have attained thermal equilibrium when the deformation transducer output is stable for at least three readings taken at equal intervals over a period of no less than 30 min. Stability is defined as a constant reading showing only the effects of normal instrument and heater unit fluctuations.

NOTE 5—It has been observed that for some rock types microfracturing will occur for heating rates above 1.8°F (1°C)/min. The operator is cautioned to select a heating rate such that microfracturing due to thermal gradients does not occur.

6.1.2.4 Rapidly raise the load without shock to the required test load within 20 s. Thereafter, the test load shall be held constant during the duration of the test.

6.2 Record the strain/deformation immediately after the required test load has been applied. Thereafter, record the strain/deformation at suitable time intervals. During the transient creep period (Fig. 4), strain/deformation readings shall be taken every few minutes to few hours until the strain/deformation rate becomes constant. Readings shall be taken at least twice daily during the steady-state phase of creep (Fig. 4) until the test is terminated. If the test extends into the tertiary creep period (Fig. 4), frequency of readings shall be increased appropriately.

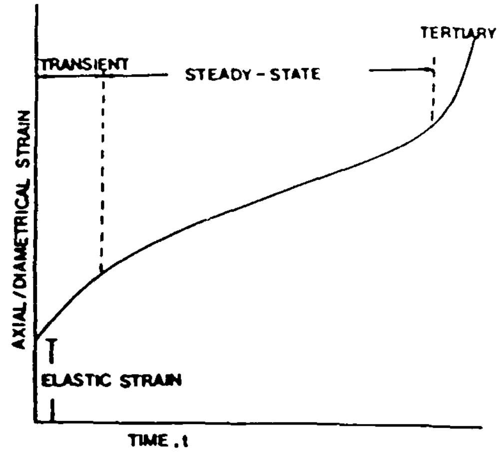


FIG. 4 Typical Creep Curve

6.3 Record the load and specimen temperature either continuously or each time the strain/deformation is read.

7. Calculation

7.1 The axial strain ϵ_a, circumferential strain, ϵ_c and diametrical strain, ϵ_d, may be obtained directly from strain-indicating equipment, or may be calculated from deformation readings, depending on the type of apparatus or instrumentation employed.

7.1.1 *Calculate the axial strain, ϵ_a, as follows:*

$$\epsilon_a = \Delta L / L$$

where:

L = original undeformed axial gage length, in. (mm), and
ΔL = change in measured axial length (negative for decrease in length) in. (mm).

7.1.2 Calculate the diametrical strain, ϵ_d, as follows:

$$\epsilon_d = \Delta D / D$$

where:

D = original undeformed diameter, in. (mm), and
ΔD = change in diameter (positive for an increase in diameter) in. (mm).

NOTE 6—Tensile stresses and strains are used as being positive herein. A consistent application of a compression-positive sign convention may be employed if desired.

7.2 Calculate the compressive stress in the test specimen from the compressive load on the specimen and the initial computed cross-sectional area as follows:

$$\sigma = -P/A$$

where:

σ = stress, psi (MPa),
P = load, lbf (N), and
A = area, in.^{2} (mm^2).

7.3 Plot the strain versus time curve as shown in Fig. 4. Frequently it may be necessary to use a logarithmic scale for the "time" if a long test period is involved.

8. Report

8.1 The report shall include the following:

8.1.1 Source of sample, including project name and location (often the location is specified in terms of the drill hole number and depth of specimen from the collar of the hole),

8.1.2 Lithologic description of the rock and load direction with respect to lithology,

8.1.3 Moisture condition of specimen before test,

8.1.4 Specimen diameter and height (conformance with dimensional requirements, see Note 4),

8.1.5 Stress level at which test was performed,

8.1.6 Temperature at which test was performed,

8.1.7 Tabulation of strain and time data,

8.1.8 Plot of the strain versus time curve (Fig. 4),

8.1.9 A description of physical appearance of specimen after test, and

8.1.10 If the actual equipment or procedure has varied from the requirements contained in this test method, each variation and the reasons for it shall be discussed.

9. Precision and Bias

9.1 *Precision*—Due to the nature of rock materials tested by this test method, it is, at this time, either not feasible or too costly to produce multiple specimens which have uniform physical properties. Therefore, since specimens which would yield the same test results cannot be tested, Subcommittee D18.12 cannot determine the variation between tests since any variation observed is just as likely to be due to specimen variation as to operator or laboratory testing variation. Subcommittee D18.12 welcomes proposals to resolve this problem that would allow for development of a valid precision statement.

9.2 *Bias*—There is no accepted reference value for this test method; therefore, bias cannot be determined.

10. Keywords

10.1 compression testing; creep; deformation; loading tests; rock

METHOD OF TEST FOR THERMAL DIFFUSIVITY OF ROCK

1. Scope

1.1 This method of test outlines a procedure for determining the thermal diffusivity of rock. The thermal diffusivity is equal to the thermal conductivity divided by the heat capacity per unit volume and may be used as an index of the facility with which the material will undergo temperature change.

2. Apparatus

2.1 The apparatus shall consist of:

2.1.1 Bath - A heating bath in which specimens can be raised to uniform high temperature (212°F (100°C)).

2.1.2 Diffusion Chamber - A diffusion chamber containing running cold water.

2.1.3 Temperature-Indicating or Recording Instrument - Consisting of iron-constantan thermocouples, Type K potentiometer, ice bath, standard cell, galvanometer, switch, and storage battery; or thermocouples and suitable recording potentiometer.

2.1.4 Timer - Timer capable of indicating minutes and seconds.

3. Procedure

3.1 Preparation of Specimen - The test specimen shall be a 6- by 12-in. (152- by 305-mm) core (for other shapes and sizes, see Section 5). A thermocouple shall be inserted in an axially drilled hole 3/8 in. (9.5 mm) in diameter and subsequently grouted.

3.2 Heating - Each specimen shall be heated to the same temperature by continuous immersion in boiling water until the temperature of the center is 212°F (100°C). The specimen shall then be transferred to a bath of running cold water and suspended in the bath so that the entire surface of the specimen is in contact with the water. The temperature of the cold water shall be determined by means of another thermocouple.

3.3 Cooling - The cooling history of the specimen shall be obtained from readings of the temperature of the interior of the specimen at 1-minute intervals from the time the temperature difference between the center and the water is 120°F (67°C) until the temperature difference between the center and water is 8°F (4°C). The data shall be recorded. Two such cooling histories shall be obtained for each test specimen, and the calculated diffusivities shall check within 40.002 ft^2/h ($0.0052 \cdot 10^{-5}\ m^2/s$).

4. Calculations

4.1 The temperature difference in degrees F shall be plotted against the time in minutes on a semilogarithmic scale. The best possible straight line shall then be drawn through the points so obtained. A typical graph is shown in Fig. 1. The time elapsed between the temperature differences of 80°F (44°C) and 20°F (11°C) shall be read from the graph, and this value inserted in the equation below, from which the thermal diffusivity, α , shall be calculated as follows:

$$\alpha = 0.812278/(t_1 - t_2)$$

where α = thermal diffusivity, ft^2/hr (Note 1)

$(t_1 - t_2)$ = elapsed time between temperature differences of 80°F (44°C) and 20°F (11°C), minutes

0.812278 = numerical factor applicable to 6- by 12-in. (152- by 305-mm) cylinder

NOTE 1--The SI equivalent of ft^2/hr is m^2/s; $ft^2/hr \cdot 2.580640\ E - 05 = m^2/s$.

5. Specimens of Other Sizes and Shapes

5.1 The method given above is directly applicable to a 6- by 12-in. (152- by 305-mm) cylinder. Specimens of other sizes and shapes may be treated in the manner described below.

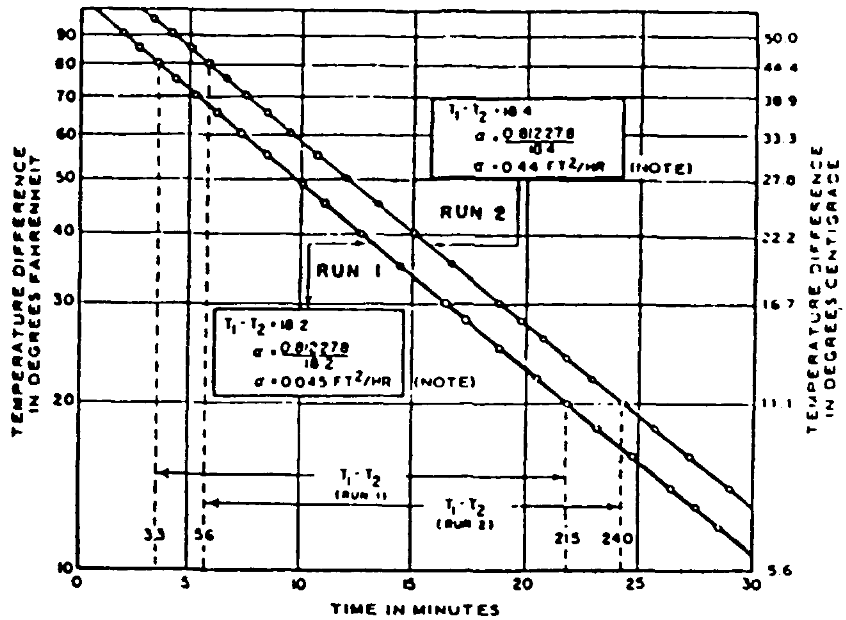


Fig. 1. Calculation of thermal diffusivity.

5.2 The thermal diffusivity of a specimen of regular shape is, to a first approximation,

$$\alpha = M/(t_2 - t_1)$$

where α = thermal diffusivity, ft^2/hr

M = a factor depending on the size and shape of the specimen

t_1, t_2 = times at which the center of the specimen reaches any specified temperature differences, minutes

5.3 For a prism,

$$M = \frac{60 \ln(T_1/T_2)}{\pi^2\left(\frac{1}{a^2} + \frac{1}{b^2} + \frac{1}{c^2}\right)}$$

where $\ln(T_1/T_2)$ = natural logarithm of the temperature difference ratio

T_1, T_2 = temperature differences at times t_1 and t_2, deg F

a, b, c = dimensions of prism, feet

5.4 For a cylinder,

$$M = \frac{60 \ln(T_1/T_2)}{\left(\frac{5.783}{r^2} + \frac{\pi^2}{l^2}\right)}$$

where $\ln(T_1/T_2)$ = natural logarithm, as above

r = radius of cylinder, feet

l = length of cylinder, feet

5.5 For specimens whose minimum dimension is more than 3 in. (76 mm), this approximate calculation will yield the required accuracy. For smaller specimens or when more precise determinations are desired, reference may be made to "Heat Conduction," by L. R. and A. C. Ingersoll, and O. J. Zebel, McGraw-Hill Book Company, Inc., 1948, pp. 183-185 and appended tables. Charts which may be used are also found in Williamson and Adams, Phys. Rev. XIV, p. 99 (1919) and "Heat Transmission," W. H. McAdams, McGraw-Hill Book Company, Inc., 1942, pp. 27-44.

Corps of Engineers, U.S. Army

ROCK TESTING HANDBOOK
(Standard and Recommended Methods)

August 1989

Geotechnical Laboratory
U.S. Army Engineer Waterways Experiment Station
P.O. Box 631, Vicksburg, MS 39181-0631

DEPARTMENT OF THE ARMY

WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
P O BOX 631
VICKSBURG, MISSISSIPPI 39180-0631

REPLY TO
ATTENTION OF

CEWES-GS-R (1110-2-1150a)

14 SEP 1989

MEMORANDUM FOR SEE DISTRIBUTION

SUBJECT: Transmittal of Rock Testing Handbook - Test Standards 1989

1. The subject handbook (encl 1) is a compilation of standards and recommended rock testing methods and has been prepared for use in laboratory and field offices of the CE. Preparation of the handbook was authorized and funded by the Office, Chief of Engineers.

2. The subject handbook supersedes the previous handbook entitled "Rock Testing Handbook (Standard and Recommended Methods), August 1980." The current Rock Testing Handbook includes nine new (RTH-89) test standards, seven revised ASTM standards, one revised ISRM standard, and small editorial changes to some of the previous standards. As before, the test standards are subject to updating and the issuance of new standards when the need arises.

3. Correspondence concerning the subject handbook should be addressed to: Commander and Director, U.S. Army Engineer Waterways Experiment Station, ATTN: CEWES-GS-R, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

Encl

LARRY B. FULTON
Colonel, Corps of Engineers
Commander and Director

DISTRIBUTION:

All Commanders, Divisions and Districts

PREFACE

This handbook is a compilation of standard and recommended rock testing methods and has been prepared for use in both the laboratory and the field.

Preparation of the handbook was authorized and funded by the Office, Chief of Engineers, U.S. Army. The cooperation of the American Society for Testing and Materials, the International Society for Rock Mechanics, and the U.S. Bureau of Reclamation in permitting the use of a number of their standards is appreciated.

Suggestions for revisions, corrections, and additions are welcomed. Correspondence concerning such matters should be addressed either to the Commander and Director, U.S. Army Engineer Waterways Experiment Station (ATTN: CEWES-GS-R), P.O. Box 631, Vicksburg, MS, 39181-0631; or to the Office, Chief of Engineers, U.S. Army (ATTN: DAEN-ECE, Engineering Division, Civil Works), Washington, DC, 20314-1000.

In order that this handbook may be of greatest service, it is intended that it be kept up to date by the issuance of supplementary items and the revision of existing items whenever necessary. To facilitate such revisions, the handbook has been issued in loose-leaf form.

The handbook has been prepared in the Geotechnical Laboratory (GL) and Structures Laboratory (SL) of the U.S. Army Engineer Waterways Experiment Station (WES). It was compiled by Mr. George H. Bragg, Jr. and Ms. M. Eileen Glynn, under the direct supervision of Messrs. Jerry S. Huie, Chief, Rock Mechanics Branch, GL, and Kenneth L. Saucier, Concrete Technology Division, SL; and the general guidance of Dr. Don C. Banks, Chief, Soil and Rock Mechanics Division, GL, and Mr. John Scanlon, Chief, Concrete Technology Division, SL.

At the time of initial publication of this handbook, Messrs. James P. Sale and Richard G. Ahlvin were Chief and Assistant Chief, respectively, of GL, and Mr. Bryant Mather was Chief of SL. Commander and Director of WES was COL Nelson P. Conover, CE. Technical Director was Mr. Fred R. Brown.

CONTENTS
Sep 1989

CONTENTS
Sep 1989

Corps of Engineers, U.S. Army

ROCK TESTING HANDBOOK
(Standard and Recommended Methods)

August 1989

Geotechnical Laboratory
U.S. Army Engineer Waterways Experiment Station
P.O. Box 631, Vicksburg, MS 39181-0631

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
P O BOX 631
VICKSBURG, MISSISSIPPI 39180-0631

REPLY TO
ATTENTION OF

CEWES-GS-R (1110-2-1150a)

1 : SEP 1989

MEMORANDUM FOR SEE DISTRIBUTION

SUBJECT: Transmittal of Rock Testing Handbook - Test Standards 1989

1. The subject handbook (encl 1) is a compilation of standards and recommended rock testing methods and has been prepared for use in laboratory and field offices of the CE. Preparation of the handbook was authorized and funded by the Office, Chief of Engineers.

2. The subject handbook supersedes the previous handbook entitled "Rock Testing Handbook (Standard and Recommended Methods), August 1980." The current Rock Testing Handbook includes nine new (RTH-89) test standards, seven revised ASTM standards, one revised ISRM standard, and small editorial changes to some of the previous standards. As before, the test standards are subject to updating and the issuance of new standards when the need arises.

3. Correspondence concerning the subject handbook should be addressed to: Commander and Director, U.S. Army Engineer Waterways Experiment Station, ATTN: CEWES-GS-R, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

Encl

LARRY B. FULTON
Colonel, Corps of Engineers
Commander and Director

DISTRIBUTION:

All Commanders, Divisions and Districts

PREFACE

This handbook is a compilation of standard and recommended rock testing methods and has been prepared for use in both the laboratory and the field.

Preparation of the handbook was authorized and funded by the Office, Chief of Engineers, U.S. Army. The cooperation of the American Society for Testing and Materials, the International Society for Rock Mechanics, and the U.S. Bureau of Reclamation in permitting the use of a number of their standards is appreciated.

Suggestions for revisions, corrections, and additions are welcomed. Correspondence concerning such matters should be addressed either to the Commander and Director, U.S. Army Engineer Waterways Experiment Station (ATTN: CEWES-GS-R), P.O. Box 631, Vicksburg, MS, 39181-0631; or to the Office, Chief of Engineers, U.S. Army (ATTN: DAEN-ECE, Engineering Division, Civil Works), Washington, DC, 20314-1000.

In order that this handbook may be of greatest service, it is intended that it be kept up to date by the issuance of supplementary items and the revision of existing items whenever necessary. To facilitate such revisions, the handbook has been issued in loose-leaf form.

The handbook has been prepared in the Geotechnical Laboratory (GL) and Structures Laboratory (SL) of the U.S. Army Engineer Waterways Experiment Station (WES). It was compiled by Mr. George H. Bragg, Jr. and Ms. M. Eileen Glynn, under the direct supervision of Messrs. Jerry S. Huie, Chief, Rock Mechanics Branch, GL, and Kenneth L. Saucier, Concrete Technology Division, SL; and the general guidance of Dr. Don C. Banks, Chief, Soil and Rock Mechanics Division, GL, and Mr. John Scanlon, Chief, Concrete Technology Division, SL.

At the time of initial publication of this handbook, Messrs. James P. Sale and Richard G. Ahlvin were Chief and Assistant Chief, respectively, of GL, and Mr. Bryant Mather was Chief of SL. Commander and Director of WES was COL Nelson P. Conover, CE. Technical Director was Mr. Fred R. Brown.

CONTENTS
Sep 1989

CONTENTS
Sep 1989

PART I. LABORATORY TEST METHODS

A. Characterization Methods

Designation: D 653 – 88

RTH 101-89

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa. 19103

Standard Terminology Relating to Soil, Rock, and Contained Fluids[1]

These definitions were prepared jointly by the American Society of Civil Engineers and the American Society for Testing and Materials.

INTRODUCTION

A number of the definitions include symbols and indicate the units of measurement. The symbols appear in italics immediately after the name of the term, followed by the unit in parentheses. No significance should be placed on the order in which the symbols are presented where two or more are given for an individual term. The applicable units are indicated by capital letters, as follows:

F—Force, such as pound-force, ton-force, newton

L—Length, such as inch, foot, centimetre

T—Time, such as second, minute

D—Dimensionless

Positive exponents designate multiples in the numerator. Negative exponents designate multiples in the denominator. Degrees of angle are indicated as "degrees."

Expressing the units either in SI or the inch-pound system has been purposely omitted in order to leave the choice of the system and specific unit to the engineer and the particular application, for example:

FL^{-2}—may be expressed in pounds-force per square inch, kilopascals, tons per square foot, etc.

LT^{-1}—may be expressed in feet per minute, centimetres per second, etc.

Where synonymous terms are cross-referenced, the definition is usually included with the earlier term alphabetically. Where this is not the case, the later term is the more significant.

Definitions marked with (ISRM) are taken directly from the publication in Ref 42 and are included for the convenience of the user.

For a list of ISRM symbols relating to soil and rock mechanics, refer to Appendix X1.

A list of references used in the preparation of these definitions appears at the end.

AASHTO compaction—see **compaction test.**

"A" Horizon—see **horizon.**

abrasion—a rubbing and wearing away. (ISRM)

abrasion—the mechanical wearing, grinding, scraping or rubbing away (or down) of rock surfaces by friction or impact, or both.

abrasive—any rock, mineral, or other substance that, owing to its superior hardness, toughness, consistency, or other properties, is suitable for grinding, cutting, polishing, scouring, or similar use.

abrasiveness—the property of a material to remove matter when scratching and grinding another material. (ISRM)

absorbed water—water held mechanically in a soil or rock mass and having physical properties not substantially different from ordinary water at the same temperature and pressure.

absorption—the assimilation of fluids into interstices.

absorption loss—that part of transmitted energy (mechanical) lost due to dissipation or conversion into other forms (heat, etc.).

accelerator—a material that increases the rate at which chemical reactions would otherwise occur.

activator—a material that causes a catalyst to begin its function.

active earth pressure—see **earth pressure.**

active state of plastic equilibrium—see **plastic equilibrium.**

additive—any material other than the basic components of a grout system.

adhesion—shearing resistance between soil and another material under zero externally applied pressure.

	Symbol	Unit
Unit Adhesion	c_a	FL^{-2}
Total Adhesion	C_a	F or FL^{-1}

adhesion—shearing resistance between two unlike materials under zero externally applied pressure.

admixture—a material other than water, aggregates, or cementitious material, used as a grout ingredient for cement-based grouts.

adsorbed water—water in a soil or rock mass attracted to the particle surfaces by physiochemical forces, having properties that may differ from those of pore water at the same temperature and pressure due to altered molecular ar-

[1] This terminology is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.93 on Terminology for Soil, Rock, and Contained Fluids.

Current edition approved June 29, 1988. Published August 1988. Originally published as D 653 – 42 T. Last previous edition D 653 – 87[ε1]

This extensive list of definitions represents the joint efforts of Subcommittee D18.93 on Terminology for Soil, Rock, and Contained Fluids of ASTM Committee D-18 on Soil and Rock, and the Committee on Definitions and Standards of the Geotechnical Engineering Division of the American Society of Civil Engineers. These two groups function together as the Joint ASCE/ASTM Committee on Nomenclature in Soil and Rock Mechanics. This list incorporates some terms from ASTM Definitions D 1707, Terms Relating to Soil Dynamics, which were discontinued in 1967.

rangement: adsorbed water does not include water that is chemically combined within the clay minerals

adsorption—the attachment of water molecules or ions to the surfaces of soil particles.

advancing slope grouting—a method of grouting by which the front of a mass of grout is caused to move horizontally by use of a suitable grout injection sequence.

aeolian deposits—wind-deposited material such as dune sands and loess deposits.

aggregate—*as a grouting material*, relatively inert granular mineral material, such as sand, gravel, slag, crushed stone, etc. "Fine aggregate" is material that will pass a No. 4 (6.4-mm) screen.

"Coarse aggregate" is material that will not pass a No. 4 (6.4-mm) screen. Aggregate is mixed with a cementing agent (such as Portland cement and water) to form a grout material.

agitator tank—a tank, usually vertical and with open top, with rotation paddles used to prevent segregation of grout after mixing.

air-space ratio, G_a (D)—ratio of: (*1*) volume of water that can be drained from a saturated soil or rock under the action of force of gravity, to (*2*) total volume of voids.

air-void ratio, G_v (D)—the ratio of: (*1*) the volume of air space, to (*2*) the total volume of voids in a soil or rock mass.

alkali aggregate reaction—a chemical reaction between Na_2O and K_2O in the cement and certain silicate minerals in the cement and certain silicate minerals in the aggregate, which causes expansion resulting in weakening and cracking of Portland cement grout. See **reactive aggregate**.

allowable bearing value (allowable soil pressure), q_a, p_a (FL^{-2})—the maximum pressure that can be permitted on foundation soil, giving consideration to all pertinent factors, with adequate safety against rupture of the soil mass or movement of the foundation of such magnitude that the structure is impaired.

allowable pile bearing load, Q_a, P_a (F)—the maximum load that can be permitted on a pile with adequate safety against movement of such magnitude that the structure is endangered.

alluvium—soil, the constituents of which have been transported in suspension by flowing water and subsequently deposited by sedimentation.

amplification factor—ratio of dynamic to static displacement.

amorphous peat—see **sapric peat**.

angle of external friction (angle of wall friction), δ (degrees)—angle between the abscissa and the tangent of the curve representing the relationship of shearing resistance to normal stress acting between soil and surface of another material.

angle of friction (angle of friction between solid bodies), ϕs (degrees)—angle whose tangent is the ratio between the maximum value of shear stress that resists slippage between two solid bodies at rest with respect to each other, and the normal stress across the contact surfaces.

angle of internal friction (angle of shear resistance), ϕ (degrees)—angle between the axis of normal stress and the tangent to the Mohr envelope at a point representing a given failure-stress condition for solid material.

angle of obliquity, α, β, ϕ, Ψ (degrees)—the angle between the direction of the resultant stress or force acting on a given plane and the normal to that plane.

angle of repose, α (degrees)—angle between the horizontal and the maximum slope that a soil assumes through natural processes. For dry granular soils the effect of the height of slope is negligible: for cohesive soils the effect of height of slope is so great that the angle of repose is meaningless.

angle of shear resistance—see **angle of internal friction**.

angle of wall friction—see **angle of external friction**.

angular aggregate—aggregate, the particles of which possess well-defined edges formed at the intersection of roughly planar faces.

anisotropic mass—a mass having different properties in different directions at any given point.

anisotropy—having different properties in different directions. (ISRM)

apparent cohesion—see **cohesion**.

aquiclude—a relatively impervious formation capable of absorbing water slowly but will not transmit it fast enough to furnish an appreciable supply for a well or spring.

aquifer—a water-bearing formation that provides a ground water reservoir.

aquitard—a confining bed that retards but does not prevent the flow of water to or from an adjacent aquifer: a leaky confining bed.

arching—the transfer of stress from a yielding part of a soil or rock mass to adjoining less-yielding or restrained parts of the mass.

area grouting—grouting a shallow zone in a particular area utilizing holes arranged in a pattern or grid.

Discussion—This type of grouting is sometimes referred to as blanket or consolidation grouting.

area of influence of a well, α (L^2)—area surrounding a well within which the piezometric surface has been lowered when pumping has produced the maximum steady rate of flow.

area ratio of a sampling spoon, sampler, or sampling tube, A_r (D)—the area ratio is an indication of the volume of soil displaced by the sampling spoon (tube), calculated as follows:

$$A_r = [(D_e^2 - D_i^2)/D_i^2] \times 100$$

where:

D_e = maximum external diameter of the sampling spoon, and

D_i = minimum internal diameter of the sampling spoon at the cutting edge.

armor—the artificial surfacing of bed, banks, shore, or embankment to resist erosion or scour.

armor stone—(generally one ton to three tons in weight) stone resulting from blasting, cutting, or by other methods to obtain rock heavy enough to require handling two individual pieces by mechanical means.

ash content—the percentage by dry weight of material remaining after an oven dry organic soil or peat is burned by a prescribed method.

attenuation—reduction of amplitude with time or distance.

"B" horizon—see **horizon**.

backpack grouting—the filling with grout of the annular space between a permanent tunnel lining and the surrounding formation.

Discussion—Same as crown grouting and backfill grouting.

back-packing—any material (usually granular) that is used to fill the empty space between the lagging and the rock surface. (ISRM)

baffle—a pier, weir, sill, fence, wall, or mound built on the bed of a stream to parry, deflect, check, or regulate the flow or to float on the surface to dampen the wave action.

base—*in grouting*, main component in a grout system.

base course (base)—a layer of specified or selected material of planned thickness constructed on the subgrade or subbase for the purpose of serving one or more functions such as distributing load, providing drainage, minimizing frost action, etc.

base exchange—the physicochemical process whereby one species of ions adsorbed on soil particles is replaced by another species.

batch—*in grouting*, quantity of grout mixed at one time.

batch method—*in grouting*, a quantity of grout materials are mixed or catalyzed at one time prior to injection.

batch mixer—*in grouting*, a machine that mixes batches of grout, in contrast to a continuous mixer.

bearing capacity—see **ultimate bearing capacity**.

bearing capacity (of a pile), Q_p, P_p (F)—the load per pile required to produce a condition of failure.

bedding—applies to rocks resulting from consolidation of sediments and exhibiting surfaces of separation (bedding planes) between layers of the same or different materials, that is, shale, siltstone, sandstone, limestone, etc. (ISRM)

bedding—collective term signifying the existence of layers of beds. Planes or other surfaces dividing sedimentary rocks of the same or different lithology.

bedrock—the more or less continuous body of rock which underlies the overburden soils. (ISRM)

bedrock (ledge)—rock of relatively great thickness and extent in its native location.

bench—(*1*) the unexcavated rock having a nearly horizontal surface which remains after a top heading has been excavated, or (*2*) step in a slope; formed by a horizontal surface and a surface inclined at a steeper angle than that of the entire slope. (ISRM)

bending—process of deformation normal to the axis of an elongated structural member when a moment is applied normal to its long axis. (ISRM)

bentonitic clay—a clay with a high content of the mineral montmorillonite, usually characterized by high swelling on wetting.

berm—a shelf that breaks the continuity of a slope.

biaxial compression—compression caused by the application of normal stresses in two perpendicular directions. (ISRM)

biaxial state of stress—state of stress in which one of the three principal stresses is zero. (ISRM)

binder (soil binder)—portion of soil passing No. 40 (425-μm) U.S. standard sieve.

binder—anything that causes cohesion in loosely assembled substances, such as clay or cement.

bit—any device that may be attached to or is an integral part of a drill string and is used as a cutting tool to bore into or penetrate rock or other materials.

blaine fineness—the fineness of powdered materials, such as cement and pozzolans, expressed as surface area usually in square centimetres per gram.

blanket grouting—a method in which relatively closely spaced shallow holes are drilled and grouted on a grid pattern over an area, for the purpose of making the upper portions of the bedrock stronger and less pervious.

blastibility—index value of the resistance of a rock formation to blasting. (ISRM)

blasting cap (detonator, initiator)—a small tube containing a flashing mixture for firing explosives. (ISRM)

bleeding—*in grouting*, the autogeneous flow of mixing water within, or its emergence from, newly placed grout caused by the settlement of the solid materials within the mass.

bleeding rate—*in grouting*, the rate at which water is released from grout by bleeding.

blocking—wood blocks placed between the excavated surface of a tunnel or shaft and the main bracing system. (ISRM)

body force—a force such as gravity whose effect is distributed throughout a material body by direct action on each elementary part of the body independent of the others. (ISRM)

bog—a peat covered area with a high water table and a surface dominated by a carpet of mosses, chiefly sphagnum. It is generally nutrient poor and acidic. It may be treed or treeless.

bond strength—*in grouting*, resistance to separation of set grout from other materials with which it is in contact; a collective expression for all forces such as adhesion, friction, and longitudinal shear.

bottom charge—concentrated explosive charge at the bottom of a blast hole. (ISRM)

boulder clay—a geological term used to designate glacial drift that has not been subjected to the sorting action of water and therefore contains particles from boulders to clay sizes.

boulders—a rock fragment, usually rounded by weathering or abrasion, with an average dimension of 12 in. (305 mm) or more.

breakwater stone—(generally three tons to twenty tons in weight) stone resulting from blasting, cutting, or other means to obtain rock heavy enough to require handling individual pieces by mechanical means.

buckling—a bulge, bend, bow, kink, or wavy condition produced in sheets, plates, columns, or beams by compressive stresses.

bulb of pressure—see **pressure bulb**.

bulkhead—a steep or vertical structure supporting natural or artificial embankment.

bulking—the increase in volume of a material due to manipulation. Rock bulks upon being excavated; damp sand bulks if loosely deposited, as by dumping, because the apparent cohesion prevents movement of the soil particles to form a reduced volume.

buoyant unit weight (submerged unit weight)—see **unit weight**.

burden—*in an explosive blasting*, the distance between the charge and the free face of the material to be blasted.

burden—distance between charge and free surface in direction of throw. (ISRM)

"C" Horizon—see **horizon.**

California bearing ratio, *CBR* (D)—the ratio of: (*1*) the force per unit area required to penetrate a soil mass with a 3-in.^{2} (19-cm)^2 circular piston (approximately 2-in. (51-mm) diameter) at the rate of 0.05 in. (1.3 mm)/min. to (*2*) that required for corresponding penetration of a standard material. The ratio is usually determined at 0.1-in. (2.5-mm) penetration, although other penetrations are sometimes used. Original California procedures required determination of the ratio at 0.1-in. intervals to 0.5 in. (12.7 mm). Corps of Engineers' procedures require determination of the ratio at 0.1 in. and 0.2 in. (5.1 mm). Where the ratio at 0.2 in. is consistently higher than at 0.1 in., the ratio at 0.2 in. is used.

camouflet—the underground cavity created by a fully contained explosive. (ISRM)

capillary action (capillarity)—the rise or movement of water in the interstices of a soil or rock due to capillary forces.

capillary flow—see **capillary action.**

capillary fringe zone—the zone above the free water elevation in which water is held by capillary action.

capillary head, h (L)—the potential, expressed in head of water, that causes the water to flow by capillary action.

capillary migration—see **capillary action.**

capillary rise (height of capillary rise), h_c (L)—the height above a free water elevation to which water will rise by capillary action.

capillary water—water subject to the influence of capillary action.

catalyst—a material that causes chemical reactions to begin.

catalyst system—those materials that, in combination, cause chemical reactions to begin; catalyst systems normally consist of an initiator (catalyst) and an activator.

cation—an ion that moves, or would move toward a cathode; thus nearly always synonymous with positive ion.

cation exchange—see **base exchange.**

cavity—a natural underground opening that may be small or large.

cavity—underground opening created by a fully contained explosive. (ISRM)

cement factor—quantity of cement contained in a unit volume of concrete or grout, expressed as weight, or volume (specify which).

cement grout—a grout in which the primary cementing agent is Portland cement.

cementitious factor—quantity of cement and other cementitious materials contained in a unit volume of concrete or grout, expressed as weight or volume (specify which).

centrifuge moisture equivalent—see **moisture equivalent.**

chamber—a large room excavated underground, for example, for a powerhouse, pump station, or for storage. (ISRM)

chamber blasting (coyotehole blasting)—a method of quarry blasting in which large explosive charges are confined in small tunnel chambers inside the quarry face. (ISRM)

chemical grout—any grouting material characterized by being a true solution; no particles in suspension. See also **particulate grout.**

chemical grout system—any mixture of materials used for grouting purposes in which all elements of the system are true solutions (no particles in suspension).

chip—crushed angular rock fragment of a size smaller than a few centimetres. (ISRM)

chisel—the steel cutting tool used in percussion drilling. (ISRM)

circuit grouting—a grouting method by which grout is circulated through a pipe extending to the bottom of the hole and back up the hole via the annular space outside the pipe. Then the excess grout is diverted back over a screen to the agitator tank by means of a packing gland at the top of the hole. The method is used where holes tend to cave and sloughing material might otherwise clog openings to be grouted.

clay (clay soil)—fine-grained soil or the fine-grained portion of soil that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength when air-dry. The term has been used to designate the percentage finer than 0.002 mm (0.005 mm in some cases), but it is strongly recommended that this usage be discontinued, since there is ample evidence from an engineering standpoint that the properties described in the above definition are many times more important.

clay size—that portion of the soil finer than 0.002 mm (0.005 mm in some cases) (see also **clay**).

clay soil—see **clay.**

cleavage—*in crystallography*, the splitting, or tendency to split, along planes determined by the crystal structure. *In petrology*, a tendency to cleave or split along definite, parallel, closely spaced planes. It is a secondary structure, commonly confined to bedded rocks.

cleavage—the tendency to cleave or split along definite parallel planes, which may be highly inclined to the bedding. It is a secondary structure and is ordinarily accompanied by at least some recrystallization of the rock. (ISRM)

cleavage planes—the parallel surfaces along which a rock or mineral cleaves or separates; the planes of least cohesion, usually parallel to a certain face of the mineral or crystal.

cleft water—water that exists in or circulates along the geological discontinuities in a rock mass.

closure—the opening is reduced in dimension to the extent that it cannot be used for its intended purpose. (ISRM)

closure—*in grouting*, closure refers to achieving the desired reduction in grout take by splitting the hole spacing. If closure is being achieved, there will be a progressive decrease in grout take as primary, secondary, tertiary, and quaternary holes are grouted.

cobble (cobblestone)—a rock fragment, usually rounded or semirounded, with an average dimension between 3 and 12 in. (75 and 305 mm).

coefficient of absolute viscosity—see **coefficient of viscosity.**

coefficient of active earth pressure—see **coefficient of earth pressure.**

coefficient of compressibility (coefficient of compression), a_v (L^2F^{-1})—the secant slope, for a given pressure increment,

of the pressure-void ratio curve. Where a stress-strain curve is used, the slope of this curve is equal to $a_v/(1 + e)$.

coefficient of consolidation, c_v (L^2T^{-1})—a coefficient utilized in the theory of consolidation, containing the physical constants of a soil affecting its rate of volume change.

$$c_v = k(1 + e)/a_v\gamma_w$$

where:

k = coefficient of permeability, LT^{-1},
e = void ratio, D,
a_v = coefficient of compressibility, L^2F^{-1}, and
γ_w = unit weight of water, FL^{-3}.

Discussion—In the literature published prior to 1935, the coefficient of consolidation, usually designated c, was defined by the equation:

$$c = k/a_v\gamma_w(1 + e)$$

This original definition of the coefficient of consolidation may be found in some more recent papers and care should be taken to avoid confusion.

coefficient of earth pressure, K (D)—the principal stress ratio at a point in a soil mass.

coefficient of earth pressure, active, K_A (D)—the minimum ratio of: (*1*) the minor principal stress, to (*2*) the major principal stress. This is applicable where the soil has yielded sufficiently to develop a lower limiting value of the minor principal stress.

coefficient of earth pressure, at rest, K_O (D)—the ratio of: (*1*) the minor principal stress, to (*2*) the major principal stress. This is applicable where the soil mass is in its natural state without having been permitted to yield or without having been compressed.

coefficient of earth pressure, passive, K_P (D)—the maximum ratio of: (*1*) the major principal stress, to (*2*) the minor principal stress. This is applicable where the soil has been compressed sufficiently to develop an upper limiting value of the major principal stress.

coefficient of friction (coefficient of friction between solid bodies), f (D)—the ratio between the maximum value of shear stress that resists slippage between two solid bodies with respect to each other, and the normal stress across the contact surfaces. The tangent of the angle of friction is ϕs.

coefficient of friction, f—a constant proportionality factor, μ, relating normal stress and the corresponding critical shear stress at which sliding starts between two surfaces: $T = \mu \cdot \sigma$. (ISRM)

coefficient of internal friction, μ (D)—the tangent of the angle of internal friction (angle of shear resistance) (see **internal friction**).

coefficient of permeability (permeability), k (LT^{-1})—the rate of discharge of water under laminar flow conditions through a unit cross-sectional area of a porous medium under a unit hydraulic gradient and standard temperature conditions (usually 20°C).

coefficient of shear resistance—see **coefficient of internal friction,** μ (D).

coefficient of subgrade reaction (modulus of subgrade reaction), k, k_s (FL^{-3})—ratio of: (*1*) load per unit area of horizontal surface of a mass of soil, to (*2*) corresponding settlement of the surface. It is determined as the slope of the secant, drawn between the point corresponding to zero settlement and the point of 0.05-in. (1.3-mm) settlement, of a load-settlement curve obtained from a plate load test on a soil using a 30-in. (762-mm) or greater diameter loading plate. It is used in the design of concrete pavements by the Westergaard method.

coefficient of transmissibility—the rate of flow of water in gallons per day through a vertical strip of the aquifer 1 ft (0.3 m) wide, under a unit hydraulic gradient.

coefficient of uniformity, C_u (D)—the ratio D_{60}/D_{10}, where D_{60} is the particle diameter corresponding to 60 % finer on the cumulative particle-size distribution curve, and D_{10} is the particle diameter corresponding to 10 % finer on the cumulative particle-size distribution curve.

coefficient of viscosity (coefficient of absolute viscosity), η (FTL^{-2})—the shearing force per unit area required to maintain a unit difference in velocity between two parallel layers of a fluid a unit distance apart.

coefficient of volume compressibility (modulus of volume change), m_v (L^2F^{-1})—the compression of a soil layer per unit of original thickness due to a given unit increase in pressure. It is numerically equal to the coefficient of compressibility divided by one plus the original void ratio, or $a_v/(1 + e)$.

cohesion—shear resistance at zero normal stress (an equivalent term in rock mechanics is intrinsic shear strength). (ISRM)

cohesion, c (FL^{-2})—the portion of the shear strength of a soil indicated by the term c, in Coulomb's equation, $s = c + p \tan \phi$. See **intrinsic shear strength.**

apparent cohesion—cohesion in granular soils due to capillary forces.

cohesionless soil—a soil that when unconfined has little or no strength when air-dried and that has little or no cohesion when submerged.

cohesive soil—a soil that when unconfined has considerable strength when air-dried and that has significant cohesion when submerged.

collar—*in grouting,* the surface opening of a borehole.

colloidal grout—*in grouting,* a grout in which the dispersed solid particles remain in suspension (colloids).

colloidal mixer—*in grouting,* a mixer designed to produce colloidal grout.

colloidal particles—particles that are so small that the surface activity has an appreciable influence on the properties of the aggregate.

communication—*in grouting,* subsurface movement of grout from an injection hole to another hole or opening.

compaction—the densification of a soil by means of mechanical manipulation.

compaction curve (Proctor curve) (moisture-density curve)—the curve showing the relationship between the dry unit weight (density) and the water content of a soil for a given compactive effort.

compaction test (moisture-density test)—a laboratory compacting procedure whereby a soil at a known water content is placed in a specified manner into a mold of given dimensions, subjected to a compactive effort of controlled magnitude, and the resulting unit weight determined. The procedure is repeated for various water contents sufficient to establish a relation between water content and unit weight.

compressibility—property of a soil or rock pertaining to its susceptibility to decrease in volume when subjected to load.

compression curve—see **pressure-void ratio curve**.

compression index, C_c (D)—the slope of the linear portion of the pressure-void ratio curve on a semi-log plot.

compression wave (irrotational)—wave in which element of medium changes volume without rotation.

compressive strength (unconfined or uniaxial compressive strength), p_c, q_u, C_o (FL^{-2})—the load per unit area at which an unconfined cylindrical specimen of soil or rock will fail in a simple compression test. Commonly the failure load is the maximum that the specimen can withstand in the test.

compressive stress—normal stress tending to shorten the body in the direction in which it acts. (ISRM)

concentration factor, n (D)—a parameter used in modifying the Boussinesq equations to describe various distributions of vertical stress.

conjugate joints (faults)—two sets of joints (faults) that formed under the same stress conditions (usually shear pairs). (ISRM)

consistency—the relative ease with which a soil can be deformed.

consistency—*in grouting*, the relative mobility or ability of freshly mixed mortar or grout to flow; the usual measurements are slump for stiff mixtures and flow for more fluid grouts.

consistency index—see **relative consistency**.

consolidated-drained test (slow test)—a soil test in which essentially complete consolidation under the confining pressure is followed by additional axial (or shearing) stress applied in such a manner that even a fully saturated soil of low permeability can adapt itself completely (fully consolidate) to the changes in stress due to the additional axial (or shearing) stress.

consolidated-undrained test (consolidated quick test)—a soil test in which essentially complete consolidation under the vertical load (in a direct shear test) or under the confining pressure (in a triaxial test) is followed by a shear at constant water content.

consolidation—the gradual reduction in volume of a soil mass resulting from an increase in compressive stress.

initial consolidation (initial compression)—a comparatively sudden reduction in volume of a soil mass under an applied load due principally to expulsion and compression of gas in the soil voids preceding primary consolidation.

primary consolidation (primary compression) (primary time effect)—the reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to a squeezing out of water from the void spaces of the mass and accompanied by a transfer of the load from the soil water to the soil solids.

secondary consolidation (secondary compression) (secondary time effect)—the reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to the adjustment of the internal structure of the soil mass after most of the load has been transferred from the soil water to the soil solids.

consolidation curve—see **consolidation time curve**.

consolidation grouting—injection of a fluid grout, usually sand and Portland cement, into a compressible soil mass in order to displace it and form a lenticular grout structure for support.

Discussion—In rock, grouting is performed for the purpose of strengthening the rock mass by filling open fractures and thus eliminating a source of settlement.

consolidation ratio, U_z (D)—the ratio of: (*1*) the amount of consolidation at a given distance from a drainage surface and at a given time, to (*2*) the total amount of consolidation obtainable at that point under a given stress increment.

consolidation test—a test in which the specimen is laterally confined in a ring and is compressed between porous plates.

consolidation-time curve (time curve) (consolidation curve) (theoretical time curve)—a curve that shows the relation between: (*1*) the degree of consolidation, and (*2*) the elapsed time after the application of a given increment of load.

constitutive equation—force deformation function for a particular material. (ISRM)

contact grouting—see **backpack grouting**.

contact pressure, p (FL^{-2})—the unit of pressure that acts at the surface of contact between a structure and the underlying soil or rock mass.

continuous mixer—a mixer into which the ingredients of the mixture are fed without stopping, and from which the mixed product is discharged in a continuous stream.

contraction—linear strain associated with a decrease in length. (ISRM)

controlled blasting—includes all forms of blasting designed to preserve the integrity of the remaining rocks, that is, smooth blasting or pre-splitting. (ISRM)

controlled-strain test—a test in which the load is so applied that a controlled rate of strain results.

controlled-stress test—a test in which the stress to which a specimen is subjected is applied at a controlled rate.

convergence—generally refers to a shortening of the distance between the floor and roof of an opening, for example, in the bedded sedimentary rocks of the coal measures where the roof sags and the floor heaves. Can also apply to the convergence of the walls toward each other. (ISRM)

core—a cylindrical sample of hardened grout, concrete, rock, or grouted deposits, usually obtained by means of a core drill.

core drilling; diamond drilling—a rotary drilling technique, using diamonds in the cutting bit, that cuts out cylindrical rock samples. (ISRM)

core recovery—ratio of the length of core recovered to the length of hole drilled, usually expressed as a percentage.

cover—the perpendicular distance from any point in the roof of an underground opening to the ground surface. (ISRM)

cover—*in grouting*, the thickness of rock and soil material overlying the stage of the hole being grouted.

crack—a small fracture, that is, small with respect to the scale of the feature in which it occurs. (ISRM)

crater—excavation (generally of conical shape) generated by an explosive charge. (ISRM)

creep—slow movement of rock debris or soil usually imperceptible except to observations of long duration. Time-dependent strain or deformation, for example, continuing strain with sustained stress.

critical circle (critical surface)—the sliding surface assumed in a theoretical analysis of a soil mass for which the factor of safety is a minimum.

critical damping—the minimum viscous damping that will allow a displaced system to return to its initial position without oscillation.

critical density—the unit weight of a saturated granular material below which it will lose strength and above which it will gain strength when subjected to rapid deformation. The critical density of a given material is dependent on many factors.

critical frequency, f_c—frequency at which maximum or minimum amplitudes of excited waves occur.

critical height, H_c (L)—the maximum height at which a vertical or sloped bank of soil or rock will stand unsupported under a given set of conditions.

critical hydraulic gradient—see **hydraulic gradient**.

critical slope—the maximum angle with the horizontal at which a sloped bank of soil or rock of given height will stand unsupported.

critical surface—see **critical circle**.

critical void ratio—see **void ratio**.

crown—also roof or back, that is, the highest point of the cross section. *In tunnel linings*, the term is used to designate either the arched roof above spring lines or all of the lining except the floor or invert. (ISRM)

cryology—the study of the properties of snow, ice, and frozen ground.

cure—*in grouting*, the change in properties of a grout with time.

cure time—*in grouting*, the interval between combining all grout ingredients or the formation of a gel and substantial development of its potential properties.

curtain grouting—injection of grout into a sub-surface formation in such a way as to create a barrier of grouted material transverse to the direction of the anticipated water flow.

cuttings—small-sized rock fragments produced by a rock drill. (ISRM)

damping—reduction in the amplitude of vibration of a body or system due to dissipation of energy internally or by radiation. (ISRM)

damping ratio—*for a system with viscous damping*, the ratio of actual damping coefficient to the critical damping coefficient.

decay time—the interval of time required for a pulse to decay from its maximum value to some specified fraction of that value. (ISRM)

decomposition—for peats and organic soils, see **humification**.

decoupling—the ratio of the radius of the blasthole to the radius of the charge. In general, a reducing of the strain wave amplitude by increasing the spacing between charge and blasthole wall. (ISRM)

deflocculating agent (deflocculant) (dispersing agent)—an agent that prevents fine soil particles in suspension from coalescing to form flocs.

deformability—*in grouting*, a measure of the elasticity of the grout to distort in the interstitial spaces as the sediments move.

deformation—change in shape or size.

deformation—a change in the shape or size of a solid body. (ISRM)

deformation resolution (deformation sensitivity), R_d (L)—ratio of the smallest subdivision of the indicating scale of a deformation-measuring device to the sensitivity of the device.

degree-days—the difference between the average temperature each day and 32°F (0°C). In common usage degree-days are positive for daily average temperatures above 32°F and negative for those below 32°F (see **freezing index**).

degree of consolidation (percent consolidation), U (D)—the ratio, expressed as a percentage, of: (*1*) the amount of consolidation at a given time within a soil mass, to (*2*) the total amount of consolidation obtainable under a given stress condition.

degrees-of-freedom—the minimum number of independent coordinates required in a mechanical system to define completely the positions of all parts of the system at any instant of time. In general, it is equal to the number of independent displacements that are possible.

degree of saturation—see **percent saturation**.

degree of saturation—the extent or degree to which the voids in rock contain fluid (water, gas, or oil). Usually expressed in percent related to total void or pore space. (ISRM)

degree of sensitivity (*sensitivity ratio*)—see **remolding index**.

delay—time interval (fraction of a second) between detonation of explosive charges. (ISRM)

density—the mass per unit, ρ (ML^{-3}) kg/m^3.

density of dry soil or rock, ρ_d (ML^{-3}) kg/m^3—the mass of solid particles per the total volume of soil or rock.

density of saturated soil or rock, ρ_{sat} (ML^{-3}) kg/m^3—the total mass per total volume of completely saturated soil or rock.

density of soil or rock (*bulk density*), ρ (ML^{-3}) kg/m^3—the total mass (solids plus water) per total volume.

density of solid particles, ρ_s (ML^{-3}) kg/m^3—the mass per volume of solid particles.

density of submerged soil or rock, ρ_{sub} (ML^{-3}) kg/m^3—the difference between the density of saturated soil or rock, and the density of water.

density of water, ρ_w (ML^{-3}) kg/m^3—the mass per volume of water.

detonation—an extremely rapid and violent chemical reaction causing the production of a large volume of gas. (ISRM)

deviator stress, $\Delta\sigma$ (FL^{-2})—the difference between the major and minor principal stresses in a triaxial test.

deviator of stress (strain)—the stress (strain) tensor obtained by subtracting the mean of the normal stress (strain) components of a stress (strain) tensor from each normal stress (strain) component. (ISRM)

differential settlement—settlement that varies in rate or amount, or both, from place to place across a structure.

dilatancy—property of volume increase under loading. (ISRM)

dilatancy—the expansion of cohesionless soils when subject to shearing deformation.

direct shear test—a shear test in which soil or rock under an applied normal load is stressed to failure by moving one section of the sample or sample container (shear box) relative to the other section.

discharge velocity, v, q (LT^{-1})—rate of discharge of water through a porous medium per unit of total area perpendicular to the direction of flow.

discontinuity surface—any surface across which some property of a rock mass is discontinuous. This includes fracture surfaces, weakness planes, and bedding planes, but the term should not be restricted only to mechanical continuity. (ISRM)

dispersing agent—*in grouting*, an addition or admixture that promotes dispersion of particulate grout ingredients by reduction of interparticle attraction.

dispersing agent—see **deflocculating agent**.

dispersion—the phenomenon of varying speed of transmission of waves, depending on their frequency. (ISRM)

displacement—a change in position of a material point. (ISRM)

displacement grouting—injection of grout into a formation in such a manner as to move the formation; it may be controlled or uncontrolled. See also **penetration grouting**.

distortion—a change in shape of a solid body. (ISRM)

divergence loss—that part of transmitted energy lost due to spreading of wave rays in accordance with the geometry of the system.

double amplitude—total or peak to peak excursion.

drag bit—a noncoring or full-hole boring bit, which scrapes its way through relatively soft strata. (ISRM)

drain—a means for intercepting, conveying, and removing water.

drainage curtain—*in grouting*, a row of open holes drilled parallel to and downstream from the grout curtain of a dam for the purpose of reducing uplift pressures.

Discussion—Depth is ordinarily approximately one-third to one-half that of the grout curtain.

drainage gallery—*in grouting*, an opening or passageway from which grout holes or drainage curtain holes, or both, are drilled. See also **grout gallery**.

drawdown (L)—vertical distance the free water elevation is lowered or the pressure head is reduced due to the removal of free water.

drill—a machine or piece of equipment designed to penetrate earth or rock formations, or both.

drillability—index value of the resistance of a rock to drilling. (ISRM)

drill carriage; jumbo—a movable platform, stage, or frame that incorporates several rock drills and usually travels on the tunnel track; used for heavy drilling work in large tunnels. (ISRM)

drilling pattern—the number, position, depth, and angle of the blastholes forming the complete round in the face of a tunnel or sinking pit. (ISRM)

drill mud—*in grouting*, a dense fluid or slurry used in rotary drilling; to prevent caving of the bore hole walls, as a circulation medium to carry cuttings away from the bit and out of the hole, and to seal fractures or permeable formations, or both, preventing loss of circulation fluid.

Discussion—The most common drill mud is a water-bentonite mixture, however, many other materials may be added or substituted to increase density or decrease viscosity.

dry pack—a cement-sand mix with minimal water content used to fill small openings or repair imperfections in concrete.

dry unit weight (*dry density*)—see **unit weight**.

ductility—condition in which material can sustain permanent deformation without losing its ability to resist load. (ISRM)

dye tracer—*in grouting*, an additive whose primary purpose is to change the color of the grout or water.

earth—see **soil**.

earth pressure—the pressure or force exerted by soil on any boundary.

	Symbol	Unit
Pressure	p	FL^{-2}
Force	P	F or FL^{-1}

active earth pressure, P_A, p_A—the minimum value of earth pressure. This condition exists when a soil mass is permitted to yield sufficiently to cause its internal shearing resistance along a potential failure surface to be completely mobilized.

earth pressure at rest, P_o, p_o—the value of the earth pressure when the soil mass is in its natural state without having been permitted to yield or without having been compressed.

passive earth pressure, P_p, p_p—the maximum value of earth pressure. This condition exists when a soil mass is compressed sufficiently to cause its internal shearing resistance along a potential failure surface to be completely mobilized.

effect diameter (effective size), D_{10}, D_e (L)—particle diameter corresponding to 10 % finer on the grain-size curve.

effective drainage porosity—see **effective porosity**.

effective force, $\bar{F}$ (F)—the force transmitted through a soil or rock mass by intergranular pressures.

effective porosity (effective drainage porosity), n_e (D)—the ratio of: (*1*) the volume of the voids of a soil or rock mass that can be drained by gravity, to (*2*) the total volume of the mass.

effective pressure—see **stress**.

effective size—see **effective diameter**.

effective stress—see **stress**.

effective unit weight—see **unit weight**.

efflux time—time required for all grout to flow from a flow cone.

elasticity—property of material that returns to its original form or condition after the applied force is removed. (ISRM)

elastic limit—point on stress strain curve at which transition from elastic to inelastic behavior takes place. (ISRM)

elastic state of equilibrium—state of stress within a soil mass when the internal resistance of the mass is not fully mobilized.

elastic strain energy—potential energy stored in a strained solid and equal to the work done in deforming the solid from its unstrained state less any energy dissipated by inelastic deformation. (ISRM)

electric log—a record or log of a borehole obtained by lowering electrodes into the hole and measuring any of the various electrical properties of the rock formations or materials traversed.

electrokinetics—involves the application of an electric field to soil for the purpose of dewatering materials of very low permeability to enhance stability. The electric field produces negative pore pressures near a grout pipe that facilitates grout injection.

emulsifier—a substance that modifies the surface tension of colloidal droplets, keeping them from coalescing, and keeping them suspended.

emulsion—a system containing dispersed colloidal droplets.

endothermic—pertaining to a reaction that occurs with the adsorption of heat.

envelope grouting—grouting of rock surrounding a hydraulic pressure tunnel for purpose of consolidation, and primarily, reduction of permeability.

epoxy—a multicomponent resin grout that usually provides very high, tensile, compressive, and bond strengths.

equipotential line—see **piezometric line**.

equivalent diameter (equivalent size), *D* (L)—the diameter of a hypothetical sphere composed of material having the same specific gravity as that of the actual soil particle and of such size that it will settle in a given liquid at the same terminal velocity as the actual soil particle.

equivalent fluid—a hypothetical fluid having a unit weight such that it will produce a pressure against a lateral support presumed to be equivalent to that produced by the actual soil. This simplified approach is valid only when deformation conditions are such that the pressure increases linearly with depth and the wall friction is neglected.

excess hydrostatic pressure—see **hydrostatic pressure**.

exchange capacity—the capacity to exchange ions as measured by the quantity of exchangeable ions in a soil or rock.

excitation (stimulus)—an external force (or other input) applied to a system that causes the system to respond in some way.

exothermic—pertaining to a reaction that occurs with the evolution of heat.

expansive cement—a cement that tends to increase in volume after it is mixed with water.

extender—an additive whose primary purpose is to increase total grout volume.

extension—linear strain associated with an increase in length. (ISRM)

external force—a force that acts across external surface elements of a material body. (ISRM)

extrados—the exterior curved surface of an arch, as opposed to intrados, which is the interior curved surface of an arch. (ISRM)

fabric—*for rock or soil*, the spatial configuration of all textural and structural features as manifested by every recognizable material unit from crystal lattices to large scale features requiring field studies.

fabric—the orientation in space of the elements composing the rock substance. (ISRM)

face (heading)—the advanced end of a tunnel, drift, or excavation at which work is progressing. (ISRM)

facing—the outer layer of revetment.

failure (in rocks)—exceeding the maximum strength of the rock or exceeding the stress or strain requirement of a specific design. (ISRM)

failure by rupture—see **shear failure**.

failure criterion—specification of the mechanical condition under which solid materials fail by fracturing or by deforming beyond some specified limit. This specification may be in terms of the stresses, strains, rate-of-change of stresses, rate-of-change of strains, or some combination of these quantities, in the materials.

failure criterion—theoretically or empirically derived stress or strain relationship characterizing the occurrence of failure in the rock. (ISRM)

false set—*in grouting*, the rapid development of rigidity in a freshly mixed grout without the evolution of much heat.

Discussion—Such rigidity can be dispelled and plasticity regained by further mixing without the addition of water; premature stiffening, hesitation set, early stiffening, and rubber set are other much used terms referring to the same phenomenon.

fatigue—the process of progressive localized permanent structural change occurring in a material subjected to conditions that produce fluctuating stresses and strains at some point or points and that may culminate in cracks or complete fracture after a sufficient number of fluctuations.

fatigue—decrease of strength by repetitive loading. (ISRM)

fatigue limit—point on stress-strain curve below which no fatigue can be obtained regardless of number of loading cycles. (ISRM)

fault—a fracture or fracture zone along which there has been displacement of the two sides relative to one another parallel to the fracture (this displacement may be a few centimetres or many kilometres). (See also **joint fault set** and **joint fault system**. (ISRM)

fault breccia—the assemblage of broken rock fragments frequently found along faults. The fragments may vary in size from inches to feet. (ISRM)

fault gouge—a clay-like material occurring between the walls of a fault as a result of the movement along the fault surfaces. (ISRM)

fiber—*for peats and organic soils*, a fragment or piece of plant tissue that retains a recognizable cellular structure and is large enough to be retained after wet sieving on a 100-mesh sieve (openings 0.15 mm).

fibric peat—peat in which the original plant fibers are slightly decomposed (greater than 67 % fibers).

fibrous peat—see **fibric peat**.

field moisture equivalent—see **moisture equivalent**.

fill—man-made deposits of natural soils or rock products and waste materials.

filling—generally, the material occupying the space between joint surfaces, faults, and other rock discontinuities. The filling material may be clay, gouge, various natural cementing agents, or alteration products of the adjacent rock. (ISRM)

filter bedding stone—(generally 6-in. minus material) stone placed under graded riprap stone or armor stone in a layer or combination of layers designed and installed in such a manner as to prevent the loss of underlying soil or finer bedding materials due to moving water.

filter (protective filter)—a layer or combination of layers of pervious materials designed and installed in such a manner as to provide drainage, yet prevent the movement of soil particles due to flowing water.

final set—*in grouting*, a degree of stiffening of a grout mixture greater than initial set, generally stated as an empirical value indicating the time in hours and minutes that is required for cement paste to stiffen sufficiently to resist the penetration of a weighted test needle.

fineness—a measure of particle-size.

fineness modulus—an empirical factor obtained by adding the total percentages of an aggregate sample retained on each of a specified series of sieves, and dividing the sum by 100; in the United States, the U.S. Standard sieve sizes are: No. 100 (149 μm), No. 50 (297 μm), No. 30 (590 μm), No. 16 (1,190 μm), No. 8 (2,380 μm), and No. 4 (4,760 μm) and ⅜ in. (9.5 mm), ¾ in. (19 mm), 1½ in. (38 mm), 3 in. (76 mm), and 6 in. (150 mm).

fines—portion of a soil finer than a No. 200 (75-μm) U.S. standard sieve.

finite element—one of the regular geometrical shapes into which a figure is subdivided for the purpose of numerical stress analysis. (ISRM)

fishing tool—*in grouting*, a device used to retrieve drilling equipment lost or dropped in the hole.

fissure—a gapped fracture. (ISRM)

flash set—*in grouting*, the rapid development of rigidity in a freshly mixed grout, usually with the evolution of considerable heat; this rigidity cannot be dispelled nor can the plasticity be regained by further mixing without addition of water; also referred to as quick set or grab set.

floc—loose, open-structured mass formed in a suspension by the aggregation of minute particles.

flocculation—the process of forming flocs.

flocculent structure—see **soil structure.**

floor—bottom of near horizontal surface of an excavation, approximately parallel and opposite to the roof. (ISRM)

flow channel—the portion of a flow net bounded by two adjacent flow lines.

flow cone—*in grouting*, a device for measurement of grout consistency in which a predetermined volume of grout is permitted to escape through a precisely sized orifice, the time of efflux (flow factor) being used as the indication of consistency.

flow curve—the locus of points obtained from a standard liquid limit test and plotted on a graph representing water content as ordinate on an arithmetic scale and the number of blows as abscissa on a logarithmic scale.

flow failure—failure in which a soil mass moves over relatively long distances in a fluid-like manner.

flow index, F_w, I_f (D)—the slope of the flow curve obtained from a liquid limit test, expressed as the difference in water contents at 10 blows and at 100 blows.

flow line—the path that a particle of water follows in its course of seepage under laminar flow conditions.

flow net—a graphical representation of flow lines and equipotential (piezometric) lines used in the study of seepage phenomena.

flow slide—the failure of a sloped bank of soil in which the movement of the soil mass does not take place along a well-defined surface of sliding.

flow value, N_ϕ (degrees)—a quantity equal to tan [45 deg + (ϕ/2)].

fluidifier—*in grouting*, an admixture employed in grout to increase flowability without changing water content.

fly ash—the finely divided residue resulting from the combustion of ground or powdered coal and which is transported from the firebox through the boiler by flue gases.

fold—a bend in the strata or other planar structure within the rock mass. (ISRM)

foliation—the somewhat laminated structure resulting from segregation of different minerals into layers parallel to the schistosity. (ISRM)

footing—portion of the foundation of a structure that transmits loads directly to the soil.

footwall—the mass of rock beneath a discontinuity surface. (ISRM)

forced vibration (forced oscillation)—vibration that occurs if the response is imposed by the excitation. If the excitation is periodic and continuing, the oscillation is steady-state.

forepoling—driving forepoles (pointed boards or steel rods) ahead of the excavation, usually over the last set erected, to furnish temporary overhead protection while installing the next set. (ISRM)

foundation—lower part of a structure that transmits the load to the soil or rock.

foundation soil—upper part of the earth mass carrying the load of the structure.

fracture—the general term for any mechanical discontinuity in the rock; it therefore is the collective term for joints, faults, cracks, etc. (ISRM)

fracture—a break in the mechanical continuity of a body of rock caused by stress exceeding the strength of the rock. Includes joints and faults.

fracture frequency—the number of natural discontinuities in a rock or soil mass per unit length, measured along a core or as exposed in a planar section such as the wall of a tunnel.

fracture pattern—spatial arrangement of a group of fracture surfaces. (ISRM)

fracturing—*in grouting*, intrusion of grout fingers, sheets, and lenses along joints, planes of weakness, or between the strata of a formation at sufficient pressure to cause the strata to move away from the grout.

fragmentation—the breaking of rock in such a way that the bulk of the material is of a convenient size for handling. (ISRM)

free water (gravitational water) (ground water) (phreatic water)—water that is free to move through a soil or rock mass under the influence of gravity.

free water elevation (water table) (ground water surface) (free water surface) (ground water elevation)—elevations at which the pressure in the water is zero with respect to the atmospheric pressure.

freezing index, F (degree-days)—the number of degree-days between the highest and lowest points on the cumulative degree-days—time curve for one freezing season. It is used as a measure of the combined duration and magnitude of below-freezing temperature occurring during any given freezing season. The index determined for air temperatures at 4.5 ft (1.4 m) above the ground is commonly

designated as the air freezing index, while that determined for temperatures immediately below a surface is known as the surface freezing index.

free vibration—vibration that occurs in the absence of forced vibration.

frequency, f (T^{-1})—number of cycles occurring in unit time.

frost action—freezing and thawing of moisture in materials and the resultant effects on these materials and on structures of which they are a part or with which they are in contact.

frost boil—(*a*) softening of soil occurring during a thawing period due to the liberation of water form ice lenses or layers.

(*b*) the hole formed in flexible pavements by the extrusion of soft soil and melt waters under the action of wheel loads.

(*c*) breaking of a highway or airfield pavement under traffic and the ejection of subgrade soil in a soft and soupy condition caused by the melting of ice lenses formed by frost action.

frost heave—the raising of a surface due to the accumulation of ice in the underlying soil or rock.

fundamental frequency—lowest frequency of periodic variation.

gage length, L (L)—distance over which the deformation measurement is made.

gage protector—*in grouting*, a device used to transfer grout pressure to a gage without the grout coming in actual contact with the gage.

gage saver—see **gage protector.**

gel—*in grouting*, the condition where a liquid grout begins to exhibit measurable shear strength.

gel time—*in grouting*, the measured time interval between the mixing of a grout system and the formation of a gel.

general shear failure—see **shear failure.**

glacial till (till)—material deposited by glaciation, usually composed of a wide range of particle sizes, which has not been subjected to the sorting action of water.

gradation (grain-size distribution) (texture)—the proportions by mass of a soil or fragmented rock distributed in specified particle-size ranges.

grain-size analysis (mechanical analysis) (particle-size analysis)—the process of determining grain-size distribution.

gravel—rounded or semirounded particles of rock that will pass a 3-in. (76.2-mm) and be retained on a No. 4 (4.75-μm) U.S. standard sieve.

gravitational water—see **free water.**

gravity grouting—grouting under no applied pressure other than the height of fluid in the hole.

groin—bank or shore-protection structure in the form of a barrier placed oblique to the primary motion of water, designed to control movement of bed load.

ground arch—the theoretical stable rock arch that develops some distance back from the surface of the opening and supports the opening. (ISRM)

ground water—that part of the subsurface water that is in the saturated zone.

Discussion—Loosely, all subsurface water as distinct from surface water.

ground-water barrier—soil, rock, or artificial material which has a relatively low permeability and which occurs below the land surface where it impedes the movement of ground water and consequently causes a pronounced difference in the potentiometric level on opposite sides of the barrier.

ground-water basin—a ground-water system that has defined boundaries and may include more than one aquifer of permeable materials, which are capable of furnishing a significant water supply.

Discussion—A basin is normally considered to include the surface area and the permeable materials beneath it. The surface-water divide need not coincide with ground-water divide.

ground-water discharge—the water released from the zone of saturation; also the volume of water released.

ground-water divide—a ridge in the water table or other potentiometric surface from which ground water moves away in both directions normal to the ridge line.

ground-water elevation—see **free water elevation.**

ground-water flow—the movement of water in the zone of saturation.

ground-water level—the level below which the rock and subsoil, to unknown depths, are saturated. (ISRM)

ground-water, perched—see **perched ground-water.**

ground-water recharge—the process of water addition to the saturated zone; also the volume of water added by this process.

ground-water surface—see **free water elevation.**

grout—*in soil and rock grouting*, a material injected into a soil or rock formation to change the physical characteristics of the formation.

groutability—the ability of a formation to accept grout.

groutability ratio of granular formations—the ratio of the 15 % size of the formation particles to be grouted to the 85 % size of grout particles (suspension-type grout). This ratio should be greater than 24 if the grout is to successfully penetrate the formation.

groutable rock bolts—rock bolts with hollow cores or with tubes adapted to the periphery of the bolts and extending to the bottom of the bolts to facilitate filling the holes surrounding the bolts with grout.

grouted-aggregate concrete—concrete that is formed by injecting grout into previously placed coarse aggregate. See also **preplaced aggregate concrete.**

grout cap—a "cap" that is formed by placing concrete along the top of a grout curtain. A grout cap is often used in weak foundation rock to secure grout nipples, control leakage, and to form an impermeable barrier at the top of a grout curtain.

grout gallery—an opening or passageway within a dam utilized for grouting or drainage operations, or both.

grout header—a pipe assembly attached to a ground hole, and to which the grout lines are attached for injecting grout. Grout injector is monitored and controlled by means of valves and a pressure gate mounted on the header; sometimes called grout manifold.

grout mix—the proportions or amounts of the various materials used in the grout, expressed by weight or volume. (The words "by volume" or "by weight" should be used to specify the mix.)

grout nipple—*in grouting*, a short length of pipe, installed at the collar of the grout hole, through which drilling is done and to which the grout header is attached for the purpose of injecting grout.

grout slope—the natural slope of grout injected into preplaced-aggregate or other porous mass.

grout system—formulation of different materials used to form a grout.

grout take—the measured quantity of grout injected into a unit volume of formation, or a unit length of grout hole.

hanging wall—the mass of rock above a discontinuity surface. (ISRM)

hardener—*in grouting*, in a two component epoxy or resin, the chemical component that causes the base component to cure.

hardness—resistance of a material to indentation or scratching. (ISRM)

hardpan—a hard impervious layer, composed chiefly of clay, cemented by relatively insoluble materials, that does not become plastic when mixed with water and definitely limits the downward movement of water and roots.

head—pressure at a point in a liquid, expressed in terms of the vertical distance of the point below the surface of the liquid. (ISRM)

heat of hydration—heat evolved by chemical reactions with water, such as that evolved during the setting and hardening of Portland cement.

heave—upward movement of soil caused by expansion or displacement resulting from phenomena such as: moisture absorption, removal of overburden, driving of piles, frost action, and loading of an adjacent area.

height of capillary rise—see **capillary rise.**

hemic peat—peat in which the original plant fibers are moderately decomposed (between 33 and 67 % fibers).

heterogeneity—having different properties at different points. (ISRM)

homogeneity—having different properties at different points. (ISRM)

homogeneity—having the same properties at all points. (ISRM)

homogeneous mass—a mass that exhibits essentially the same physical properties at every point throughout the mass.

honeycomb structure—see **soil structure.**

horizon (soil horizon)—one of the layers of the soil profile, distinguished principally by its texture, color, structure, and chemical content.

"A" horizon—the uppermost layer of a soil profile from which inorganic colloids and other soluble materials have been leached. Usually contains remnants of organic life.

"B" horizon—the layer of a soil profile in which material leached from the overlying "A" horizon is accumulated.

"C" horizon—undisturbed parent material from which the overlying soil profile has been developed.

humic peat—see **sapric peat.**

humification—a process by which organic matter decomposes.

Discussion—The degree of humification for peats is indicated by the state of the fibers. In slightly decomposed material, most of the volume consists of fibers. In moderately decomposed material, the fibers may be preserved but may break down with disturbance, such as rubbing between the fingers. In highly decomposed materials, fibers will be virtually absent: see von Post **humification scale.**

humus—a brown or black material formed by the partial decomposition of vegetable or animal matter: the organic portion of soil.

hydration—formation of a compound by the combining of water with some other substance.

hydraulic conductivity—see **coefficient of permeability.**

hydraulic fracturing—the fracturing of an underground strata by pumping water or grout under a pressure in excess of the tensile strength and confining pressure: also called hydrofracturing.

hydraulic gradient, i, s (D)—the loss of hydraulic head per unit distance of flow, dh/dL.

critical hydraulic gradient, i_c (D)—hydraulic gradient at which the intergranular pressure in a mass of cohesionless soil is reduced to zero by the upward flow of water.

hydrostatic head—the fluid pressure of formation water produced by the height of water above a given point.

hydrostatic pressure, u_o (FL^{-2})—a state of stress in which all the principal stresses are equal (and there is no shear stress), as in a liquid at rest: the product of the unit weight of the liquid and the different in elevation between the given point and the free water elevation.

excess hydrostatic pressure (hydrostatic excess pressure), $\bar{u}$, u (FL^{-2})—the pressure that exists in pore water in excess of the hydrostatic pressure.

hydrostatic pressure—a state of stress in which all the principal stresses are equal (and there is no shear stress). (ISRM)

hygroscopic capacity (hygroscopic coefficient), w_e (D)—ratio of: (*1*) the weight of water absorbed by a dry soil or rock in a saturated atmosphere at a given temperature, to (*2*) the weight of the oven-dried soil or rock.

hygroscopic water content, w_H (D)—the water content of an air-dried soil or rock.

hysteresis—incomplete recovery of strain during unloading cycle due to energy consumption. (ISRM)

impedance, acoustic—the product of the density and sonic velocity of a material. The extent of wave energy transmission and reflection at the boundary of two media is determined by their acoustic impedances. (ISRM)

inelastic deformation—the portion of deformation under stress that is not annulled by removal of stress. (ISRM)

inert—not participating in any fashion in chemical reactions.

influence value, I (D)—the value of the portion of a mathematical expression that contains combinations of the independent variables arranged in dimensionless form.

inhibitor—a material that stops or slows a chemical reaction from occurring.

initial consolidation (initial compression)—see **consolidation.**

initial set—a degree of stiffening of a grout mixture generally stated as an empirical value indicating the time in hours and minutes that is required for a mixture to stiffen sufficiently to resist the penetration of a weighted test needle.

injectability—see **groutability**.

inorganic silt—see **silt.**

in situ—applied to a rock or soil when occurring in the situation in which it is naturally formed or deposited.

intergranular pressure—see **stress.**

intermediate principal plane—see **principal plane.**

intermediate principal stress—see **stress.**

internal friction (shear resistance), (FL^{-2})—the portion of the shearing strength of a soil or rock indicated by the terms $p \tan \phi$ in Coulomb's equation $s = c + p \tan \phi$. It is usually considered to be due to the interlocking of the soil or rock grains and the resistance to sliding between the grains.

interstitial—occurring between the grains or in the pores in rock or soil.

intrinsic shear strength, S_o (FL^{-2})—the shear strength of a rock indicated by Coulomb's equation when $p \tan \phi$ (shear resistance or internal friction) vanishes. Corresponds to cohesion, c, in soil mechanics.

invert—on the cross section, the lowest point of the underground excavation or the lowest section of the lining. (ISRM)

isochrome—a curve showing the distribution of the excess hydrostatic pressure at a given time during a process of consolidation.

isotropic mass—a mass having the same property (or properties) in all directions.

isotropic material—a material whose properties do not vary with direction.

isotropy—having the same properties in all directions. (ISRM)

jackhammer—an air driven percussion drill that imparts a rotary hammering motion to the bit and has a passageway to the bit for the injection of compressed air for cleaning the hole of cuttings.

Discussion—These two characteristics distinguish it from the pavement breaker which is similar in size and general appearance.

jack-leg—a portable percussion drill of the jack-hammer type, used in underground work; has a single pneumatically adjustable leg for support.

jet grouting—technique utilizing a special drill bit with horizontal and vertical high speed water jets to excavate alluvial soils and produce hard impervious columns by pumping grout through the horizontal nozzles that jets and mixes with foundation material as the drill bit is withdrawn.

jetty—an elongated artificial obstruction projecting into a body of water form a bank or shore to control shoaling and scour by deflection of the force of water currents and waves.

joint—a break of geological origin in the continuity of a body of rock occurring either singly, or more frequently in a set or system, but not attended by a visible movement parallel to the surface of discontinuity. (ISRM)

joint diagram— a diagram constructed by accurately plotting the strike and dip of joints to illustrate the geometrical relationship of the joints within a specified area of geologic investigation. (ISRM)

joint pattern—a group of joints that form a characteristic geometrical relationship, and which can vary considerably from one location to another within the same geologic formation. (ISRM)

joint (fault) set—a group of more or less parallel joints. (ISRM)

joint (fault) system—a system consisting of two or more joint sets or any group of joints with a characteristic pattern, that is radiating, concentric, etc. (ISRM)

jumbo—a specially built mobile carrier used to provide a work platform for one or more tunneling operations, such as drilling and loading blast holes, setting tunnel supports, installing rock bolts, grouting, etc.

kaolin—a variety of clay containing a high percentage of kaolinite.

kaolinite—a common clay mineral having the general formula $Al_2(Si_2O_5)(OH_4)$; the primary constituent of kaolin.

karst—a geologic setting where cavities are developed in massive limestone beds by solution of flowing water. Caves and even underground river channels are produced into which surface runoff drains and often results in the land above being dry and relatively barren. (ISRM)

kelly—a heavy-wall tube or pipe, usually square or hexagonal in cross section, which works inside the matching center hole in the rotary table of a drill rig to impart rotary motion to the drill string.

lagging, *n—in mining or tunneling,* short lengths of timber, sheet steel, or concrete slabs used to secure the roof and sides of an opening behind the main timber or steel supports. The process of installation is also called lagging or lacing.

laminar flow (streamline flow) (viscous flow)—flow in which the head loss is proportional to the first power of the velocity.

landslide—the perceptible downward sliding or movement of a mass of earth or rock, or a mixture of both. (ISRM)

landslide (slide)—the failure of a sloped bank of soil or rock in which the movement of the mass takes place along a surface of sliding.

leaching—the removal in solution of the more soluble materials by percolating or moving waters. (ISRM)

leaching—the removal of soluble soil material and colloids by percolating water.

lime—specifically, calcium oxide (CaO_2); also loosely, a general term for the various chemical and physical forms of quicklime, hydrated lime, and hydraulic hydrated lime.

ledge—see **bedrock**.

linear (normal) strain—the change in length per unit of length in a given direction. (ISRM)

line of creep (path of percolation)—the path that water follows along the surface of contact between the foundation soil and the base of a dam or other structure.

line of seepage (seepage line) (phreatic line)—the upper free water surface of the zone of seepage.

linear expansion, L_e (D)—the increase in one dimension of a soil mass, expressed as a percentage of that dimension at the shrinkage limit, when the water content is increased from the shrinkage limit to any given water content.

linear shrinkage, L_s (D)—decrease in one dimension of a soil mass, expressed as a percentage of the original dimension, when the water content is reduced from a given value to the shrinkage limit.

lineation— the parallel orientation of structural features that are lines rather than planes; some examples are parallel orientation of the long dimensions of minerals; long axes of pebbles; striae on slickensides; and cleavage-bedding plane intersections. (ISRM)

liquefaction—the process of transforming any soil from a solid state to a liquid state, usually as a result of increased pore pressure and reduced shearing resistance.

liquefaction potential—the capability of a soil to liquefy or develop cyclic mobility.

liquefaction (spontaneous liquefaction)—the sudden large decrease of the shearing resistance of a cohesionless soil. It is caused by a collapse of the structure by shock or other type of strain and is associated with a sudden but temporary increase of the prefluid pressure. It involves a temporary transformation of the material into a fluid mass.

liquid, limit, *LL*, L_w, w_L (D)—(*a*) the water content corresponding to the arbitrary limit between the liquid and plastic states of consistency of a soil.

(*b*) the water content at which a pat of soil, cut by a groove of standard dimensions, will flow together for a distance of ½ in. (12.7 mm) under the impact of 25 blows in a standard liquid limit apparatus.

liquidity index (water-plasticity ratio) (relative water content), *B*, R_w, I_L (D)—the ratio, expressed as a percentage, of: (*1*) the natural water content of a soil minus its plastic limit, to (*2*) its plasticity index.

liquid-volume measurement—*in grouting*, measurement of grout on the basis of the total volume of solid and liquid constituents.

lithology—the description of rocks, especially sedimentary clastics and especially in hand specimens and in outcrops, on the basis of such characteristics as color, structures, mineralogy, and particle size.

loam—a mixture of sand, silt, or clay, or a combination of any of these, with organic matter (see **humus**).

Discussion—It is sometimes called topsoil in contrast to the subsoils that contain little or no organic matter.

local shear failure—see **shear failure**.

loess—a uniform aeolian deposit of silty material having an open structure and relatively high cohesion due to cementation of clay or calcareous material at grain contacts.

Discussion—A characteristic of loess deposits is that they can stand with nearly vertical slopes.

logarithmic decrement—the natural logarithm of the ratio of any two successive amplitudes of like sign, in the decay of a single-frequency oscillation.

longitudinal rod wave—see **compression wave**.

longitudinal wave, v_l (LT^{-1})—wave in which direction of displacement at each point of medium is normal to wave front, with propagation velocity, calculated as follows:

$$v_l = \sqrt{(E/\rho)[(1-\nu)/(1+\nu)(1-2\nu)]} = \sqrt{(\lambda + 2\mu)/\rho}$$

where:

E = Young's modulus,
ρ = mass density,
λ and μ = Lamé's constants, and
ν = Poisson's ratio.

long wave (quer wave), W (LT^{-1})—dispersive surface wave with one horizontal component, generally normal to the direction of propagation, which decreases in propagation velocity with increase in frequency.

lubricity—*in grouting*, the physico-chemical characteristic of a grout material flow through a soil or rock that is the inverse of the inherent friction of that material to the soil or rock; comparable to "wetness."

lugeon—a measure of permeability defined by a pump-in test or pressure test, where one Lugeon unit is a water take of 1 L/min per metre of hole at a pressure of 10 bars.

major principal plane—see **principal plane**.

major principal stress—see **stress**.

manifold—see **grout header**.

marl—calcareous clay, usually containing from 35 to 65 % calcium carbonate ($CaCO_3$).

marsh—a wetland characterized by grassy surface mats which are frequently interspersed with open water or by a closed canopy of grasses, sedges, or other herbacious plants.

mass unit weight—see **unit weight**.

mathematical model—the representation of a physical system by mathematical expressions from which the behavior of the system can be deduced with known accuracy. (ISRM)

matrix—*in grouting*, a material in which particles are embedded, that is, the cement paste in which the fine aggregate particles of a grout are embedded.

maximum amplitude (L, LT^{-1}, LT^{-2})—deviation from mean or zero point.

maximum density (maximum unit weight)—see **unit weight**.

mechanical analysis—see **grain-size analysis**.

mesic peat—see **hemic peat**.

metering pump—a mechanical arrangement that permits pumping of the various components of a grout system in any desired proportions or in fixed proportions. (*Syn.* proportioning pump, variable proportion pump.)

microseism—seismic pulses of short

tude, often occurring previous to failure of a material or structure. (ISRM)

minor principal plane—see **principal plane**.

minor principal stress—see **stress**.

mixed-in-place pile—a soil-cement pile, formed in place by forcing a grout mixture through a hollow shaft into the ground where it is mixed with the in-place soil with an auger-like head attached to the hollow shaft.

mixer—a machine employed for blending the constituents of grout, mortar, or other mixtures.

mixing cycle—the time taken for the loading, mixing, and unloading cycle.

mixing speed—the rotation rate of a mixer drum or of the paddles in an open-top, pan, or trough mixer, when mixing a batch; expressed in revolutions per minute.

modifier—*in grouting*, an additive used to change the normal chemical reaction or final physical properties of a grout system.

modulus of deformation—see **modulus of elasticity**.

modulus of elasticity (modulus of deformation), *E*, *M* (FL^{-2})—the ratio of stress tostrain for a material under given loading conditions; numerically equal to the slope of the tangent or the secant of a stress-strain curve. The use of the term **modulus of elasticity** is recommended for materials that deform in accordance with Hooke's law; the term **modulus of deformation** for materials that deform otherwise.

modulus of subgrade reaction—see **coefficient of subgrade reaction**.

modulus of volume change—see **coefficient of volume compressibility**.

Mohr circle—a graphical representation of the stresses acting on the various planes at a given point.

Mohr circle of stress (strain)—a graphical representation of the components of stress (strain) acting across the various planes at a given point, drawn with reference to axes of normal stress (strain) and shear stress (strain). (ISRM)

Mohr envelope—the envelope of a sequence of Mohr circles representing stress conditions at failure for a given material. (ISRM)

Mohr envelope (rupture envelope) (rupture line)—the envelope of a series of Mohr circles representing stress conditions at failure for a given material.

Discussion—According to Mohr's rupture hypothesis, a rupture envelope is the locus of points the coordinates of which represent the combinations of normal and shearing stresses that will cause a given material to fail.

moisture content (water content), *w* (D)—the ratio, expressed as a percentage, of: (*1*) the weight of water in a given soil mass, to (*2*) the weight of solid particles.

moisture content—the percentage by weight of water contained in the pore space of a rock or soil with respect to the weight of the solid material. (ISRM)

moisture-density curve—see **compaction curve.**

moisture-density test—see **compaction test.**

moisture equivalent:

centrifuge moisture equivalent, W_e, *CME* (D)—the water content of a soil after it has been saturated with water and then subjected for 1 h to a force equal to 1000 times that of gravity.

field moisture equivalent, FME—the minimum water content expressed as a percentage of the weight of the oven-dried soil, at which a drop of water placed on a smoothed surface of the soil will not immediately be absorbed by the soil but will spread out over the surface and give it a shiny appearance.

montmorillonite—a group of clay minerals characterized by a weakly bonded sheet-like internal molecular structure; consisting of extremely finely divided hydrous aluminum or magnesium silicates that swell on wetting, shrink on drying, and are subject to ion exchange.

muck—stone, dirt, debris, or useless material; or an organic soil of very soft consistency.

mud—a mixture of soil and water in a fluid or weakly solid state.

mudjacking—see **slab jacking.**

multibench blasting—the blasting of several benches (steps) in quarries and open pits, either simultaneously or with small delays. (ISRM)

multiple-row blasting—the drilling, charging, and firing of several rows of vertical holes along a quarry or opencast face. (ISRM)

muskeg—level, practically treeless areas supporting dense growth consisting primarily of grasses. The surface of the soil is covered with a layer of partially decayed grass and grass roots which is usually wet and soft when not frozen.

mylonite—a microscopic breccia with flow structure formed in fault zones. (ISRM)

natural frequency—the frequency at which a body or system vibrates when unconstrained by external forces. (ISRM)

natural frequency (displacement resonance), f_n—frequency for which phase angle is 90° between the direction of the excited force (or torque) vector and the direction of the excited excursion vector.

neat cement grout—a mixture of hydraulic cement and water without any added aggregate or filler materials.

Discussion—This may or may not contain admixture.

neutral stress—see **stress.**

newtonian fluid—a true fluid that tends to exhibit constant viscosity at all rates of shear.

node—point, line, or surface of standing wave system at which the amplitude is zero.

normal force—a force directed normal to the surface element across which it acts. (ISRM)

normal stress—see **stress.**

normally consolidated soil deposit—a soil deposit that has never been subjected to an effective pressure greater than the existing overburden pressure.

no-slump grout—grout with a slump of 1 in. (25 mm) or less according to the standard slump test (Test Method C 143).[2] See also **slump** and slump test.

open cut—an excavation through rock or soil made through a hill or other topographic feature to facilitate the passage of a highway, railroad, or waterway along an alignment that varies in topographic relief. An open cut can be comprised of single slopes or multiple slopes, or multiple slopes and horizontal benches, or both. (ISRM)

optimum moisture content (optimum water content), *OMC,* w_o (D)—the water content at which a soil can be compacted to a maximum dry unit weight by a given compactive effort.

organic clay—a clay with a high organic content.

organic silt—a silt with a high organic content.

organic soil—soil with a high organic content.

Discussion—In general, organic soils are very compressible and have poor load-sustaining properties.

organic terrain—see **peatland.**

oscillation—the variation, usually with time, of the magnitude of a quantity with respect to a specified reference when the magnitude is alternately greater and smaller than the reference.

outcrop—the exposure of the bedrock at the surface of the ground. (ISRM)

overbreak—the quantity of rock that is excavated or breaks out beyond the perimeter specified as the finished excavated tunnel outline. (ISRM)

overburden—the loose soil, sand, silt, or clay that overlies bedrock. In some usages it refers to all material overlying the point of interest (tunnel crown), that is, the total cover of soil and rock overlying an underground excavation. (ISRM)

overburden load—the load on a horizontal surface underground due to the column of material located vertically above it. (ISRM)

overconsolidated soil deposit—a soil deposit that has been subjected to an effective pressure greater than the present overburden pressure.

---

[2] *Annual Book of ASTM Standards*, Vol 04.02.

overconsolidation ratio. OCR—the ratio of preconsolidation vertical stress to the current effective overburden stress.

packer—*in grouting*, a device inserted into a hole in which grout or water is to be injected which acts to prevent return of the grout or water around the injection pipe; usually an expandable device actuated mechanically, hydraulically, or pneumatically.

paddle mixer—a mixer consisting essentially of a trough within which mixing paddles revolve about the horizontal axis, or a pan within which mixing blades revolve about the vertical axis.

pan mixer—a mixer comprised of a horizontal pan or drum in which mixing is accomplished by means of the rotating pan of fixed or rotating paddles, or both; rotation is about a vertical axis.

parent material—material from which a soil has been derived.

particle-size analysis—see **grain-size analysis.**

particle-size distribution—see **gradation, grain-size distribution.**

particulate grout—any grouting material characterized by undissolved (insoluble) particles in the mix. See also **chemical grout.**

passive earth pressure—see **earth pressure.**

passive state of plastic equilibrium—see **plastic equilibrium.**

path percolation (line of creep)—the path that water follows along the surface of contact between the foundation soil or rock and the base of a dam or other structure.

pavement pumping—ejection of soil and water mixtures from joints, cracks, and edges of rigid pavements, under the action of traffic.

peak shear strength—maximum shear strength along a failure surface. (ISRM)

peat—a naturally occurring highly organic substance derived primarily from plant materials.

Discussion—Peat is distinguished from other organic soil materials by its lower ash content (less than 25 % ash by dry weight) and from other phytogenic material of higher rank (that is, lignite coal) by its lower calorific value on a water saturated basis.

peatland—areas having peat-forming vegetation on which peak has accumulated or is accumulating.

penetrability—a grout property descriptive of its ability to fill a porous mass; primarily a function of lubricity and viscosity.

penetration—depth of hole cut in rock by a drill bit. (ISRM)

penetration grouting—filling joints or fractures in rock or pore spaces in soil with a grout without disturbing the formation; this grouting method does not modify the solid formation structure. See also **displacement grouting.**

penetration resistance (standard penetration resistance) (Proctor penetration resistance), p_R, N (FL^{-2} or Blows L^{-1})—(*a*) number of blows of a hammer of specified weight falling a given distance required to produce a given penetration into soil of a pile, casing, or sampling tube.

(*b*) unit load required to maintain constant rate of penetration into soil of a probe or instrument.

(*c*) unit load required to produce a specified penetration into soil at a specified rate of a probe or instrument. For a Proctor needle, the specified penetration is 2½ in. (63.5 mm) and the rate is ½ in. (12.7 mm)/s.

penetration resistance curve (Proctor penetration curve)—the curve showing the relationship between: (*1*) the penetration resistance, and (*2*) the water content.

percent compaction—the ratio, expressed as a percentage, of: (*1*) dry unit weight of a soil, to (*2*) maximum unit weight obtained in a laboratory compaction test.

percent consolidation—see **degree of consolidation.**

percent fines—amount, expressed as a percentage by weight, of a material in aggregate finer than a given sieve, usually the No. 200 (74 μm) sieve.

percent saturation (degree of saturation), S, S, (D)—the ratio, expressed as a percentage, of: (*1*) the volume of water in a given soil or rock mass, to (*2*) the total volume of intergranular space (voids).

perched ground water—confined ground water separated from an underlying body of ground water by an unsaturated zone.

perched water table—a water table usually of limited area maintained above the normal free water elevation by the presence of an intervening relatively impervious confining stratum.

perched water table—groundwater separated from an underlying body of groundwater by unsaturated soil or rock. Usually located at a higher elevation than the groundwater table. (ISRM)

percolation—the movement of gravitational water through soil (see **seepage**).

percolation—movement, under hydrostatic pressure of water through the smaller interstices of rock or soil, excluding movement through large openings such as caves and solution channels. (ISRM)

percussion drilling—a drilling technique that uses solid or hollow rods for cutting and crushing the rock by repeated blows (ISRM)

percussion drilling—a drilling process in which a hole is advanced by using a series of impacts to the drill steel and attached bit; the bit is normally rotated during drilling. See **rotary drilling.**

period—time interval occupied by one cycle.

permafrost—perennially frozen soil.

permanent strain—the strain remaining in a solid with respect to its initial condition after the application and removal of stress greater than the yield stress (commonly also called "residual" strain). (ISRM)

permeability—see **coefficient of permeability.**

permeability—the capacity of a rock to conduct liquid or gas. It is measured as the proportionality constant, k, between flow velocity, v, and hydraulic gradient, I: $v = k \cdot I$. (ISRM)

permeation grouting—filling joints or fractures in rock or pore spaces in soil with a grout, without disturbing the formation.

pH, pH (D)—an index of the acidity or alkalinity of a soil in terms of the logarithm of the reciprocal of the hydrogen ion concentration.

phase difference—difference between phase angles of two waves of same frequency.

phase of periodic quantity—fractional part of period through which independent variable has advanced, measured from an arbitrary origin.

phreatic line—the trace of the phreatic surface in any selected plane of reference.

phreatic line—see **line of seepage**.

phreatic surface—sse **free water elevation**.

phreatic water—see **free water**.

piezometer—an instrument for measuring pressure head.

piezometric line (equipotential line)—line along which water will rise to the same elevation in piezometric tubes.

piezometric surface—the surface at which water will stand in a series of piezometers.

piezometric surface—an imaginary surface that everywhere coincides with the static level of the water in the aquifer. (ISRM)

pile—relatively slender structural element which is driven, or otherwise introduced, into the soil, usually for the purpose of providing vertical or lateral support.

pillar—in-situ rock between two or more underground openings: crown pillars; barrier pillars; rib pillars; sill pillars; chain pillars; etc. (ISRM)

pilot drift (pioneer tunnel)—a drift or tunnel first excavated as a smaller section than the dimensions of the main tunnel. A pilot drift or tunnel is usually used to investigate rock conditions in advance of the main tunnel, to permit installation of bracing before the principal mass of rock is removed, or to serve as a drainage tunnel. (ISRM)

piping—the progressive removal of soil particles from a mass by percolating water, leading to the development of channels.

pit—an excavation in the surface of the earth from which ore is obtained as in large open pit mining or as an excavation made for test purposes, that is, a testpit. (ISRM)

plane of weakness—surface or narrow zone with a (shear or tensile) strength lower than that of the surrounding material. (ISRM)

plane stress (strain)—a state of stress (strain) in a solid body in which all stress (strain) components normal to a certain plane are zero. (ISRM)

plane wave—wave in which fronts are parallel to plane normal to direction of propagation.

plastic deformation—see **plastic flow**.

plastic equilibrium—state of stress within a soil or rock mass or a portion thereof, which has been deformed to such an extent that its ultimate shearing resistance is mobilized.

active state of plastic equilibrium—plastic equilibrium obtained by an expansion of a mass.

passive state of plastic equilibrium—plastic equilibrium obtained by a compression of a mass.

plastic flow (plastic deformation)—the deformation of a plastic material beyond the point of recovery, accompanied by continuing deformation with no further increase in stress.

plasticity—the property of a soil or rock which allows it to be deformed beyond the point of recovery without cracking or appreciable volume change.

plasticity—property of a material to continue to deform indefinitely while sustaining a constant stress. (ISRM)

plasticity index, I_p, *PI*, I_w (D)—numerical difference between the liquid limit and the plastic limit.

plasticizer—*in grouting*, a material that increases the plasticity of a grout, cement paste, or mortar.

plastic limit, w_p, *PL*, P_w (D)—(*a*) the water content corresponding to an arbitrary limit between the plastic and the semisolid states of consistency of a soil. (*b*) water content at which a soil will just begin to crumble when rolled into a thread approximately ⅛ in. (3.2 mm) in diameter.

plastic soil—a soil that exhibits plasticity.

plastic state (plastic range)—the range of consistency within which a soil or rock exhibits plastic properties.

Poisson's ratio, (ν)—ratio between linear strain changes perpendicular to and in the direction of a given uniaxial stress change.

pore pressure (*pore water pressure*)—see **neutral stress** under stress.

pore water—water contained in the voids of the soil or rock.

porosity, *n* (D)—the ratio, usually expressed as a percentage, of: (*1*) the volume of voids of a given soil or rock mass, to (*2*) the total volume of the soil or rock mass.

porosity—the ratio of the aggregate volume of voids or interstices in a rock or soil to its total volume. (ISRM)

portal—the surface entrance to a tunnel. (ISRM)

positive displacement pump—a pump that will continue to build pressure until the power source is stalled if the pump outlet is blocked.

potential drop, Δh (L)—the difference in total head between two equipotential lines.

power spectral density—the limiting mean-square value (for example, of acceleration, velocity, displacement, stress, or other random variable) per unit bandwidth, that is the limit of the mean-square value in a given rectangular bandwidth divided by the bandwidth, as the bandwidth approaches zero.

pozzolan—a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties.

preconsolidation pressure (prestress), p_c (FL^{-2})—the greatest effective pressure to which a soil has been subjected.

preplaced aggregate concrete—concrete produced by placing coarse aggregate in a form and later injecting a portland cement-sand or resin grout to fill the interstices.

pressure, *p* (FL^{-2})—the load divided by the area over which it acts.

pressure bulb—the zone in a loaded soil or rock mass bounded by an arbitrarily selected isobar of stress.

pressure testing—a method of permeability testing with water or grout pumped downhole under pressure.

pressure-void ratio curve (compression curve)—a curve representing the relationship between effective pressure and void ratio of a soil as obtained from a consolidation test. The curve has a characteristic shape when plotted on semilog paper with pressure on the log scale. The various parts of the curve and extensions to the parts of the curve and extensions to the parts have been designated as recompression, compression, virgin compression, expansion, rebound, and other descriptive names by various authorities.

pressure washing—the cleaning of soil or rock surfaces accomplished by injection of water, air, or other liquids, under pressure.

primary consolidation (primary compression) (primary time effect)—see **consolidation.**

primary hole—*in grouting*, the first series of holes to be drilled and grouted, usually at the maximum allowable spacing.

primary lining—the lining first placed inside a tunnel or shaft, usually used to support the excavation. The primary lining may be of wood or steel sets with steel or wood lagging or rock bolts and shot-crete. (ISRM)

primary permeability—internal permeability of intack rock; intergranular permeability (not permeability due to fracturing).

primary porosity—the porosity that developed during the final stages of sedimentation or that was present within sedimentary particles at the time of deposition.

primary state of stress—the stress in a geological formation before it is disturbed by man-made works. (ISRM)

principal plane—each of three mutually perpendicular planes through a point in a soil mass on which the shearing stress is zero.

intermediate principal plane—the plane normal to the direction of the intermediate principal stress.

major principal plane—the plane normal to the direction of the major principal stress.

minor principal plane—the plane normal to the direction of the minor principal stress.

principal stress—see **stress.**

principal stress (strain)—the stress (strain) normal to one of three mutually perpendicular planes on which the shear stresses (strains) at a point in a body are zero. (ISRM)

Proctor compaction curve—see **compaction curve.**

Proctor penetration curve—see **penetration resistance curve.**

Proctor penetration resistance—see **penetration resistance.**

profile—see **soil profile.**

progressive failure—failure in which the ultimate shearing resistance is progressively mobilized along the failure surface.

progressive failure—formation and development of localized fractures which, after additional stress increase, eventually form a continuous rupture surface and thus lead to failure after steady deterioration of the rock. (ISRM)

proportioning pump—see **metering pump.**

proprietary—made and marketed by one having the exclusive right to manufacture and sell; privately owned and managed.

protective filter—see **filter.**

pumpability—*in grouting*, a measure of the properties of a particular grout mix to be pumped as controlled by the equipment being used, the formation being injected, and the engineering objective limitations.

pumping of pavement (pumping)—see **pavement pumping.**

pumping test—a field procedure used to determine in situ permeability or the ability of a formation to accept grout.

pure shear—a state of strain resulting from that stress condition most easily described by a Mohr circle centered at the origin. (ISRM)

quarry—an excavation in the surface of the earth from which stone is obtained for crushed rock or building stone. (ISRM)

Quer-wave (love wave), W—dispersive surface wave with one horizontal component, generally normal to the direction of propagation, which decreases in propagation velocity with increase in frequency.

quick condition (quicksand)—condition in which water is flowing upwards with sufficient velocity to reduce significantly the bearing capacity of the soil through a decrease in intergranular pressure.

quick test—see **unconsolidated undrained test.**

radius of influence of a well—distance from the center of the well to the closest point at which the piezometric surface is not lowered when pumping has produced the maximum steady rate of flow.

raise—upwardly constructed shaft; that is, an opening, like a shaft, made in the roof of one level to reach a level above. (ISRM)

range (of a deformation-measuring instrument)—the amount between the maximum and minimum quantity an instrument can measure without resetting. In some instances provision can be made for incremental extension of the range.

Rayleigh wave, v_R (LT^{-1})—dispersive surface wave in which element has retrograding elliptic orbit with one major vertical and one minor horizontal component both in plane of propagation velocity:

$$v_R = \alpha v_t \text{ with } 0.910 < \alpha < 0.995 \text{ for } 0.25 < \nu < 0.5$$

reactant—*in grouting*, a material that reacts chemically with the base component of grout system.

reactive aggregate—an aggregate containing siliceous material (usually in amorphous or crypto-crystalline state) which can react chemically with free alkali in the cement.

Discussion—The reaction can result in expansion of the hardened material, frequently to a damaging extent.

reflected (or refracted) wave—components of wave incident upon second medium and reflected into first medium (or refracted) into second medium.

reflection and refraction loss—that part of transmitted energy lost due to nonuniformity of mediums.

refusal—*in grouting*, when the rate of grout take is low, or zero, at a given pressure.

relative consistency, I_c, C_r (D)—ratio of: (*1*) the liquid limit minus the natural water content, to (*2*) the plasticity index.

relative density, D_d, I_D (D)—the ratio of (*1*) the difference between the void ratio of a cohesionless soil in the loosest state and any given void ratio, to (*2*) the difference between the void ratios in the loosest and in the densest states.

relative water content—see **liquidity index.**

remodeled soil—soil that has had its natural structure modified by manipulation.

remolding index, I_R (D)—the ratio of: (*1*) the modulus of deformation of a soil in the undisturbed state, to (*2*) the modulus of deformation of the soil in the remolded state.

remodeling sensitivity (sensitivity ratio), S_t (D)—the ratio of: (*1*) the unconfined compressive strength of an undisturbed specimen of soil, to (*2*) the unconfined compressive strength of a specimen of the same soil after remolding at unaltered water content.

residual soil—soil derived in place by weathering of the underlying material.

residual strain—the strain in a solid associated with a state of residual stress. (ISRM)

residual stress—stress remaining in a solid under zero external stress after some process that causes the dimensions of the various parts of the solid to be incompatible under zero stress, for example, (*1*) deformation under the action of external stress when some parts of the body suffer permanent strain; or (*2*) heating or cooling of a body in which the thermal expansion coefficient is not uniform throughout the body. (ISRM)

resin—*in grouting*, a material that usually constitutes the base of an organic grout system.

resin grout—a grout system composed of essentially resinous materials such as epoxys, polyesters, and urethanes.

Discussion—In Europe, this refers to any chemical grout system regardless of chemical origin.

resolution (of a deformation-measuring instrument)—the ratio of the smallest divisional increment of the indicating scale to the sensitivity of the instrument. Interpolation within the increment may be possible, but is not recommended in specifying resolution.

resonance—the reinforced vibration of a body exposed to the vibration, at about the frequency, of another body.

resonant frequency—a frequency at which resonance exists.

response—the motion (or other output) in a device or system resulting from an excitation (stimulus) under specified conditions.

retard—bank-protection structure designed to reduce the riparian velocity and induce silting or accretion.

retardation—delay in deformation. (ISRM)

retarder—a material that slows the rate at which chemical reactions would otherwise occur.

reverse circulation—a drilling system in which the circulating medium flows down through the annulus and up through the drill rod, that is, in the reverse of the normal direction of flow.

revetment—bank protection by armor, that is, by facing of a bank or embankment with erosion-resistant material.

riprap stone—(generally less than 1 ton in weight) specially selected and graded quarried stone placed to prevent erosion through wave action, tidal forces, or strong currents and thereby preserve the shape of a surface, slope, or underlying structure.

rise time (pulse rise time)—the interval of time required for the leading edge of a pulse to rise from some specified small fraction to some specified larger fraction of the maximum value.

rock—natural solid mineral matter occurring in large masses or fragments.

rock—any naturally formed aggregate of mineral matter occurring in large masses or fragments. (ISRM)

rock anchor—a steel rod or cable installed in a hole in rock; in principle the same as rock bolt, but generally used for rods longer than about four metres. (ISRM)

rock bolt—a steel rod placed in a hole drilled in rock used to tie the rock together. One end of the rod is firmly anchored in the hole by means of a mechanical device or grout, or both, and the threaded projecting end is equipped with a nut and plate that bears against the rock surface. The rod can be pretensioned. (ISRM)

rock burst—a sudden and violent expulsion of rock from its surroundings that occurs when a volume of rock is strained beyond the elastic limit and the accompanying failure is of such a nature that accumulated energy is released instantaneously.

rock burst—sudden explosive-like release of energy due to the failure of a brittle rock of high strength. (ISRM)

rock flour—see **silt**.

rock mass—rock as it occurs in situ, including its structural discontinuities. (ISRM)

rock mechanics—the application of the knowledge of the mechanical behavior of rock to engineering problems dealing with rock. Rock mechanics overlaps with structural geology, geophysics, and soil mechanics.

rock mechanics—theoretical and applied science of the mechanical behaviour of rock. (ISRM)

roof—top of excavation or underground opening, particularly applicable in bedded rocks where the top surface of the opening is flat rather than arched. (ISRM)

rotary drilling—a drilling process in which a hole is advanced by rotation of a drill bit under constant pressure without impact. See **percussion drilling**.

round—a set of holes drilled and charged in a tunnel or quarry that are fired instantaneously or with short-delay detonators. (ISRM)

running ground—*in tunneling*, a granular material that tends to flow or "run" into the excavation.

rupture—that stage in the development of a fracture where instability occurs. It is not recommended that the term rupture be used in rock mechanics as a synonym for fracture. (ISRM)

rupture envelope (*rupture line*)—see **Mohr envelope**.

sagging—usually occurs in sedimentary rock formations as a separation and downward bending of sedimentary beds in the roof of an underground opening. (ISRM)

sand—particles of rock that will pass the No. 4 (4.75-mm) sieve and be retained on the No. 200 (75-μm) U.S. standard sieve.

sand boil—the ejection of sand and water resulting from piping.

sand equivalent—a measure of the amount of silt or clay contamination in fine aggregate as determined by test (Test Method D 2419).[3]

sanded grout—grout in which sand is incorporated into the mixture.

sapric peat—peat in which the original plant fibers are highly decomposed (less than 33 % fibers).

saturated unit weight—see **unit weight**.

saturation curve—see **zero air voids curve**.

scattering loss—that part of transmitted energy lost due to roughness of reflecting surface.

schistosity—the variety of foliation that occurs in the coarser-grained metamorphic rocks and is generally the result of the parallel arrangement of platy and ellipsoidal mineral grains within the rock substance. (ISRM)

secant modulus—slope of the line connecting the origin and a given point on the stress-strain curve. (ISRM)

---

[3] *Annual Book of ASTM Standards*, Vol 04.03

secondary consolidation (secondary compression) (secondary time effect)—see **consolidation.**

secondary hole—*in grouting*, the second series of holes to be drilled and grouted usually spaced midway between primary holes.

secondary lining—the second-placed, or permanent, structural lining of a tunnel, which may be of concrete, steel, or masonry. (ISRM)

secondary state of stress—the resulting state of stress in the rock around man-made excavations or structures. (ISRM)

sediment basin—a structure created by construction of a barrier or small dam-like structure across a waterway or by excavating a basin or a combination of both to trap or restrain sediment.

seep—a small area where water oozes from the soil or rock.

seepage—the infiltration or percolation of water through rock or soil to or from the surface. The term seepage is usually restricted to the very slow movement of ground water. (ISRM)

seepage (percolation)—the slow movement of gravitational water through the soil or rock.

seepage force—the frictional drag of water flowing through voids or interstices in rock, causing an increase in the intergranular pressure, that is, the hydraulic force per unit volume of rock or soil which results from the flow of water and which acts in the direction of flow. (ISRM)

seepage force, J (F)—the force transmitted to the soil or rock grains by seepage.

seepage line—see **line of seepage.**

seepage velocity, V_s, V_1 (LT^{-1})—the rate of discharge of seepage water through a porous medium per unit area of void space perpendicular to the direction of flow.

segregation—*in grouting*, the differential concentration of the components of mixed grout, resulting in nonuniform proportions in the mass.

seismic support—mass (heavy) supported on springs (weak) so that mass remains almost at rest when free end of springs is subjected to sinusoidal motion at operating frequency.

seismic velocity—the velocity of seismic waves in geological formations. (ISRM)

seismometer—instrument to pick up linear (vertical, horizontal) or rotational displacement, velocity, or acceleration.

self-stressing grout—expansive-cement grout in which the expansion induces compressive stress in grout if the expansion movement is restrained.

sensitivity—the effect of remolding on the consistency of a cohesive soil.

sensitivity (of an instrument)—the differential quotient dQ_0/dQ_1, where Q_0 is the scale reading and Q_1 is the quantity to be measured.

sensitivity (of a transducer)—the differential quotient dQ_0/dQ_1, where Q_0 is the output and Q_1 is the input.

series grouting—similar to stage grouting, except each successively deeper zone is grouted by means of a newly drilled hole, eliminating the need for washing grout out before drilling the hole deeper.

set—*in grouting*, the condition reached by a cement paste, or grout, when it has lost plasticity to an arbitrary degree, usually measured in terms of resistance to penetration or deformation; initial set refers to first stiffening and final set refers to an attainment of significant rigidity.

setting shrinkage—*in grouting*, a reduction in volume of grout prior to the final set of cement caused by bleeding, by the decrease in volume due to the chemical combination of water with cement, and by syneresis.

set time—(*1*) the hardening time of portland cement; or (*2*) the gel time for a chemical grout.

shaft—generally a vertical or near vertical excavation driven downward from the surface as access to tunnels, chambers, or other underground workings. (ISRM)

shaking test—a test used to indicate the presence of significant amounts of rock flour, silt, or very fine sand in a fine-grained soil. It consists of shaking a pat of wet soil, having a consistency of thick paste, in the palm of the hand; observing the surface for a glossy or livery appearance; then squeezing the pat; and observing if a rapid apparent drying and subsequent cracking of the soil occurs.

shear failure (failure by rupture)—failure in which movement caused by shearing stresses in a soil or rock mass is of sufficient magnitude to destroy or seriously endanger a structure.

general shear failure—failure in which the ultimate strength of the soil or rock is mobilized along the entire potential surface of sliding before the structure supported by the soil or rock is impaired by excessive movement.

local shear failure—failure in which the ultimate shearing strength of the soil or rock is mobilized only locally along the potential surface of sliding at the time the structure supported by the soil or rock is impaired by excessive movement.

shear force—a force directed parallel to the surface element across which it acts. (ISRM)

shear plane—a plane along which failure of material occurs by shearing. (ISRM)

shear resistance—see **internal friction.**

shear strain—the change in shape, expressed by the relative change of the right angles at the corner of what was in the undeformed state an infinitesimally small rectangle or cube. (ISRM)

shear strength, s, T_f (FL^{-2})—the maximum resistance of a soil or rock to shearing stresses. See **peak shear strength.**

shear stress—stress directed parallel to the surface element across which it acts. (ISRM)

shear stress (shearing stress) (tangential stress)—see **stress.**

shear wave (rotational, equivoluminal)—wave in which medium changes shape without change of volume (shear-plane wave in isotropic medium is transverse wave).

shelf life—maximum time interval during which a material may be stored and remain in a usable condition; usually related to storage conditions.

shock pulse—a substantial disturbance characterized by a rise of acceleration from a constant value and decay of acceleration to the constant value in a short period of time.

shock wave—a wave of finite amplitude characterized by a shock front, a surface across which pressure, density, and internal energy rise almost discontinuously, and which travels with a speed greater than the normal speed of sound. (ISRM)

shotcrete—mortar or concrete conveyed through a hose and pneumatically projected at high velocity onto a surface. Can be applied by a "wet" or "dry" mix method. (ISRM)

shrinkage-compensating—*in grouting*, a characteristic of grout made using an expansive cement in which volume increase, if restrained, induces compressive stresses that are intended to offset the tendency of drying shrinkage to induce tensile stresses. See also **self-stressing grout**.

shrinkage index, *SI* (D)—the numerical difference between the plastic and shrinkage limits.

shrinkage limit, *SL*, w_s (D)—the maximum water content at which a reduction in water content will not cause a decrease in volume of the soil mass.

shrinkage ratio, *R* (D)—the ratio of: (*1*) a given volume change, expressed as a percentage of the dry volume, to (*2*) the corresponding change in water content above the shrinkage limit, expressed as a percentage of the weight of the oven-dried soil.

sieve analysis—determination of the proportions of particles lying within certain size ranges in a granular material by separation on sieves of different size openings.

silt (inorganic silt) (rock flour)—material passing the No. 200 (75-μm) U.S. standard sieve that is nonplastic or very slightly plastic and that exhibits little or no strength when air-dried.

silt size—that portion of the soil finer than 0.02 mm and coarser than 0.002 mm (0.05 mm and 0.005 mm in some cases).

simple shear—shear strain in which displacements all lie in one direction and are proportional to the normal distances of the displaced points from a given reference plane. The dilatation is zero. (ISRM)

single-grained structure—see **soil structure**.

size effect—influence of specimen size on its strength or other mechanical parameters. (ISRM)

skin friction, *f* (FL^{-2})—the frictional resistance developed between soil and an element of structure.

slabbing—the loosening and breaking away of relatively large flat pieces of rock from the excavated surface, either immediately after or some time after excavation. Often occurring as tensile breaks which can be recognized by the subconchoidal surfaces left on remaining rock surface. (ISRM)

slabjacking—*in grouting*, injection of grout under a concrete slab in order to raise it to a specified grade.

slaking—deterioration of rock on exposure to air or water.

slaking—the process of breaking up or sloughing when an indurated soil is immersed in water.

sleeved grout pipe—see **tube A manchette**.

sliding—relative displacement of two bodies along a surface, without loss of contact between the bodies. (ISRM)

slope—the excavated rock surface that is inclined to the vertical or horizontal, or both, as in an open-cut. (ISRM)

slow test—see **consolidated-drain test**.

slump—a measure of consistency of freshly mixed concrete or grout. See also **slump test**.

slump test—the procedure for measuring slump (Test Method C 143).[2]

slurry cutoff wall—a vertical barrier constructed by excavating a vertical slot under a bentonite slurry and backfilling it with materials of low permeability for the purpose of the containment of the lateral flow of water and other fluids.

slurry grout—a fluid mixture of solids such as cement, sand, or clays in water.

slurry trench—a trench that is kept filled with a bentonite slurry during the excavation process to stabilize the walls of the trench.

slush grouting—application of cement slurry to surface rock as a means of filling cracks and surface irregularities or to prevent slaking; it is also applied to riprap to form grouted riprap.

smooth (-wall) blasting—a method of accurate perimeter blasting that leaves the remaining rock practically undamaged. Narrowly spaced and lightly charged blastholes, sometimes alternating with empty dummy holes, located along the breakline and fired simultaneously as the last round of the excavation. (ISRM)

soil (earth)—sediments or other unconsolidated accumulations of solid particles produced by the physical and chemical disintegration of rocks, and which may or may not contain organic matter.

soil binder—see **binder**.

soil-forming factors—factors, such as parent material, climate, vegetation, topography, organisms, and time involved in the transformation of an original geologic deposit into a soil profile.

soil horizon—see **horizon**.

soil mechanics—the application of the laws and principles of mechanics and hydraulics to engineering problems dealing with soil as an engineering material.

soil physics—the organized body of knowledge concerned with the physical characteristics of soil and with the methods employed in their determinations.

soil profile (profile)—vertical section of a soil, showing the nature and sequence of the various layers, as developed by deposition or weathering, or both.

soil stabilization—chemical or mechanical treatment designed to increase or maintain the stability of a mass of soil or otherwise to improve its engineering properties.

soil structure—the arrangement and state of aggregation of soil particles in a soil mass.

flocculent structure—an arrangement composed of flocs of soil particles instead of individual soil particles.

honeycomb structure—an arrangement of soil particles having a comparatively loose, stable structure resembling a honeycomb.

single-grained structure—an arrangement composed of individual soil particles: characteristic structure of coarse-grained soils.

soil suspension—highly diffused mixture of soil and water.

soil texture—see **gradation**.

solution cavern—openings in rock masses formed by moving water carrying away soluble materials.

sounding well—*in grouting*, a vertical conduit in a mass of coarse aggregate for preplaced aggregate concrete which contains closely spaced openings to permit entrance of grout.

Discussion—The grout level is determined by means of a measuring line on a float within the sounding well.

spacing—the distance between adjacent blastholes in a direction parallel to the face. (ISRM)

spalling—(*1*) longitudinal splitting in uniaxial compression, or (*2*) breaking-off of plate-like pieces from a free rock surface. (ISRM)

specific gravity:

specific gravity of solids, G, G_s, S_s (D)—ratio of: (*1*) the weight in air of a given volume of solids at a stated temperature to (*2*) the weight in air of an equal volume of distilled water at a stated temperature.

apparent specific gravity, G_a, S_a (D)—ratio of: (*1*) the weight in air of a given volume of the impermeable portion of a permeable material (that is, the solid matter including its impermeable pores or voids) at a stated temperature to (*2*) the weight in air of an equal volume of distilled water at a stated temperature.

bulk specific gravity (specific mass gravity), G_m, S_m (D)—ratio of: (*1*) the weight in air of a given volume of a permeable material (including both permeable and impermeable voids normal to the material) at a stated temperature to (*2*) the weight in air of an equal volume of distilled water at a stated temperature.

specific surface (L^{-1})—the surface area per unit of volume of soil particles.

spherical wave—wave in which wave fronts are concentric spheres.

split spacing grouting—a grouting sequence in which initial (primary) grout holes are relatively widely spaced and subsequent grout holes are placed midway between previous grout holes to "split the spacing."; this process is continued until a specified hole spacing is achieved or a reduction in grout take to a specified value occurs, or both.

spring characteristics, c (FL^{-1})—ratio of increase in load to increase in deflection:

$$c = l/C$$

where:

C = compliance.

stability—the condition of a structure or a mass of material when it is able to support the applied stress for a long time without suffering any significant deformation or movement that is not reversed by the release of stress. (ISRM)

stability factor (stability number), N_s (D)—a pure number used in the analysis of the stability of a soil embankment, defined by the following equation:

$$N_s = H_c \gamma_e / c$$

where:

H_c = critical height of the sloped bank,
γ_e = effective unit of weight of the soil, and
c = cohesion of the soil

NOTE—Taylor's "stability number" is the reciprocal of Terzaghi's "stability factor."

stabilization—see **soil stabilization**.

stage—*in grouting*, the length of hole grouted at one time. See also **stage grouting**.

stage grouting—sequential grouting of a hole in separate steps or stages in lieu of grouting the entire length at once; holes may be grouted in ascending stages by using packers or in descending stages downward from the collar of the hole.

standard compaction—see **compaction test**.

standard penetration resistance—see **penetration resistance**.

standing wave—a wave produced by simultaneous transmission in opposite directions of two similar waves resulting in fixed points of zero amplitudes called nodes.

steady-state vibration—vibration in a system where the velocity of each particle is a continuing periodic quantity.

stemming—(*1*) the material (chippings, or sand and clay) used to fill a blasthole after the explosive charge has been inserted. Its purpose is to prevent the rapid escape of the explosion gases. (*2*) the act of pushing and tamping the material in the hole. (ISRM)

stick-slip—rapid fluctuations in shear force as one rock mass slides past another, characterized by a sudden slip between the rock masses, a period of no relative displacement between the two masses, a sudden slip, etc. The oscillations may be regular as in a direct shear test, or irregular as in a triaxial test.

sticky limit, T_w (D)—the lowest water content at which a soil will stick to a metal blade drawn across the surface of the soil mass.

stiffness—the ratio of change of force (or torque) to the corresponding change in translational (or rotational) deflection of an elastic element.

stiffness-force—displacement ratio. (ISRM)

stone—crushed or naturally angular particles of rock.

stop—*in grouting*, a packer setting at depth.

stop grouting—the grouting of a hole beginning at the lowest packer setting (stop) after the hole is drilled to total depth.

Discussion—Packers are placed at the top of the zone being grouted. Grouting proceeds from the bottom up. Also called upstage grouting.

strain, ϵ (D)—the change in length per unit of length in a given direction.

strain (linear or normal), ϵ (D)—the change in length per unit of length in a given direction.

strain ellipsoid—the representation of the strain in the form of an ellipsoid into which a sphere of unit radius deforms and whose axes are the principal axes of strain. (ISRM)

strain (stress) rate—rate of change of strain (stress) with time. (ISRM)

strain resolution (strain sensitivity), R_s (D)—the smallest subdivision of the indicating scale of a deformation-measuring device divided by the product of the sensitivity of the device and the gage length. The deformation resolution, R_d, divided by the gage length.

strain (stress) tensor—the second order tensor whose diagonal elements consist of the normal strain (stress) components with respect to a given set of coordinate axes and whose off-diagonal elements consist of the corresponding shear strain (stress) components. (ISRM)

streamline flow—see **laminar flow**.

strength—maximum stress which a material can resist without failing for any given type of loading. (ISRM)

stress, σ, p, f (FL^{-2})—the force per unit area acting within the soil mass.

effective stress (effective pressure) (intergranular pressure), $\bar{\sigma}$, $\bar{f}$ (FL^{-2})—the average normal force per unit area transmitted from grain to grain of a soil mass. It is the stress that is effective in mobilizing internal friction.

neutral stress (pore pressure) (pore water pressure), u, u_w

(FL^{-2})—stress transmitted through the pore water (water filling the voids of the soil).

normal stress, σ, p (FL^{-2})—the stress component normal to a given plane.

principal stress, σ_1, σ_2, σ_3 (FL^{-2})—stresses acting normal to three mutually perpendicular planes intersecting at a point in a body, on which the shearing stress is zero.

major principal stress, σ_1 (FL^{-2})—the largest (with regard to sign) principal stress.

minor principal stress, σ_3 (FL^{-2})—the smallest (with regard to sign) principal stress.

intermediate principal stress, σ_2 (FL^{-2})—the principal stress whose value is neither the largest nor the smallest (with regard to sign) of the three.

shear stress (shearing stress) (tangential stress), τ, s FL^{-2})—the stress component tangential to a given plane.

total stress, σ, f (FL^{-2})—the total force per unit area acting within a mass of soil. It is the sum of the neutral and effective stresses.

stress ellipsoid—the representation of the state of stress in the form of an ellipsoid whose semi-axes are proportional to the magnitudes of the principal stresses and lie in the principal directions. The coordinates of a point *P* on this ellipse are proportional to the magnitudes of the respective components of the stress across the plane normal to the direction *OP*, where *O* is the center of the ellipsoid. (ISRM)

stress (strain) field—the ensemble of stress (strain) states defined at all points of an elastic solid. (ISRM)

stress relaxation—stress release due to creep. (ISRM)

strike—the direction or azimuth of a horizontal line in the plane of an inclined stratum, joint, fault, cleavage plane, or other planar feature within a rock mass. (ISRM)

structure—one of the larger features of a rock mass, like bedding, foliation, jointing, cleavage, or brecciation: also the sum total of such features as contrasted with texture. Also, in a broader sense, it refers to the structural features of an area such as anti-clines or synclines. (ISRM)

structure—see **soil structure**.

subbase—a layer used in a pavement system between the subgrade and base coarse, or between the subgrade and portland cement concrete pavement.

subgrade—the soil prepared and compacted to support a structure or a pavement system.

subgrade surface—the surface of the earth or rock prepared to support a structure or a pavement system.

submerged unit weight—see **unit weight**.

subsealing—*in grouting*, grouting under concrete slabs for the purpose of filling voids without raising the slabs.

subsidence—the downward displacement of the overburden (rock or soil, or both) lying above an underground excavation or adjoining a surface excavation. Also the sinking of a part of the earth's crust. (ISRM)

subsoil—(*a*) soil below a subgrade of fill. (*b*) that part of a soil profile occurring below the "A" horizon.

sulfate attack—*in grouting*, harmful or deleterious reactions between sulfates in soil or groundwater and the grout.

support—structure or structural feature built into an underground opening for maintaining its stability. (ISRM)

surface force—any force that acts across an internal or external surface element in a material body, not necessarily in a direction lying in the surface. (ISRM)

surface wave—a wave confined to a thin layer at the surface of a body. (ISRM)

suspension—a mixture of liquid and solid materials.

suspension agent—an additive that decreased the settlement rate of particles in liquid.

swamp—a forested or shrub covered wetland where standing or gently flowing water persists for long periods on the surface.

syneresis—*in grouting*, the exudation of liquid (generally water) from a set gel which is not stressed, due to the tightening of the grout material structure.

take—see **grout take**.

talus—rock fragments mixed with soil at the foot of a natural slope from which they have been separated.

tangential stress—see **stress**.

tangent modulus—slope of the tangent to the stress-strain curve at a given stress value (generally taken at a stress equal to half the compressive strength). (ISRM)

tensile strength (unconfined or uniaxial tensile strength), T_o (FL^{-2})—the load per unit area at which an unconfined cylindrical specimen will fail in a simple tension (pull) test.

tensile stress—normal stress tending to lengthen the body in the direction in which it acts. (ISRM)

tertiary hole—*in grouting*, the third series of holes to be drilled and grouted usually spaced midway between previously grouted primary and secondary holes.

texture—*of soil and rock*, geometrical aspects consisting of size, shape, arrangement, and crystallinity of the component particles and of the related characteristics of voids.

texture—the arrangement in space of the components of a rock body and of the boundaries between these components. (ISRM)

theoretical time curve—see **consolidation time curve**.

thermal spalling—the breaking of rock under stresses induced by extremely high temperature gradients. High-velocity jet flames are used for drilling blast holes with this effect. (ISRM)

thermo-osmosis—the process by which water is caused to flow in small openings of a soil mass due to differences in temperature within the mass.

thickness—the perpendicular distance between bounding surfaces such as bedding or foliation planes of a rock. (ISRM)

thixotropy—the property of a material that enables it to stiffen in a relatively short time on standing, but upon agitation or manipulation to change to a very soft consistency or to a fluid of high viscosity, the process being completely reversible.

throw—the projection of broken rock during blasting. (ISRM)

thrust—force applied to a drill in the direction of penetration. (ISRM)

tight—rock remaining within the minimum excavation lines after completion of a blasting record. (ISRM)

till—see **glacial till**.

time curve—see **consolidation time curve**.

time factor, T_v, T (D)—dimensionless factor, utilized in the theory of consolidation, containing the physical constants of a soil stratum influencing its time-rate of consolidation, expressed as follows:

$$T = k(1 + e)t/(a_v \gamma_w \cdot H^2) = (c_v \cdot t)/H^2$$

where:

k = coefficient of permeability (LT^{-1}),
e = void ratio (dimensionless),
t = elapsed time that the stratum has been consolidated (T),
a_v = coefficient of compressibility (L^2F^{-1}),
γ_w = unit weight of water (FL^{-3}),
H = thickness of stratum drained on one side only. If stratum is drained on both sides, its thickness equals $2H$ (L), and
c_v = coefficient of consolidation (L^2T^{-1}).

topsoil—surface soil, usually containing organic matter.

torsional shear test—a shear test in which a relatively thin test specimen of solid circular or annular cross-section, usually confined between rings, is subjected to an axial load and to shear in torsion. In-place torsion shear tests may be performed by pressing a dentated solid circular or annular plate against the soil and measuring its resistance to rotation under a given axial load.

total stress—see **stress**.

toughness index, I_T, T_w—the ratio of: (*1*) the plasticity index, to (*2*) the flow index.

traction, S_1, S_2, S_3 (FL^{-2})—applied stress.

transformed flow net—a flow net whose boundaries have been properly modified (transformed) so that a net consisting of curvilinear squares can be constructed to represent flow conditions in an anisotropic porous medium.

transported soil—soil transported from the place of its origin by wind, water, or ice.

transverse wave, v_t (LT^{-1})—wave in which direction of displacement of element of medium is parallel to wave front. The propagation velocity, v_t, is calculated as follows:

$$v_t = \sqrt{G/\rho} = \sqrt{\mu/\rho} = \sqrt{(E/\rho)[1/2(1 + \nu)]}$$

where:

G = shear modulus,
ρ = mass density,
ν = Poisson's ratio, and
E = Young's modulus.

transverse wave (shear wave)—a wave in which the displacement at each point of the medium is parallel to the wave front. (ISRM)

tremie—material placed under water through a tremie pipe in such a manner that it rests on the bottom without mixing with the water.

trench—usually a long, narrow, near vertical sided cut in rock or soil such as is made for utility lines. (ISRM)

triaxial compression—compression caused by the application of normal stresses in three perpendicular directions. (ISRM)

triaxial shear test (triaxial compression test)—a test in which a cylindrical specimen of soil or rock encased in an impervious membrane is subjected to a confining pressure and then loaded axially to failure.

triaxial state of stress—state of stress in which none of the three principal stresses is zero. (ISRM)

true solution—one in which the components are 100 % dissolved in the base solvent.

tube A manchette—*in grouting*, a grout pipe perforated with rings of small holes at intervals of about 12 in. (305 mm).

Discussion—Each ring of perforations is enclosed by a short rubber sleeve fitting tightly around the pipe so as to act as a one-way valve when used with an inner pipe containing two packer elements that isolate a stage for injection of grout.

tunnel—a man-made underground passage constructed without removing the overlying rock or soil. Generally nearly horizontal as opposed to a shaft, which is nearly vertical. (ISRM)

turbulent flow—that type of flow in which any water particle may move in any direction with respect to any other particle, and in which the head loss is approximately proportional to the second power of the velocity.

ultimate bearing capacity, q_o, q_{ult} (FL^{-2})—the average load per unit of area required to produce failure by rupture of a supporting soil or rock mass.

unconfined compressive strength—the load per unit area at which an unconfined prismatic or cylindrical specimen of material will fail in a simple compression test without lateral support.

unconfined compressive strength—see **compressive strength**.

unconsolidated-undrained test (quick test)—a soil test in which the water content of the test specimen remains practically unchanged during the application of the confining pressure and the additional axial (or shearing) force.

undamped natural frequency—*of a mechanical system*, the frequency of free vibration resulting from only elastic and inertial forces of the system.

underconsolidated soil deposit—a deposit that is not fully consolidated under the existing overburden pressure.

undisturbed sample—a soil sample that has been obtained by methods in which every precaution has been taken to minimize disturbance to the sample.

uniaxial (unconfined) compression—compression caused by the application of normal stress in a single direction. (ISRM)

uniaxial state of stress—state of stress in which two of the three principal stresses are zero. (ISRM)

unit weight, γ (FL^{-3})—weight per unit volume (with this, and all subsequent unit-weight definitions, the use of the term weight means force).

dry unit weight (unit dry weight), γ_d, γ_e (FL^{-3})—the weight of soil or rock solids per unit of total volume of soil or rock mass.

effective unit weight, γ_e (FL^{-3})—that unit weight of a soil or rock which, when multiplied by the height of the overlying column of soil or rock, yields the effective pressure due to the weight of the overburden.

maximum unit weight, γ_{max} (FL^{-3})—the dry unit weight defined by the peak of a compaction curve.

saturated unit weight, γ_G, γ_{sat} (FL^{-3})—the wet unit weight of a soil mass when saturated.

submerged unit weight (buoyant unit weight), γ_m, γ', γ_{sub} (FL^{-3})—the weight of the solids in air minus the weight of water displaced by the solids per unit of volume of soil or rock mass; the saturated unit weight minus the unit weight of water.

unit weight of water, γ_w (FL^{-3})—the weight per unit volume of water; nominally equal to 62.4 lb/ft^3 or 1 g/cm^3.

wet unit weight (mass unit weight), γ_m, γ_{wet} (FL^{-3})—the weight (solids plus water) per unit of total volume of soil or rock mass, irrespective of the degree of saturation.

zero air voids unit weight, γ_z, γ_s (FL^{-3})—the weight of solids per unit volume of a saturated soil or rock mass.

unloading modulus—slope of the tangent to the unloading stress-strain curve at a given stress value. (ISRM)

uplift—the upward water pressure on a structure.

	Symbol	Unit
unit symbol	u	FL^{-2}
total symbol	U	F or FL^{-1}

uplift—the hydrostatic force of water exerted on or underneath a structure, tending to cause a displacement of the structure. (ISRM)

uplift—*in grouting*, vertical displacement of a formation due to grout injection.

vane shear test—an in-place shear test in which a rod with thin radial vanes at the end is forced into the soil and the resistance to rotation of the rod is determined.

varved clay—alternating thin layers of silt (or fine sand) and clay formed by variations in sedimentation during the various seasons of the year, often exhibiting contrasting colors when partially dried.

vent hole—*in grouting*, a hole drilled to allow the escape of air and water and also used to monitor the flow of grout.

vent pipe—*in grouting*, a small-diameter pipe used to permit the escape of air, water, or diluted grout from a formation.

vibrated beam wall (injection beam wall)—barrier formed by driving an H-beam in an overlapping pattern of prints and filling the print of the beam with cement-bentonite slurry or other materials as it is withdrawn.

vibration—an oscillation wherein the quantity is a parameter that defines the motion of a mechanical system (see **oscillation**).

virgin compression curve—see **compression curve**.

viscoelasticity—property of materials that strain under stress partly elastically and partly viscously, that is, whose strain is partly dependent on time and magnitude of stress. (ISRM)

viscosity—the internal fluid resistance of a substance which makes it resist a tendency to flow.

viscous damping—the dissipation of energy that occurs when a particle in a vibrating system is resisted by a force that has a magnitude proportional to the magnitude of the velocity of the particle and direction opposite to the direction of the particle.

viscous flow—see **laminar flow**.

void—space in a soil or rock mass not occupied by solid mineral matter. This space may be occupied by air, water, or other gaseous or liquid material.

void ratio, e (D)—the ratio of: (*1*) the volume of void space, to (*2*) the volume of solid particles in a given soil mass.

critical void ratio, e_c (D)—the void ratio corresponding to the critical density.

volumetric shrinkage (volumetric change), V_s (D)—the decrease in volume, expressed as a percentage of the soil mass when dried, of a soil mass when the water content is reduced from a given percentage to the shrinkage limit.

von Post humification scale—a scale describing various stages of decomposition of peat ranging from H1, which is completely undecomposed, to H10, which is completely decomposed.

wall friction, f' (FL^{-2})—frictional resistance mobilized between a wall and the soil or rock in contact with the wall.

washing—*in grouting*, the physical act of cleaning the sides of a hole by circulating water, water and air, acid washes, or chemical substances through drill rods or tremie pipe in an open hole.

water-cement ratio—the ratio of the weight of water to the weights of Portland cement in a cement grout or concrete mix. See also **grout mix**.

water content—see **moisture content**.

water gain—see **bleeding**.

water-holding capacity (D)—the smallest value to which the water content of a soil or rock can be reduced by gravity drainage.

water-plasticity ratio (relative water content) (liquidity index)—see **liquidity index**.

water table—see **free water elevation**.

wave—disturbance propagated in medium in such a manner that at any point in medium the amplitude is a function of time, while at any instant the displacement at point is function of position of point.

wave front—moving surface in a medium at which a propagated disturbance first occurs.

wave front—(*1*) a continuous surface over which the phase of a wave that progresses in three dimensions is constant, or (*2*) a continuous line along which the phase of a surface wave is constant. (ISRM)

wave length—normal distance between two wave fronts with periodic characteristics in which amplitudes have phase difference of one complete cycle.

weathering—the process of disintegration and decomposition as a consequence of exposure to the atmosphere, to chemical action, and to the action of frost, water, and heat. (ISRM)

wetland—land which has the water table at, near, or above the land surface, or which is saturated for long enough periods to promote hydrophylic vegetation and various kinds of biological activity which are adapted to the wet environment.

wetting agent—a substance capable of lowering the surface tension of liquids, facilitating the wetting of solid surfaces, and facilitating the penetration of liquids into the capillaries.

wet unit weight—see **unit weight**.

working pressure—the pressure adjudged best for any particular set of conditions encountered during grouting.

Discussion—Factors influencing the determination are size of voids to be filled, depth of zone to be grouted, lithology of area to be grouted, grout viscosity, and resistance of the formation to fracture.

yield—*in grouting*, the volume of freshly mixed grout produced from a known quantity of ingredients.

yielding arch—type of support of arch shape, the joints of which deform plastically beyond a certain critical load, that is, continue to deform without increasing their resistance. (ISRM)

yield stress—the stress beyond which the induced deformation is not fully annulled after complete destressing. (ISRM)

Young's modulus—the ratio of the increase in stress on a test specimen to the resulting increase in strain under constant transverse stress limited to materials having a linear stress-strain relationship over the range of loading. Also called elastic modulus.

zero air voids curve (saturation curve)—the curve showing the zero air voids unit weight as a function of water content.

zero air voids density (*zero air voids unit weight*)—see **unit weight.**

APPENDIX

(Nonmandatory Information)

X1. ISRM SYMBOLS RELATING TO SOIL AND ROCK MECHANICS

NOTE—These symbols may not correlate with the symbols appearing in the text.

X1.1 Space

Ω, ω	solid angle
l	length
b	width
h	height or depth
r	radius
A	area
V	volume
t	time
v	velocity
ω	angular velocity
g	gravitational acceleration

X1.2 Periodic and Related Phenomena

T	periodic time
f	frequency
ω	angular frequency
λ	wave length

X1.3 Statics and Dynamics

m	mass
ρ	density (mass density)
G_m	mass specific gravity
G_s	specific gravity of solids
G_w	specific gravity of water
F	force
T	tangential force
W	weight
γ	unit weight
γ_d	dry unit weight
γ_w	unit weight of water
γ'	buoyant unit weight
γ_s	unit of solids
T	torque
I	moment of inertia
W	work
W	energy

X1.4 Applied Mechanics

e	void ratio
n	porosity
w	water content
S_r	degree of saturation
p	pressure
u	pore water pressure
σ	normal stress
σ_x, σ_y, σ_z	stress components in rectangular coordinates
σ_1, σ_2, σ_3	principal stresses
S_1, S_2, S_3	applied stresses (and reactions)
σ_h	horizontal stress
σ_v	vertical stress
τ	shear stress
τ_{xy}, τ_{yz}, τ_{zx}	shear stress components in rectangular coordinates
ϵ	strain
ϵ_x, ϵ_y, ϵ_z	strain components in rectangular coordinates
γ_{xy}, γ_{yz}, γ_{zx}	shear strain components in rectangular coordinates
θ	volume strain
E	Young's modulus; modulus of elasticity $E = \sigma/\epsilon$
ϵ_1, ϵ_2, ϵ_3	principal strains
G	shear modulus; modulus of rigidity $G = \tau/\gamma$
c	cohesion
ϕ_μ	angle of friction between solid bodies
ϕ	angle of shear resistance (angle of internal friction)
h	hydraulic head
i	hydraulic gradient
J	seepage force per unit volume or seepage pressure per unit length
k	coefficient of permeability
η	viscosity
η_{pl}	plasticity (viscosity of Bingham body)
t_{rd}	retardation time
t_{rl}	relaxation time
T_s	surface tension
q	quantity rate of flow; rate of discharge
Q	quantity of flow
FS	safety factor

X1.5 Heat

T	temperature
β	coefficient of volume expansion

X1.6 Electricity

I	electric current
Q	electric charge
C	capacitance
L	self-inductance
R	resistance
ρ	resistivity

REFERENCES

(1) Terzaghi, *Theoretical Soil Mechanics*, John Wiley & Sons, Inc., New York, NY (1943).

(2) Terzaghi and Peck, *Soil Mechanics in Engineering Practice*, John Wiley & Sons, Inc., New York, NY (1948).

(3) Taylor, D. W., *Fundamentals of Soil Mechanics*, John Wiley & Sons, Inc., New York, NY (1948).

(4) Krynine, D. P., *Soil Mechanics*, 2nd Edition, McGraw-Hill Book Co., Inc., New York, NY (1947).

(5) Plummer and Dore, *Soil Mechanics and Foundations*, Pitman Publishing Corp., New York, NY (1940).

(6) Tolman, C. F., *Ground Water*, McGraw-Hill Book Co., Inc., New York, NY (1937).

(7) Stewart Sharpe, C. F., *Land Slides and Related Phenomena*, Columbia University Press, New York, NY (1938).

(8) "Letter Symbols and Glossary for Hydraulics with Special Reference to Irrigation." Special Committee on Irrigation Hydraulics, *Manual of Engineering Practice*, Am. Soc. Civil Engrs., No. 11 (1935).
(9) "Soil Mechanics Nomenclature," Committee of the Soil Mechanics and Foundations Division on Glossary of Terms and Definitions and on Soil Classification. *Manual of Engineering Practice*. Am. Soc. Civil Engrs., No. 22 (1941).
(10) "Pile Foundations and Pile Structures," Joint Committee on Bearing Value of Pile Foundations of the Waterways Division, Construction Division, and Soil Mechanics and Foundations Division. *Manual of Engineering Practice*, Am. Soc. Civil Engrs., No. 27 (1956).
(11) Webster's New International Dictionary of the English Language, unabridged, 2nd Edition, G. and C. Merriam Co., Springfield, MA (1941).
(12) Baver, L. D., *Soil Physics*, John Wiley & Sons, Inc., New York, NY (1940).
(13) Longwell, Knopf and Flint, "Physical Geology," *Textbook of Geology*, Part I, 2nd Edition, John Wiley & Sons, Inc., New York, NY (1939).
(14) Runner, D. G., *Geology for Civil Engineers*, Gillette Publishing Co., Chicago, IL (1939).
(15) Leggett, R. F., *Geology and Engineering*, McGraw-Hill Book Co., Inc., New York, NY (1939).
(16) Holmes, A., *The Nomenclature of Petrology*, Thomas Murby and Co., London, England (1920).
(17) Meinzer, O. E., "Outline of Ground Water Hydrology with Definitions," *U. S. Geological Survey Water Supply Paper 494* (1923).
(18) "Reports of the Committee on Sedimentation of the Division of Geology and Geography of the National Research Council," Washington, DC (1930–1938).
(19) Twenhofel, W. H., *A Treatise on Sedimentation*, 2nd Edition, Williams & Wilkins Co., Baltimore, MD (1932).
(20) Hogentogler, C. A., *Engineering Properties of Soils*, McGraw Hill Book Co., Inc., New York, NY (1937).
(21) *Special Procedures for Testing Soil and Rock for Engineering Purposes, ASTM STP 479*, ASTM, 1970.
(22) "Glossary of Terms and Definitions," Preliminary Report of Subcommittee G-3 on Nomenclature and Definitions of ASTM Committee D-18 on Soils for Engineering Purposes.
(23) Sowers and Sowers, *Introductory Soil Mechanics and Foundations*, The Macmillan Co., New York, NY (1951).
(24) Lambe, T. William, *Soil Testing for Engineers*, John Wiley & Sons, Inc., New York, NY (1951).
(25) Capper and Cassie, *The Mechanics of Engineering Soils*, McGraw-Hill Book Co., Inc., New York, NY (1949).
(26) Dunham, C. W., *Foundations of Structures*, McGraw-Hill Book Co., Inc., New York, NY (1950).
(27) Casagrande, A., "Notes on Soil Mechanics," Graduate School of Engineering, Harvard University (1938).
(28) Tschebotarioff, G. P., *Soil Mechanics, Foundations, and Earth Structures*, McGraw-Hill Book Co., Inc., New York, (1951).
(29) Rice, C. M., "Dictionary of Geological Terms," Edwards Bros., Inc., Ann Arbor, MI (1940).
(30) Creager, Justin and Hinds, *Engineering for Dams*, John Wiley & Sons, Inc., New York, NY (1945).
(31) Krumbein and Sloss, *Stratigraphy and Sedimentation*, W. H. Freeman and Co., San Francisco, CA (1951).
(32) Pettijohn, F. J. *Sedimentary Rocks*, Harper and Bros., New York, NY (1949).
(33) Reiche, Parry, *A Survey of Weathering Processes and Products*, University of New Mexico Press, Albuquerque, NM (1945).
(34) Garrels, R. M., *A Textbook of Geology*, Harper and Bros., New York, NY (1951).
(35) Ries and Watson, *Engineering Geology*, John Wiley & Sons, Inc., New York, NY (1936).
(36) Ross and Hendricks, *Minerals of the Montmorillonite Group*, U. S. Geological Survey Professional Paper 205-B (1945).
(37) Hartman, R. J., *Colloid Chemistry*, Houghton Mifflin Co., New York, NY (1947).
(38) "Frost Investigations," Corps of Engineers, Frost Effects Laboratory, Boston, MA, June 1951.
(39) "Standard Specifications for Highway Materials and Methods of Sampling and Testing," Parts I and II, adopted by the American Association of State Highway Officials (1950).
(40) Coates, D. G., "Rock Mechanics Principles," rev ed. Mines Br., Dept. Mines and Tech. Surv., Ottawa, Mines Br. Mon. 874 (1970).
(41) Gar· M., McAfee, R., Jr., and Wolf, C. L., (eds.), *Glossary of Geology*, American Geological Institute (1972).
(42) International Society for Rock Mechanics, Commission on Terminology, Symbols and Graphic Representation, Final Document on Terminology, English Version, 1972, and List of Symbols, 1970.
(43) Jaeger, J. C., and Cook, N. G. W., *Fundamentals of Rock Mec!..ics*, Methuen, London (1969).
(44) Nelson, A., and Nelson, K. D., *Dictionary of Applied Geology* (Mining and Civil Engineering), Philosophical Library, Inc., New York, NY (1967).
(45) Obert, L. A., and Duvall, W. I., *Rock Mechanics and the Design of Structures in Rock*, John Wiley & Sons, New York, NY (1967).
(46) SME Mining Engineering Handbook: Society Mining Engineers, Vol 2, New York, NY (1973).
(47) Thrush, R. P. (ed), et al., *A Dictionary of Mining, Mineral and Related Terms*, U. S. Bureau of Mines (1968).
(48) Lohman, W. W., and others, *Definitions of Selected Ground-Water Terms—Revisions and Conceptual Refinements*, U. S. Geological Survey Water-Supply Paper 1988, 21 pp., 1972.
(49) *Glossary of Soil Science Terms*, Madison, WI, Soil Science Society of America, 34 pp. (1975).
(50) *International Glossary of Hydrology*, Geneva, Switzerland, World Meterological Organization, WMO No. 385, 393 pp., (1974).

RTH 102-80

RECOMMENDED PRACTICE FOR
PETROGRAPHIC EXAMINATION OF ROCK CORES

1. Scope

1.1 This recommended practice outlines procedures for the petrographic examination of rock cores whose engineering properties can be determined by selected tests. The specific procedures employed in the petrographic examination of any sample will depend to a large extent on the purpose of the examination and the nature of the sample. Complete petrographic examination may require use of such procedures as light microscopy, x-ray diffraction analysis, differential thermal analysis, infrared spectroscopy, or others; in some instances, such procedures are more rapid and more definitive than are microscopical methods. Petrographic examinations are made for the following purposes:

(a) To determine the physical and chemical properties of a material, by petrographic methods, that have a bearing on the quality of the material for its intended use.

(b) To describe and classify the constituents of the sample. (Note 1)

(c) To determine the relative amounts of the constituents of the sample, which is essential for proper evaluation of the sample, when the constituents differ significantly in properties that have a bearing on the quality of the material for its intended use.

NOTE 1—It is recommended that the rock and mineral names in "Descriptive Nomenclature of Constituents of Natural Mineral Aggregates" (ASTM Designation: C 294) be used insofar as they are appropriate in reports prepared according to this recommended practice.

1.2 Detection of structural features and identification of the constituents of a sample are usually necessary steps toward recognition of the properties that may be expected to influence the behavior of the material in its intended use. However, the value of any petrographic examination will depend to a large extent on the representativeness of the samples examined, the completeness and accuracy of the information provided to the petrographer concerning the source and proposed use of the material, and the petrographer's ability to correlate these data with the findings of the examination.

1.3 This recommended practice does not attempt to outline the techniques of petrographic work since it is assumed that the method will be used by persons who are qualified by education and/or experience to employ such techniques for the recognition of the characteristic properties of rocks and minerals and to describe and classify the constituents of a sample. It is intended to outline the extent to which such techniques should be used, the selection of properties that should be looked for, and the manner in which such techniques may best be employed in the examination. These objectives will have been attained if engineers responsible for the application of the results of petrographic examinations have reasonable assurance that such results, wherever and whenever obtained, may confidently be compared.

2. Sampling and Examination Procedure

2.1 The purpose in specifying a sampling procedure is to ensure the selection of an adequate group of test specimens from each mechanically different rock type that forms an essential part of the core or cores that will be tested. The sampling procedure should also be guided by the project objectives and directed at obtaining the properties of the rock that eventually will comprise the roof, side walls, foundation, or other specific parts of the project structure. Samples for petrographic examination should be taken by or under the direct supervision of a geologist familiar with the requirements of the project. The exact location from which the sample was taken, the geology of the site, and other pertinent data should be submitted with the sample. The amount of material actually studied in the petrographic examination will be determined by the nature of the material to be examined. Areas to be studied should be sampled by means of cores drilled through the entire depth required for project investigation. Drilling of such cores should be in a direction that is essentially normal to the dominant structural feature of the rock. Massive material may be sampled by "NX" (2-1/8-in.-diam) (54-mm-diam) cores. Thinly bedded or complex material should be represented by cores not less than 4 in. (101.6 mm) in diameter, preferably 6 in. (152.4 mm). There should be an adequate number of cores to cover the limits of the rock mass under consideration.

2.2 The following is considered a preferable but not a mandatory procedure. A petrographer should inspect all of the rock core before any tests are made. Each core should be logged to show footage of core recovered, core loss, and location; location and spacing of fractures and parting planes; lithologic type or types; alternation of types; physical conditions and variations in conditions; toughness, hardness, coherence; obvious porosity; grain size, texture, variations in grain size and texture; type or types of breakage. If the surface of the core being examined is wetted, it is usually easier to recognize significant features and changes in lithology. Most of the information usually required can be obtained by careful visual examination, scratch and acid tests, and hitting the core with a hammer. A preliminary analysis of the test results may indicate that the results from one or another of the subdivisions are not significantly different and the groups may be combined. On the other hand, the analysis may disclose significant differences within a given group of specimens and a further subdivision may be required. Most rock is anisotropic and, if the core stock and sample procedure permit, a group of specimens should be obtained from the three mutually perpendicular directions. Usually these directions are oriented with respect to some petrographic property of the rocks such as bedding, schistosity, cleavage, or fabric. In bedded rock the greatest difference in properties occurs in specimens taken perpendicular and parallel to the bedding, and generally this type of rock is sampled only in these two directions. The petrographic examination may disclose mineral components that are soluble or that expand or soften in water, as for example, bentonites or other clays. The intent of this inspection is to provide a basis for the selection of samples for engineering tests. This basis will be rock types, amounts of rock types, differences within a rock type, etc. At this point, the petrographer, in conjunction with the project leader, should select the sections of core(s) that will be subjected to engineering tests. The detailed petrographic examination will usually be made on unused portions of some or all of the test pieces so that the petrographic data can be matched to the engineering data. This matching should mean that, in addition to petrographic characterization, the petrographic data should serve as a basis for understanding the physical test data within and between sample groups.

3. Apparatus and Supplies

3.1 The apparatus and supplies listed in the following subparagraphs (a) and (b) comprise a recommended selection which will permit the use of all of the procedures described in this recommended practice. All specific items have been used in connection with the performance of petrographic examinations by the procedures described herein; it is not, however, intended to imply that other items cannot be substituted to serve similar functions. Whenever possible the selection of particular apparatus and supplies should be left to the judgment of the petrographer who is to perform the work so that items obtained will be those with which he has the greatest experience and familiarity. The minimum equipment regarded as essential to the making of petrographic examinations are those items, or equivalent apparatus or supplies that will serve the same purpose, that are indicated by asterisks in the lists in subparagraphs (a) and (b).

(a) Apparatus and Supplies for Preparation of Specimens:

(1) Rock-Cutting Saw,* preferably with a 20-in.-(508-mm-) diam blade.

(2) Horizontal Grinding Wheel,* preferably 16 in. (406.4 mm) in diameter.

(3) Polishing Wheel, preferably 8 to 12 in. (203.2 to 304.8 mm) in diameter.

(4) Abrasives:* silicon carbide grit Nos. 100, 220, 320, 600, and 800; optical finishing alumina.

(5) Prospector's Pick.

(6) Microscope Slides,* clear, noncorrosive, 25 by 45 mm in size.

(7) Canada Balsam,* neutral, in xylene or other material to cement cover slips.

(8) Xylene.*

(9) Mounting Medium,* suitable for mounting rock slices for thin sections (preferably flexibilized epoxy).

(10) Laboratory Oven.*

(11) Plate-Glass Squares,* about 12 in. (304.8 mm) on an edge for thin-section grinding.

(12) Micro Cover Glasses,* No. 1 noncorrosive, square, 12 to 18 mm, 25 mm, etc.

(13) Plattner mortar.

(b) Apparatus and Supplies for Examination of Specimens:

(1) Polarizing Microscope* with mechanical stage; low-, medium-, and high-power objectives, and objective centering devices; eyepieces of various powers; full- and quarter-wave compensators; quartz wedge.

(2) Microscope Lamps* (preferably including a sodium arc lamp).

(3) Stereoscopic Microscope* with objectives and oculars to give final magnifications from about 7X to about 140X.

(4) Magnet,* preferably Alnico, or an electromagnet.

(5) Needleholder and Points.*

(6) Dropping Bottles, 60-ml capacity.

(7) Forceps, smooth, straight-pointed.

(8) Lens Paper.*

(9) Immersion Media,* n = 1.410 to n = 1.785 in steps of 0.005. (Note 2)

(10) Counter.

(11) Photomicrographic Camera and accessories. (Note 3)

(12) X-ray diffraction equipment.

(13) Differential thermal analysis equipment.

(14) Infrared absorption spectrometer equipment.

NOTE 2--It is necessary that facilities be available to the petrographer to check the index of refraction of the immersion media. If accurate identification of materials is to be attempted, as for example the differentiation of quartz and chalcedony or the differentiation of basic from intermediate volcanic glass, the indices of refraction of the media need to be known with precision. Media will not be stable for very long periods of time and are subject to considerable variation due to temperature change. In laboratories not provided with close temperature control, it is often necessary to recalibrate immersion media several times during the course of a single day when accurate identifications are required. The equipment needed for checking immersion media consists of and Abbe Refractometer. The refractometer should be equipped with compensating prisms to read indices for sodium light from white light, or it should be used with a sodium arc lamp.

NOTE 3--It is believed that a laboratory that undertakes any considerable amount of petrographic work should be provided with facilities to make photomicrographic records of such features as cannot adequately be described in words. Photomicrographs can be taken using standard microscope lamps for illumination; however, it is recommended that whenever possible a zirconium arc lamp be provided for this purpose. For illustrations of typical apparatus, reference may be made to the paper by Mather and Mather.[1]

---

[1] This recommended practice is modified from the "Method of Petrographic Examination of Aggregates for Concrete," by Katharine Mather and Bryant Mather. Proceedings, American Society for Testing Materials, ASTEA, Vol 50, 1950, pp 1288-1312.

4. Report

4.1 First and foremost the report should be clear and useful to the engineer for whom it is intended. It should identify samples, give their source as appropriate, describe test procedures and equipment used as appropriate, describe the samples, and list the petrographic findings. Tabulations of data and photographs should be included as needed. Results that may bear on the engineering test data and the potential performance of the material should be clearly stated and their significance should be emphasized. It may also be appropriate to mention past performance records of the same or similar materials if such information is available. In general, the report should be an objective statement. If any opinion is presented it should be clearly indicated to be an opinion. Finally, the petrographic report should make recommendations if and as appropriate.

PREPARATION OF TEST SPECIMENS

1. Scope

1.1 In order to obtain valid results from tests on brittle materials, careful and precise specimen preparation is required. This method outlines preparatory procedures recommended for normal rock mechanics test progress.

2. Collection and Storage

2.1 Test material is normally collected from the field in the form of drilled cores. Field sampling procedures should be rational and systematic, and the material should be marked to indicate its original position and orientation relative to identifiable boundaries of the parent rock mass. Ideally, samples should be moistureproofed immediately after collection either by waxing, spraying, or packing in polyethylene bags or sheet. (Example: For moistureproofing by waxing, the following procedure can be used for core that will not fall apart in handling:

(a) Wrap core in a clear thin polyethylene such as GLAD WRAP or SARAN WRAP, or

(b) Wrap in cheese cloth.

(c) Coat wrapped core with a lukewarm wax mixture to an approximate 1/4-in. (6.4-mm) thickness. The wax should consist of a 1 to 1 mixture of paraffin and microcrystalline wax, such as Sacony Vacuum Mobil Wax No. 2300 and 2305, Gulf Oil Corporation Petrowax A, and Humble Oil Company Microvan No. 1650.

Cores that could easily be broken by handling should be prepared using the soil sampling technique described on pages 4-20 and 4-21 of EM 1110-2-1907, 31 March 1972[9.1]). They should be transported as a fragile material and protected from excessive changes in humidity and temperature. The identification markings of all samples should be verified immediately upon their receipt at the laboratory, and an inventory of the samples received should be maintained. Samples should be examined and tested as soon as possible after receipt; however, it is often necessary to store samples for several days or even weeks to complete a large testing program. Every care must be taken to protect stored samples against damage. Core logs of samples should be available.

3. Avoidance of Contamination

3.1 The deformation and fracture properties of rock may be influenced by air, water, and other fluids in contact with their internal (crack and pore) surfaces. If these internal surfaces are contaminated by oils or other substances, their properties may be altered appreciably and give misleading test results. Of course, a cutting fluid is required with many types of specimen preparation equipment. Clean water is the preferred fluid. Even so, one must be cognizant of the effect of moisture on the test specimens. While it may be impossible to exactly duplicate the in situ conditions even if they were known, a concerted effort should be made to simulate the environment from which the samples came. Generally there are three conditions to be considered:

(a) Hard, dense rock of low porosity will not normally be affected by moisture. This type of material is normally allowed to air-dry prior to testing to bring all samples to an equilibrium condition. Drying at temperatures above 120°F (49°C) is not recommended as excessive heat may cause an irreversible change in rock properties.

(b) Some shales and rocks containing clay will disintegrate if allowed to dry. Usually the disintegration of diamond drill cores can be prevented by wrapping the cores as they are drilled in a moistureproof material such as aluminum foil or chlorinated rubber, or sealing them in moistureproof containers.

(c) Mud shales and rock containing bentonites (e.g., tuff) may soften if the moisture content is too high. Most of the softer rocks can be cored or cut using compressed air to clear cuttings and to cool the bit or saw.

It is imperative to determine very early in the test program the moisture sensitivity of all types of material to be tested and to take steps to accommodate the requirements throughout the test life of the selected specimens.

4. Selection

4.1 Under the most favorable circumstances, a laboratory determination of the engineering properties of a small specimen gives an approximate guide to the behavior of an extensive, nonhomogeneous geological formation under the complex system of induced stresses. No other aspect of laboratory rock testing is as important as the selection of test specimens to best represent those features of a foundation which influence the analysis or design of a project. Closest teamwork of the laboratory personnel and the project engineer/geologist must be continued throughout the testing program since, as quantitative data become available, changes in the initial allocation of samples or the securing of additional samples may be necessary. Second in importance only to the selection of the most representative undisturbed material is the preparation and handling of the test specimens to preserve in every way possible the natural structure of the material. Indifferent handling of undisturbed rock can result in erroneous test data.

5. Coring

5.1 Virtually all laboratory coring is done with thin-wall diamond rotary bits, which may be detachable or integral to the core barrel. The usual size range for laboratory core drills is from 6-in.-diam (152.4-mm) down to 1-in. (25.4-mm) outside diameter. Typical sample diameters for uniaxial testing are 2.125 in. (54 mm). Drilling machines range from small quarry drills to modified machine shop drill presses. Almost any kind can be adapted for rock work by fitting a water swivel, but a heavy, rigid machine is desirable in order to assure consistent production of high quality core. The work block must be clamped tightly to a strong base or table so as to prevent any tilting, oscillation, or other shifting. To avoid unnecessary unclamping and rearrangement of the work block, it is desirable to have provision for traversing the drill head or the work block. Traversing devices must lock securely to eliminate any play between drill and work. The drill travel should be sufficient to permit continuous runs of at least 10 to 12 in. (254 to 304.8 mm), without need for stopping the machine. Optimum drilling speeds vary with bit size and rock type, and to some extent with condition of the bit and the characteristics of the machine. The general trend is that drill speed increases as drill diameter decreases; also, higher drill speeds are sometimes used on softer rocks. The broad range of drill speeds lies mainly between 200 and 2,000 rpm. No hard-and-fast rules can be given, but an experienced operator can easily choose a suitable speed by trial. Some core drills are hand-fed, but it is desirable to have some provision for automatic feed. The ideal feed arrangement is a constant-force hydraulic feed which can be set for each bit size and rock type, but such machines are quite rare. Constant-force feed can be improvised by means of a weight and pulley arrangement. On adapted metal-working drill presses, the automatic feed rate for a given drill size and rock type can be determined; however, since there is a danger of damaging the machine or the core barrel if too high a feed rate is used, an electrical overload breaker should be provided.

6. Sawing

6.1 For heavy sawing, a slabbing saw is adequate for most purposes. For exact sawing, a precision cutoff machine, with a diamond abrasive wheel about 10 in. (254 mm) in diameter, and a table with two-way screw traversing and provision for rotation are recommended. The speed of the wheel is usually fixed, but the feed rate of the wheel through the work can be controlled. Clean water, either direct from house supply or recirculated through a settling tank, is the standard cutting and cooling fluid. For crosscutting, core should be clamped in a vee-block slotted to permit passage of the wheel. By supporting the core on both sides of the cut, the problem of spalling and lip formation at the end of the cut is largely avoided. Saw cuts should be relatively smooth and perpendicular to the core axis in order to minimize the grinding or lapping needed to produce end conditions required for the various tests.

7. End Preparation

7.1 Due to the rather large degree of flatness required on bearing surfaces for many tests, end grinding or lapping is required. Conventional surface grinders provide the most practical means of preparing flat surfaces, especially on core samples with diameters greater than approximately 2 in. (50.8 mm). Procedures are essentially comparable to metal working. Quite often a special jig is constructed to hold one or more specimens in the grinding operation. The lathe can also be used for end-grinding cylindrical samples. A sample is held directly in the chuck, rotated at 200 to 300 rpm, and the grinding wheel, its axis inclined some 15 deg (0.26 radian) to the sample axis, is passed across end of the sample while rotating at 6,000 to 8,000 rpm. The "bite" ranges from about 0.003 in. (0.0762-mm) maximum to less than 0.001 in. (0.0254 mm) for finishing, and the grinding wheel is passed across the sample at about 0.5 in. (12.7 mm) per minute. For core diameters of 2-1/8 in. (54 mm) or less, a lap can be used for grinding flat end surfaces on specimens, although producing a sufficiently flat surface by this method is an art. To end-grind on the lap, a cylindrical specimen is placed in a steel-carrying tube which is machined to accept core with a clearance of about 0.002 in. (0.0508 mm). At the lower end of this tube is a steel collar which rests on the lapping wheel. The method requires use of grinding compounds and, hence, is not recommended where other methods are available.

8. Specimen Check

8.1 In general terms, test specimens should be straight, their diameter should be constant, and the ends should be flat, parallel, and normal to the long axis. Sample dimensions should be checked during machining with a micrometer or vernier caliper; final dimensions are normally measured with a micrometer and reported to the nearest 0.01 in. (0.254 mm). Tolerances are best checked on a comparator fitted with a dial micrometer reading to 0.0001 in. (0.00254 mm). There is a technique for revealing the roughness and planes qualitatively. Impressions are made by sandwiching a sheet of carbon paper and a sheet of white paper between the sample end and a smooth surface. The upper end of the sample is given a light blow with a rubber or plastic hammer, and an imprint is formed on the white paper. Areas where no impressions are made indicate dished or uneven surfaces. The importance of proper specimen preparation cannot be overemphasized. Specimens should not be tested which do not meet the dimensional tolerances specified in the respective test methods.

9. References

9.1 Department of the Army, Office, Chief of Engineers, "Soil Sampling," EM 1110-2-1907, Washington, D. C., 1972.

RTH 104-80

STATISTICAL CONSIDERATIONS

1. Scope

1.1 The purpose of this recommended practice is to outline some general statistical concepts which may be applied to small sample sizes typical of rock test data. It is not the intent to deal with accuracy or precision considerations, but rather to assess the results from the assumed point that the test has been conducted as specified in the respective test methods.

2. Variation and Sample Size

2.1 Most physical tests involve tabulation of a series of readings, with computation of an average said to be representative of the whole. The question arises as to how representative this average is as the measure of the characteristic under test. Three important factors introduce uncertainties in the result:

(a) Instrument and procedural errors.

(b) Variations in the sample being tested.

(c) Variations between the sample and the other samples that might have been drawn from the same source.

If a number of identical specimens were available for tests, or if the tests were nondestructive and could be repeated a number of times on the same specimen, determination of the procedural and instrument errors would be comparatively simple, because in such a test the sample variation would be zero. Periodic tests on this specimen or group of specimens could be used to check the performance of the test procedure and equipment. However, as most of the tests used in determining the mechanical properties of rock are destructive, the instrument and sample variations cannot be separated. Nevertheless, we may apply some elementary statistical concepts and still have confidence in the test results. Of course, the more test data available the more reliable the results. Due to the expense of testing and occasionally the shortage of test specimens, rock test data almost always require treatment as groups of small samples. As a general rule at least 10 tests are recommended for any one condition of each individual test with an absolute minimum of 5.

3. Measures of Central Tendency and Deviation

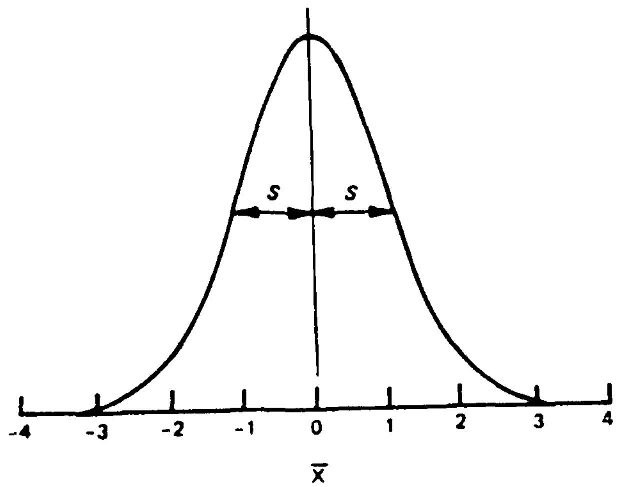
3.1 The most commonly used measure of the central tendency of a sample of n specimens is the arithmetic mean or average, X. The standard deviation, s, of the sample S is a measure of the sample variability. The range, r, is defined as the difference between the highest and lowest of the n values. For small samples ($3 \leq n \leq 12$)

$$s = r/\sqrt{n}$$

When more than 12 bits of data are available, more refined calculations are required for computation of the standard deviation.

4. The Normal Distribution

4.1 A series of tests for any one property will, of course, have some variation in the individual determinations. The distribution of these bits of data quite often follows a pattern of normal distribution, i.e., a large portion of the data bits will be closely grouped to the mean on either side with progressively fewer bits distributed farther from the mean. The normal distribution is usually portrayed graphically as shown below.



Methods are available to check the normality of the distribution and to deal with those which are not normal (skewed).[7.1-7.3]

5. Small Sampling Theory

5.1 A study of sampling distributions of statistics for small samples ($n < 30$) is called small sampling theory. Statistical tables are available for use with the proper relationship to develop confidence in test data.[7.1] For small samples the confidence limits for a population are given by:

$$\bar{X} \pm t_c \frac{s}{\sqrt{n-1}}$$

$\bar{X}$, n , and s are determined as given in paragraph 3.1. t_c is called the confidence coefficient and depends on the level of confidence desired and the sample size. Values of t_c for 90, 95, and 99 percent confidence limits are given below for n from 3 to 12.

		t_c, %		
n	DF*	90	95	99
3	2	2.92	4.30	9.92
4	3	2.35	3.18	5.84
5	4	2.13	2.78	4.60
6	5	2.02	2.57	4.03
7	6	1.94	2.45	3.71
8	7	1.90	2.36	3.50
9	8	1.86	2.31	3.36
10	9	1.83	2.26	3.25
11	10	1.81	2.23	3.17
12	11	1.80	2.20	3.11

* Degrees of freedom

Statistical treatment of very small samples, i.e. $n < 7$, is questionable and of limited value.

6. Example

6.1 For the purposes of illustration, assume there is a normal distribution of compressive strength data[7.4] for a particular rock type yielding the following individual specimen strengths: 18,000; 18,700; 19,200; 19,600; 20,000; 20,100; 20,500; 20,800; 21,100; and 22,000 psi. Find the 95 percent confidence limits for the mean strength.

By computation:

$$n = 10$$

$$\bar{X} = 20{,}000 \text{ psi}$$

$$r = 22{,}000 - 18{,}000 = 4000 \text{ psi}$$

$$S = \frac{r}{\sqrt{n}} = \frac{4000}{3.162} = 4000 \text{ psi}$$

$$95\% \text{ confidence limits} = \bar{X} \pm t_{95}\ (s/\sqrt{n-1});\ \text{thus,}$$

$$\bar{X} \pm 2.26\ (1260/\sqrt{10-1}) =$$

$$20{,}000 \pm 2.26\ (420) = 20{,}000 \pm 950 \text{ psi}$$

Thus, we can be 95 percent confident that the true mean lies between 19,050 and 20,950 psi.

6.2 Also, observations can be expected to fall within some range of the mean. If a new effect is added, or a bias occurs, deviations from the mean that would be expected to occur with extremely small frequency under normal circumstances will be observed. When such deviations occur they may be discarded on the grounds that (a) they represent the effect of variables that are not under study, or (b) they are the result of absolute errors and do not represent the actual observations, or (c) the large deviation can be considered the result of a combination of factors that is not representative of usual operating conditions. The confidence coefficient information can be used to provide a measure of confidence in the individual test data. The relation is:

$$\text{Confidence range (90, 95, 99, etc.)} = \bar{X} \pm t_c s$$

$$\text{For the example: Confidence range (95\%)} = 20{,}000 \pm 2.26\ (1260)$$
$$= 17{,}150 \text{ to } 22{,}850 \text{ psi}$$

Thus, we can say that there is only 1 chance in 20 of observing a breaking strength as low as 17,150 psi or as high as 22,850 psi by using the test method employed on this particular method. If a strength determination is obtained above or below this range, there is a high probability that additional factors come into play and the extraneous strength value may thus be considered nonrepresentative of the material.

7. References

7.1 Spiegel, M. R., Theory and Problems of Statistics, Schaun's Outline Series, McGraw-Hill Company, New York, 1961.

7.2 Volk, William, Applied Statistics for Engineers, McGraw-Hill Company, New York, 1958.

7.3 Obert, Leonard and Duvall, W. I., Rock Mechanics and the Design of Structures in Rock, John Wiley and Sons, Inc., New York, 1967.

7.4 "Engineering Geology; Special Issue, Uniaxial Testing in Rock Mechanics Laboratories," Elsevier Publishing Company, Amsterdam, Vol 4, No. 3, July 1970.

METHOD FOR DETERMINATION OF REBOUND NUMBER OF ROCK

1. Scope

1.1 This method provides instructions for the determination of a rebound number of rock using a spring-driven steel hammer (Fig. 1).

1 Impact plunger
3 Housing compl.
4 Rider with guide rod
5 Scale (starting with serial No. 230 printed on window No. 19)
6 Pushbutton compl.
7 Hammer guide bar
8 Disk
9 Cap
10 Two-part ring
11 Rear cover
12 Compression spring
13 Pawl
14 Hammer mass
15 Retaining spring
16 Impact spring
17 Guide sleeve
18 Felt washer
19 Plexiglass window
20 Trip screw
21 Lock nut
22 Pin
23 Pawl spring

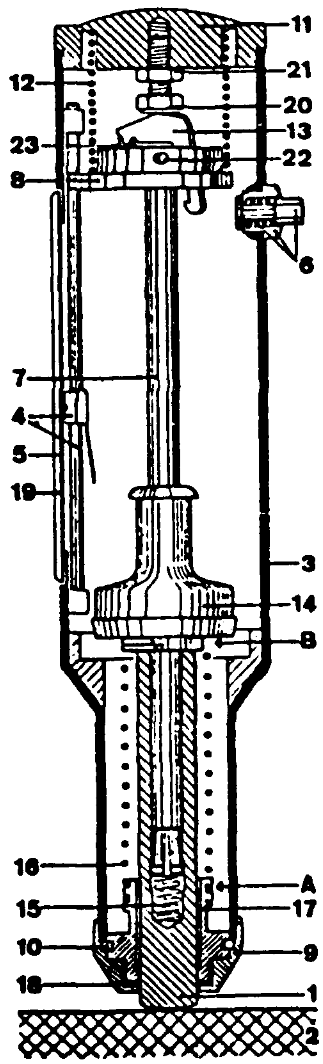


Fig. 1. Spring-driven steel hammer.

2. Significance

2.1 The rebound number determined by this method may be used to assess the uniformity of rock in situ, or on cored samples to indicate hardness characteristics of the rock.

3. Apparatus

3.1 Rebound Hammer - The rebound hammer consists of a spring-loaded steel hammer which when released strikes a steel plunger which is in contact with the test surface. The spring-loaded hammer must travel with a fixed and reproducible velocity. The rebound distance of the steel hammer from the steel plunger is measured by means of a linear scale attached to the frame of the instrument (Note 1).

NOTE 1--Several types and sizes of rebound hammers are commercially available. Hammers with an energy impact of 0.075 m-kg (0.542 ft-lb) have been found satisfactory for rock testing.

3.2 Abrasive Stone - An abrasive stone consisting of medium grain texture silicon carbide or equivalent material shall be provided.

4. Test Area

4.1 Selection of Test Surface - Surfaces to be tested shall be at least 2 in. (50 mm) thick and fixed within a stratum. Specimens should be rigidly supported. Some companies market a "rock cradle" for this purpose. Areas exhibiting scaling, rough texture, or high porosity should be avoided. Dry rocks give higher rebound numbers than wet.

4.2 Preparation of Test Surface - Heavily textured soft surfaces or surfaces with loose particles shall be ground smooth with the abrasive stone described in Section 3.2. Smooth surfaces shall be tested without grinding. The effects of drying and carbonation can be minimized by thoroughly wetting the surfaces for 24 hours prior to testing.

4.3 Factors Affecting Test Results - Other factors related to test circumstances may affect the results of the test:

(a) Rock at 32°F (0°C) or less may exhibit very high rebound values. Rock should be tested only after it has thawed.

(b) The temperature of the rebound hammer itself may affect the rebound number (Note 2).

NOTE 2—Rebound hammers at 0°F (-18°C) may produce rebound numbers reduced by as much as 2 or 3 units.

(c) For readings to be compared, the direction of impact—horizontal, downward, upward, etc.—must be the same.

(d) Different hammers of the same nominal design may give rebound numbers differing by 1 to 3 units, and therefore, to be compared, tests should be made with a single hammer. If more than one hammer is to be used, a sufficient number of tests must be made on typical rock surfaces to determine the magnitude of the differences to be expected. (Note 3)

NOTE 3—Rebound hammers require periodic servicing and verification annually for hammers in heavy use, biennially for hammers in less frequent use, and whenever there is reason to question their proper operation. Metal anvils are available for verification and are recommended. However, verification on an anvil will not guarantee that different hammers will yield the same results at other points on the rebound scale. Some users compare several hammers on surfaces encompassing the usual range of rebound values encountered in the field.

5. Test Procedure

5.1 The instrument shall be firmly held in a position which allows the plunger to strike perpendicular to the surface tested. The pressure on the plunger shall be gradually increased until the hammer impacts. After impact, the rebound number should be recorded to two significant figures. Ten readings shall be taken from each test area with no two impact tests being closer together than 1 in. (25.4 mm). Examine the impression made on the surface after impact and disregard the reading if the impact crushes or breaks through the surface.

6. Calculation

6.1 Readings differing from the average of 10 readings by more than 7 units are to be discarded and the average of the remaining readings determined. If more than 2 readings differ from the average by 7 units, the entire set of readings should be discarded.

7. Precision

7.1 The single-specimen-operator-machine-day precision is 2.5 units (1S) as defined in ASTM Recommended Practice E 177, "Use of the Terms Precision and Accuracy as Applied to Measurement of a Property of a Material."

8. Use and Interpretation of Rebound Hammer Results

8.1 Optimally, rebound numbers may be correlated with core testing information. There is a relationship between rebound number and strength and deformation and the relationship is normally provided by the rebound hammer manufacturer.

9. Report

9.1 The report should include the following information for each test area:

9.1.1 Rock identification.

9.1.2 Location of rock stratum.

9.1.3 Description and composition of rock if known.

9.1.4 Average rebound number for each test area or specimen.

9.1.5 Approximate angular direction of rebound hammer impact, with horizontal being considered 0, vertically upward being +90 deg (1.57 radians), and vertically downward being -90 deg (1.57 radians).

9.1.6 Hammer type and serial number.

RTH 106-80

METHOD FOR DETERMINATION OF THE WATER CONTENT OF A ROCK SAMPLE

1. Scope

1.1 This test is intended to measure the weight of water contained in a rock sample as a percentage of the ovendry sample weight. The water content, or moisture content, is designated by the symbol w.

2. Apparatus

(a) An oven capable of maintaining a temperature of $105^{\circ}C$ to within $3^{\circ}C$ for a period of at least 24 hours.

(b) A sample container of noncorrodible material, including an airtight lid.

(c) A desiccator to hold sample containers during cooling.

(d) A balance of adequate capacity, capable of weighing to an accuracy of 0.01 percent of the sample weight.

3. Procedure

(a) The container with its lid is cleaned and dried, and its weight X measured.

(b) A representative sample is selected, preferably comprising at least ten rock lumps each weighing at least 50 g to give a total sample weight of at least 500 g. For in situ water content determination, sampling, storage, and handling precautions should retain water content to within 1 percent of its in situ value.

(c) The sample is placed in the container, the lid replaced, and the weight Y of sample plus container determined.

(d) The lid is removed and the sample dried to constant weight at a temperature of $105^{\circ}C$.

(e) The lid is replaced and the sample allowed to cool in the desiccator for 30 minutes. The weight Z of sample plus container is determined.

4. Calculation

$$\text{Water content } w = \frac{\text{pore water weight}}{\text{grain weight}} \cdot 100\% = \frac{Y-Z}{Z-X} \cdot 100\%$$

5. Reporting of Results

5.1 The water content should be reported to the nearest 0.1 percent stating whether this corresponds to in situ water content, in which case precautions taken to retain water during sampling and storage should be specified.

STANDARD METHOD OF TEST FOR
SPECIFIC GRAVITY AND ABSORPTION OF ROCK

1. Scope

1.1 This method covers the determination of bulk and apparent specific gravities, 73.4/73.4°F (23/23°C), and absorption of rock samples. This test method is based on ASTM Designation C 127-73.

1.2 This method determines (after 24 hours in water) the bulk specific gravity and the apparent specific gravity as defined in RTH 101 and absorption as defined in ASTM Designation C 125, "Definitions of Terms Relating to Concrete and Concrete Aggregates."

2. Apparatus

2.1 Balance - A balance or scale having a capacity of 5 kg or more, sensitive to 0.5 g or less, and accurate within 0.1 percent of the test load at any point within the range used for this test. Within any 500 g range of test load, a difference between readings shall be accurate within 0.5 g.

2.2 Sample Container - A wire basket of No. 5 (3 mm) or finer mesh, or a bucket, of approximately equal breadth and height, with a capacity of 4000 to 7000 cm^3.

3. Sample

3.1 Select a representative sample of the rock to be tested not to exceed 5 kg.

4. Procedure

4.1 After thoroughly washing to remove dust or other coatings from the surface of the particles, dry the sample to constant weight at a temperature of 212 to 230°F (100 to 110°C), cool in air at room temperature for 1 to 3 hours, and then immerse in water at room temperature for a period of 24 ± 4 hours. (Note 1)

NOTE 1—Where the absorption and specific gravity values in the sample's naturally moist condition are specified, the requirement for initial drying to constant weight may be eliminated, and if the surfaces of the particles in the sample have been kept continuously wet until test, the 24-hour soaking may also be eliminated. Values for absorption and for specific gravity in the saturated-surface-dry condition may be significantly higher for rock not ovendried before soaking. Therefore, any exceptions to the procedure of Section 4.1 should be noted in reporting the results.

4.2 Remove the sample from the water and roll it in a large absorbent cloth until all visible films of water are removed. Wipe the larger particles individually. Take care to avoid evaporation of water from pores during the operation of surface-drying. Weigh the sample in the saturated-surface-dry condition. Record this and all subsequent weights to the nearest 0.5 g.

4.3 After weighing, immediately place the saturated-surface-dry sample in the sample container and determine its weight in water at 73.4 ± 3°F (23 ± 1.7°C), having a density of 0.997 ± 0.002 g per cc. Take care to remove all entrapped air before weighing by shaking the container while it is immersed. (Note 2)

NOTE 2--The container should be immersed to a depth sufficient to cover it and the test sample during weighing. Wire suspending the container should be of the smallest practical size to minimize any possible effects of a variable immersed length.

4.4 Dry the sample to constant weight at a temperature of 212 to 230°F (100 to 110°C), cool in air at room temperature for 1 to 3 hours, and weigh.

5. Bulk Specific Gravity

5.1 Calculate the bulk specific gravity, 73.4/73.4°F (23/23°C), as follows:

$$\text{Bulk sp gr} = G_m = \frac{A}{B-C}$$

where A = weight of ovendry sample in air, g

B = weight of saturated-surface-dry sample in air, g

C = weight of saturated sample in water, g

6. Bulk Specific Gravity (Saturated-Surface-Dry Basis)

6.1 Calculate the bulk specific gravity, 73.4/73.4°F (23/23°C), on the basis of weight of saturated-surface-dry rock as follows:

$$\text{Bulk sp gr (saturated-surface-dry basis)} = \frac{B}{B-C}$$

7. Apparent Specific Gravity

7.1 Calculate the apparent specific gravity, 73.4/73.4°F (23/23°C), as follows:

$$\textit{Apparent sp gr} = G_a = \frac{A}{A-C}$$

8. Absorption

8.1 Calculate the percentage of absorption as follows:

$$\text{Absorption, percent} = \frac{B-A}{A} \cdot 100$$

9. Precision

9.1 Data from carefully conducted tests on normal weight rock at one laboratory yielded the following for tests on the same sample. Different samples from the same source may vary more.

9.1.1 For specific gravity, single-operator and multioperator precision (2S limits) less than ±0.01 from the average specific gravity. Differences greater than 0.01 between duplicate tests on the same sample by the same or different operators should occur by chance less than 5 percent of the time (D2S limit less than 0.01).

9.1.2 For absorption, single-operator and multioperator precision ±0.09 from the average percent absorption 95 percent of the time (2S limits). The difference between single tests by the same or different operators on the same sample should not exceed 0.13 more than 5 percent of the time (D2S limit).

METHOD OF DETERMINING DENSITY OF SOLIDS

1. Scope and Definition

1.1 This method covers procedures for determining the density of solids. The density of solids is the ratio of the mass in air of a given volume of crushed solids to the total volume of solids.

2. Apparatus

2.1 The apparatus shall consist of the following:

(a) Volumetric flask, 500-mL capacity.

(b) Vacuum pump or aspirator connected to vacuum line.

(c) Oven of the forced draft type, automatically controlled to maintain a uniform temperature of 110 ± 5°C throughout the oven.

(d) Balance, sensitive to 0.01 g, capacity 500 g or more.

(e) Thermometer, range 0 to 50°C, graduated in 0.1 deg. C.

(f) Evaporating dish.

(g) Water bath.

(h) Sieves, U.S. Standard 4.75-mm (No. 4) and 600-μm (No. 30) conforming to ASTM Designation E 11, "Specifications for Wire-Cloth Sieves for Testing Purposes."

(i) Sample splitter suitable for splitting material passing 4.75-mm (No. 4) and 600-μm (No. 30) sieves.

3. Calibration of Volumetric Flask

3.1 The volumetric flask shall be calibrated for the mass of the flask and water at various temperatures. The flask and water are calibrated by direct determination of mass at the range of temperatures likely to be encountered in the laboratory. The calibration procedure is as follows.

3.2 Fill the flask with de-aired, distilled, and demineralized water to slightly below the calibration mark and place in a water bath which is at a temperature between 30 and 35°C. Allow the flask to remain in the bath until the water in the flask reaches the temperature of the water bath. This may take several hours. Remove the flask from the water bath and adjust the water level in the flask so that the bottom of the meniscus is even with the calibration mark on the neck of the flask. Thoroughly dry the outside of the flask and remove any water adhering to the inside of the neck above the graduation, then determine the mass of the flask and water to the nearest 0.01 g. Immediately after determination of mass, shake the flask gently and determine the temperature of the water to the nearest 0.1°C by immersing a thermometer to the middepth of the flask. Repeat the procedure outlined above at approximately the same temperature, then make two more determinations, one at room temperature and the other at approximately 5°C less than room temperature. Draw a calibration curve showing the relation between temperature and corresponding values of mass of the flask plus water. Prepare a calibration curve for each flask used for density determination and maintain the curves as a permanent record. A typical calibration curve is shown in Fig. 1.

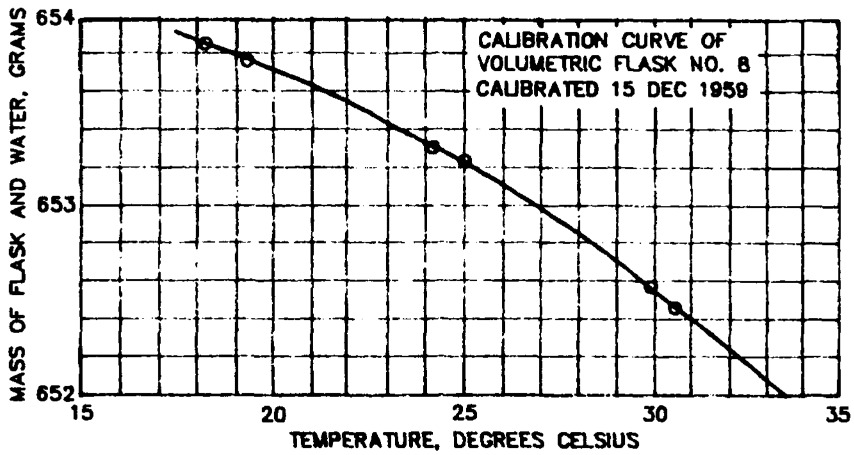


Fig. 1. **Typical calibration curve of volumetric flask.**

4. Sample

4.1 Crush the sample until it all passes a 4.75-mm (No. 4) sieve. With a sample splitter, separate out 120 to 150 g of representative crushed material. Pulverize this material to pass a 600-μm (No. 30) sieve. Oven-dry the crushed material to constant mass, determine the mass of the material to the nearest 0.01 g, and record the mass.

5. Procedure

5.1 After determination of mass, transfer the crushed material to a volumetric flask, taking care not to lose any material during this operation. To reduce possible error due to loss of material of known mass, the sample may have its mass determined after transfer to the flask. Fill the flask approximately half full with de-aired, distilled water. Shake the mixture well and allow it to stand overnight.

5.2 Then connect the flask to the vacuum line and apply a vacuum of approximately 99.99 Pa (750 mm of mercury) for approximately 4 to 6 h, agitating the flask at intervals during the evacuation process. Again allow the flask to stand overnight. Finally, fill the flask with de-aired, distilled water to about 3/4 in. (19 mm) below the 500-mL graduation and again apply a vacuum to the flask until the suspension is de-aired, slowly and carefully remove the stopper from the flask, and observe the lowering of the water surface in the neck. If the water surface is lowered less than 1/8 in. (3.2 mm), the suspension can be considered sufficiently de-aired. Fill the flask until the bottom of the meniscus is coincident with the calibration line of the neck of the flask. Thoroughly dry the outside of the flask and remove the moisture on the inside of the neck by wiping with a paper towel. Determine the mass of the flask and contents to the nearest 0.01 g. Immediately after determination of mass, stir the suspension to assure uniform temperature, and determine the temperature of the suspension to the nearest 0.1°C by immersing a thermometer to the middepth of the flask. Record the mass and temperature.

5.3 Compute the density of the solid, G_s, from the following formula:

$$G_s = \frac{W_s \quad \gamma_w}{W_s + W_{fw} - W_{fws}}$$

where W_s = the ovendry mass of the crushed rock sample, g

γ_w = density of water at test temperature, M /m³ g (Table 1)

W_{fw} = mass of flask plus water at test temperature g (from calibration curve, Fig. 1)

W_{fws} = mass of flask plus water plus solids at test temperature, g

6. Report

6.1 The report shall include the following:

(a) The density of the solid.

(b) Ovendry mass of test sample.

(c) Water temperature during test.

METHOD OF DETERMINING EFFECTIVE (AS RECEIVED) AND DRY UNIT WEIGHTS AND TOTAL POROSITY OF ROCK CORES

1. Scope and Definition

1.1 This method covers the procedure for determining the effective unit weight, dry unit weight, and porosity of rock cores as defined in RTH 101. (This method covers determination of "total" porosity of a rock sample. Porosity calculated from the bulk volume and grain volume using the pulverization method is termed "total" since the pore volume obtained includes that of all closed pores. Other techniques give "effective" porosity since they measure the volume of interconnected pores only.)

2. Apparatus

2.1 The apparatus shall consist of the following:

(a) Balance having a capacity of 5 kg or more and accurate to 1 g.

(b) Wire basket of No. 4 mesh, diameter at least 2 in. (50.8 mm) greater than that of the core to be tested, walls at least one-half the height of the cylinder, and bail clearing the top of the core by at least 1 in. (25.4 mm) at all points.

(c) Watertight container in which the wire basket may be suspended with a constant-level overflow spout at such a height that the wire basket, when suspended below the spout, will be at least 1 in. (25.4 mm) from the bottom of the container.

(d) Suspending apparatus suitable for suspending the wire basket in the container from the center of the balance platform or pan so that the basket will hang completely below the overflow spout and not be less than 1 in. (25.4 mm) from the bottom of the container.

(e) Thermometer, range 0 to 50°C, graduated to 0.1°.

(f) Caliper or suitable measuring device capable of measuring lengths and diameters of test cores to the nearest 0.1 mm.

(g) Oven of the forced draft type, automatically controlled to maintain a uniform temperature of 110 ± 5°C throughout.

3. Sample

3.1 Select representative samples from the population and identify each sample. Individual samples should not exceed 5 kg in weight.

4. Effective Unit Weight (As Received)

4.1 The test procedure for determining the effective unit weight of rock cores shall consist of the following steps:

(a) Weigh the core as received to the nearest gram (0.1 g for 3-in. (76.2 -mm) and smaller cores) (W_a) and record this weight and the temperature in the working area near the core surface.

(b) Determine the bulk volume of the core in cubic centimetres by one of the following two methods:

(1) Determine the average length and diameter of the core from measurements of each of these dimensions at evenly spaced intervals covering the surface of the specimen. These measurements should be made to the nearest 0.1 mm. Calculate the volume using the formula $V = \frac{\pi}{4} d^2 L$, where V = volume, d = diameter of the core, and L = length of the core. (Note 1)

NOTE 1--If this method is used, the specimen should be sawed and machined to conform closely to the shape of a right cylinder or prism prior to weighing as in 4.1(a) above.

(2) Coat the surface of the core with wax or other suitable coating until it is watertight, making sure that the coating material does not measurably penetrate the pores of the core. Weigh the specimen, after coating, to the nearest gram (or 0.1 g). The density of the coating material shall be determined. The volume of the coating on the core shall be determined by dividing the weight of coating by the density of the coating. Determine the volume of the coated core in cubic centimetres by liquid displacement. Subtract the volume of the coating material from the volume of the coated core to obtain the volume of the core (V) in cubic centimetres.

(c) Calculate the effective unit weight of the core from the following formula:

$$\gamma_e = \frac{W_a}{V}$$

where γ_e = effective unit weight of the core, as received
W_a = weight of the core, in grams, as received
V = volume of the core, in cubic centrimetres

5. Dry Unit Weight

5.1 The test procedure for determining the dry unit weight of rock cores shall consist of the following steps:

(a) If a coating was utilized to waterproof the specimen as in 4.1(b)(2), remove it and, if applicable, brush to remove dust or elements of the coating. Then weigh the core. (Note 2)

NOTE 2--If there is no weight loss in stripping or loss or gain in moisture, this weight should equal W_a.

(b) Crush the sample until it all passes a No. 4 sieve, taking care not to lose any material.

(c) Oven-dry the crushed material to constant weight W_b (constant weight is achieved when the weight does not change as much as 0.1 percent during any 4-hour drying period), cool to room temperature, then record all weights and room temperature in the area of the test on the data sheet.

(d) Calculate the dry unit weight of the core from the following formula:

$$\gamma_d = \frac{W_b}{V}$$

where γ_d = dry unit weight of the core
W_b = weight of the crushed, dried core, in grams
V = volume of the core, in cubic centimetres

6. Porosity

6.1 The total porosity, n , may be determined from the dry unit weight and the gram unit weight of a sample. Determine the specific of the solids, G_s , according to RTH 108. Determine the grain unit weight of the sample by the relation:

$$\text{Grain unit weight, } \gamma_g = G_s \cdot 623 \text{ lb/cu ft}$$

Determine total porosity by:

$$n = \frac{100(\gamma_g - \gamma_d)}{\gamma_g}\%$$

where n = total porosity

γ_g = grain unit weight

γ_d = dry unit weight

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Method for
LABORATORY DETERMINATION OF PULSE VELOCITIES AND ULTRASONIC ELASTIC CONSTANTS OF ROCK[1]

This standard is issued under the fixed designation D 2845; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This method describes equipment and procedures for laboratory measurements of the pulse velocities of compression waves and shear waves in rock (**1**)[2] (Note 1) and the determination of ultrasonic elastic constants (Note 2) of an isotropic rock or one exhibiting slight anisotropy.

NOTE 1—The compression wave velocity as defined here is the dilatational wave velocity. It is the propagation velocity of a longitudinal wave in a medium which is effectively infinite in lateral extent. It should not be confused with the bar or rod velocity.

NOTE 2—The elastic constants determined by this method are termed ultrasonic since the pulse frequencies used are above the audible range. The terms sonic and dynamic are sometimes applied to these constants but do not describe them precisely (**2**). It is possible that the ultrasonic elastic constants may differ from those determined by other dynamic methods.

1.2 This method is valid for wave velocity measurements in both anisotropic and isotropic rocks although the velocities obtained in grossly anisotropic rocks may be influenced by such factors as direction, travel distance, and diameter of transducers.

1.3 The ultrasonic elastic constants are calculated from the measured wave velocities and the bulk density. The limiting degree of anisotropy for which calculations of elastic constants are allowed and procedures for determining the degree of anisotrophy are specified.

1.4 The values stated in U.S. customary units are to be regarded as the standard. The metric equivalents of U.S. customary units may be approximate.

2. Summary of Method

2.1 Details of essential procedures for the determination of the ultrasonic velocity, measured in terms of travel time and distance, of compression and shear waves in rock specimens include requirements of instrumentation, suggested types of transducers, methods of preparation, and effects of specimen geometry and grain size. Elastic constants may be calculated for isotropic or slightly anisotropic rocks, while anisotropy is reported in terms of the variation of wave velocity with direction in the rock.

3. Significance

3.1 The primary advantages of ultrasonic testing are that it yields compression and shear wave velocities, and ultrasonic values for the elastic constants of intact homogeneous isotropic rock specimens (**3**). Elastic constants are not to be calculated for rocks having pronounced anisotropy by procedures described in this method. The values of elastic constants often do not agree with those determined by static laboratory methods or the *in situ* methods. Measured wave velocities likewise may not agree with seismic velocities, but offer good approximations. The ultrasonic evaluation of rock properties is useful for preliminary prediction of static properties. The method is useful for evaluating the effects of uniaxial stress and water saturation on pulse velocity. These properties are in turn useful in engineering design.

3.2 The method as described herein is not adequate for measurement of stress-wave attenuation. Also, while pulse velocities can be em-

[1] This method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock.

Current edition approved Aug. 17, 1983. Published October 1983. Last previous edition D 2845 – 69 (1976).

[2] The bold face numbers in parentheses refer to the list of references at the end of this method.

ployed to determine the elastic constants of materials having a high degree of anisotropy, these procedures are not treated herein.

4. Apparatus

4.1 *General*—The testing apparatus (Fig. 1) should have impedance matched electronic components and shielded leads to ensure efficient energy transfer. To prevent damage to the apparatus allowable voltage inputs should not be exceeded.

4.2 *Pulse Generator Unit*—This unit shall consist of an electronic pulse generator and external voltage or power amplifiers if needed. A voltage output in the form of either rectangular pulse or a gated sine wave is satisfactory. The generator shall have a voltage output with a maximum value after amplification of at least 50 V into a 50-Ω impedance load. A variable pulse width, with a range of 1 to 10 μs is desirable. The pulse repetition rate may be fixed at 60 repetitions per second or less although a range of 20 to 100 repetitions per second is recommended. The pulse generator shall also have a trigger-pulse output to trigger the oscilloscope. There shall be a variable delay of the main-pulse output with respect to the trigger-pulse output, with a minimum range of 0 to 20 μs.

4.3 *Transducers*—The transducers shall consist of a transmitter which converts electrical pulses into mechanical pulses and a receiver which converts mechanical pulses into electrical pulses. Environmental conditions such as ambient temperature, moisture, humidity, and impact should be considered in selecting the transducer element. Piezoelectric elements are usually recommended, but magnetostrictive elements may be suitable. Thickness-expander piezoelectric elements generate and sense predominately compression-wave energy; thickness-shear piezoelectric elements are preferred for shear-wave measurements. Commonly used piezoelectric materials include ceramics such as lead-zirconate-titanate for either compression or shear, and crystals such as a-c cut quartz for shear. To reduce scattering and poorly defined first arrivals at the receiver, the transmitter shall be designed to generate wavelengths at least three times the average grain size of the rock.

NOTE 3—Wavelength is the wave velocity in the rock specimen divided by the resonance frequency of the transducer. Commonly used frequencies range from 75 kHz to 3 MHz.

4.3.1 In laboratory testing, it may be convenient to use unhoused transducer elements. But if the output voltage of the receiver is low, the element should be housed in metal (grounded) to reduce stray electromagnetic pickup. If protection from mechanical damage is necessary, the transmitter as well as the receiver may be housed in metal. This also allows special backings for the transducer element to alter its sensitivity or reduce ringing **(4)**. The basic features of a housed element are illustrated in Fig. 2. Energy transmission between the transducer element and test specimen can be improved by (*1*) machining or lapping the surfaces of the face plates to make them smooth, flat, and parallel, (*2*) making the face plate from a metal such as magnesium whose characteristic impedance is close to that of common rock types, (*3*) making the face plate as thin as practicable, and (*4*) coupling the transducer element to the face plate by a thin layer of an electrically conductive adhesive, an epoxy type being suggested.

4.3.2 Pulse velocities may also be determined for specimens subjected to uniaxial states of stress. The transducer housings in this case will also serve as loading platens and should be designed with thick face plates to assure uniform loading over the ends of the specimen **(5)**.

NOTE 4—The state of stress in many rock types has a marked effect on the wave velocities. Rocks *in situ* are usually in a stressed state and therefore tests under stress have practical significance.

4.4 *Preamplifier*—A voltage preamplifier is required if the voltage output of the receiving transducer is relatively low or if the display and timing units are relatively insensitive. To preserve fast rise times, the frequency response of the preamplifier shall drop no more that 2 dB over a frequency range from 5 kHz to 4 × the resonance frequency of the receiver. The internal noise and gain must also be considered in selecting a preamplifier. Oscilloscopes having a vertical-signal output can be used to amplify the signal for an electronic counter.

4.5 *Display and Timing Unit*—The voltage pulse applied to the transmitting transducer and the voltage output from the receiving transducer shall be displayed on a cathode-ray oscilloscope for visual observation of the waveforms. The oscilloscope shall have an essentially flat response between a frequency of 5 kHz and 4 × the resonance frequency of the transducers. It shall have dual beams or dual traces so that the two waveforms may be displayed simultaneously and their amplitudes separately controlled. The oscilloscope shall be triggered by a triggering pulse from the pulse generator. The timing unit shall be capable of measuring intervals between 2 μs and 5 ms to an accuracy of 1 part in 100. Two alternative classes of timing units are suggested, the respective positions of each being shown as dotted outlines in the block diagram in Fig. 1: (*1*) an electronic counter with provisions for time interval measurements, or (*2*) a time-delay circuit such as a continuously variable-delay generator, or a delayed-sweep feature on the oscilloscope. The travel-time measuring circuit shall be calibrated periodically with respect to its accuracy and linearity over the range of the instrument. The calibration shall be checked against signals transmitted by the National Bureau of Standards radio station WWV, or against a crystal controlled time-mark or frequency generator which can be referenced back to the signals from WWV periodically. It is recommended that the calibration of the time measuring circuit be checked at least once a month and after any severe impact which the instrument may receive.

5. Test Specimens

5.1 *Preparation*—Exercise care in core drilling, handling, sawing, grinding, and lapping the test specimen to minimize the mechanical damage caused by stress and heat. It is recommended that liquids other than water be prevented from contacting the specimen, except when necessary as a coupling medium between specimen and transducer during the test. The surface area under each transducer shall be sufficiently plane that a feeler gage 0.001 in. (0.025 mm) thick will not pass under a straightedge placed on the surface. The two opposite surfaces on which the transducers will be placed shall be parallel to within 0.005 in./in. (0.1 mm/20 mm) of lateral dimension (Fig. 3). If the pulse velocity measurements are to be made along a diameter of a core, the above tolerance then refers to the parallelism of the lines of contact between the transducers and curved surface of the rock core. Moisture content of the test specimen can affect the measured pulse velocities (see 6.2). Pulse velocities may be determined on the velocity test specimen for rocks in the oven-dry state (0 % saturation), in a saturated condition (100 % saturation), or in any intermediate state. If the pulse velocities are to be determined with the rock in the same moisture condition as received or as existing underground, care must be exercised during the preparation procedure so that the moisture content does not change. In this case it is suggested that both the sample and test specimen be stored in moisture-proof bags or coated with wax and that dry surface-preparation procedures be employed. If results are desired for specimens in the oven-dried condition, the oven temperature shall not exceed 150°F (66°C). The specimen shall remain submerged in water up to the time of testing when results are desired for the saturated state.

5.2 *Limitation on Dimensions*—It is recommended that the ratio of the pulse-travel distance to the minimum lateral dimension not exceed 5. Reliable pulse velocities may not be measurable for high values of this ratio. The travel distance of the pulse through the rock shall be at least 10 × the average grain size so that an accurate average propagation velocity may be determined. The grain size of the rock sample, the natural resonance frequency of the transducers, and the minimum lateral dimension of the specimen are interrelated factors which affect test results. The wavelength corresponding to the dominant frequency of the pulse train in the rock is approximately related to the natural resonance frequency of the transducer and the pulse-propagation velocity, (compression or shear) as follows:

$$\lambda \approx V/f, \qquad (1)$$

where:

λ = dominant wavelength of pulse train, in. (or m),

V = pulse propagation velocity (compression or shear), in./s (or m/s), and

f = natural resonance frequency of transducers, Hz.

The minimum lateral dimension of the test specimen shall be at least 5 × the wavelength of the compression wave so that the true dilational wave velocity is measured (Note 5), that is,

$$D \geq 5\lambda, \qquad (2)$$

where:

D = minimum lateral dimension of test specimen, in. (or m).

The wavelength shall be at least 3 × the average grain size (See 4.3) so that

$$\Lambda \geq 3d, \quad (3)$$

where:

d = average grain size, in. (or m).

Equations 1, 2, and 3 can be combined to yield the relationship for compression waves as follows:

$$D \geq 5(V_p/f) \geq 15\ d, \quad (4)$$

where:

V_p = pulse propagation velocity (compression), in./s (or m/s).

Since V_p and d are inherent properties of the material, f and D shall be selected to satisfy Eq 4 (Fig. 4) for each test specimen. For any particular value of V_p/f the permissible values of specimen diameter D lie above the diagonal line in Fig. 4, while the permissible values of grain size d lie below the diagonal line. For a particular diameter, the permissible values for specimen length L lie to the left of the diagonal line.

NOTE 5—Silaeva and Shamina **(6)** found the limiting ratio of diameter to wavelength to be about 2 for metal rods. Data obtained by Cannady **(3)** on rock indicate the limiting ratio is at least 8 for a specimen length-to-diameter ratio of about 8.

6. Procedure

6.1 *Determination of Travel Distance and Density*—Mark off the positions of the transducers on the specimen so that the line connecting the centers of the transducer contact areas is not inclined more than 2° (approximately 0.1 in. in 3 in. (1 mm in 30 mm)) with a line perpendicular to either surface. Then measure the pulse-travel distance from center to center of the transducer contact area to within 0.1 %. The density of the test specimen is required in the calculation of the ultrasonic elastic constants (see 7.2). Determine the density of the test specimen from measurements of its mass and its volume calculated from the average external dimensions. Determine the mass and average dimensions within 0.1 %. Calculate the density as follows:

$$\rho = m/V$$

where:

ρ = density, lb sec^2/in.^{4} (or kg/m^3),

m = mass of test specimen, lb sec^2/386.4 in. (or kg), and

V = volume of test specimen, in^3 (or m^3).

6.2 *Moisture Condition*—The moisture condition of the sample shall be noted and reported as 8.1.3.

6.3 *Determination of Pulse-Travel Time:*

6.3.1 Increase the voltage output of the pulse generator, the gain of the amplifier, and the sensitivity of the oscilloscope and counter to an optimum level, giving a steeper pulse front to permit more accurate time measurements. The optimum level is just below that at which electromagnetic noise reaches an intolerable magnitude or triggers the counter at its lowest triggering sensitivity. The noise level shall not be greater than one tenth of the amplitude of the first peak of the signal from the receiver. Measure the travel time to a precision and accuracy of 1 part in 100 for compression waves and 1 part in 50 for shear waves by (*1*) using the delaying circuits in conjunction with the oscilloscope (see 6.1.1) or (*2*) setting the counter to its highest usable precision, (see 6.3.2).

6.3.1.1 The oscilloscope is used with the time-delay circuit to display both the direct pulse and the first arrival of the transmitted pulse, and to measure the travel time. Characteristically, the *first arrival displayed on the oscilloscope consists* of a curved transition from the horizontal zero-voltage trace followed by a steep, more or less linear, trace. Select the first break in a consistent manner for both the test measurement and the zero-time determination. Select it either at the beginning of the curved transition region or at the zero-voltage intercept of the straight line portion of the first arrival.

6.3.1.2 The counter is triggered to start by the direct pulse applied to the transmitter and is triggered to stop by the first arrival of the pulse reaching the receiver. Because a voltage change is needed to trigger the counter, it can not accurately detect the first break of a pulse. To make the most accurate time interval measurements possible, increase the counter's triggering sensitivity to an optimum without causing spurious triggering by extraneous electrical noise.

6.3.2 Determine the zero time of the circuit including both transducers and the travel-time measuring device and apply the correction to the measured travel times. This factor will remain constant for a given rock and stress level if the circuit characteristics do not change. Determine the zero time accordingly to detect any changes. Determine it by (*1*) placing the transducers in direct contact with each other and measuring the delay time directly, or (2) measuring the apparent travel time of some uniform material (such as steel) as a function of length, and then using the zero-length intercept of the line through the data points as the correction factor.

6.3.3 Since the first transmitted arrival is that of the compression wave, its detection is relatively easy. The shear-wave arrival, however, may be obscured by vibrations due to ringing of the transducers and reflections of the compression wave. The amplitude of the shear wave relative to the compression wave may be increased and its arrival time determined more accurately by means of thickness shear-transducer elements. This type of element generates some compressional energy, so that both waves may be detected. Energy transmission between the specimen and each transducer may be improved by using a thin layer of a coupling medium such as phenyl salicylate, high-vacuum grease, or resin, and by pressing the transducer against the specimen with a small seating force.

6.3.4 For specimens subjected to uniaxial stress fields, first arrivals of compression waves are usually well defined. However, the accurate determination of shear-wave first arrivals for specimens under stress is complicated by mode conversions at the interfaces on either side of the face plate and at the free boundary of the specimen **(4)**. Shear-wave arrivals are therefore difficult to determine and experience is required for accurate readings.

6.4 *Ultrasonic Elastic Constants*—The rock must be isotropic or possess only a slight degree of anisotropy if the ultrasonic elastic constants are to be calculated (Section 7). In order to estimate the degree of anisotropy of the rock, measure the compression-wave velocity in three orthogonal directions, and in a fourth direction oriented at 45° from any one of the former three directions if required as a check. Make these measurements with the same geometry, that is, all between parallel flat surfaces or all across diameters. The equations in 7.2 for an isotropic medium shall not be applied if any of the three compression-wave velocities varies by more than 2 % from their average value. The error in E and G (see 7.2) due to both anisotropy and experimental error will then normally not exceed 6 %. The maximum possible error in μ, λ, and K depends markedly upon the relative values of V_p and V_s as well as upon testing errors and anisotropy. In common rock types the respective percent of errors for μ, λ, and K may be large as or even higher than 24, 36, and 6. For greater anisotropy, the possible percent of error in the elastic constants would be still greater.

7. Calculations

7.1 Calculate the propagation velocities of the compression and shear waves, V_p and V_s respectively, as follows:

$$V_p = L_p/T_p$$

$$V_s = L_s/T_s$$

where:

V = pulse-propagation velocity, in./s (or m/s),

L = pulse-travel distance, in. (or m),

T = effective pulse-travel time (measured time minus zero time correction), s,

and subscripts $_p$ and $_s$ denote the compression wave and shear wave, respectively.

7.2 If the degree of velocity anisotropy is 2 % or less, as specified in 6.4, calculate the ultrasonic elastic constants as follows:

$$E = [\rho V_s^2(3V_p^2 - 4V_s^2)]/(V_p^2 - V_s^2)$$

where:

E = Young's modulus of elasticity, psi (or Pa), and

ρ = density, lb/in.^{3} (or kg/m^3);

$$G = \rho V_s^2$$

where:

G = modulus of rigidity or shear modulus, psi (or Pa);

$$\mu = (V_p^2 - 2V_s^2)/[2(V_p^2 - V_s^2)]$$

where:

μ = Poisson's ratio;

$$\lambda = \rho(V_p^2 - 2V_s^2)$$

where:

λ = Lamé's constant, psi (or Pa); and

$$K = \rho(3V_p^2 - 4V_s^2)/3$$

where:

K = bulk modulus, psi (or Pa).

8. Report

8.1 The report shall include the following:

8.1.1 Identification of the test specimen including rock type and location,

8.1.2 Density of test specimen,

8.1.3 General indication of moisture condition of sample at time of test such as as received, saturated, laboratory air dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation.

8.1.4 Degree of anisotropy expressed as the maximum percent deviation of compression-pulse velocity from the average velocity determined from measurements in three directions,

8.1.5 Stress level of specimens,

8.1.6 Calculated pulse velocities for compression and shear waves with direction of measurement,

8.1.7 Calculated ultrasonic elastic constants (if desired and if degree of anisotropy is not greater than specified limit),

8.1.8 Coupling medium between transducers and specimen, and

8.1.9 Other data such as physical properties, composition, petrography, etc., if determined.

9. Precision and Accuracy

9.1 At present, data for determining the precision and accuracy of ultrasonic pulse velocity measurements are not available.

9.2 Inaccuracies in the calculation of the ultrasonic elastic constants occur when the rock is anisotropic (see 6.4).

REFERENCES

(1) Simmons, Gene, "Ultrasonics in Geology," *Proceedings, Inst. Electrical and Electronic Engineers*, Vol 53, No. 10, 1965, pp. 1337–1345.

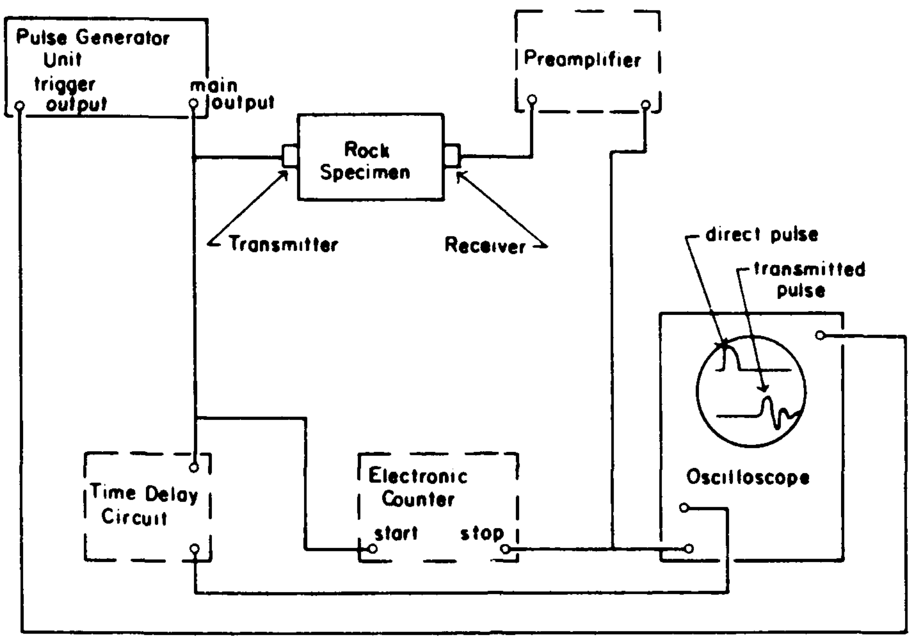
(2) Whitehurst, E. A., *Evaluation of Concrete Properties from Sonic Tests*, Am. Concrete Inst., Detroit, Mich., and the Iowa State Univ. Press, Ames, Iowa, 1966, pp. 1–2.

(3) Cannaday, F. X., "Modulus of Elasticity of a Rock Determined by Four Different Methods," *Report of Investigations U.S. Bureau of Mines 6533*, 1964.

(4) Thill, R. E., McWilliams, J. R., and Bur, T. R., "An Acoustical Bench for an Ultrasonic Pulse System," *Report of Investigations U.S. Bureau of Mines 7164*, 1968.

(5) Gregory, A. R., "Shear Wave Velocity Measurements of Sedimentary Rock Samples under Compression," *Rock Mechanics*, Pergamon Press, New York, N.Y., 1963, pp. 439–471.

(6) Silaeva, O. I., and Shamina, O. G., "The Distribution of Elastic Pulses in Cylindrical Specimens," *USSR Academy of Sciences (Izvestiya), Geophysics Series*, 1958, pp. 32–43, (English ed., Vol 1, No. 1, 1958, pp. 17–24).



NOTE—Components shown by dashed lines are optional, depending on method of travel-time measurement and voltage sensitivity of oscilloscope.

FIG. 1 Schematic Diagram of Typical Apparatus

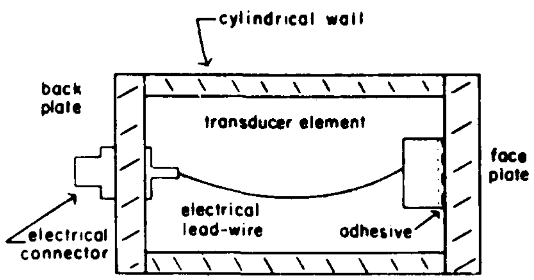
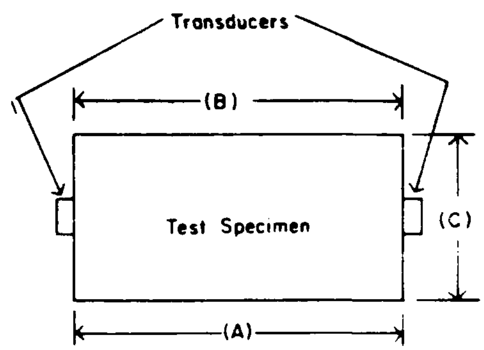


FIG. 2 Basic Features of a Housed Transmitter or Receiver



NOTE—(A) must be within 0.1 mm of (B) for each 20 mm of width (C).

FIG. 3 Specification for Parallelism

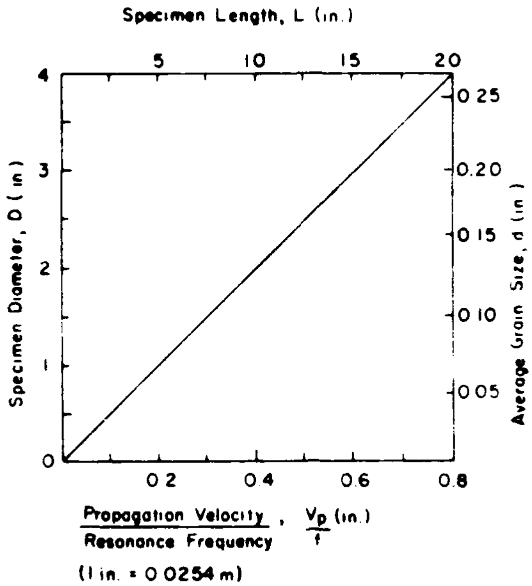


FIG. 4 Graph Showing Allowable Values of Specimen Diameter and Grain Size Versus the Ratio of Propagation Velocity to Resonance Frequency

Designation: D 2938 - 86 RTH 111-89

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa. 19103
Reprinted from the Annual Book of ASTM Standards. Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Test Method for UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMENS[1]

This standard is issued under the fixed designation D 2938; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This test method covers the determination of the unconfined compressive strength of intact cylindrical rock specimens.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

D 4543 Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances[2]

E 4 Practices for Load Verification of Testing Machines[3]

E 122 Recommended Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[4]

3. Apparatus

3.1 *Loading Device*, to apply and measure axial load on the specimens, of sufficient capacity to apply load at a rate conforming to the requirements set forth in 5.3. It shall be verified at suitable time intervals in accordance with the procedures given in Practices E 4, and comply with the requirements prescribed therein.

3.2 *Bearing Surfaces*—The testing machine shall be equipped with two steel bearing blocks having a Rockwell hardness of not less than HRC 58 (Note 1). One of the blocks shall be spherically seated and the other a plain rigid block. The bearing faces shall not depart from a plane by more than 0.0005 in. (0.013 mm) when the blocks are new and shall be maintained within a permissible variation of 0.001 in. (0.025 mm). The diameter of the spherically seated bearing face shall be at least as large as that of the test specimen but shall not exceed twice the diameter of the test specimen. The center of the sphere for the spherically seated block shall coincide with the bearing face of the specimen. The movable portion of the bearing block shall be held closely in the spherical seat, but the design shall be such that the bearing face can be rotated and tilted through small angles in any direction.

NOTE 1—False platens, with plane bearing faces conforming to the requirements of this method may be used. These shall consist of disks about ½ to ¾ in. (12.7 to 19.05 mm) thick, oil-hardened to more than HRC 58, and surface ground. With abrasive rocks, these platens tend to roughen after a number of specimens have been tested, and hence need to be resurfaced from time to time.

4. Test Specimens

4.1 Prepare test specimens in accordance with Practice D 4543.

4.2 The moisture condition of the specimen

[1] This method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Oct. 31, 1986. Published December 1986. Originally published as D 2938 – 71. Last previous edition D 2938 – 79.

[2] *Annual Book of ASTM Standards*, Vol 4.08.

[3] *Annual Book of ASTM Standards*, Vols 03.01, 04.01, 07.01, and 08.03.

[4] *Annual Book of ASTM Standards*, Vol 14.02.

at time of test can have a significant effect upon the indicated strength of the rock. Good practice generally dictates that laboratory tests be made upon specimens representative of field conditions. Thus it follows that the field moisture condition of the specimen should be preserved until time of test. On the other hand, there may be reasons for testing specimens at other moisture contents including zero. In any case, tailor the moisture content of the test specimen to the problem at hand and report it in accordance with 7.1.7.

5. Procedure

5.1 Check the ability of the spherical seat to rotate freely in its socket before each test.

5.2 Wipe clean the bearing faces of the upper and lower bearing blocks and of the test specimen and place the test specimen on the lower bearing block. As the load is gradually brought to bear on the specimen, adjust the movable portion of the spherically seated block so that uniform seating is obtained.

5.3 Many rock types fail in a violent manner when loaded to failure in compression. A protective shield should be placed around the test specimen to prevent injury from flying rock fragments.

5.4 Apply the load continuously and without shock to produce an approximately constant rate of load or deformation such that failure will occur within 5 to 15 min of loading.

NOTE 2—Results of tests by several investigators have shown that strain rates within this range will provide strength values that are reasonably free of rapid loading effects and reproducible within acceptable tolerances.

6. Calculation

6.1 Calculate the unconfined compressive strength of the specimen by dividing the maximum load carried by the specimen during the test by the cross-sectional area calculated and express the result to the nearest 10 psi (68.9 kPa).

7. Report

7.1 The report should include the following:

7.1.1 Source of sample including project name and location, and, if known, storage environment. The location is frequently specified in terms of the borehole number and depth of specimen from collar of hole.

7.1.2 Physical description of sample including rock type; location and orientation of apparent weakness planes, bedding planes, and schistosity; large inclusions or inhomogeneities, if any.

7.1.3 Date of sampling and testing.

7.1.4 Specimen diameter and length, conformance with dimensional requirements (see Note 4).

7.1.5 Rate of loading or deformation rate.

7.1.6 General indication of moisture condition of sample at time of test such as as-received, saturated, laboratory air dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation.

7.1.7 Unconfined compressive strength for each specimen as calculated average unconfined compressive strength of all specimens tested, standard deviation or coefficient of variation.

7.1.8 Type and location of failure. A sketch of the fractured sample is recommended.

7.1.9 Other available physical data.

NOTE 3—If only cores with a length-to-diameter ratio (L/D) of less than 2 are available, these may be used for the test and this fact noted in the report. However, a length-to-diameter ratio (L/D) as close to 2 as possible should be used. This apparent compressive strength may then be corrected in accordance with the following equation:

$$C = C_a/(0.88 + (0.24b/h))$$

where:

C = computed compressive strength of an equivalent L/D = 2 specimen,
C_a = measured compressive strength of the specimen tested,
b = test core diameter, and
h = test core height.

NOTE 4—The number of specimens tested may depend upon availability of specimens, but normally a minimum of ten is preferred. The number of specimens tested should be indicated. The statistical basis of relating the required number of specimens to the variability of measurements is given in Recommended Practice E 122.

8. Precision and Bias

8.1 The variability of rock and resultant in-

ability to determine a true reference value prevent development of a meaningful statement of bias. Data are being evaluated to determine the precision of this test method. In addition, the subcommittee is seeking pertinent data from users of the method.

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa. 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition.

Standard Test Method for DIRECT TENSILE STRENGTH OF INTACT ROCK CORE SPECIMENS[1]

This standard is issued under the fixed designation D 2936; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This test method covers the determination of the direct tensile strength of intact cylindrical rock specimens.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Applicable Documents

2.1 *ASTM Standards:*

E 4 Methods of Load Verification of Testing Machines[2]

E 122 Recommended Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[3]

3. Significance and Use

3.1 Rock is much weaker in tension than in compression. Thus, in determining the failure condition for a rock structure, many investigators employ the tensile strength of the component rock as the failure strength for the structure. Direct tensile stressing of rock is the most basic test for determining the tensile strength of rock.

4. Apparatus

4.1 *Loading Device*, to apply and measure axial load on the specimen, of sufficient capacity to apply the load at a rate conforming to the requirements of 6.2. The device shall be verified at suitable time intervals in accordance with the procedures given in Methods E 4 and shall comply with the requirements prescribed therein.

4.2 *Caps*—Cylindrical metal caps that, when cemented to the specimen ends, provide a means through which the direct tensile load can be applied. The diameter of the metal caps shall not be less than that of the test specimen, nor shall it exceed the test specimen diameter by more than 0.0625 in. (1.6 mm). Caps shall have a thickness of at least 1¼ in. (32 mm). Caps shall be provided with a suitable linkage system for load transfer from the loading device to the test specimen. The linkage system shall be so designed that the load will be transmitted through the axis of the test specimen without the application of bending or torsional stresses. The length of the linkages at each end shall be at least two times the diameter of the metal end caps. One such system is shown in Fig. 1.

NOTE 1—Roller or link chain of suitable capacity has been found to perform quite well in this application. Because roller chain flexes in one plane only, the upper and lower segments should be positioned at right angles to each other to effectively reduce bending in the specimen. Ball-and-socket, cable, or similar arrangements have been found to be generally unsuitable as their tendency for bending and twisting makes the assembly unable to transmit a purely direct tensile stress to the test specimen.

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Sept. 28, 1984. Published November 1984. Originally published as D 2936 – 71. Last previous edition D 2936 – 71.

[2] *Annual Book of ASTM Standards*, Vols 04.02, 07.01, and 08.03.

[3] *Annual Book of ASTM Standards*, Vol 14.02.

5. Test Specimens

5.1 Test specimens shall be right circular cylinders within the following tolerances:

5.1.1 The sides of the specimen shall be generally smooth and free of abrupt irregularities with all the elements straight to within 0.020 in. (0.50 mm) over the full length of the specimen. The deviation from straightness of the elements shall be determined by either Method A or Method B as follows:

5.1.1.1 *Method A*—Roll the cylindrical specimen on a smooth flat surface and measure the height of the maximum gap between the specimen and the flat surface with a feeler gage. If the maximum gap exceeds 0.020 in. (0.50 mm), the specimen does not meet the required tolerance for straightness of the elements. The flat test surface on which the specimen is rolled shall not depart from a plane by more than 0.0005 in. (15 μm).

5.1.1.2 *Method B*:

5.1.1.2.1 Place the cylindrical surface of the specimen on a V-block that is laid flat on a surface. The smoothness of the surface shall not depart from a plane by more than 0.0005 in. (15 μm).

5.1.1.2.2 Place a dial indicator in contact with the top of the specimen as shown in Fig. 2, and observe the dial reading as the specimen is moved from one end of the V-block to the other along a straight line.

5.1.1.2.3 Record the maximum and minimum readings on the dial gage and calculate the difference, Δ_0. Repeat the same operations by rotating the specimen for every 90°, and obtain the differences, Δ_{90}, Δ_{180}, and Δ_{270}. The maximum value of these four differences shall be less than 0.020 in. (0.50 mm).

5.1.2 Cut the ends of the specimen parallel to each other, generally smooth (Note 2), and at right angles to the longitudinal axis. The ends shall not depart from perpendicularity to the axis of the specimen by more than 0.25°, approximately 0.01 in. (0.3 mm) in 2 in. (50.0 mm). The perpendicularity of the end surfaces to the longitudinal axis shall be determined by the similar setup as for the cylindrical surface (Fig. 3), except that the dial gage is mounted near the end of the V-block. Move the mounting pad horizontally so that the dial gage runs across the end surface of the specimen along a diametral direction. Take care to ensure that one end of the mounting pad maintains intimate contact with the end surface of the V-block during moving. Record the dial gage readings and calculate the difference between the maximum and the minimum values, Δ_1. Rotate the specimen 90° and repeat the same operations and calculate the difference, Δ_2. Turn the specimen around and repeat the same measurement procedures for the other end surface and obtain the difference values Δ'_1 and Δ'_2. The perpendicularity will be considered to have been met when:

$$\frac{\Delta_I}{D} \text{ and } \frac{\Delta'_I}{D} \leq 0.005$$

where:

I = 1 or 2, and

D = diameter.

The smoothness of the end surfaces can be determined by taking dial gage readings for every ⅛ in. (3.2 mm) during the perpendicularity measurements. The closeness of the readings are expected to provide a smooth curve of the end surface along the specific diametral plane. The smoothness requirement is met when the slope along any part of the curve is less than 0.25°.

NOTE 2—In this test method, the condition of the specimen ends with regard to the degree of flatness and smoothness is not as critical as it is, for example, in compression tests where good bearing is a prerequisite. In direct-tension tests it is more important that the ends be parallel to each other and perpendicular to the longitudinal axis of the specimen in order to facilitate the application of a direct tensile load. End surfaces, such as result from sawing with a diamond cut-off wheel, are entirely adequate. Grinding, lapping, or polishing beyond this point serves no useful purpose, and in fact, may adversely affect the adhesion of the cementing medium.

5.1.3 The specimen shall have a length-to-diameter ratio (L/D) of 2.0 to 2.5 (Note 3) and a diameter of not less than NX wireline core size, approximately 1⅞ in. (48 mm) (Note 4).

NOTE 3—In direct-tension tests, the specimen should be free to select and fail on the weakest plane within its length. This degree of freedom becomes less as the specimen length diminishes. When cores of shorter than standard length must be tested, make suitable notation of this fact in the test report.

NOTE 4—It is desirable that the diameter of rock tension specimens be at least ten times greater than the diameter of the largest mineral grain. It is considered that the specified minimum specimen diameter of approximately 1⅞ in. (48 mm) will satisfy this criterion in the majority of cases. It may be necessary in some instances to test specimens that do not comply with this criterion. In this case, and particularly when cores of diameter smaller than the specified minimum must be tested because of the unavailability of larger size specimens (as is often the case in the mining industry) make suitable notation of these facts in the test report, and mention the grain size.

5.1.4 Determine the diameter of the test specimen to the nearest 0.01 in. (0.25 mm) by averaging two diameters measured at right angles to each other at about midlength of the specimen. Use this average diameter for calculating the cross-sectional area. Determine the length of the test specimen to the nearest 0.01 in. by averaging two height measurements along the diameter.

5.1.5 The moisture condition of the specimen at the time of test can have a significant effect upon the indicated strength of the rock. Good practice generally dictates that laboratory tests be made upon specimens representative of field conditions or conditions expected under the operating environment. However, consider also that there may be reasons for testing specimens at other moisture contents, or with none. In any case, relate the moisture content of the test specimen to the problem at hand and report the content in accordance with 8.1.6.

6. Procedure

6.1 Cement the metal caps to the test specimen to ensure alignment of the cap axes with the longitudinal axis of the specimen (Note 5). The thickness of the cement layer should not exceed $^1/_{16}$ in. (1.6 mm) at each end. The cement layer must be of uniform thickness to ensure parallelism between the top surfaces of the metal caps attached to both ends of the specimens. This should be checked before the cement is hardened (Note 5) by measuring the length of the specimen and end-cap assembly at three locations 120° apart and near the edge. The maximum difference between these measurements should be less than 0.005 in. (0.13 mm) for each 1.0 in. (25.0 mm) of specimen diameter. After the cement has hardened sufficiently to exceed the tensile strength of the rock, place the test specimen in the testing machine, making certain that the load transfer system is properly aligned.

NOTE 5—In cementing the metal caps to the test specimens, use jigs and fixtures of suitable design to hold the caps and specimens in proper alignment until the cement has hardened. The chucking arrangement of a machine lathe or drill press is also suitable. The cement used should be one that sets at room temperature. Epoxy resin formulations of rather stiff consistency and similar to those used as a patching and filling compound in automobile body repair work have been found to be a suitable cementing medium.

6.2 Apply the tensile load continuously and without shock to failure. Apply the load or deformation at an approximately constant rate such that failure will occur in not less than 5 nor more than 15 min. Note and record the maximum load carried by the specimen during the test.

NOTE 6—In this test arrangement failure often occurs near one of the capped ends. Discard the results for those tests in which failure occurs either partly or wholly within the cementing medium.

7. Calculation

7.1 Calculate the tensile strength of the rock by dividing the maximum load carried by the specimen during test by the cross-sectional area computed in accordance with 5.3; express the result to the nearest 5 psi (35.0 kPa).

8. Report

8.1 The report shall include the following:

8.1.1 Source of sample including project name and location, and, if known, storage environment (the location is frequently specified in terms of the borehole number and depth of specimen from collar of hole),

8.1.2 Physical description of sample including: rock type, location and orientation of apparent planes, bedding planes, and schisotosity; and large inclusions or inhomogeneities, if any,

8.1.3 Date of sampling and testing,

8.1.4 Specimen length and diameter, also conformance with dimensional requirements as stated in Notes 3 and 4,

8.1.5 Rate of loading or deformation rate,

8.1.6 General indication of moisture condition of sample at time of test, such as as-received, saturated, laboratory air dry, or oven dry (It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation),

8.1.7 Direct tensile strength for each specimen as calculated, average direct tensile strength of all specimens, standard deviation or coefficient of variation,

8.1.8 Type and location of failure (A sketch of the fractured sample is recommended), and

8.1.9 Other available physical data.

NOTE 7—The number of specimens tested may depend upon the availability of specimens, but normally a minimum of ten is preferred. The number of specimens tested should be indicated. The statistical basis for relating the required number of specimens to the variability of measurements is given in Recommended Practice E 122.

9. Precision and Bias

9.1 The precision and bias of this test method have not been determined. Data are being sought to develop a precision and bias statement.

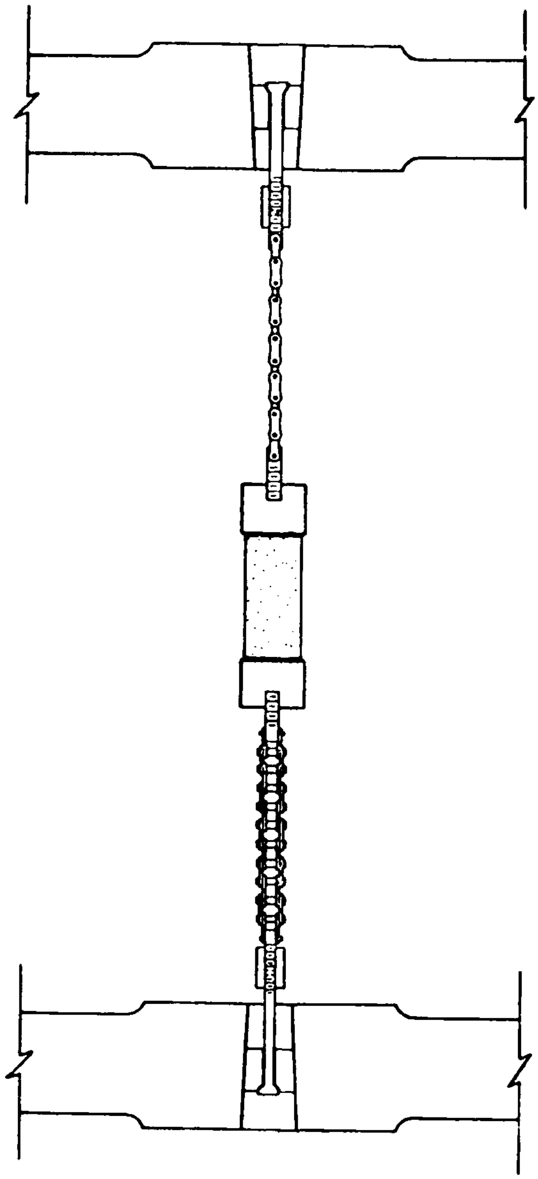


FIG. 1 Direct Tensile-Strength Test Assembly

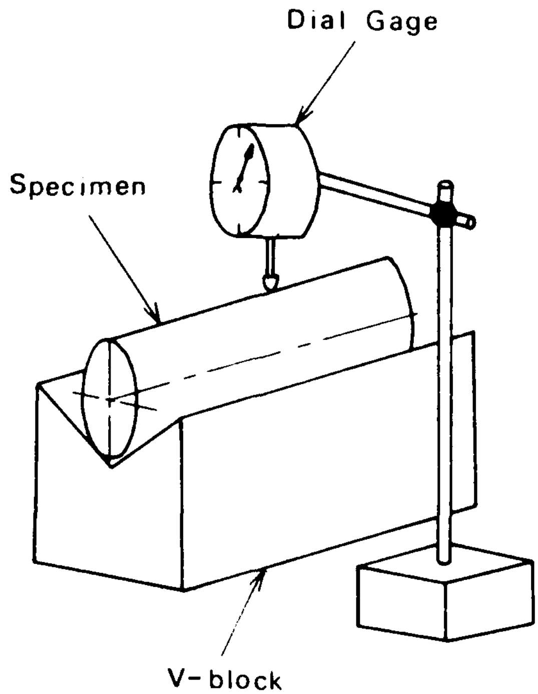


FIG. 2 Assembly for Determining the Straightness of the Cylindrical Surface

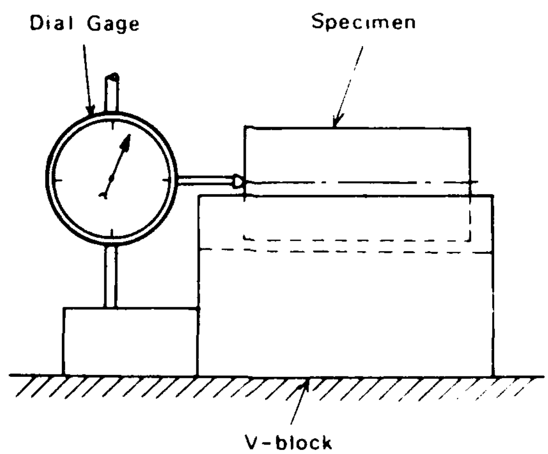


FIG. 3 Assembly for Determining the Perpendicularity of End Surfaces to the Specimen Axis

The American Society for Testing and Materials takes no position respecting the validity of any patent rights asserted in connection with any item mentioned in this standard. Users of this standard are expressly advised that determination of the validity of any such patent rights, and the risk of infringement of such rights, are entirely their own responsibility.

This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 1916 Race St., Philadelphia, Pa 19103.

STANDARD METHOD OF TEST FOR DETERMINING THE SPLITTING STRENGTH OF ROCK (BRAZILIAN METHOD)

1. Scope

1.1 This method covers the specimen preparation and testing procedure for determining the splitting tensile strength of rock by diametrical compression of a disk.

NOTE 1—The tensile strength of rock determined by tests other than the straight pull test is designated as "indirect" tensile strength and, specifically, the test values obtained in Section 5 are termed "splitting tensile strength."

NOTE 2—In this method the values stated in U. S. customary units are to be regarded as the standard. The metric equivalents of U. S. customary units given in the method are rounded to reasonable numbers in accordance with the ASTM Metric Practice Guide.

2. Apparatus

2.1 Testing Machine - The testing machine can be of any type with sufficient capacity to meet specifications prescribed in Section 4.2. The testing machine shall be verified at suitable time intervals in accordance with the procedure given in ASTM Method E4, "Verification of Testing Machines," and shall comply with the requirements prescribed therein.

2.2 Bearing Surfaces - Same as described in RTH 111.

2.3 Bearing Strips - No bearing strip shall be used.

2.4 Supplementary Bearing Plates - Supplementary bearing plates made of material recommended in RTH 111 may be used during test.

3. Test Specimens

3.1 Dimensions - The test specimen shall be a circular disk with length-to-diameter ratio L/D = 1/2 cut from a drilled core. The length of the specimen should be at least 10 times greater than the largest mineral grain constituent. The minimum dimensions of the disk shall not be less than 2 in. (50.8 mm) in diameter and 1 in. (25.4 mm) in thickness.

NOTE 3—When cores smaller than the specified minimum must be tested because of the unavailability of material, notation of that fact shall be made in the test report.

3.2 Number of Specimens - Ten specimens should be tested in order to obtain a meaningful average test value. The mean value, standard deviation, and/or coefficient of variation of splitting tensile strength shall be included in the test report.

3.3 Tolerance - The sides of the specimen shall be smooth within 0.005 in. (0.0127 mm) and the diameter of the test specimen shall be measured to the nearest 0.01 in. (0.254 mm) by the average of at least three measurements. The ends of the specimen shall be cut perpendicular to the axis of the core sample not departing from perpendicularity by more than 0.25 degree or approximately 0.004 in. per inch diameter. The end roughness shall not exceed 0.01 in. (0.254 mm). The use of a surface grinder is recommended to achieve required dimensional tolerance.[1] The thickness of the specimen shall be measured to the nearest 0.01 in. (0.254 mm) by averaging at least three thickness measurements taken between the lines marked on both ends of the specimen (Section 4.1).

NOTE 4—The application of dye penetrant is recommended to detect existing defects of the specimens.[2] The penetrant shall be removed from the surfaces of the sample prior to preconditioning or testing.

---

[1] Podnicks, E. R., P. G. Chamberlain, and R. E. Thrill, "Environmental Effects on Rock Properties," to be published in the Proceedings of the Tenth Symposium on Rock Mechanics, The University of Texas, Austin, Texas.

[2] Gardner, R. D. and H. J. Pincus, "Fluorescent Dye Penetrants Applied to Rock Fractures," International Journal of Rock Mechanics and Mineralogical Science, Vol 5, No. 2, 1968, pp 155-156.

4. Procedure

4.1 Marking - The desired vertical orientation of the specimen in the test apparatus shall be indicated by marking a diametric line on each end of the specimen. These lines shall be used in centering the specimen in the loading machine to ensure proper orientation, and they also are used as reference lines for thickness and diameter measurements.

4.2 Positioning - Position the test sample to ensure the following conditions:

4.2.1 The projection of the diametrical plane of the two lines marked on the ends of the specimen should intersect the center of the upper bearing plate.

4.2.2 The supplementary bearing plate, when used, and the center of the specimen should be directly beneath the center of thrust of the spherical bearing block (Fig. 1).

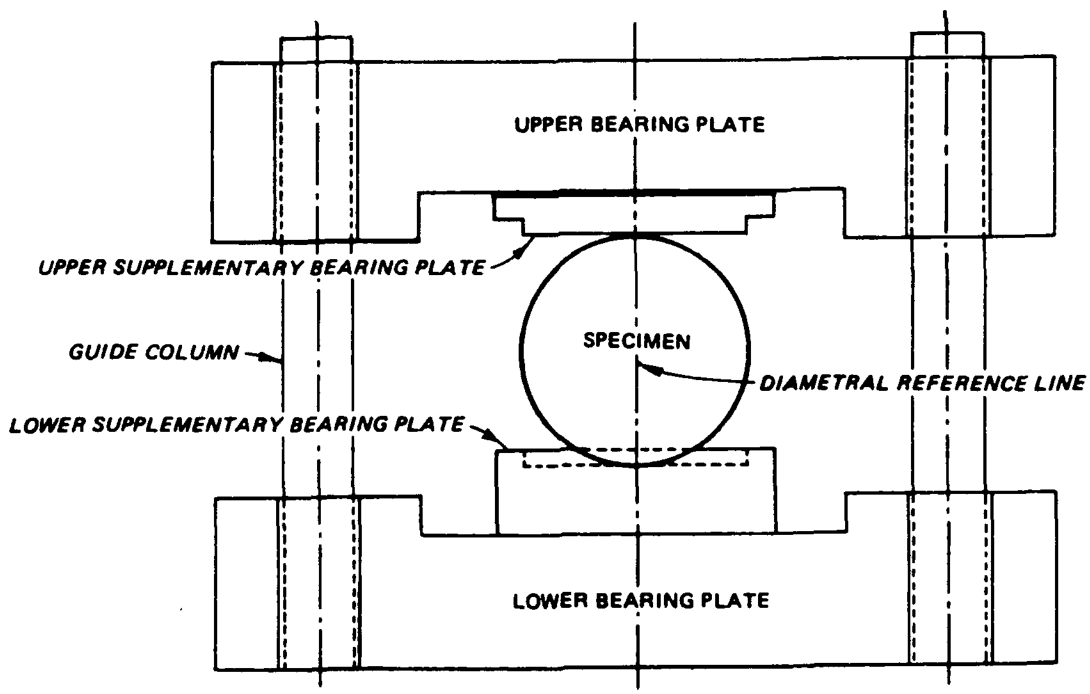


Fig. 1. Loading assembly for determination of splitting tensile strength.

4.3 Loading - Apply a continuously increasing compressive load without any shock at a constant rate of approximately 100 psi/min (11.3 $kN/m^2/sec$) splitting tensile stress until failure of the specimen. Record the maximum applied load indicated by the testing machine at failure.

4.4 Safety Precautions - The upper and lower platens, testing fixture, load-deformation measuring device, and other moving parts shall be enclosed in a cage to arrest the particles flying from a broken specimen. The cage shall be constructed of transparent material to facilitate observation of the specimen during the test.

5. Calculations

5.1 The splitting tensile strength of the specimen shall be calculated as follows:

$$\sigma_t = \frac{2P}{\pi t d}$$

and the result shall be expressed to the appropriate number of significant figures (usually 3) where:

σ_t = splitting tensile strength, psi (N/mm^2)

P = maximum applied load indicated by the testing machine, pounds (newtons)

t = thickness, inches (mm)

d = diameter, inches (mm)

6. Report

6.1 The report shall include the following:

6.1.1 Lithologic description of rock.

6.1.2 Source of material, including depth and the orientation of the specimens with respect to the field orientation.

6.1.3 Number of specimens.

6.1.4 Specimen diameter and thickness.

6.1.5 Specimen moisture content or test environment. Mention should be made of preconditioning treatment, if any.

6.1.6 Loading rate.

6.1.7 If available, petrographic description and physical properties data such as density and porosity.

PROPOSED METHOD OF TEST FOR GAS PERMEABILITY OF ROCK CORE SAMPLES

1. Scope

1.1 This method describes procedures for determining gas permeabilities of rock core samples. Test procedures are described for both large-diameter cores (NX size--5.4 cm and larger) and small-diameter cores which include plugs taken from larger diameter cores. Samples are normally right cylinders although cube samples may be used with a suitably modified sample holder.

1.2 The permeability measurement will be standardly made with dry air as the gas. The value obtained with this fluid may then be corrected to a corresponding value for a nonreacting liquid using a standard table of Klinkenberg corrections.[1] The use of such corrections should be specifically noted in reporting results. Determination of permeability of rock specimens may be made with other gases or with liquids. However, liquid permeabilities are not considered to be routine because of possible interaction between the rock constituents and the liquid.

1.3 Permeability measurements made parallel to the bedding planes of sedimentary rocks shall be reported as horizontal permeability while those measured perpendicular to the bedding shall be reported as vertical permeability. Structural features other than bedding may be used to describe the direction of flow during measurement. These shall be specifically noted in reporting results.

1.4 Samples of hard, consolidated rock may be cut to shape and tested without artificial support. Friable, soft, shaly, or otherwise weak rock may require additional support to resist deformation or alteration during testing. Deformation or alteration will affect test results. Such samples shall be supported by mounting in a suitable

---

[1] "Recommended Practice for Determining Permeability of Porous Media," 3rd ed., American Petroleum Institute RP 27 (1952).

potting plastic or optical pitch exercising care not to alter the surfaces through which fluid flow will occur. Artificially supported samples are tested in the conventional manner.

1.5 The conventional direction of permeability measurement for small cylindrical samples is parallel to the axis of the cylinder. Large-diameter core samples are conventionally measured by one of two methods. In one method, referred to as the "linear permeability measurement," gas flow is either across the core parallel to a plane formed by a diameter of the core and the vertical axis or parallel to the core axis as with the small-diameter samples. When flow is across the sample, screens are used over diametrically opposite quadrants of the core circumference to uniformly distribute the gas flow. The second method is referred to as "radial permeability measurement" in which gas flows from the outside surface of the core radially through the core to a small-diameter hole drilled concentric with the axis of the core sample.

1.6 The permeability test is applicable to a wide range of rock types with a correspondingly wide range of permeabilities. With proper selection of test equipment components, permeabilities as low as 0.01 millidarcys and as high as 10 darcys can be measured accurately.

2. Summary of Method

2.1 The method for small samples consists of (a) placing the prepared sample in a Hassler- or Fancher-type core holder, Figs. 1 and 2, (b) applying the air pressure required to seal the sleeve around the core for the Hassler-type holder or loading the rubber compression ring for the Fancher holder, and (c) initiating dry gas flow through the sample. Because of the sensitivity of permeability to minor changes in lithology, no prescribed number of samples can be recommended to define the permeability of a given rock stratum. Reproducibility of approximately ±2 percent for samples of 0.1 millidarcy or greater permeability should be obtained for a given sample.

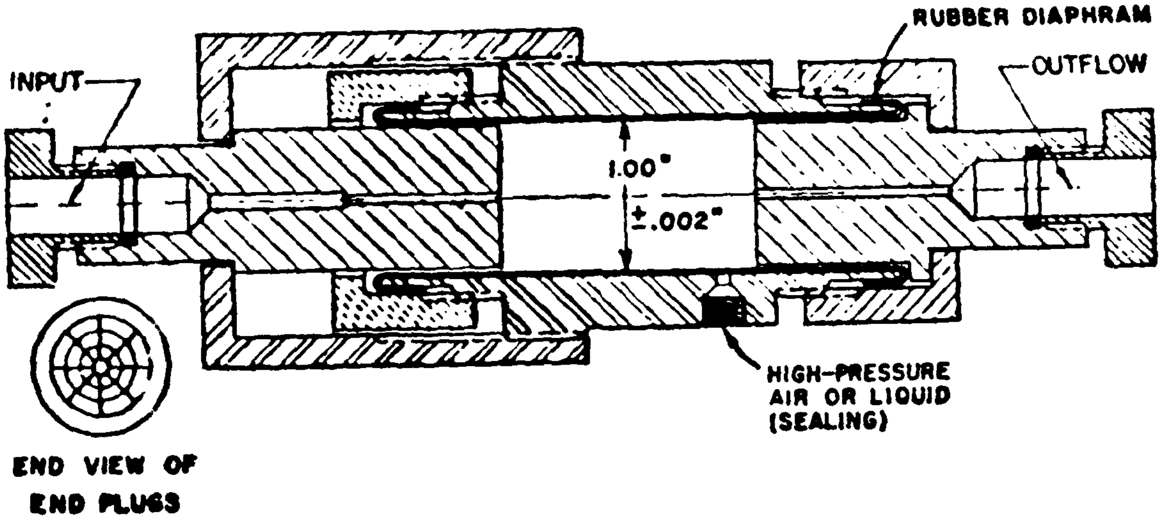


Fig. 1. Hassler-type permeability cell.

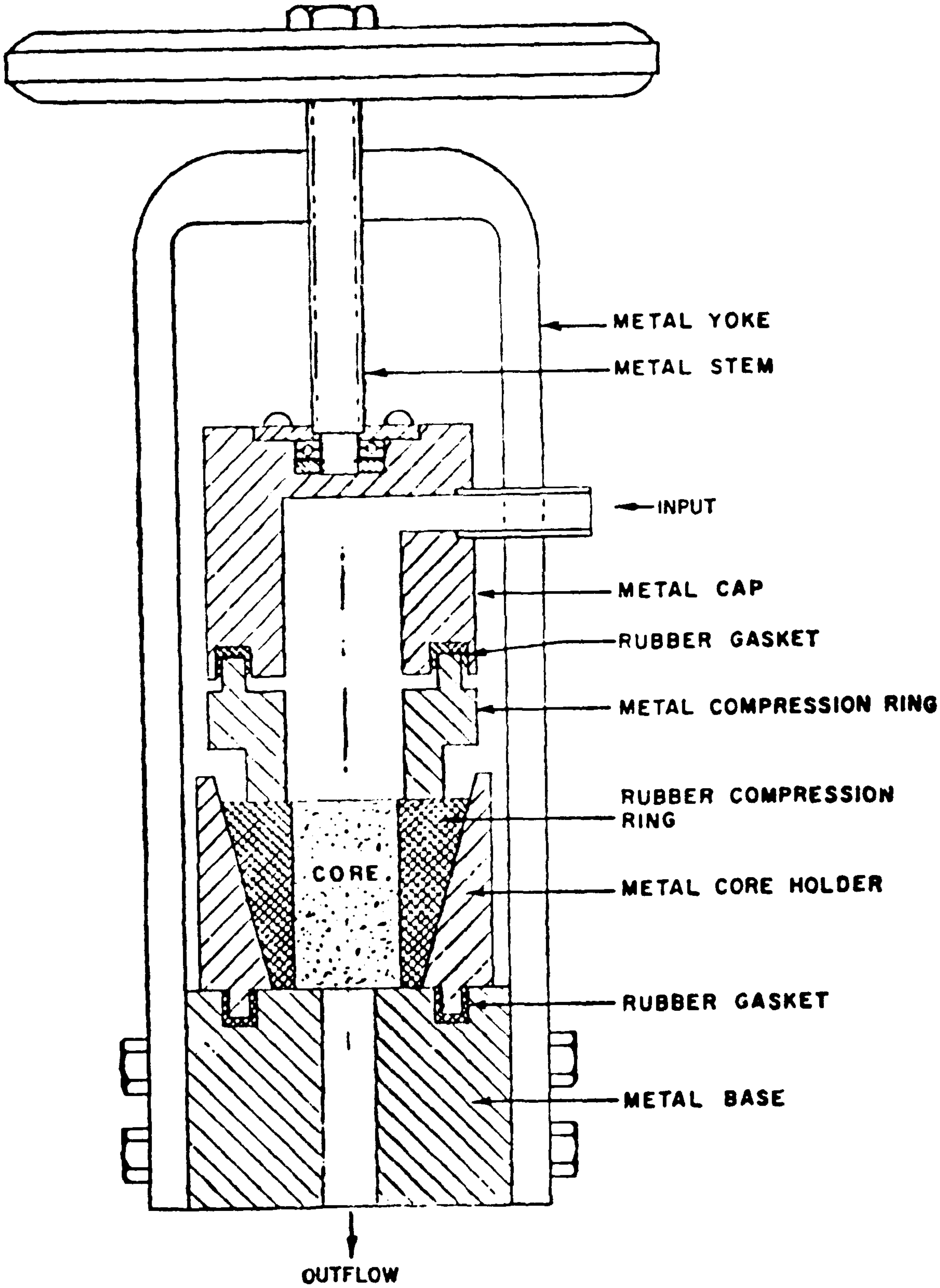


Fig. 2. Fancher-type core holder.

2.2 The method used to measure permeability for large-diameter cylindrical samples with vertical gas flow is the same as that described in 2.1.

2.3 The method for large-diameter samples with horizontal flow consists of (a) positioning appropriate-size screens diametrically opposite each other, (b) attaching the screens to the sample by light rubber bands, (c) placing the prepared sample in a Hassler-type or compression (ram) core holder, Figs. 3 and 4, (d) compressing the rubber gaskets which seal the ends of the samples in the Hassler-type holder (the ends of the samples used in the compression holder are presealed with plastic which is overlapped by the compression halves), (e) applying the air pressure (Hassler type) or hydraulic force (compression) necessary to seal the sides of the sample except the area covered by the screens, and (f) initiating dry gas flow through the sample. Permeability is normally measured in two directions across the core: one is in the direction of apparent maximum permeability and the other is perpendicular to the first.

2.4 The method for large-diameter specimens with radial flow consists of (a) positioning the prepared sample in the radial flow core holder, Fig. 5, (b) raising the core against the closed lid by means of a piston, and (c) initiating dry gas flow through the sample.

3. Apparatus

3.1 Components - The apparatus used in dry air permeability testing consists of the following major components:

(a) A source of dry air
(b) Pressure regulator
(c) Inlet-pressure measuring device
(d) Core holder
(e) Outlet-pressure measuring device
(f) A dry air flow-rate metering device

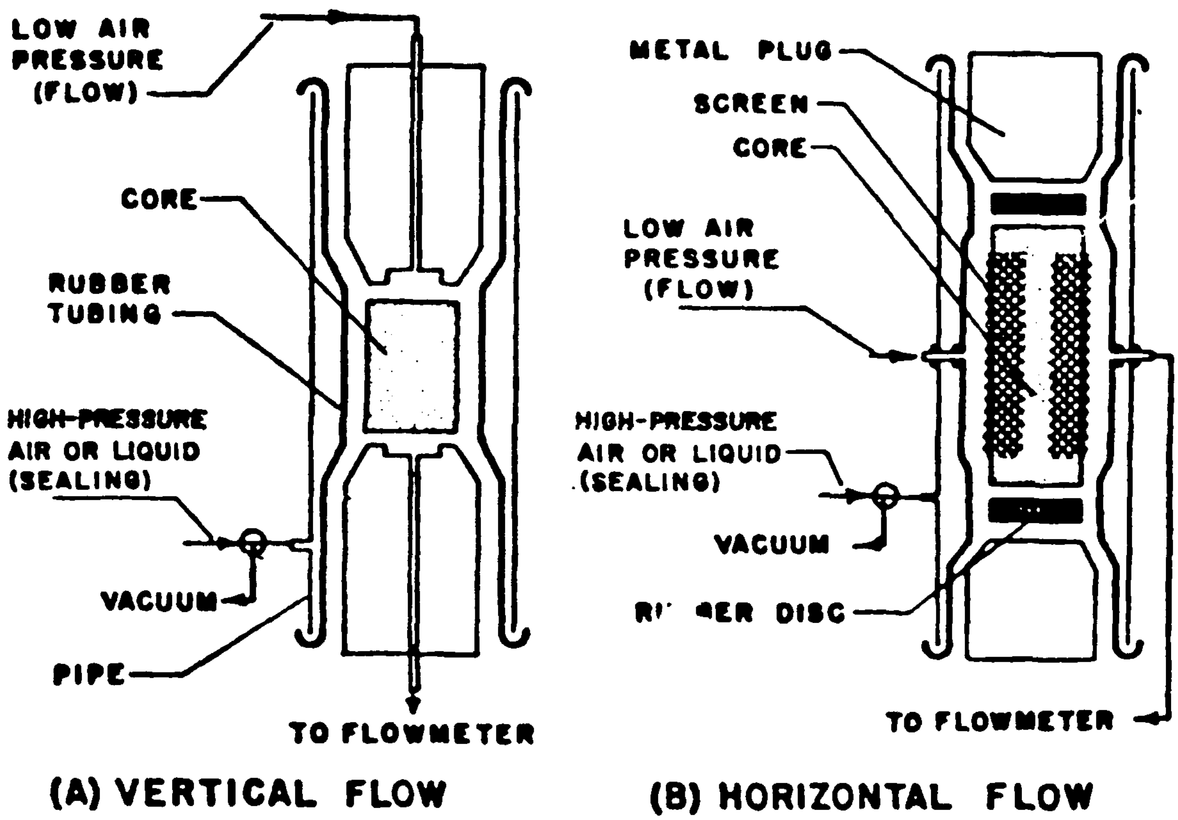


Fig. 3. Hassler-type permeameter.

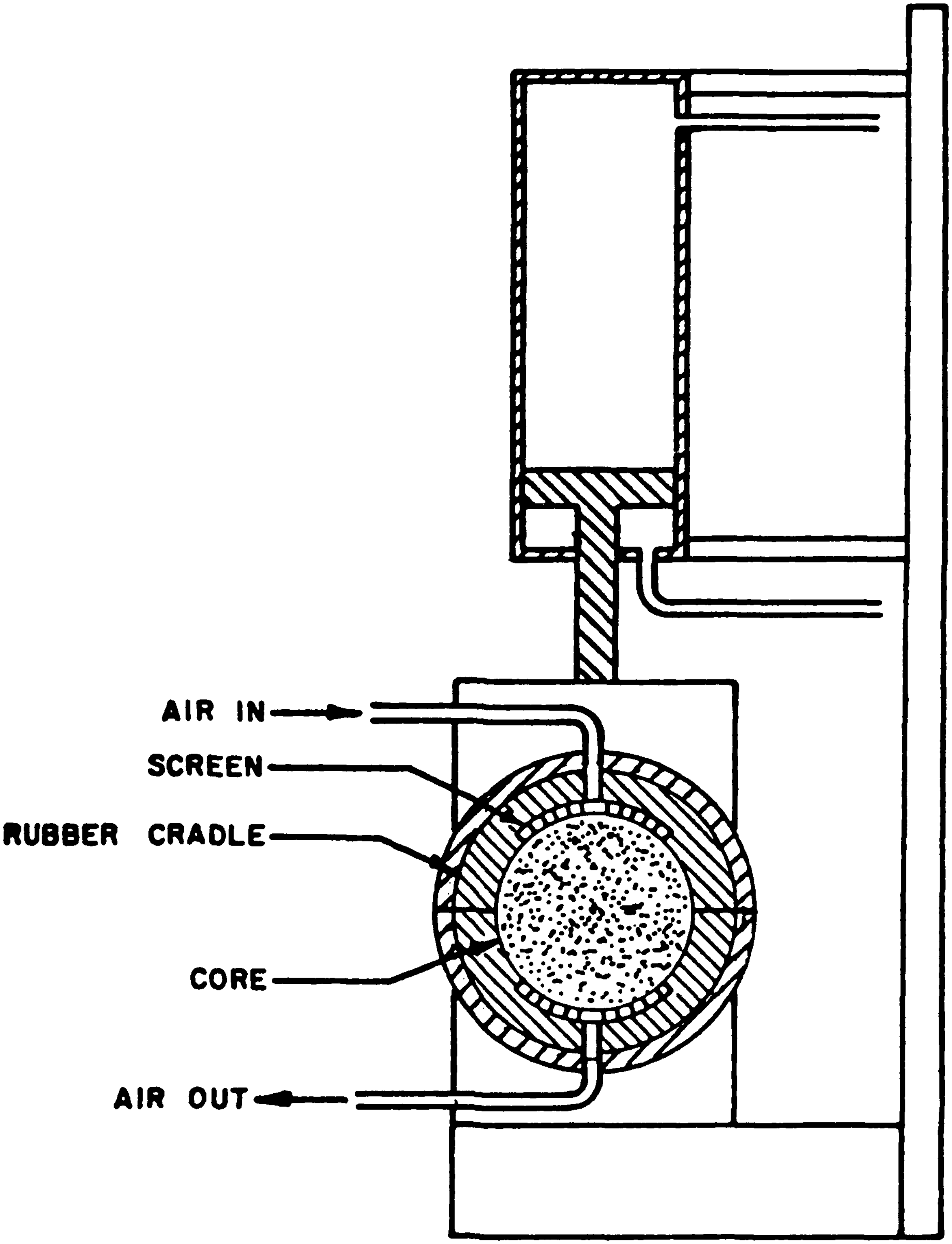


Fig. 4. Compression (RAM) permeameter.

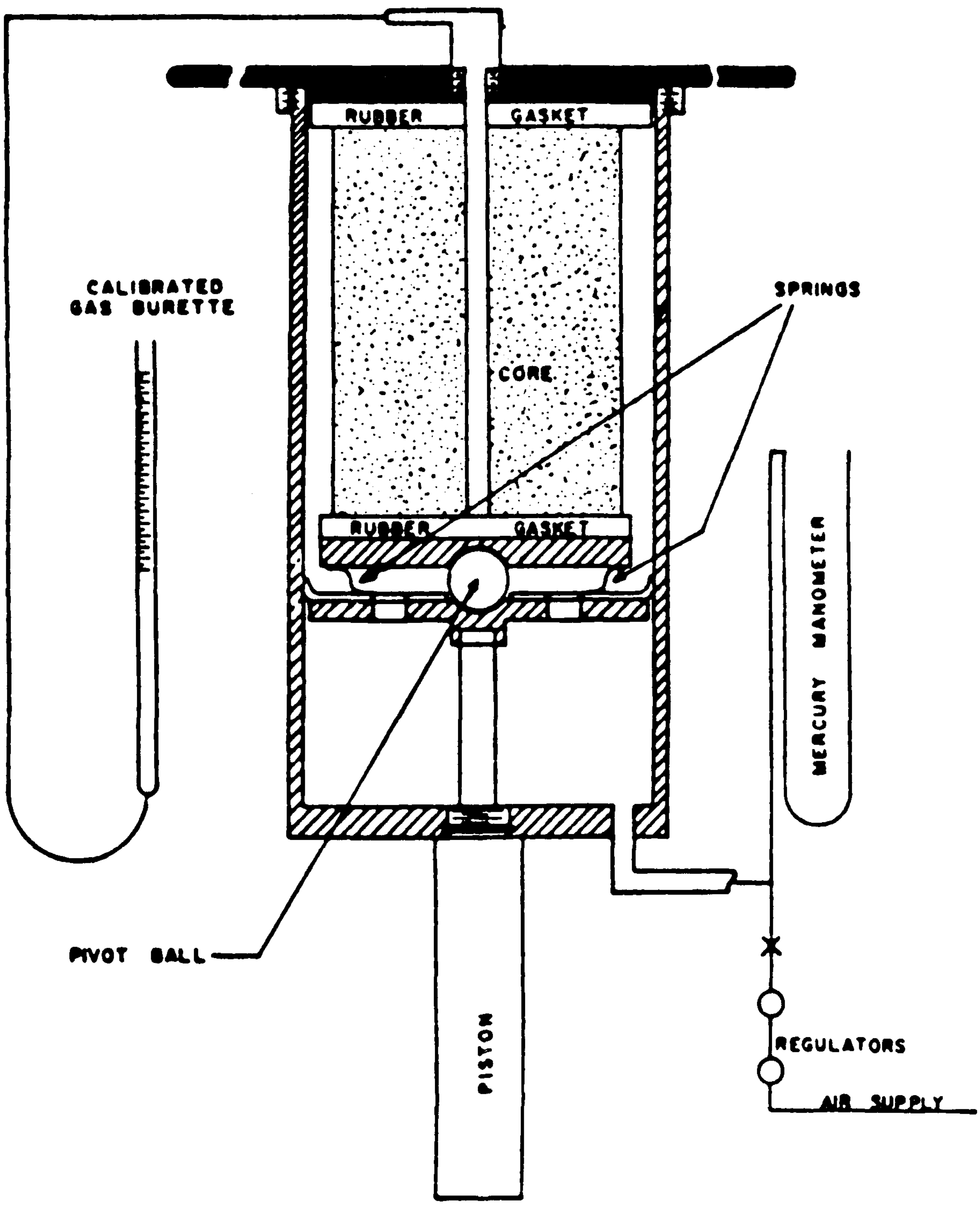


Fig. 5. Full-diameter radial permeameter.

3.2 Source of Air - The source of air for permeability measurements can be either the normal laboratory air supply or cylinders. Provisions should be made to filter particulate matter, absorb oil vapor, and remove water vapor. These devices should be periodically checked to insure proper operation.

3.3 Pressure Regulator - A suitable pressure regulator should be provided for the source of dry air. This regulator should supply air at a constant pressure and should be capable of doing so over a range of pressures between 1 and 80 cm of mercury (Note 1) which will produce the desired flow rate (Note 1) of approximately 1 cu cm per second. Regulators of the nullmatic type are suitable for this purpose.

NOTE 1--The pressures required to produce the desired flow rate depend on the permeability and dimensions of the rock sample. Since the dimensions and permeability range of samples are not specified, the range of pressures over which pressure control is required cannot be specified. Equipment in common use operates over the pressure range of 1 to 80 cm of mercury. This pressure range is capable of producing laminar flow. This is the flow region required for permeability measurements. It is usually observed at flow rates up to 1 cu cm per second.

3.4 Pressure Measuring Devices - Inlet- and outlet-pressure measuring devices are manometers. These are water-, oil-, or mercury-filled and of a convenient length, usually 80 cm or less, with selection of length and fluid depending upon pressures to be measured. Manometers may be used in parallel to obtain the necessary accuracy over a range of pressures. Manometers are either open to the atmosphere or connected across the core. Connected across the core, they measure "differential" pressure. Where the pressure is in excess of 80 cm of mercury, a bourdon-type gage may be employed. This type of gage is normally only used in measuring extremely low permeabilities where high inlet pressures are required.

3.5 Core Holders

3.5.1 Three types of core measurements are commonly made:

(a) Axial flow in both small- and large-diameter cores

(b) Diametric flow in large-diameter cores

(c) Radial flow in large-diameter cores

3.5.2 Except for radial flow, more than one type of core holder may be used for permeability determinations. Irrespective of direction of gas flow or type of core holder used, the core holder must be such that when pressure is applied to one end of the system, all flow is through the sample. Care must be taken that no fluid bypasses the sample either through an imperfect seal between the core holder and sample or between the sample and the supporting material if the sample is mounted. Holders which accommodate several cores for simultaneous or sequential testing using matched pairs of inlet-outlet valves must be designed so that no fluid can leak between samples.

3.5.3 Selection of the type of core measurement is based on both the size of the sample available and the desired representativeness of the permeability value. For uniform, homogeneous rock small-diameter cores taken parallel and perpendicular to the bedding should provide representative permeability values. Rock which is nonhomogeneous, vugular, fractured, or laminated provides more representative values when subjected to the large-diameter core methods of measurement. The radial method provides more representative data than the horizontal flow method since the entire surface of the core rather than a quadrant is involved in flow.

3.6 Screens to Distribute Gas Flow - When flow is across the core parallel to a diameter, screens should be of a wire diameter and mesh size such t ıt a uniform gas distribution is obtained. Care should be exercised in positioning the screens prior to placing the sample in the holder to ensure that they maintain their proper position while the core is being seated.

3.7 Air Flow-Rate Metering Device - Three types of dry air flow-rate metering devices may be used:

(a) Calibrated orifices (a capillary tube which is calibrated for the conditions of testing, so that the pressure drop across the orifice is small compared to the core)

(b) Soap bubble in a calibrated burette

(c) Water-displacement meters

The calibrated orifice is the most commonly used type of flow metering device. Timing the movement of a soap bubble in a burette is also frequently used. Differential pressures across the core are adjusted to minimize turbulence in the flow of gas through the sample. Flow rates of 1 cu cm per second or less are used.

3.8 Sample Preparation Equipment - Sample preparation equipment shall include the following:

3.8.1 Diamond coring equipment for taking small-diameter cylinders from larger samples.

3.8.2 Diamond saw for trimming ends of samples. Ends should be sufficiently flat and parallel for leakfree seating of rubber end seals.

3.8.3 Drill press and diamond drill for drilling axial holes for radial flow measurement samples. The holes should be concentric with the axis of the cylindrical sample. Care should be exercised in drilling to prevent cracking of the sample.

3.8.4 Sample cleaning to remove original fluids from the cores, external coatings such as drilling muds, and, in the case of highly saline original fluid, deposited salts. Where the original fluids are hydrocarbons, cleaning may be accomplished by solvent extraction, gas-driven solvent extraction, distillation-extraction, or other suitable method.[2] Drilling muds may be removed from the surface with water washing. If the interstitial water was very saline a freshwater wash may be used to remove deposited salts.

---

[2]Darcy, H., The Public Fountain of the Village of Dijon, Paris: Victor Dalmont, 1856 (French text).

3.8.5 Drying oven that can be maintained at 110 ± 5°C.

3.8.6 Micrometer or vernier caliper for measuring length and diameter of test specimen. Micrometer or caliper should be direct reading to 1/50 mm.

3.8.7 Equipment as required for mounting samples of weak rock in optical pitch or suitable potting plastic.

3.8.8 Miscellaneous equipment such as timing devices, magnifying lenses, etc., used in preparing the sample and measuring pressures and gas flow rates.

4. Calibration

4.1 The permeameter should be calibrated regularly by means of capillary tubes of various known permeabilities or with standard plugs.

4.2 Orifices used to measure gas flow rates should be calibrated by allowing air to flow from the orifice to a burette containing a soap bubble.

4.3 Micrometers or vernier calipers used to measure sample dimensions should be checked against length standards.

5. Sample Preparation

5.1 There are no standard sample sizes. Small-diameter samples are commonly 1.9 to 3.81 cm in diameter. Large-diameter cores are arbitrarily 5.4 cm (equivalent to NX core) in diameter and above. Cores as large as 15.24 cm in diameter are commonly tested. Core holders are designed for a specific size or sizes of cores. The core holder may be modified to accommodate a smaller sized sample by placing the core in a rubber sleeve whose inner diameter is that of the core and whose outer diameter is that for which the permeameter was designed. Thus a 1.9-cm-diam sample can be tested in a permeameter designed for 2.54-cm samples. Sample length is also not standardized. Sample lengths vary from 2.54 cm for small-diameter samples to 60.96 cm for large-diameter cores. Core holding devices are designed to accept different length samples. Where a sleeve is used to adapt a core, differences in sleeve and sample length may be compensated for by means of spacer rings placed on top of the sample.

5.2 Sample length to diameter ratio is also not standardized. A ratio of 1:1 is recommended as a minimum for small-diameter samples. For large-diameter samples, a minimum L/D ratio of 1:2 is recommended.

5.3 The diameter of the test specimen is measured to the nearest 0.1 mm by averaging two diameters measured at right angles to each other at about midlength of the specimen. This average diameter is used for calculating the cross-sectional area. The length of the test specimen is determined to the nearest 0.1 mm by averaging three length measurements taken at third points around the circumference.

5.4 If the sample requires artificial support, the mounting material and method of mounting should be such that penetration into the sample is minimized, the sample does not extend beyond the surface of the mounting material, and mounting material does not cover any surface perpendicular to the direction of flow. Specific details of the mounting procedures, as well as mounting materials, are given in reference 1.

5.5 In cleaning and mounting the sample prior to testing, care should be used to prevent any alteration of the minerals comprising the rock sample which may produce changes in permeability. Samples containing clays or other hydratable minerals are especially susceptible.

5.6 Before measuring permeability any sample which is suspected of being cracked should be tested by coating with a liquid while air is passing through the sample. A row of bubbles on the downstream surface will serve to indicate the presence of a crack parallel to the direction of flow. Such samples should either be discarded as nonrepresentative or, if retained, note should be made of the crack in reporting permeability values.

6. Procedure

6.1 The prepared sample is mounted in the specified core holder as follows:

6.1.1 Hassler-type Holder - Retract rubber diaphragm or sleeve, Fig. 1, by applying a vacuum to the space between the diaphragm and the body of the core holder. (Note 2)

NOTE 2--A vacuum source of 5 cm of mercury is sufficient to retract the diaphragm.

Remove one end of the core holder and insert the cylindrical sample. Reinsert the end of the core holder, applying sufficient force to seat the sample. Remove the vacuum from the space between the body and the diaphragm, thus allowing the diaphragm to constrict around the sample. (Note 3)

NOTE 3--Fine-grained, smoothly formed samples can be effectively sealed at 690 newtons per square metre pressure. Coarse-grained samples will require from 1035 to 1370 newtons per square metre sealing pressure.

The pressure between the diaphragm and core holder body is maintained during the permeability measurement.

6.1.2 Fancher-type Holder - Select a rubber stopper drilled to the diameter and length of the sample. Carefully push the sample plug into the tapered stopper base until it is flush with the small end surface. Remove all loose sand grains. Place the stopper containing the test sample inside the tapered metal core holder, Fig. 2. Turn the ram hand wheel to move the upper ram plug down to seal tightly against the top of the holder. Compress the tapered rubber stopper with the ram to effect a seal around the sample perimeter.

6.1.3 Compression (Ram) Holder - Place the sample in the lower half of the rubber cradle, Fig. 4, making certain that the screen is centered over the air outlet and the ends of the sample are overlapped by the rubber cradle. Lower the upper compression half by applying air or hydraulic pressure above the piston. After seating the two halves, apply sufficient pressure to compress the rubber cradle with the ram to effect the seal around the sample perimeter. (see Note 3).

6.1.4 Radial Flow Holder - Place the core on a 1-in. (2.54-cm) solid rubber gasket which is attached to the lower floating plate, Fig. 5. Raise the core by means of the piston against the closed lid with the center hole of the core matching that of the upper gaskets. After the core contacts the upper gasket, increase the piston pressure slightly to adjust the lower floating plate if the core ends are not parallel. Sufficient piston pressure is then applied to effect a seal of the upper and lower core surfaces. (Note 4)

NOTE 4—With gas flowing through the core at a constant rate, the piston pressure is increased. Decreased flow rate with increasing piston pressure indicates a leak at the rubber gasket. Repeat the test until no change in the flow rate is noted.

6.2 Connect the source of dry gas and inlet pressure measuring devices to the inlet fitting on the core holder. Connect the outlet pressure measuring devices and flow rate metering device to the outlet fitting on the core holder.

6.3 Initiate dry gas flow through the sample. Control the flow rate to minimize turbulence. Measure the flow rate through the sample intermittently for a period of 3 to 10 minutes until the flow becomes constant. Record the constant flow rate and pressures.

7. Calculations

7.1 The calculation of permeability is based on the empirical expression of Darcy known as Darcy's law.[2] The coefficient, k, of proportionality is the permeability.

7.2 The unit of the permeability coefficient, k, is the darcy. For convenience the subunit millidarcy may be used where 1 millidarcy equals 0.001 darcy. A porous medium has a permeability of one darcy when a single-phase fluid of one centipoise viscosity that completely fills the voids of the medium will flow through it under "conditions of viscous flow" at a rate of 1 cu cm per second per square centimetre of cross-sectional area under a pressure gradient of one atmosphere per centimetre.

"Conditions of viscous flow" mean that the rate of flow is sufficiently low to be directly proportional to the pressure gradient. The permeability coefficient so defined has the units of length squared or area.

7.3 Vertical Flow Parallel to Core Axis - Darcy's law in differential form for linear flow is

$$q = \frac{k}{\mu}\frac{dp}{dL}$$

where q = macroscopic velocity of flow, in centimetres per second

k = permeability coefficient, in darcys

μ = viscosity of the fluid that is flowing, in centipoises

dp/dL = pressure gradient in the direction of flow, in atmospheres per centimetres

Relating the velocity of flow to the volume rate of flow through the cross-sectional area and performing the indicated integration produces the following working equation for permeability coefficient in millidarcys:

$$k = (2000\ Q_o O_o\ L\ \mu)/(P_i^{\,2} - P_o^{\,2})$$

where Q_o = rate of flow of outlet air, in cubic centimetres per second

P_o = outlet pressure, in atmospheres (absolute)

P_i = inlet pressure, in atmospheres (absolute)

L = length of sample, in centimetres

A = cross-sectional area perpendicular to direction of flow, in square centimetres

Several methods of simplification exist for calculating the permeability coefficient. Two such methods are given below:

7.3.1 Method 1 - For this method inlet pressure and pressure drop across the rate measuring orifice during the test are selected so that the outlet pressure is essentially one atmosphere. The working equation then reduces to

$$k = Q\ CL/A$$

where $C = (2000\ \mu)/(P_i^2 - 1)$

μ = viscosity of air under the conditions used to calibrate the orifice

C then is a constant for each fixed inlet pressure since the fact that the same air flows through both the core and the orifice means that any change in air viscosity resulting from temperature changes or water vapor will have no effect on the relative pressure readings.

7.3.2 Method 2 - For this method calibration charts or tables of permeance (Note 5) versus outlet pressure for given inlet pressures and orifices are prepared based on the following equation

$$k_c = \frac{L_c}{A_c}\left(\frac{k_{or}}{L_{or}/A_{or}} \cdot \frac{\Delta P_{or} Q_c}{\Delta P_c Q_{or}}\right)$$

where

k_c, k_{or} = permeability of the core and the equivalent permeability of the orifice, respectively, in millidarcys

L_c, L_{or} = length of core and orifice, respectively, in centimetres

A_c, A_{or} = cross-sectional area of core and orifice, respectively, in square centimetres

ΔP_c, ΔP_{or} = pressure drop across the core and orifice, respectively, in atmospheres

Q_c, Q_{or} = flow rate through the core and orifice, respectively, in cubic centimetres per second

NOTE 5--Permeance or apparent permeability is the proper term for flow capacity. The term as used is analogous to the term conductance for the flow of current through an electrolyte solution. Permeance and permeability, therefore, are related in the same way as conductance and conductivity.

This working equation can be simplified to

$$k_c = \frac{L_c}{A_c} \frac{Q_c}{\Delta P_c} \cdot L$$

where L = orifice constant

L may be determined directly by use of a known permeability plug. When tables or nomographs are used to calculate the permeability coefficient from measured outlet pressure for given inlet pressures and orifices, the working equation reduces to

$$k_c = \frac{L_c}{A_c} k_c{}^{\theta}$$

where $k_c{}^{\theta}$ = permeance of the core

The permeance as calculated when multiplied by the L/A ratio gives the core permeability coefficient, k_c.

7.4 Horizontal Flow Parallel to Core Diameter - The same differential form of Darcy's law and working equation as used for vertical flow are applied. The working equation is modified by a factor for shape to the following form:

$$k = (Q_m \, \mu/L \, \Delta P)(1000)(G)$$

where k = permeability, in millidarcys

Q_m = volume rate of air flow at mean core pressure, in cubic centimetres per second

μ = viscosity of flowing fluid, in centipoises

L = length of sample, in centimetres

ΔP = pressure drop across the core, in atmospheres

G = shape factor,[3] Fig. 6

7.5 Radial Flow - The radial permeability is calculated directly from the integrated form of Darcy's law for radial flow. The equation in terms of permeability coefficient is

$$k = (\mu Q_a)(\ln d_e/d_w)(P_o)/(\pi h)(P_i^2 - P_o^2) \cdot 1000$$

where k = permeability, in millidarcys

μ = viscosity of flowing fluid at test temperature, in centipoises

Q_a = measured flow rate at test temperature and pressure = P_o, in cubic centimetres per second

$\ln$ = logarithm to the base e

d_e = outside diameter of sample, in centimetres

d_w = inside diameter of inner hole, in centimetres

P_o = outlet pressure, in atmospheres (absolute)

h = height of sample, in centimetres

P_i = inlet pressure, in atmospheres (absolute)

As with vertical flow, if the outlet pressure is atmospheric and the orifices are calibrated over the range of inlet pressures, the working equation simplifies to

$$k = \mu Q_m(\ln d_e/d_w)/2\pi h\Delta P \cdot 1000$$

where Q_m = volume rate of air flow at mean core pressure, in cubic centimetres per second

ΔP = pressure drop across sample, in atmospheres (absolute)

[3] Collins, R. E., "Determination of the Transverse Permeabilities of Large Core Samples from Petroleum Reservoirs," Journal of Applied Physics 23, 681-84 (1952).

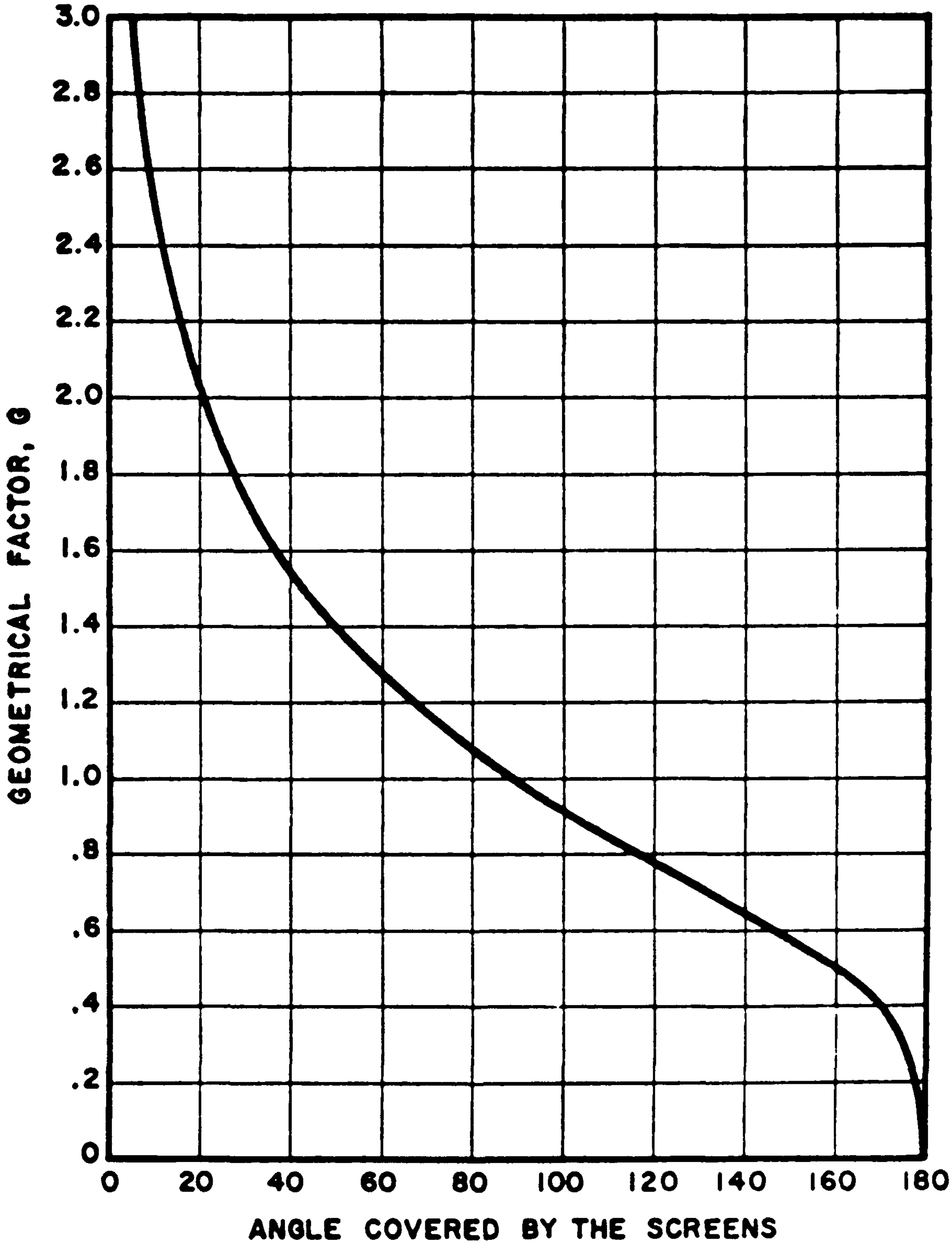


Fig. 6. Theoretical curve relating the geometric factor and the angular segment of the core covered by the screens (Collins, 1952).

8. Report

8.1 The report shall include the following:

8.1.1 A lithologic description of the rock tested.

8.1.2 Source of sample including depth and orientation, dates of sampling and testing, and storage environment.

8.1.3 Methods used for sample cleaning.

8.1.4 Methods used for sample support, capping, or other preparation such as sawing, grinding, or drilling.

8.1.5 Specimen length and diameter.

8.1.6 Type of core holder used.

8.1.7 Pressures used to seal core surfaces in core holder.

8.1.8 Flowing fluid used and direction of flow.

8.1.9 Method used for calculating permeability coefficient.

8.1.10 Results of other physical tests, citing the method of determination for each.

8.1.11 Permeability corrections used.

STANDARD METHOD OF TEST FOR RESISTANCE TO ABRASION OF ROCK BY USE OF THE LOS ANGELES MACHINE

1. Scope

1.1 This method covers a procedure for testing rock samples larger than 3/4 in. (19 mm) for resistance to abrasion using the Los Angeles testing machine.

2. Apparatus

(a) Los Angeles machine conforming to the requirements of ASTM Designation C 131.

(b) Sieves, conforming to the Specifications for Wire-Cloth Sieves for Testing Purposes (ASTM Designation: E 11).

(c) A balance or scale accurate within 0.1 percent of test load over the range required for this test.

3. Abrasive Charge

3.1 The abrasive charge shall consist of 12 steel spheres averaging approximately 1-27/32 in. (46.831 mm) in diameter, each weighing between 390 and 445 g, and having a total weight of 5000 ± 25 g.

NOTE 1--Steel ball bearings 1-13/16 in. (46,038 mm) and 1-7/8 in. (47.625 mm) in diameter, weighing approximately 400 and 440 g each, respectively, are readily available. Steel spheres 1-27/32 in. (46.831 mm) in diameter weighing approximately 420 g may also be obtainable. The abrasive charge may consist of a mixture of these sizes.

4. Test Sample

4.1 The test sample shall be prepared from rock representative of that being furnished. The sample shall be washed and oven-dried at 221 to 230°F (105 to 110°C) to substantially constant weight (Note 3), separated into individual size fractions, and recombined to one of the gradings of Table I. To secure a proper grading, some preliminary crushing may be necessary. The weight of the sample prior to test shall be recorded to the nearest 1g.

TABLE I

Gradings of Test Samples

Sieve Size, in. (Square Openings)		Weights of Indicated Sizes, g					
		Grading					
Passing	Retained on	1		2		3	
3	2-1/2	2 500	50	---		---	
2-1/2	2	2 500	50	---		---	
2	1-1/2	5 000	50	5 000	50	---	
1-1/2	1	---		5 000	25	5 000	25
1	3/4	---		---		5 000	25
Total		10 000	100	10 000	75	10 000	50

5. Procedure

5.1 The test sample and abrasive charge shall be placed in the Los Angeles abrasion testing machine and the machine rotated at 30 to 33 rpm for 1,000 revolutions. The machine shall be so driven and so counter-balanced as to maintain a substantially uniform peripheral speed (Note 2). If an angle is used as the shelf (see Appendix A1), the direction of rotation shall be such that the charge is caught on the outside surface of the angle. After the prescribed number of revolutions, the material shall be discharged from the machine and a preliminary separation of the sample made on a sieve coarser than the No. 12 (1.68 mm). The finer portion shall then be sieved on a No. 12 sieve in a manner conforming to Section 5.2 of the Method of Test for Sieve or Screen Analysis of Fine and Coarse Aggregates (ASTM Designation:

C 136). The material coarser than the No. 12 sieve shall be washed (Note 3), oven-dried at 221 to 230°F (105 to 110°C) to substantially constant weight, and weighed to the nearest 5 g (Note 4).

NOTE 2--Backlash or slip in the driving mechanism is very likely to cause test results which are not duplicated by other Los Angeles abrasion machines producing constant peripheral speed.

NOTE 3--If the rock is essentially free from adherent coatings and dust, the requirement for washing before and after test may be waived. Elimination of washing after test will seldom reduce the measured loss by more than about 0.2 percent of the original sample weight.

NOTE 4--Valuable information concerning the uniformity of the sample under test may be obtained by determining the loss after 200 revolutions. This loss should be determined without washing the material coarser than the No. 12 sieve. The ratio of the loss after 200 revolutions to the loss after 1,000 revolutions should not greatly exceed 0.20 for material of uniform hardness. When this determination is made, care should be taken to avoid losing any part of the sample; the entire sample, including the dust of abrasion, shall be returned to the testing machine for the final 800 revolutions required to complete the test.

6. Calculation

6.1 The difference between the original weight and the final weight of the test sample shall be expressed as a percentage of the original weight of the test sample. This value shall be reported as the percentage of wear (Note 5).

NOTE 5--The percentage of wear determined by this method has no known consistent relationship to the percentage of wear for the same material when tested by Method C 131.

APPENDIX

A1. Maintenance of Shelf

A1.1 The shelf of the Los Angeles machine is subject to severe surface wear and impact. With use, the working surface of the shelf is peened by the balls and tends to develop a ridge of metal parallel to and about 1-1/4 in. (32 mm) from the junction of the shelf and the inner surface of the cylinder. If the shelf is made from a section of rolled angle, not only may this ridge develop but the shelf itself may be bent longitudinally or transversely from its proper position.

A1.2 The shelf should be inspected periodically to determine that it is not bent either lengthwise or from its normal radial position with respect to the cylinder. If either condition is found, the shelf should be repaired or replaced before further abrasion tests are made. The influence on the test result of the ridge developed by peening of the working face of the shelf is not known. However, for uniform test conditions, it is recommended that the ridge be ground off if its height exceeds 0.1 in. (2 mm).

PART I. LABORATORY TEST METHODS

B. Engineering Design Tests

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Test Method for Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression[1]

This standard is issued under the fixed designation D 3148; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

ε1 NOTE—Figure 2 was corrected editorially in December 1987.

INTRODUCTION

The deformation and strength properties of rock cores measured in the laboratory usually do not accurately reflect large scale *in situ* properties because the latter are strongly influenced by joints, faults, inhomogeneities, weakness planes, and other factors. Therefore, laboratory values for intact specimens must be employed with proper judgment in engineering applications.

1. Scope

1.1 This test method covers the determination of elastic moduli of intact rock core specimens in uniaxial compression. Procedure A specifies the apparatus, instrumentation, and procedures for determining the axial stress - strain curve and Young's modulus, E, of cylindrical rock specimens loaded in uniaxial compression. Method B specifies the additional apparatus, instrumentation, and procedures which are necessary also to determine the lateral stress - strain curve and Poisson's ratio, ν.

NOTE 1—Some applications require the value of Young's modulus, but not Poisson's ratio. Thus, the decision to use Procedure A or Procedure A and B shall be determined by the engineer in charge of the project.

NOTE 2—This test method does not include the procedures necessary to obtain a stress - strain curve beyond the ultimate strength.

1.2 Test methods are not normally specified for rock moduli in tension because its low tensile strength does not permit sufficient data points to be obtained to be significant. However, the basic principles given here may be applied to tension testing.

1.3 The relation between the three elastic constants,

$$G = E/2(1 + \nu)$$

where:
G = modulus of rigidity,
E = Young's modulus, and
ν = Poisson's ratio,

and most elastic design equations are based on the assumption of isotropy. The engineering applicability of these equations is therefore decreased if the rock is anisotropic. When possible, it is desirable to conduct tests in the plane of foliation, bedding, etc., and at right angles to it to determine the degree of anisotropy. It is noted that equations developed for isotropic materials may give only approximate calculated results if the difference in elastic moduli in any two directions is greater than 10 % for a given stress level.

NOTE 3—Elastic moduli measured by sonic methods may often be employed as preliminary measures of anisotropy.

1.4 The values stated in inch-pound units are to be regarded as the standard.

1.5 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use*

2. Referenced Documents

2.1 *ASTM Standards:*

D 2938 Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens[2]
D 4543 Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances[2]
E 4 Practices for Load Verification of Testing Machines
E 122 Recommended Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[4]

3. Apparatus

3.1 *Loading Device* (Procedures A and B), for applying and measuring axial load to the specimen and of sufficient capacity to apply load at a rate in accordance with the requirements prescribed in 5.4. It shall be verified at suitable time intervals in accordance with the procedures given in Practices E 4, and comply with the requirements prescribed therein.

NOTE 4—The loading apparatus employed for this test is the same as that required for Test Method D 2938.

3.2 *Bearing Surfaces* (Procedures A and B)—The testing machine shall be equipped with two steel bearing blocks having a Rockwell hardness of not less than 58 HRC One of the blocks shall be spherically seated and the other a plain

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Oct. 31, 1986. Published December 1986. Originally published as D 3148 – 72. Last previous edition D 3148 – 80.

[2] *Annual Book of ASTM Standards*, Vol 04.08
[3] *Annual Book of ASTM Standards*, Vols 03.01, 04.02, 07.01, and 08.03
[4] *Annual Book of ASTM Standards*, Vol 14.02

rigid block. The bearing faces shall not depart from a plane by more than 0.0006 in. (15 μm) when the blocks are new and shall be maintained within a permissible variation of 0.001 in. (25 μm). The diameter of the spherically seated bearing face shall be at least as large as that of the test specimen but shall not exceed twice the diameter of the test specimen. The center of the sphere for the spherically seated block shall coincide approximately with the center of the bearing face of the specimen. The movable portion of the bearing block shall fit closely in the spherical seat, but the design shall be such that the bearing face can be rotated and tilted through small angles in any direction. Accomplish seating by rotating the movable bearing block while the specimen is held in contact with the fixed block.

NOTE 5—False platens with plane bearing faces conforming to the requirements of this method may be used. These shall consist of disks about 0.6 to 0.8 in. (15 to 20 mm) thick, oil-hardened preferably through the disks to more than 58 HRC and surface ground. With abrasive rocks these platens tend to roughen after a number of specimens have been tested, and hence need to be resurfaced from time to time.

3.3 *Axial Strain Determination* (Procedure A)—The axial deformations or strains may be determined from data obtained by electrical resistance strain gages, compressometers, optical devices, or other suitable means. The design of the measuring device shall be such that the average of at least two axial strain measurements can be determined for each increment of load. Measuring positions shall be equally spaced around the circumference of the specimen close to midheight. The gage length over which the axial strains are determined shall be at least 10 grain diameters in magnitude. The axial strains shall be determined with an accuracy of 2 % of the reading and a precision of 0.2 % of full-scale.

NOTE 6—Accuracy should be within 2 % of value of readings above 250 μm/m strain and within 5 μm/m strain for readings lower than 250 μm/m strain.

3.4 *Lateral Strain Determination* (Procedure B)—The lateral deformations or strains may be measured by any of the methods mentioned in 3.3. Either circumferential or diametric deformations (or strains) may be measured. At least two lateral deformation sensors shall be used. These shall be equally spaced around the circumference of the specimen close to midheight. The average deformation (or strain) from the two sensors shall be recorded at each load increment. The use of a single transducer that wraps all the way around the specimen to measure the total change in circumference is also permitted. The gage length and the accuracy and precision of the lateral strain measurement system shall be the same as those specified in 3.3 for the axial direction.

4. Test Specimens

4.1 Test specimens shall be prepared in accordance with Practice D 4543.

4.2 The moisture condition of the sample shall be noted and reported in 8.1.9.

NOTE 7—The moisture condition of the specimen at time of test can have a significant effect upon the strength of the rock and, hence, upon the shape of the deformation curves. Good practice generally dictates that laboratory tests be made upon specimens representative of field conditions. Thus, it follows that the field moisture condition of the specimen should be preserved until time of test. On the other hand, there may be reasons for testing specimens at other moisture contents from saturation to dry. In any case the moisture content of the test specimen should be tailored to the problem at hand. Excess moisture will affect the adhesion of strain gages, if used, and the accuracy of their performance.

5. Procedure (Procedures A and B)

5.1 Check the ability of the spherical seat to rotate freely in its socket before each test.

5.2 Wipe clean the bearing faces of the upper and lower bearing blocks and of the test specimen and place the test specimen with the strain-measuring device attached on the lower bearing block. Carefully align the axis of the specimen with the center of thrust of the spherically seated block. Make electrical connections or adjustments to the strain- or deformation-measuring device. As the load is gradually brought to bear on the specimen, adjust the movable portion of the spherically seated block so that uniform seating is obtained.

5.3 Many rock types fail in a violent manner when loaded to failure in compression. A protective shield should be placed around the test specimen to prevent injury from flying rock fragments.

5.4 Apply the load continuously and without shock to produce an approximately constant rate of load or deformation such that failure would occur within 5 to 15 min from initiation of loading, if carried to failure (Note 8). Record the load and the axial strain or deformation frequently at evenly spaced load intervals during Procedure A of Practice D 4543. Take at least ten readings over the load range to define the axial stress-strain curve. Also record the circumferential or diametric strains (or deformations) at the same increments of load for Procedure B of Practice D 4543. Continuous recording of data with strip chart or X-Y recorders is permitted as long as the precision and accuracy of the recording system meets the requirements in 3.3.

NOTE 8—Results of tests by several investigators have shown that strain rates within this range will provide strength values that are reasonably free from rapid loading effects and reproducible within acceptable tolerances.

6. Calculation (Procedure A)

6.1 The axial strain, ϵ_a, may be recorded directly from strain-indicating equipment, or may be calculated from deformation readings depending upon type of apparatus or instrumentation employed. Calculate the axial strain, ϵ_a, as follows:

$$\epsilon_a = \Delta l / l$$

where:

l = original undeformed axial gage length, in. (mm), and
Δl = change in measured axial length (negative for a decrease in length) in. (mm).

6.2 Calculate the compressive stress in the test specimen from the compressive load on the specimen and the initial computed cross-sectional area as follows:

$$\sigma = -P/A$$

where:

σ = stress, psi (MPa),
P = load, lbf (N), and
A = area, in.^{2} (mm^2).

NOTE 9—Tensile stresses and strains are used as being positive herein. A consistent application of a compression-positive sign convention may be employed if desired.

6.3 Plot the stress versus strain curve for the axial direction

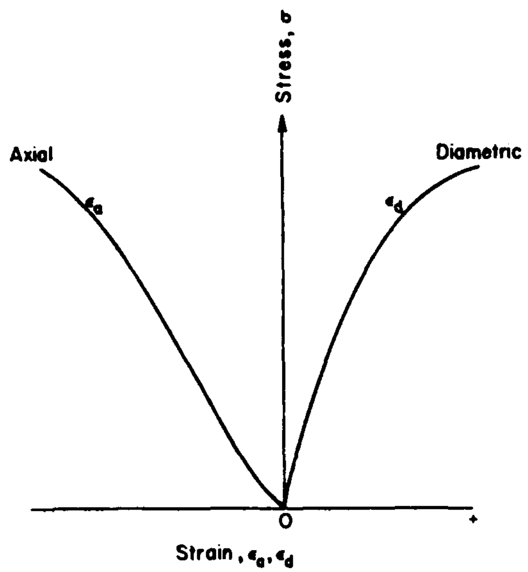


FIG. 1 Format for Graphical Presentation of Data

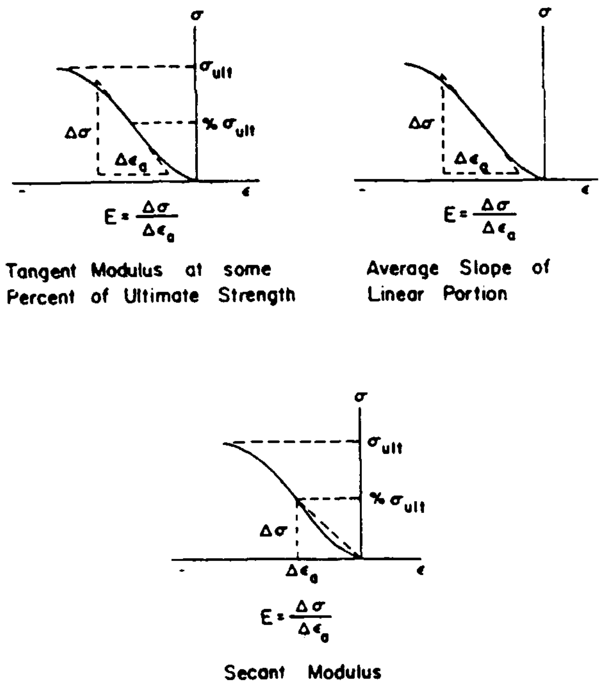


FIG. 2 Methods for Calculating Young's Modulus, E

(Fig. 1). The complete curve gives the best description of the deformation behavior of rocks having nonlinear stress-strain relationships at low and high stress levels.

6.4 The axial Young's modulus, E, may be calculated using any one of several methods employed in engineering practice. The most common methods, described in Fig. 2, are as follows:

6.4.1 Tangent modulus at a stress level which is some fixed percentage of the maximum strength.

6.4.2 Average slope of the more-or-less straight line portion of the stress-strain curve.

6.4.3 Secant modulus, usually from zero stress to some fixed percentage of maximum strength.

7. Calculation (Procedure B)

7.1 The circumferential or diametric strain, ϵ_d, may be recorded directly from strain-indicating equipment, or may be calculated from deformation readings depending upon the type of apparatus or instrumentation employed.

7.1.1 Calculate the diametric strain, ϵ_d, from the following equation:

$$\epsilon_d = \Delta d/d$$

where:

d = original undeformed diameter, in. (mm), and
Δd = change in diameter (positive for an increase in diameter), in. (mm).

NOTE 10—It should be noted that the circumferentially applied electrical resistance strain gages reflect diametric strain, the value necessary in computing Poisson's ratio, ν. Since $C = \pi d$; and $\Delta C = \pi\Delta d$, the circumferential strain, ϵ_c, is related to the diametric strain, ϵ_d, through the relation:

$$\epsilon_c = \Delta C/C = \pi\Delta d/\pi d = \Delta d/d$$

so that $\epsilon_c = \epsilon_d$ where C and d are the specimen circumference and diameter, respectively.

7.2 Plot the stresses calculated in 6.2 versus the corresponding diametric strains determined in 7.1.

7.3 The value of Poisson's ratio, ν, is greatly affected by nonlinearities at low stress levels in the axial and lateral stress-strain curves. It is suggested that Poisson's ratio be calculated from the equation:

$$\nu = -\text{slope of axial curve/slope of lateral curve}$$
$$= -E/\text{slope of lateral curve}$$

where the slope of the lateral curve is determined in the same manner as was done in 6.4 for Young's modulus, E.

NOTE 11—The denominator in the equation in 7.3 will have a negative value if the sign convention is applied properly. The negative sign in the equation thereby assures a positive value for ν.

8. Report

8.1 *Procedure A*—The report shall include the following:

8.1.1 Source of sample including project name and location. Often the location is specified in terms of the drill hole number and depth of specimen from collar of hole.

8.1.2 Date test is performed.

8.1.3 Specimen diameter and height, conformance with dimensional requirements.

8.1.4 Rate of loading or deformation rate.

8.1.5 Values of applied load, stress and axial strain as tabulated results or as recorded on a chart.

8.1.6 Plot of stress versus axial strain as shown in Fig. 1, if data are tabulated in 8.1.5. If data are recorded directly on a chart, the load and deformation axes may be scaled to give stress and strain without replotting the curve.

8.1.7 Young's modulus, E, method of determination as given in Fig. 2, and at what stress level or levels determined.

8.1.8 Physical description of sample including rock type such as sandstone, limestone, granite, etc.; location and orientation of apparent weakness planes, bedding planes, and schistosity; and large inclusions or inhomogeneities, if any.

8.1.9 General indication of moisture condition of sample

at time of test such as as-received, saturated, laboratory air-dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation.

8.2 *Procedure B*—Procedure B of the test method must always accompany Procedure A in order to determine Poisson's ratio. The report for Procedure B, which is in addition to that for Procedure A, shall include the following:

8.2.1 Values of applied load, stress, and diametric strain as tabulated results or as recorded on a chart.

8.2.2 Plot of stress versus diametric strain as shown in Fig. 1 if data is tabulated in 8.2.1. If data is recorded directly on a chart, the load and deformation axes may be scaled to give stress and strain without replotting the curve.

8.2.3 Poisson's ratio, ν, method of determination in 7.3, and at what stress level or levels determined.

9. Precision and Bias

9.1 The variability of rock and resultant inability to determine a true reference value prevent development of a meaningful statement of bias. Data are being evaluated to determine the precision of this test method. In addition, the subcommittee is seeking pertinent data from users of the method.

Designation: D 2664 - 86 RTH 202-89

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa. 19103
Reprinted from the Annual Book of ASTM Standards. Copyright ASTM
If not listed in the current combined index, will appear in the next edition

Standard Test Method for TRIAXIAL COMPRESSIVE STRENGTH OF UNDRAINED ROCK CORE SPECIMENS WITHOUT PORE PRESSURE MEASUREMENTS[1]

This standard is issued under the fixed designation D 2664; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This test method covers the determination of the strength of cylindrical rock core specimens in an undrained state under triaxial compression loading. The test provides data useful in determining the strength and elastic properties of rock, namely: shear strengths at various lateral pressures, angle of internal friction, (angle of shearing resistance), cohesion intercept, and Young's modulus. It should be observed that this method makes no provision for pore pressure measurements. Thus the strength values determined are in terms of total stress, that is, not corrected for pore pressures.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

D4543 Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances[2]

E 4 Practices for Load Verification of Testing Machines[3]

E 122 Recommended Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process[4]

3. Significance and Use

3.1 Rock is known to behave as a function of the confining pressure. The triaxial compression test is commonly used to simulate the stress conditions under which most underground rock masses exist.

4. Apparatus

4.1 *Loading Device*—A suitable device for applying and measuring axial load to the specimen. It shall be of sufficient capacity to apply load at a rate conforming to the requirements specified in 7.2. It shall be verified at suitable time intervals in accordance with the procedures given in Practices E 4 and comply with the requirements prescribed in the method.

4.2 *Pressure-Maintaining Device*—A hydraulic pump, pressure intensifier, or other system of sufficient capacity to maintain constant the desired lateral pressure, σ_3.

NOTE 1—A pressure intensifier as described by Leonard Obert in U.S. Bureau of Mines Report of Investigations No. 6332, "An Inexpensive Triaxial Apparatus for Testing Mine Rock," has been found to fulfill the above requirements.

4.3 *Triaxial Compression Chamber*[5]—An apparatus in which the test specimen may be en-

---

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Oct. 31, 1986. Published December 1986. Originally published as D 2664 - 67. Last previous edition D 2664 - 80.

[2] *Annual Book of ASTM Standards*, Vol 04.08.

[3] *Annual Book of ASTM Standards*, Vols 03.01, 04.02, 07.01, and 08.03.

[4] *Annual Book of ASTM Standards*, Vol 14.02.

[5] Assembly and detail drawings of an apparatus that meets these requirements and which is designed to accommodate 2⅛-in. (53.975-mm) diameter specimens and operate at a lateral fluid pressure of 10 000 psi (689 MPa) are available from Headquarters. Request Adjunct No. 12-426640-00.

closed in an impermeable flexible membrane; placed between two hundred platens, one of which shall be spherically seated; subjected to a constant lateral fluid pressure; and then loaded axially to failure. The platens shall be made of tool steel hardened to a minimum of Rockwell 58 HRC, the bearing faces of which shall not depart from plane surfaces by more than 0.0005 in. (0.0127 mm) when the platens are new and which shall be maintained within a permissible variation of 0.001 in. (0.025 mm). In addition to the platens and membrane, the apparatus shall consist of a high-pressure cylinder with overflow valve, a base, suitable entry ports for filling the cylinder with hydraulic fluid and applying the lateral pressure, and hoses, gages, and valves as needed.

4.4 *Deformation and Strain-Measuring Devices*—High-grade dial micrometers or other measuring devices graduated to read in 0.0001-in. (0.0025-mm) units, and accurate within 0.0001 in. (0.0025 mm) in any 0.0010-in. (0.025-mm) range, and within 0.0002 in. (0.005 mm) in any 0.0100-in. (0.25-mm) range shall be provided for measuring axial deformation due to loading. These may consist of micrometer screws, dial micrometers, or linear variable differential transformers securely attached to the high pressure cylinder.

4.4.1 Electrical resistance strain gages applied directly to the rock specimen in the axial direction may also be used. In addition, the use of circumferentially applied strain gages will permit the observation of data necessary in the calculation of Poisson's ratio. In this case two axial (vertical) gages should be mounted on opposite sides of the specimen at mid-height and two circumferential (horizontal) gages similarly located around the circumference, but in the direction perpendicular to the axial gages.

4.5 *Flexible Membrane*—A flexible membrane of suitable material to exclude the confining fluid from the specimen, and that shall not significantly extrude into abrupt surface pores. It should be sufficiently long to extend well onto the platens and when slightly stretched be of the same diameter as the rock specimen.

NOTE 2—Neoprene rubber tubing of 1/16-in. (1.588-mm) wall thickness and of 40 to 60 Durometer hardness, Shore Type A or various sizes of bicycle inner tubing, have been found generally suitable for this purpose.

5. Sampling

5.1 The specimen shall be selected from the cores to represent a true average of the type of rock under consideration. This can be achieved by visual observations of mineral constituents, grain sizes and shape, partings and defects such as pores and fissures.

6. Test Specimens

6.1 *Preparation*—The test specimens shall be prepared in accordance with Practice D 4543.

6.2 Moisture condition of the specimen at the time of test can have a significant effect upon the indicated strength of the rock. Good practice generally dictates that laboratory tests be made upon specimens representative of field conditions. Thus it follows that the field moisture condition of the specimen should be preserved until the time of test. On the other hand, there may be reasons for testing specimens at other moisture contents, including zero. In any case the moisture content of the test specimen should be tailored to the problem at hand and reported in accordance with 9.1.6.

7. Procedure

7.1 Place the lower platen on the base. Wipe clean the bearing faces of the upper and lower platens and of the test specimen, and place the test specimen on the lower platen. Place the upper platen on the specimen and align properly. Fit the flexible membrane over the specimen and platen and install rubber or neoprene O-rings to seal the specimen from the confining fluid. Place the cylinder over the specimen, ensuring proper seal with the base, and connect the hydraulic pressure lines. Position the deformation measuring device and fill the chamber with hydraulic fluid. Apply a slight axial load, approximately 25 lbf (110 N), to the triaxial compression chamber by means of the loading device in order to properly seat the bearing parts of the apparatus. Take an initial reading on the deformation device. Slowly raise the lateral fluid pressure to the predetermined test level and at the same time apply sufficient axial load to prevent the deformation measuring device from deviating from the initial reading. When the predetermined test level of fluid pressure is reached note and record the axial load registered by the loading device. Consider this load to be the zero or starting load for the test.

7.2 Apply the axial load continuously and without shock until the load becomes constant, or reduces, or a predetermined amount of strain is achieved. Apply the load in such a manner as to produce a strain rate as constant as feasible throughout the test. Do not permit the strain rate at any given time to deviate by more than 10 % from that selected. The strain rate selected should be that which will produce failure of a similar test specimen in unconfined compression, in a test time of between 2 and 15 min. The selected strain rate for a given rock type shall be adhered to for all tests in a given series of investigation (Note 3). Maintain constant the predetermined confining pressure throughout the test and observe and record readings of deformation as required.

NOTE 3—Results of tests by other investigators have shown that strain rates within this range will provide strength values that are reasonably free from rapid loading effects and reproducible within acceptable tolerances.

7.3 To make sure that no testing fluid has penetrated into the specimen, the specimen membrane shall be carefully checked for fissures or punctures at the completion of each triaxial test. If in question, weigh the specimen before and after the test.

8. Calculations

8.1 Make the following calculations and graphical plots:

8.1.1 Construct a stress difference versus axial strain curve (Note 5). Stress difference is defined as the maximum principal axial stress, σ_1, minus the lateral pressure, σ_3. Indicate the value of the lateral pressure, σ_3, on the curve.

NOTE 4—If the specimen diameter is not the same as the piston diameter through the chamber, a correction must be applied to the measured load to account for differences in area between the specimen and the loading piston where it passes through the seals into the chamber.

NOTE 5—If the total deformation is recorded during the test, suitable calibration for apparatus deformation must be made. This may be accomplished by inserting into the apparatus a steel cylinder having known elastic properties and observing differences in deformation between the assembly and steel cylinder throughout the loading range. The apparatus deformation is then subtracted from the total deformation at each increment of load in order to arrive at specimen deformation from which the axial strain of the specimen is computed.

8.1.2 Construct the Mohr stress circles on an arithmetic plot with shear stresses as ordinates and normal stresses as abscissas. Make at least three triaxial compression tests, each at a different confining pressure, on the same material to define the envelope to the Mohr stress circles.

NOTE 6—Because of the heterogeneous nature of rock and the scatter in results often encountered, it is considered good practice to make at least three tests of essentially identical specimens at each confining pressure or single tests at nine different confining pressures covering the range investigated. Individual stress circles shall be plotted and considered in drawing the envelope.

8.1.3 Draw a "best-fit", smooth curve (the Mohr envelope) approximately tangent to the Mohr circles as in Fig. 1. The figure shall also include a brief note indicating whether a pronounced failure plane was or was not developed during the test and the inclination of this plane with reference to the plane of major principal stress.

NOTE 7—If the envelope is a straight line, the angle the line makes with the horizontal shall be reported as the angle of interval friction, ϕ (or the slope of the line as tan ϕ depending upon preference) and the intercept of this line at the vertical axis reported as the cohesion intercept, C. If the envelope is not a straight line, values of ϕ (or tan ϕ) should be determined by constructing a tangent to the Mohr circle for each confining stress at the point of contact with the envelope and the corresponding cohesion intercept noted.

9. Report

9.1 The report shall include as much of the following as possible:

9.1.1 Sources of the specimen including project name and location, and if known, storage environment. The location is frequently specified in terms of the borehole number and depth of specimen from collar of hole.

9.1.2 Physical description of the specimen including rock type; location and orientation of apparent weakness planes, bedding planes, and schistosity; large inclusions or inhomogeneities, if any.

9.1.3 Dates of sampling and testing.

9.1.4 Specimen diameter and length, conformance with dimensional requirements.

9.1.5 Rate of loading or deformation or strain rate.

9.1.6 General indication of moisture condition of the specimen at time of test such as: as-received, saturated, laboratory air-dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible

and reported as either water content or degree of saturation.

9.1.7 Type and location of failure. A sketch of the fractured specimen is recommended.

NOTE 8—If it is a ductile failure and $\sigma_1 - \sigma_3$, is still increasing when the test is terminated, the maximum strain at which $\sigma_1 - \sigma_3$ is obtained shall be clearly stated.

10. Precision and Bias

10.1 The variability of rock and resultant inability to determine a true reference value prevent development of a meaningful statement of bias. Data are being evaluated to determine the precision of this test method. In addition, the subcommittee is seeking pertinent data from users of the method.

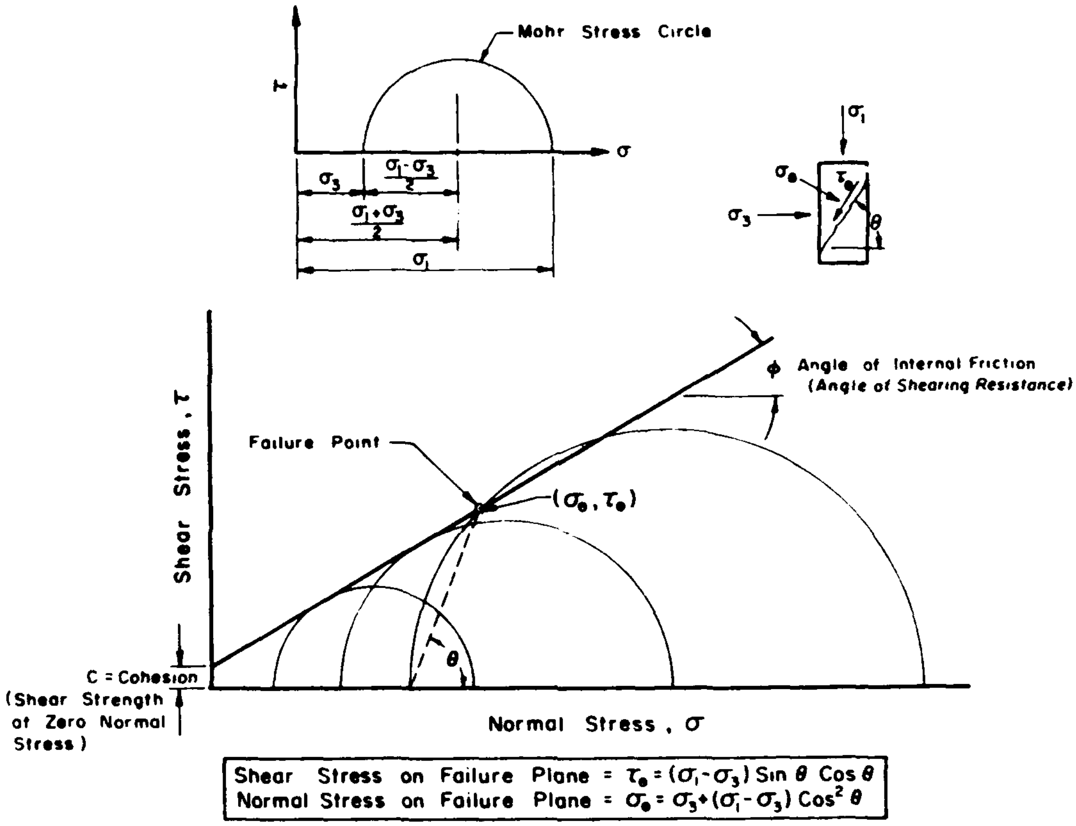


FIG. 1 Typical Mohr Stress Circles

DIRECT SHEAR STRENGTH OF ROCK CORE SPECIMENS

1. Scope

1.1 This method describes apparatus and procedures for determining the shear strength of a rock material in direct shear. The test can be made on rock core specimens from 2 to 6 in. (5 to 15 cm) in diameter. The test can be made on intact specimens to determine intact shear strength, on intact specimens with recognizable thin weak planes to determine the shearing resistance along these planes, on presawn shear surfaces to determine lower bound residual shear strengths, and on rock core to concrete bond specimens to determine the shearing resistance between the bond. The principle of the rock core direct shearing is illustrated schematically in Fig. 1.

1.2 A minimum of three test specimens of any rock type are subjected to different but constant normal stresses during the shearing process. For each type of intact rock, cohesion and an angle of internal friction are determined. For each type of rock with sawn failure surfaces, a lower bound residual angle of internal friction is determined.

1.3 The test is not suited to the development of exact stress-strain relationships within the test specimen because of the nonuniform distribution of shearing stresses and displacements. Care should be taken so that the testing conditions represent those being investigated. The results of these tests are used where field design requirements dictate unconsolidated, undrained parameters.

2. Apparatus

2.1 Test Specimen Saw - For cores of 3 to 6 in. (7.5 to 15 cm) in diameter, use a rock saw with 20-in.- (50-cm-) diam safety abrasive blade fitted for dry and for wet cutting. Alternatively for wet cutting, a diamond blade may be used. For cores 2 to 2-1/2 in. (5 to 6.25 cm) in diameter, a rock saw with 12-in.- (60-cm-) diam blade should be used.

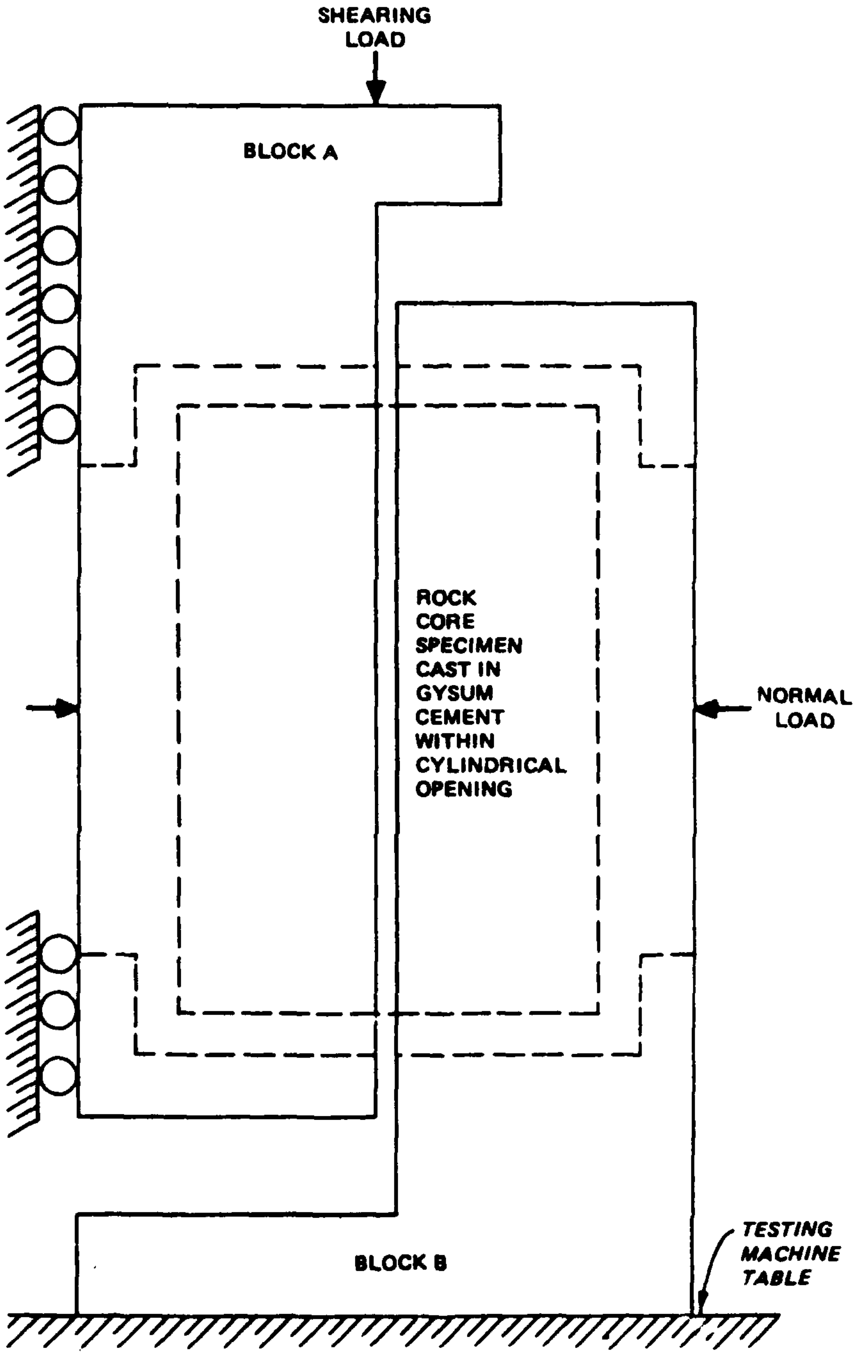


Fig 1. Schematic showing direct shear of rock core.

2.2 Shear Device - The shear device shall consist of a pair of shear boxes constructed so as to provide a means of applying a normal stress to the face of the specimen while applying a force to shear the specimen along a predetermined plane parallel to the vertical axis of the specimen. The device shall securely hold the specimen in such a way that torque cannot be applied to the specimen. The shear boxes that hold the specimen shall be sufficiently rigid to prevent their distortion during shearing. The various parts of the shear device shall be made of material not subject to corrosion by substances within the rock or moisture within the rock.

2.2.1 Shear boxes suitable for testing specimens from 3 to 6 in. (7.5 to 15 cm) in diameter should each have a recess 6-5/8 in. (16.6 cm) in diameter and 2-1/4 in. (5.67 cm) deep. Smaller shear boxes for 2- to 2-1/2-in.- (5- to 6.25-cm-) diam specimens should each have a recess 2-7/8 in. (7.36 cm) in diameter and 2 in. (5 cm) in depth. The shear boxes should be designed for a shear travel greater than 10 percent of the specimen shear plane length.

2.2.2 In both cases the two shear boxes assembled with the specimen shall be placed within a framework constructed so as to hold the boxes in proper position during testing. The framework shall include a pair of hardened stainless steel plates machined to accommodate roller bearings or ball bearings for minimizing friction of the moving shear box as indicated in Fig. 1. The roller plate device should ensure that resistance of the equipment to shear displacement is less than 1 percent of the maximum shear force applied in the test.

2.2.3 The shear device framework shall include capability of providing a submerging tank for tests in which maintaining specimen saturation is important to duplicate field conditions.

2.3 Loading Devices

2.3.1 Normal Force - The normal force device shall be capable of applying the specified force quickly without exceeding it and capable of maintaining it with an accuracy of ±2 percent for the duration of the

test. The device shall have a travel greater than the amount of dilation or compression to be expected.

2.3.2 Shear Force - The device for applying the shear force shall distribute the load uniformly along one-half face of the specimen with the resultant applied shear force acting in the plane of shearing. The required capabilities will depend upon whether a controlled-displacement test or a controlled-stress test is used. Controlled-displacement equipment shall be capable of shearing the specimen at a uniform rate of displacement with less than ±15 percent deviation and shall permit adjustment of the rate of displacement over a relatively wide range. Controlled-stress equipment shall be capable of applying the shear force in increments to the specimen in the same manner and to the same degree of accuracy as that described in 2.3.1.

2.4 Displacement Indicators - Equipment for measuring shear and normal displacements may consist of mechanical devices, such as dial gages or electric transducers. Displacement indicators shall have a sensitivity of at least 0.001 in. (0.025 mm). The shear displacement measuring system shall have a travel greater than 10 percent of the specimen shear plane length. Normal displacement systems shall have the capability of measuring both dilation and compression of the specimen. Resetting of gages during the test should if possible be avoided. If electric transducers or an automatic recording system is used, a recent calibration shall be included in the report.

2.5 Casting Compound - High-strength gypsum cement (such as hydrostone) or a capping compound (such as leadite) should be used to hold the test specimen in the recesses of the test device.

2.6 Spacer Plate - The spacer plate separating the shear boxes for development of the shear zone shall be 1/16 in. (1.6 mm) thick and constructed of a noncorrosive material.

3. Test Specimen

3.1 Intact Specimens

3.1.1 Test specimens shall be prepared by sawing rock cores into 3- to 4-in. (7.5- to 10-cm) lengths (Note 1). The diameter of each specimen shall be measured to the nearest 0.01 in. (0.025 mm) at several different positions along the length of the specimen axis. The average diameter shall be used to compute the cross-sectional area of the specimen. The volume of the specimen shall be determined by the volumetric or displacement method presented in EM 1110-2-1906, "Laboratory Soils Testing." The initial weight of the specimen shall be determined to the nearest 0.1 g for subsequent use in determining initial moisture content and density.

NOTE 1—Soft rock such as clay shales may be as short as 2-1/2 in. (6.25 cm) if material is scarce. Although helpful in the setup, ends of the test specimens need not be smooth, flat, nor square with the axis of the core. Generally, harder rocks are best cut in the wet; softer rocks are best cut in the dry, depending on fissility and reaction to pressure of cutting water.

3.1.2 A block of the shear box shall be set on a flat surface with the shear surface up. The inside of the recess shall be lightly coated with lubricant. A grout of the gypsum cement (hydrostone) and water shall be placed in the recess to approximately the one-third or midpoint. After approximately three minutes of setting, the specimen shall be set or pushed into the grout until the approximate midpoint (desired shear plane) of the specimen is opposite the top recess (Note 2). Excess grout shall be screeded off at the shear plane (Note 3).

NOTE 2—To prepare intact test specimens for testing along recognizable thin weak planes, orient the specimen so that the plane of weakness is parallel with the 1/16-in. (1.6-mm) shear gap provided by the spacer plate.

NOTE 3—Gypsum cement grout has only a few minutes pot life; hence a fresh mix will have to be prepared for each block. An alternative to gypsum cement grout for holding the test specimen in the recesses is capping compound, such as leadite. The procedure for preparing specimens with a capping compound is essentially the same as for gypsum cement. Capping compound has a shorter pot life after pouring than gypsum cement and must be heated to proper temperature and handled quickly and with great care. An overnight curing period is generally required. Capping compound is stronger in compression and shear than gypsum cement and is preferred for hard rock testing. Because capping compound must be placed hot, it should not be used to secure specimens subject to structural damage with loss of natural moisture or for tests in which it is desirable to maintain natural moisture.

3.1.3 A 1/16-in.- (1.6-mm-) thick spacer plate having a hole equal to the diameter of the specimen and split on a diameter from the front to the back of the block shall be coated with lubricant and placed on the block around the specimen. The spacer separates the two blocks of the shear box to prevent friction between the blocks during shearing. The recess of the remaining shear box block shall be lightly coated with lubricant and the block placed over the now protruding half of the specimen. The two blocks shall be aligned and temporarily clamped together with C clamps. The recess between the top block and specimen shall then be filled with the gypsum cement grout using appropriate tools to rod the grout thoroughly around the specimen. For soft rock such as clay shale, a 2-hour curing is usually sufficient before loading. For hard rocks, the grout must be allowed to cure overnight.

3.1.4 At the end of the curing period, the two halves of the spacer shall be pulled out and the C clamps removed. The specimens secured in the shear boxes are then ready for further assembly and shear testing.

3.2 Presawn Shear Surfaces - Test specimens shall be prepared the same as presented in paragraphs 3.1.1 to 3.1.4, except that the specimen shall be sawn in half near the center length before grouting the

specimen in the shear box blocks. The presawn shear surface shall be smooth and oriented in the shear box so as to be centered within the 1/16-in. (1.6-mm) shear gap provided by the spacer plate.

3.3 Concrete to Rock Core Bond

3.3.1 Test specimens shall be prepared by sawing rock cores into 1.5- to 2-in. (3.75- to 5-cm) length. The sawn specimen shall be tightly encased in the bottom of a 3- to 4-in.- (7.5- to 10-cm-) high mold (Note 4) with the smooth sawn surface (shear plane) facing upward and perpendicular to the axis of the mold. The remaining portion of the mold shall be filled with concrete, which is then consolidated and cured according to the procedures presented in CRD-C 10-73 (Note 5). The concrete mix design shall be compatible in consistency and strength with the anticipated field design mix and have a maximum aggregate no larger than 1/6 of the specimen diameter.

NOTE 4—Molds shall be made of steel, cast iron, or other nonabsorbent material, nonreactive with concrete containing portland or other hydraulic cements. Mold diameters shall conform to the dimensions of the rock core test specimen. Molds shall hold their dimensions and shape and be watertight under conditions adverse to use.

NOTE 5—"Handbook for Concrete and Cement," U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, published in quarterly supplements.

3.3.2 Procedures for measuring specimen weight, diameter, and volume are the same as presented in paragraph 3.1.1. Procedures for securing the test specimen in the shear box are the same as presented in paragraphs 3.1.2 to 3.1.4.

4. Procedure

4.1 Following the removal of the spacer plates and C clamps, transfer final assembly operations to the test shear and normal load area. Final assembly of the testing apparatus, to include orientation of the resultant normal and shear loads, will depend on the equipment utilized

in the testing. In general, the resultant of the normal load shall react through the axial center of the specimen, and the shear load shall react through the radial center of the specimen so as to pass through the shear plane. Position or activate, or both, the displacement indicators for measuring shear deformation and changes in specimen thickness.

4.2 Apply the selected normal force (normal stress " cross-section area) to the specimen as rapidly as practical (Note 6). Record and allow any initial elastic compression of the specimen to reach equilibrium. For those tests where applicable, as soon as possible after applying the initial normal force, fill the water reservoir to at least submerge the shear plane.

NOTE 6—The normal force used for each of the three or more specimens will depend upon the input information required for field analysis and/or design.

4.3 Shear the specimen.

4.3.1 After any elastic compression has reached equilibrium, apply the shearing force and shear the specimen. In a controlled-displacement test, the rate of displacement shall be less than 0.004 in./min (0.1 mm/min) until peak strength is reached. Approximately 10 sets of readings should be taken before reaching peak strength. If it is desired to determine the ultimate strength, the normal load shall be relieved and the specimen recentered. The normal load is then reapplied and the specimen sheared again. The rate of shear displacement to determine the ultimate strength shall be no greater than 0.01 in./min (0.25 mm/min). Readings should be taken at increments of from 0.02- to 0.2-in. (0.5- to 5-mm) shear displacement as required to adequately define the force-displacement curves.

4.3.2 In a controlled-stress test the rate of stress application should not exceed 5 psi/min (34.47 kPa/min) for soft rock (such as clay shale) and up to 100 psi (689.4 kPa) for the very hardest rock.

Concurrent time, shear load, and deformation readings shall be taken at convenient intervals (a minimum of 10 readings before reaching peak strength). After reaching peak strength, the ultimate strength may be determined as presented in paragraph 4.3.1.

5. Calculations

5.1 Calculate the following:

5.1.1 Initial cross-sectional area.

5.1.2 Initial water content.

5.1.3 Initial wet and dry unit weights.

5.1.4 Shear stress data.

5.1.5 Initial and final degrees of saturation, if desirable.

6. Report

6.1 The report shall include the following:

6.1.1 Description of type of shear device used in the test.

6.1.2 Identification and description of the sample.

6.1.3 Description of the shear surface.

6.1.4 Initial water content.

6.1.5 Initial wet and dry unit weights.

6.1.6 Initial and final degrees of saturation, if desirable.

6.1.7 All basic test data including normal stress, shear displacement, corresponding shear resistance values, and specimen thickness changes.

6.1.8 For each test specimen, a plot of shear and specimen thickness change versus shear displacement and a plot of composite maximum and ultimate shear stress versus normal stress.

6.1.9 Departures from the procedure outlined, such as special loading sequences or special wetting requirements.

RTH 204-80

STANDARD METHOD OF TEST FOR MULTISTAGE TRIAXIAL STRENGTH OF UNDRAINED ROCK CORE SPECIMENS WITHOUT PORE PRESSURE MEASUREMENTS

1. Scope

1.1 This method covers the determination of the strength of cylindrical rock specimens in an undrained state under multistage triaxial loading. The test provides data useful in delineating the strength of joints, seams, bedding planes, etc. This method makes no provision for pore pressure measurements. Thus, the strength values determined are in forms of total stress, i.e. not corrected for pore pressures.

2. Apparatus

2.1 The apparatus is identical with that used in RTH 202, "Triaxial Compressive Strength of Undrained Rock Core Specimens Without Pore Pressure Measurements."

3. Test Specimens

3.1 In the case of intact specimens that develop a well-defined shear failure plane, it is possible to continue testing beyond the first failure; that is, the confining pressure can be raised to a higher level and another peak stress recorded. This may be done immediately following the completion of a conventional triaxial test as conducted according to RTH 202.

3.2 Multistage testing can also be used with cores that are intact initially. The key factor in making these studies is the use of specimens with the failure plane preestablished to cause failure along an inherent weakness, such as seams, open joints, bedding planes, faults, schistosity bands, or laminations. These planes of weakness, when tested, should be oriented at 45 to 65 deg (0.79 to 1.14 radians) from the horizontal, which will normally produce a failure in the preoriented zone. When including these specific geologic features in an NX size core, the specimens can be drilled from 6-in. (15-cm) or larger cores by suitable orientation in the drilling apparatus. Care must be taken when coring these specimens to prevent breakage. However, if the core is

broken along a weakness plane, it may still be tested as an open joint. Broken cores can be taped together with plastic tape, only sufficiently to maintain the matching contact between the broken parts. The test specimens are then prepared in the manner described in Section 3 of RTH 202.

4. Procedure

4.1 The test procedure described in Section 4 of RTH 202 shall be used for the first stage of test. Subsequent stages should be achieved in like manner by applying progressively higher levels of lateral fluid pressure. This may be done as many times as desired, provided the total strain does not cause excessive misalignment of the steel platens and an eccentric loading. This procedure is referred to as multistage testing. Fig. 1 illustrates this loading sequence.

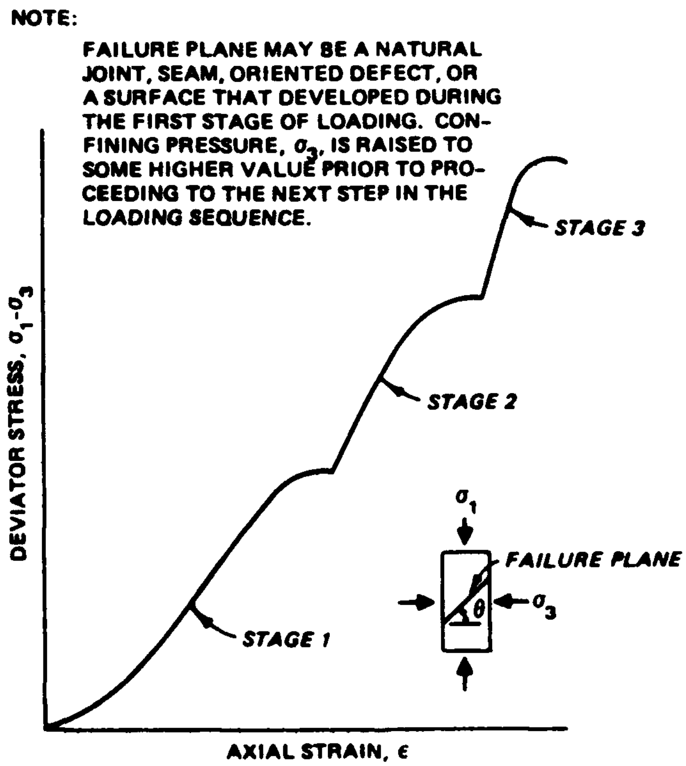


Fig. 1. Stress-strain curve for multistage triaxial test.

5. Calculations

5.1 The shear strength on the joint can be determined graphically by constructing a Mohr circle as shown in Fig. 2. Proof of this construction may be found in most soil mechanics texts.[1] For a multistage test on a given specimen, several Mohr's circles can be drawn and the same angle used to plot the stresses on the failure plane. A strength envelope for this condition, Fig. 3, which is the average of the results determined for each Mohr's circle, is considered to be the joint friction angle.

5.2 There are variations of this plotting technique that may also be employed. The multistage test described above produces a joint friction angle from a single specimen. For a strength envelope derived from tests of several intact specimens, plotting failure plane stresses would yield a higher limiting strength criterion than that of open joints. To report this type of data properly, all orientation data must be carefully and fully stated to ensure proper interpretation of results.

5.3 Various orientations of seams may also be tested to determine that which is most critical. Direct tension and unconfined compression tests may be included to completely define the strength envelope as shown in Fig. 4.

6. Report

6.1 In addition to the plots discussed in Section 5, the report should include the following:

6.1.1 Lithologic description of the rock, including the type of joint, seam, etc., tested.

6.1.2 Source of sample including depth and orientation, dates of sampling and testing, and storage environment.

6.1.3 Specimen diameter and height.

---

[1] Taylor, D., "Fundamentals of Soil Mechanics," John Wiley and Sons, Inc., p 317, or Spangler, M., "Soil Engineering," International Textbook Co., p 277.

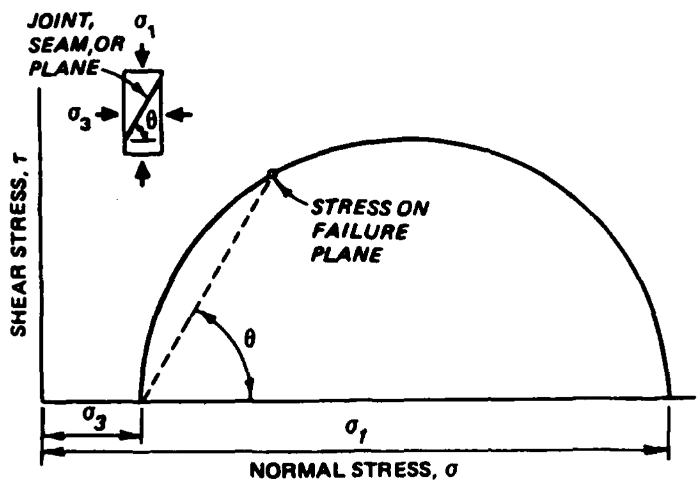


Fig. 2. Mohr circle showing method of construction for locating stresses in failure plane.

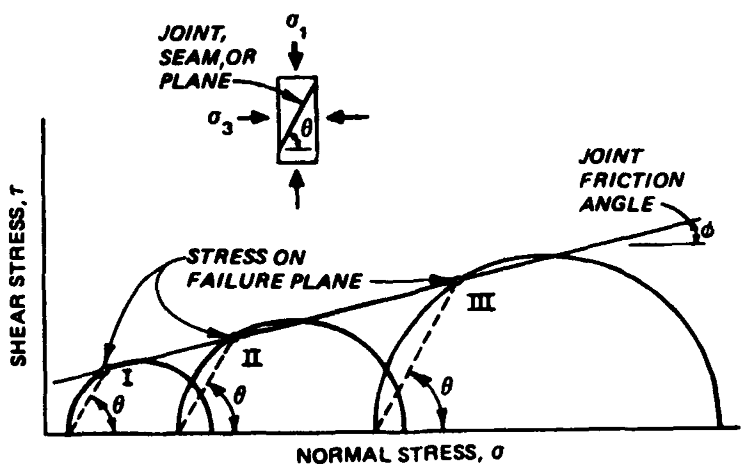


Fig. 3. Mohr diagram for locating stresses on failure plane in a multistage test.

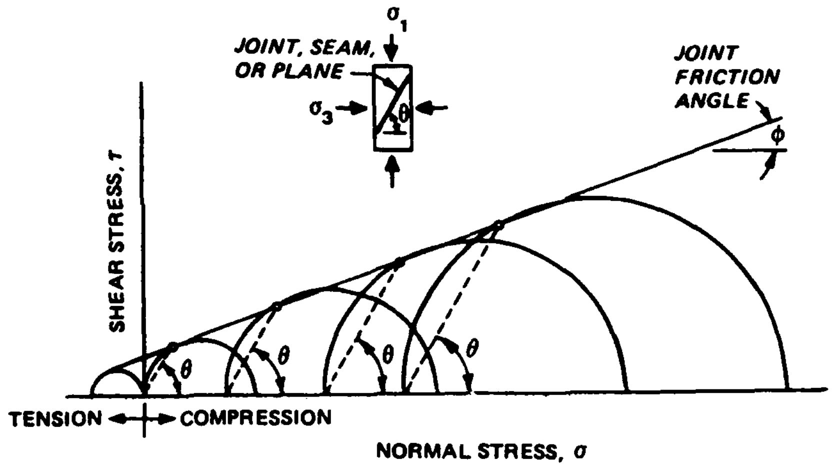


Fig. 4. Mohr diagram for locating stresses on failure plane, including direct tension and unconfined compression test data.

6.1.4 Moisture content and degree of saturation at time of test.

6.1.5 Other physical data, such as specific gravity, absorption, porosity, and permeability, citing the method of determination for each.

STANDARD METHOD OF TEST FOR CREEP OF ROCK IN COMPRESSION

1. Scope

1.1 This method covers the determination of the creep of rock cores subjected to sustained longitudinal compressive load. It is used to compare specimens under controlled conditions.

2. Apparatus

2.1 Loading Frame - The loading frame shall be capable of applying and maintaining the required load on the specimen, despite any change in the dimension of the specimen. In its simplest form the loading frame consists of header plates bearing on the ends of the loaded specimens, a load-maintaining element that may be either a spring of a hydraulic capsule or ram, and threaded rods to take the reaction of the loaded system. Bearing surfaces of the header plates shall not depart from a plane by more than 0.001 in. (0.025 mm). In any loading frame, several specimens may be stacked for simultaneous loading. The length between header plates shall not exceed 70 in. (1780 mm). When a hydraulic load-maintaining element is used, several frames may be loaded simultaneously through a central hydraulic pressure-regulating unit consisting of an accumulator, a regulator, indicating gages, and a source of high pressure, such as a cylinder of nitrogen or a high-pressure pump. Springs such as railroad car springs may be used to maintain the load in frames similar to those described above; the initial compression shall be applied by means of a portable jack or testing machine. When springs are used, care should be taken to provide a spherical head or ball joint, and end plates rigid enough to ensure uniform loading of the specimens. Means shall be provided for measuring the load to the nearest 2 percent of total applied load. This may be a permanently installed hydraulic pressure gage or a hydraulic jack and a load cell inserted in the frame when the load is applied or adjusted.

2.2 Strain-Measuring Device - Suitable apparatus shall be provided for the measurement of longitudinal strain in the specimen to the nearest 10 millionths. The apparatus may be attached or portable. If a portable apparatus is used, gage points shall be attached to the specimen in a positive manner. Attached gages relying on friction contact are not permissible. Strains shall be measured on not less than two gage lines spaced uniformly around the periphery of the specimen. The gages may be instrumented so that the average strain on all gage lines can be read directly. The effective gage length shall be at least 10 times the maximum grain size of the rock. The strain-measuring device shall be capable of measuring strains for at least 1 year without change in calibration (Note 1).

NOTE 1--Systems in which the varying strains are compared with a constant-length standard bar are considered most reliable, but unbonded electrical strain gages are satisfactory.

3. Test Specimens

3.1 Specimen Size - The diameter of each specimen shall be at least 2-1/8 in. (55 mm), and the length shall be at least twice the diameter. When the ends of the specimen are in contact with steel bearing plates, the specimen length shall be at least equal to the gage length of the strain-measuring apparatus plus the diameter of the specimen. When the ends of the specimen are in contact with other specimens similar to the test specimen, the specimen length shall be at least equal to the gage length of the strain-measuring apparatus plus 1-1/2 in. (38 mm). Between the test specimen and the steel bearing plate at each end of a stack, a supplementary noninstrumental core whose diameter is equal to that of the test specimen and whose length is at least half its diameter shall be installed.

3.2 Preparation - The specimens shall be prepared in the manner prescribed in paragraph 3.1 of RTH 111.

4. Procedure

4.1 To prevent change of moisture condition during test, the specimens should be covered or coated before loading. Load the specimens as quickly as practical (Note 2). Take strain readings immediately before and after loading, 2 to 6 hours later, then daily for 1 week, weekly until the end of 1 month, and monthly until the end of 1 year. Before taking each strain reading, measure the load. If the load varies more than 2 percent from the correct value, it must be adjusted (Note 3).

NOTE 2--If loading is not accomplished expeditiously, some creep may occur before the after-loading strain is observed. In placing creep specimens in the frame, take care in aligning the specimens to avoid eccentric loading. When specimens are stacked, it may be helpful to apply a small preload such that the resultant stress does not exceed 200 psi (1380 kPa) and note the strain variation around each specimen, after which the load may be removed and the specimens realigned for greater strain uniformity.

NOTE 3--Where springs are used to maintain the load, the adjustment can be accomplished by applying the correct load and tightening the nuts on the threaded reaction rods.

5. Calculations

5.1 Calculate the total load-induced strain per pound per square inch (or kilopascal) at any time as the difference between the average strain values of the loaded and control specimens divided by the average stress. To determine creep strain per pound per square inch (or kilopascal) for any age, subtract from the total strain per pound per square inch (or kilopascal) at that age the strain per pound per square inch (or kilopascal) immediately after loading. Depending on the type of relationship, plot on linear or semilog coordinate paper the creep strain-time function and determine the creep rate (Note 4).

NOTE 4--Treatment of creep data is not a standardized technique. The experimenter is referred to the references in paragraph 7 for further information.

6. Report

6.1 The report should include the following:

6.1.1 Lithologic description of the rock and direction of test with respect to lithology.

6.1.2 Source of sample including depth and orientation and dates of sampling and testing.

6.1.3 Storage environment and moisture content during storage and test.

6.1.4 Specimen diameter and height.

6.1.5 Type of strain-measuring device.

6.1.6 Intensity of applied load.

6.1.7 Initial elastic strain.

6.1.8 Creep strain per pound per square inch (or kilopascal) at designated ages up to 1 year.

6.1.9 Creep rate.

7. References

7.1 The following references contain information on the treatment of various types of rock creep:

(a) U. S. Army Engineer Waterways Experiment Station, CE, "The Creep of Rocks" (Le Fluage des Roches), Translation No. 67-8, Sep 1967, translated by W. W. Geddings, Jr., from Annales de L'Institut Technique du Batement et das Travaux Publics, Vol 19, No. 217, Jan 1966, pp 89-112.

(b) Iida, K., Wada, T., Aida, Y., Shichi, R., "Measurements of Creep in Igneous Rocks," Journal of Earth Sciences Najoya University, Vol 8, No. 1, Mar 1960, pp 1-16.

(c) Lomnitz, C., "Creep Measurement in Igneous Rocks," Journal of Geology, Vol 64, No. 5, pp 473-479.

(d) Griggs, David, "Creep of Rocks," Journal of Geology, Vol 47, No. 3, Apr-May 1939, pp 225-251.

METHOD OF TEST FOR THERMAL DIFFUSIVITY OF ROCK

1. Scope

1.1 This method of test outlines a procedure for determining the thermal diffusivity of rock. The thermal diffusivity is equal to the thermal conductivity divided by the heat capacity per unit volume and may be used as an index of the facility with which the material will undergo temperature change.

2. Apparatus

2.1 The apparatus shall consist of:

2.1.1 Bath - A heating bath in which specimens can be raised to uniform high temperature (212°F (100°C)).

2.1.2 Diffusion Chamber - A diffusion chamber containing running cold water.

2.1.3 Temperature-Indicating or Recording Instrument - Consisting of iron-constantan thermocouples, Type K potentiometer, ice bath, standard cell, galvanometer, switch, and storage battery; or thermocouples and suitable recording potentiometer.

2.1.4 Timer - Timer capable of indicating minutes and seconds.

3. Procedure

3.1 Preparation of Specimen - The test specimen shall be a 6- by 12-in. (152- by 305-mm) core (for other shapes and sizes, see Section 5). A thermocouple shall be inserted in an axially drilled hole 3/8 in. (9.5 mm) in diameter and subsequently grouted.

3.2 Heating - Each specimen shall be heated to the same temperature by continuous immersion in boiling water until the temperature of the center is 212°F (100°C). The specimen shall then be transferred to a bath of running cold water and suspended in the bath so that the entire surface of the specimen is in contact with the water. The temperature of the cold water shall be determined by means of another thermocouple.

3.3 Cooling - The cooling history of the specimen shall be obtained from readings of the temperature of the interior of the specimen at 1-minute intervals from the time the temperature difference between the center and the water is 120°F (67°C) until the temperature difference between the center and water is 8°F (4°C). The data shall be recorded. Two such cooling histories shall be obtained for each test specimen, and the calculated diffusivities shall check within 40.002 ft^2/h ($0.0052 \cdot 10^{-5}$ m^2/s).

4. Calculations

4.1 The temperature difference in degrees F shall be plotted against the time in minutes on a semilogarithmic scale. The best possible straight line shall then be drawn through the points so obtained. A typical graph is shown in Fig. 1. The time elapsed between the temperature differences of 80°F (44°C) and 20°F (11°C) shall be read from the graph, and this value inserted in the equation below, from which the thermal diffusivity, α , shall be calculated as follows:

$$\alpha = 0.812278/(t_1 - t_2)$$

where

α = thermal diffusivity, ft^2/hr (Note 1)

$(t_1 - t_2)$ = elapsed time between temperature differences of 80°F (44°C) and 20°F (11°C), minutes

0.812278 = numerical factor applicable to 6- by 12-in. (152- by 305-mm) cylinder

NOTE 1--The SI equivalent of ft^2/hr is m^2/s; $ft^2/hr \cdot 2.580640$ $E - 05 = m^2/s$.

5. Specimens of Other Sizes and Shapes

5.1 The method given above is directly applicable to a 6- by 12-in. (152- by 305-mm) cylinder. Specimens of other sizes and shapes may be treated in the manner described below.

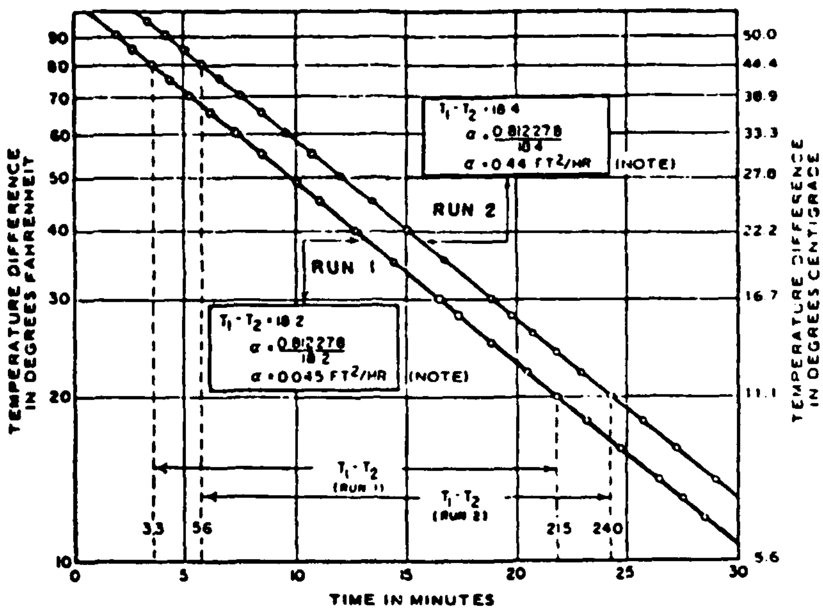


Fig. 1. Calculation of thermal diffusivity.

5.2 The thermal diffusivity of a specimen of regular shape is, to a first approximation,

$$\alpha = M/(t_2 - t_1)$$

where α = thermal diffusivity, ft^2/hr

M = a factor depending on the size and shape of the specimen

t_1, t_2 = times at which the center of the specimen reaches any specified temperature differences, minutes

5.3 For a prism,

$$M = \frac{60 \ln(T_1/T_2)}{\pi^2\left(\frac{1}{a^2} + \frac{1}{b^2} + \frac{1}{c^2}\right)}$$

where $\ln(T_1/T_2)$ = natural logarithm of the temperature difference ratio

T_1, T_2 = temperature differences at times t_1 and t_2, deg F

a, b, c = dimensions of prism, feet

5.4 For a cylinder,

$$M = \frac{60 \ln(T_1/T_2)}{\left(\frac{5.783}{r^2} + \frac{\pi^2}{l^2}\right)}$$

where $\ln(T_1/T_2)$ = natural logarithm, as above

r = radius of cylinder, feet

l = length of cylinder, feet

5.5 For specimens whose minimum dimension is more than 3 in. (76 mm), this approximate calculation will yield the required accuracy. For smaller specimens or when more precise determinations are desired, reference may be made to "Heat Conduction," by L. R. and A. C. Ingersoll, and O. J. Zebel, McGraw-Hill Book Company, Inc., 1948, pp. 183-185 and appended tables. Charts which may be used are also found in Williamson and Adams, Phys. Rev. XIV, p. 99 (1919) and "Heat Transmission," W. H. McAdams, McGraw-Hill Book Company, Inc., 1942, pp. 27-44.

PART II. IN SITU TEST METHODS

A. Rock Mass Monitoring

USE OF INCLINOMETERS FOR ROCK MASS MONITORING

1. Scope

1.1 This method describes the use of inclinometers for rock mass monitoring, lists some available instruments, outlines operating techniques and maintenance requirements, and presents data reduction methods.

2. Apparatus

2.1 An inclinometer is a device for measuring the deviation from the vertical of a flexible casing installed in a borehole. Deviations can be converted to displacements by trigonometric functions. Successive measurements enable the determination of the depth, magnitude, and rate of lateral movement. Fig. 1 shows a typical inclinometer installation.[6.1]

2.2 Many types of inclinometers are commercially available (Tables 1 and 2); however, the most commonly used is the probe type. This type consists of a control box and a probe which is lowered into the casing on a cable. In some probes a cantilevered pendulum with resistance strain gages, vibrating wire, or inductive transducers is used to measure cantilever deflection.[6.2] Other probes use the Wheatstone bridge principle (Slope Inclinometer Model 200 B), the servo accelerometer principle (Slope Indicator Digitilt), or a differential transformer (Dames and Moore, EDR). The probe generally requires a special flexible casing as indicated in Table 2. The electrical output from the probe is measured at the control box and converted to visual display, punched tape, or graphic form.

3. Procedure

3.1 Installation - Inclinometer casing should be installed in a near-vertical hole that intersects the zone of suspected movement.* The hole should extend beyond the zone of expected movement and at least

* Measurements in nonvertical holes can be made with some inclinometers; however, before planning such holes manufacturers' specifications should be checked to determine the limitations of the particular instrument being used.

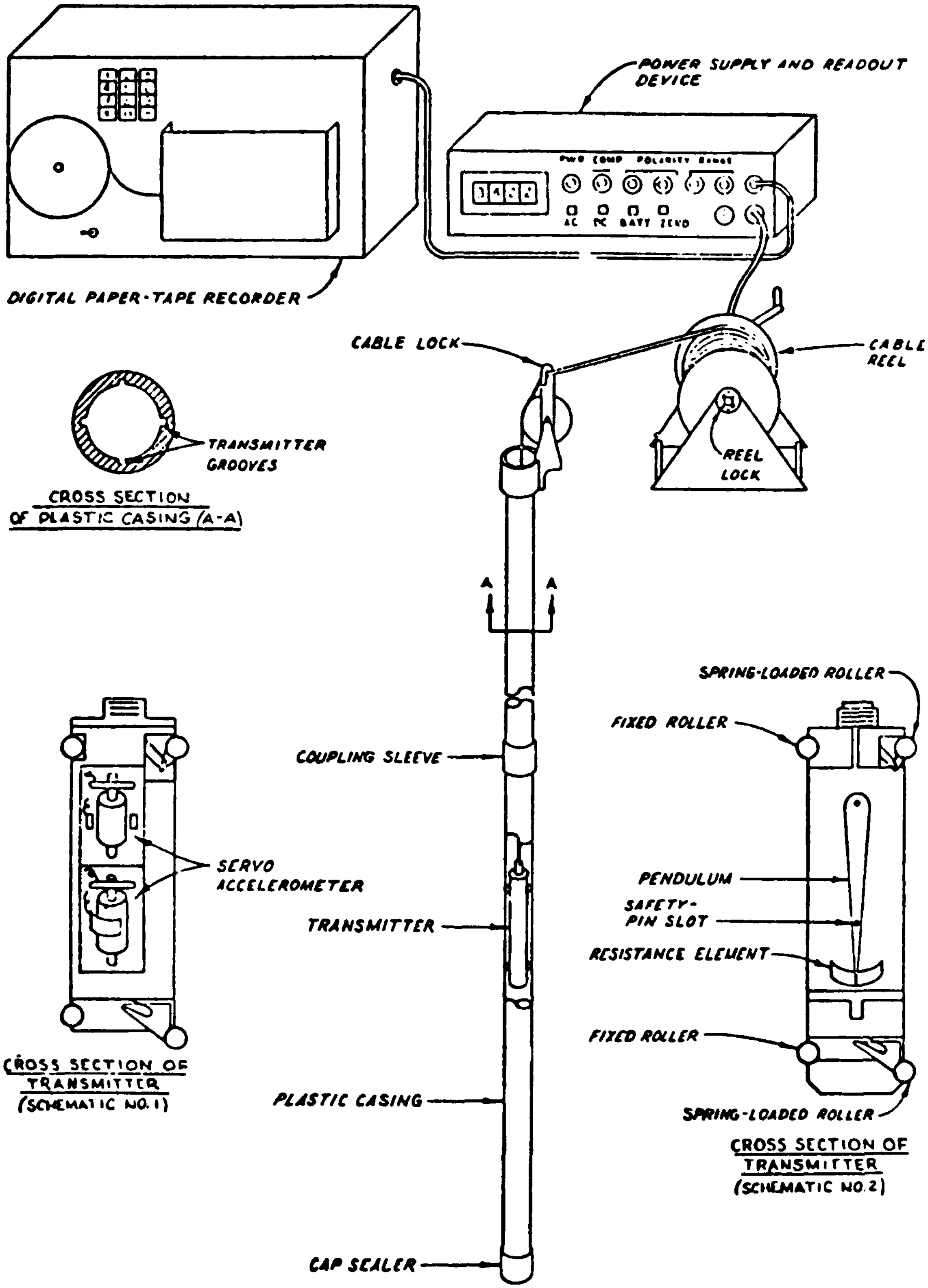


Fig. 1. Typical inclinometer installation (Leach, 1976).

Table 1. *Fixed-Position Inclinometers* (Franklin and Denton, 1973).

Type	Trade name	Drillhole diameter (mm)	Maximum No. and type of anchors	Range		Sensitivity		Manufacturer
				mm/m	mins	mm/m	secs	
Anchored chain of rods with transducers at pivots	Lateral deformation indicator/chain deflectometer	116–146 (cased)	Not determined	±10	35′	0·1–0·01	20–2	Eastman Interfels
Pivoted rod and proximity transducer	Multiple position deflectometer	75–100	Not determined	±12	40′	0·03	6	Terrametrics
Flexible steel strip with strain gauges in parallel to monitor	Strip gauge	75 or larger as required	Continuous	60 mm radius subject to metal thickness	5400	6·0	1200	Savage (not ye marketed)
Tiltmeter incorporating pendulum and vibrating wire measurement for mounting on retaining walls etc. or rods in drill hole	MDS 81 MDS 81B MDS 82B		As required	±3 ±6 ±12	10′ 20′ 40′	0·01–0·002 0·025–0·004 0·050–0·007	2–0·3 5–0·7 10–1·4	Maihak
Flexible breakable strip with resistors in series to detect depth of movement horizon	Shear strips	76 or larger	60 m strip lengths in series	Shear detection movement only		2–50 mm		Terrametrics

Table 2. Probe Inclinometers (EM 1110-2-1908, 1975, and Franklin and Denton, 1973).

Type	Trade Name	Approximate Casing Size mm	Casing Type	Range mm/m	Range deg	Sensitivity mm/m	Sensitivity sec	Manufacturer
Strain-gaged pendulum	CRL Inclinometer	45 x 45	Square aluminum duct	±88 from vertical	±5	0.075	15	Cementation Research
	Inclinometer	50	Aluminum tubing with keyways	360 from vertical	±20	0.2	36	Soil Instruments
	Borehole clinometer	76 x 76	Square steel tube	±175 from vertical	±10	0.1	20	Structural Behavior Eng. Lab.
	C-350 slope meter	45 x 45	Square steel tube	±577 from vertical	±30	0.075	15	Soiltest
Pendulum with rheostat	Series 200-B slope indicator	81	Aluminum tubing	±467 ±87 from vertical	±25 ±5	1.0	180	Slope Indicator
2 electrolevels at 90 deg, servomotor and compass	Slope reader	51	Plastic	±175 from vertical	±10	0.1	20	Eastman
Servo accelerometers	Digitilt	30/70/81	Aluminum/ plastic tube	±577 infinite	±30 ±90	0.1	18	Slope Indicator
Pendulum with vibrating wire, 2 direction, compass or keyway	MDS 83	50 or larger	Aluminum or plastic, keyways optional	±290	±15	0.05	10	Maihak
Pendulum with vibrating wire	68-062 inclinometer	50	Aluminum alloy	±792	±45	0.15	30	ELE/Geonor
Pendulum with differential transformer, automatic recorder	Earth deformation recorder (EDR)	89	Plastic with grooves			0.3% for angles up to 4 deg; 0.15% for angles up to 8 deg		Dames & Moore
Pendulum with vibrating wire	MPF clinometer				15	1		Telemac

15 ft (4.5 m) into soil or rock in which no movement is anticipated. Allowance should be made for loss of the bottom 5 ft (1.5 m) of the hole where sediment accumulation may occur. Casing should be held in place with a sand backfill or a weak cement grout. Casings over 50 ft (15 m) deep should be checked for twist using equipment described in paragraph 4.6 since some of the casings may be received with a built-in twist which would cause considerable errors in observations.[6.3]

3.1.1 Inclinometer casings are commonly installed in either 5- or 10-ft (1.5- or 3.0-m) lengths and are available in either plastic or aluminum. Plastic casing joints are glued. Aluminum casings are coupled with aluminum couplings and riveted, Fig. 2. Care should be taken to ensure that all joints are sealed since leakage can introduce fines into grooves and cause errors in readings. Joints can be sealed with caulking and taped. Greater installation details can be obtained from manufacturers' literature[6.4, 6.5, 6.6, 6.7] or from other sources.[6.3, 6.8]

3.2 <u>Observations</u> - Initial observations should be made after allowing sufficient setting time for the grout around the casing or time for the backfill to settle where sand or gravel is used. Since all displacements are computed based on the position of the casing when installed, the initial position should be verified with at least three separate sets of observations. These observations should be checked closely to see that they agree within the accuracy of the inclinometer being used. Observations should be repeated until satisfactory agreement is obtained. When initial observations are made, the top of the casing should be located with respect to a point outside the zone of expected movement by conventional surveying means and its elevation determined.

3.2.1 The frequency of observations depends upon several factors, the most important of which is the rate of movement. It is necessary to read inclinometers frequently just after installation and, based on

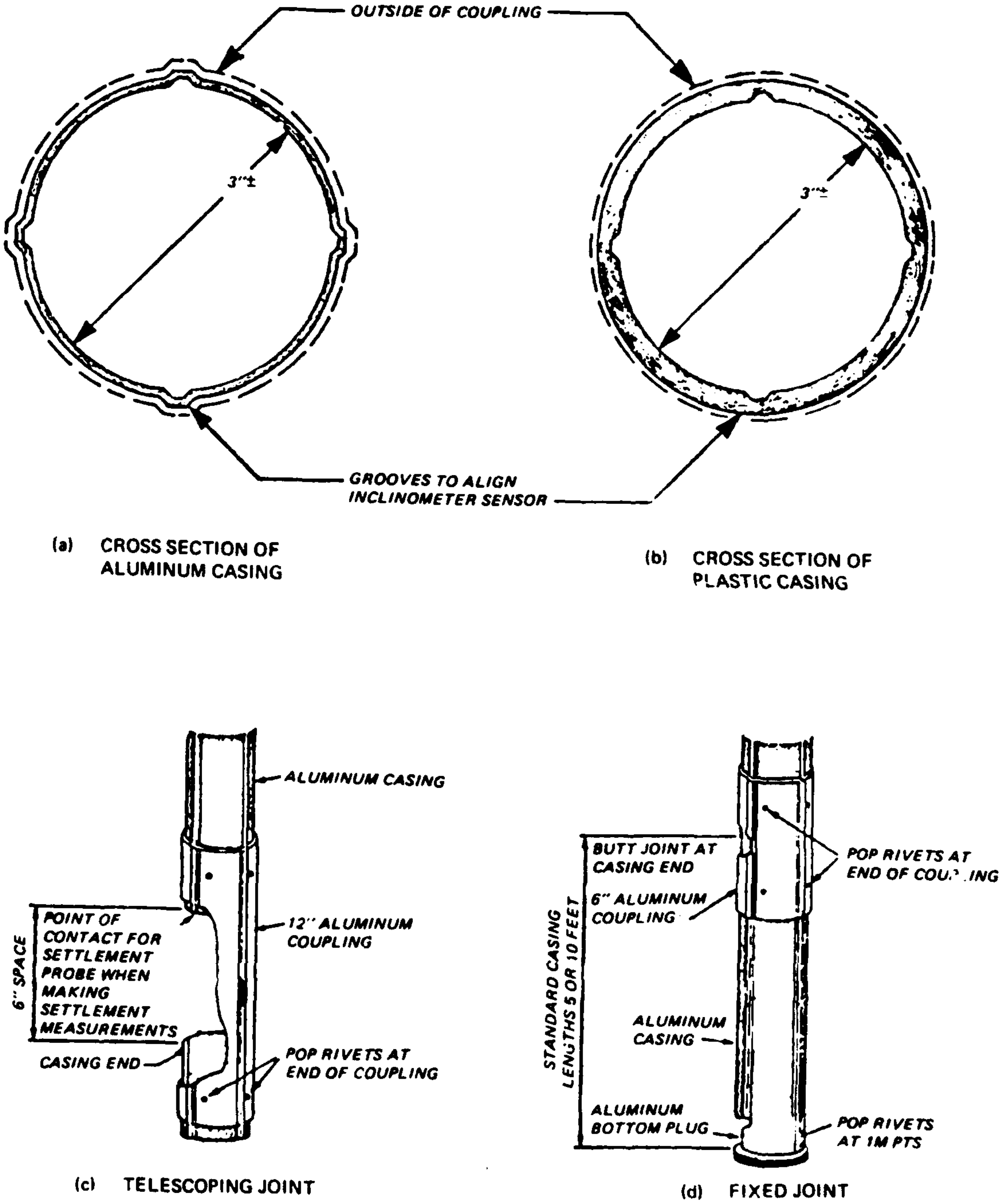


Fig. 2. Inclinometer casing details (Slope Indicator Co.).

these results, to adjust the interval of observations. Observations should coincide with observations of other instrumentation such as extensometers, piezometers, settlement devices, movement surveys, etc.

3.2.2 The procedure for obtaining readings with various inclinometers may vary slightly and manufacturers' literature should be consulted for the current procedure for a particular instrument. However, the general procedure consists of lowering the inclinometer to the bottom of the borehole and beginning the readings. The inclinometer is raised a specified interval,* readings are made, and the procedure repeated until the top of the hole is reached. The inclinometer is removed from the casing, inserted again with the guide wheels in a different groove, lowered to the bottom of the casing, and readings are again made to the top of the hole. This procedure is repeated until a set of readings is obtained for all four grooves. A field check is made by comparing the value of the sum of each set of readings (opposite grooves) and the mean of all sets of readings for the length of the casing. When variations greater than specified by the manufacturer are found, the inclinometer is relocated at that depth and an additional reading is taken. Care should be taken to ensure that readings are obtained at the same depths each time observations are made.

3.3 Maintenance - Maintenance that can be performed in the field on inclinometers is very limited. On probes using O ring connections between the probe and the cable, the O ring should be checked and replaced as necessary. Electrical connections should be kept clean and dry. On probes using batteries, the battery should be checked and charged when necessary. Manufacturers' literature should be consulted for other maintenance operations and precautions to be exercised in operation of inclinometers.

4. Data Reduction

4.1 General - The numerical values of the readings (R) obtained from observations with most inclinometers are equal to plus or minus an

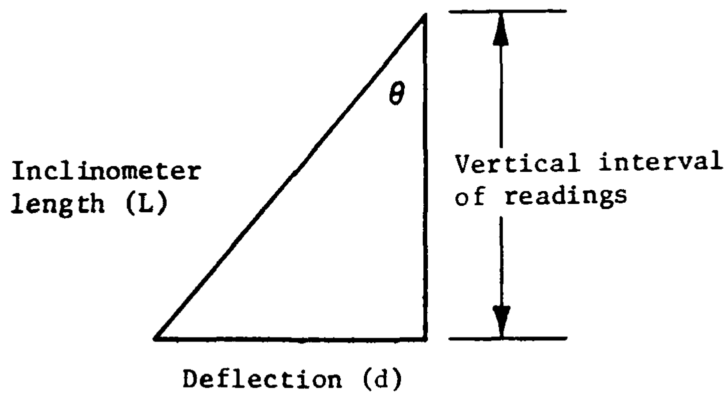
* Greatest accuracy is obtained when the interval of observations equals the wheel spacing of the probe.

instrument constant (K) times the sine of the inclination angle (θ). Expressed mathematically, this is:

$$R = \pm K \sin \theta \tag{1}$$

where the plus or minus sign indicates the direction of movement--plus away from the groove in which the measuring wheel is located and minus toward the groove.

4.1.1 To compute the deflection of the casing from the vertical at any measurement point, the right triangle depicted below is solved:



where L is the distance between measuring wheels. This results in the following expression:

$$d = L \sin \theta \tag{2}$$

The algebraic difference in readings ($R_1 - R_3$) in opposite grooves (180 deg apart) can be used to minimize errors contributed by casing and instrument irregularities.

---

* The formula is true for the Hall Inclinometer and the Digitilt. For the Model 200-B, the reading is equal to a constant times the tangent of the inclination angle; however, for the range of operation (±12 deg) the tangent is approximately equal to the sine. Manufacturers' literature should be consulted for applicability of this discussion to a particular instrument.

$$\text{Difference} = (R_1 - R_3) = \pm\ 2K \sin\theta \tag{3}$$

Solving for sin θ

$$\sin\theta = \frac{\text{Difference}}{2K}$$

and substituting into Equation 2 we have

$$d = \frac{L}{2K} \cdot \text{Difference} \tag{4}$$

4.1.2 Because the prime interest is not the magnitude of the deflection (d) but the change in deflection or magnitude of movement since the initial readings, the initial deflection of the casing must be subtracted from the deflection at some later time.

$$d = d_t - d_i = \frac{L}{2K}\,(\text{Difference}_t - \text{Difference}_i) \tag{5}$$

It is also desirable to know the deflected shape of the casing with reference to a fixed point or length. This fixed length is normally considered to be the bottom of the casing so that the formula now becomes:

$$D = \sum_{m=o}^{n} (d_t - d_i) = \frac{1}{2K} \sum_{m=o}^{n} (\text{Difference}_t - \text{Difference}_i)$$

$$D = \frac{1}{2K} \sum_{m=o}^{n} \text{Change} \tag{6}$$

where m = o is at the bottom of the casing or first measuring point and m = n is at the top of the casing or last measuring point. It is obvious from the above that the initial deflection of the casing need not be computed. Only the Difference is needed.*

---

* As stated previously, several observations should be taken initially. The Difference used above is an average of these observations.

4.2 Hand Calculations - Hand calculations of the deflections in a borehole can be made using the above formula (Equation 6) and the data sheets in Figs. 3 and 4. However, the calculations would require checking of many additions, subtractions, and multiplications for each vertical plane in which deflection measurements were made. Where several boreholes are observed, this would be a long and tedious operation and therefore computer reduction of data is usually performed.

4.3 Computer Data Reduction - Equation 6 is readily adaptable to computer reduction either from data recorded by hand or with automatic data recording devices. Computer programs are available that reduce the data, tabulate, and plot the results. Documentation of two such programs is contained in reference 6.1.

4.4 Twist Corrections - In casings over 50 ft (15 m) in length, accumulated twist can cause significant errors in the assumed direction of movement. Casing should therefore be checked for twist using commercially available equipment. If the twist is found to be significant, readings can be corrected using computer programs currently available.[6.1]

5. Reporting Results

5.1 Results of inclinometer measurements are usually reported in two ways: in tabulations of deflections with depth (Fig. 4) and in plots (Fig. 5) showing movement versus depth in relation to the structure, tunnel, or embankment near which movement is being monitored.

6. References

6.1 Leach, Roy E., "Evaluation of Some Inclinometers, Related Instruments, and Data Reduction Techniques," Miscellaneous Paper S-76-12, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi, 1976.

6.2 Franklin, J. A. and Denton, P. E., "The Monitoring of Rock Slopes," The Quarterly Journal of Engineering Geology, Vol 6, No. 3-4, 1973.

FIELD SHEET INCLINOMETER WELL NO. IN 3

Project X - Mining of Tunnel 2
After Shove 216, Face + 3

SHEET 1 OF 1

DATE 9/15/72 0900 0915 INSTR 1Z0021 READ WH/FS CALC WH CHKD HM

DEPTH	DIFF	AZ = SPRING N		DIFF	CHANGE	DIFF	AZ = SPRING S		DIFF	CHANGE
		AN	AS				BN	BS		
74		+137	-126	+263	+11		+121	-141	+262	-20
72		+136	-126	+262	+10		+142	-150	+292	-8
70		+021	-019	+040	+2		+103	-108	+211	-5
68		-045	+058	-103	+13		+132	-145	+277	-13
66		-070	+083	-153	+13		+136	-154	+290	-18
64		+068	-058	+126	+10		+380	-395	+775	-15
62		+104	-090	+194	+14		+405	-423	+828	-18
60		+104	-081	+185	+23		+373	-406	+779	-33
58		+060	-048	+108	+12		+264	-278	+542	-14
56		+053	-039	+092	+14		+245	-265	+510	-20
54		+031	-026	+057	+05		+262	-267	+529	-5
52		+066	-054	+120	+11		+186	-206	+392	-20
50		+004	+011	-007	+15		+191	-211	+402	-20
48		-010	+028	-038	+18		+315	-339	+654	-24
46		+173	-161	+334	+12		+334	-353	+687	-19
44		+145	-134	+279	+11		+376	-370	+746	+6
42		+118	-107	+227	+11		+353	-388	+741	-35
40		+092	-080	+172	+12		+326	-339	+665	-13
38		+080	-073	+153	+7		+321	-364	+685	-43
36		+038	-027	+065	+11		+293	-307	+600	-14
34		-078	+092	-170	+14		+371	-384	+755	-13
32		+083	-062	+145	+21		+318	-325	+643	-7
30		+179	-165	+344	+14		+191	-212	+403	-21
28		+192	-178	+370	+14		+154	-173	+327	-19
26		+245	-231	+476	+14		+000	-022	+022	-22
24		+256	-242	+498	+14		-079	+068	-147	-11
22		+185	-172	+357	+13		-132	+101	-233	-31
20		+114	-107	+221	+7		-044	+037	-081	-7
18		+043	-031	+074	+12		-032	+015	-047	-17
16		+009	-000	+009	+9		-014	-003	-011	-17
14		+141	-128	+269	+13		-001	-021	+020	-22
12		+124	-110	+234	+14		+009	-028	+037	-19
10		+009	+013	-004	+22		-026	+001	-027	-25
8		-076	+087	-163	+11		+021	-039	+060	-18
6		-046	+060	-106	+14		+025	-044	+069	-19
4		+038	-030	+068	+8		+040	-047	+087	-7
2		+057	-044	+101	+13		+011	-028	+039	-17
					Σ180°					Σ180°

Fig. 3. Field data sheet for borehole inclinometer measurements (Cording, 1975).

JOB NO 121 Sht 5 of 5

INCLINOMETER WELL NO. IN3

SLOT BEARING B Axis: East(-) West(+)

PROJECT X - MINING of TUNNEL 2

DEPTH	INITIAL	DATE 9/15/72 INST 120021 R: .0006 C:				DATE 9/15/72 INST 120021 R: .0006 C:				DATE INST R: C:				DATE INST R: C:				DATE INST R: C:				DATE INST R: C:			
	8/21/71	DIFF	CHANGE	ΣCHANGE	DEFL	DIFF	CHANGE	ΣCHANGE	DEFL	DIFF	CHANGE	ΣCHANGE	DEFL	DIFF	CHANGE	ΣCHANGE	DEFL	DIFF	CHANGE	ΣCHANGE	DEFL	DIFF	CHANGE	ΣCHANGE	DEFL
2'	+128	+039	-89	-650	-.29"	+050	-78	-711	-.43"																
4	+156	+087	-69	-561	-.24	+090	-66	-633	-.38																
6	+129	+069	-60	-492	-.30	+070	-59	-567	-.34																
8	+117	+060	-57	-432	-.26	+057	-60	-508	-.30																
10	+088	-027	-60	-375	-.22	-032	-65	-448	-.27																
12	+083	+037	-46	-315	-.19	+033	-50	-383	-.23																
14	+066	+020	-46	-269	-.16	+014	-52	-333	-.20																
16	+026	-011	-37	-223	-.13	-015	-41	-281	-.17																
18	-020	-047	-27	-186	-.11	-049	-29	-240	-.14																
20	-068	-081	-13	-159	-.10	-083	-15	-211	-.13																
22	-211	-233	-22	-146	-.09	-235	-24	-196	-.12																
24	-131	-147	-16	-126	-.08	-149	-18	-172	-.10																
26	+035	+022	-13	-110	-.07	+028	-7	-154	-.09																
28	+324	+327	+3	-97	-.06	+333	+9	-147	-.09																
30	+413	+403	-10	-100	-.06	+405	-8	-156	-.09																
32	+682	+643	-39	-90	-.05	+637	-45	-148	-.09																
34	+800	+755	-45	-51	-.03	+753	-47	-103	-.06																
36	+640	+600	-40	-6	0	+600	-40	-56	-.03																
38	+730	+685	-45	+34	+.02	+685	-45	-16	-.01																
40	+700	+665	-35	+79	+.05	+661	-39	+29	+.02																
42	+770	+741	-29	+114	+.07	+739	-31	+68	+.04																
44	+740	+746	+6	+192	+.09	+750	+10	+99	+.06																
46	+677	+687	+10	+137	+.08	+685	+8	+89	+.05																
48	+641	+634	+13	+127	+.08	+649	+8	+81	+.05																
50	+383	+402	+19	+114	+.07	+405	+22	+73	+.04																
52	+375	+392	+17	+95	+.06	+395	+20	+51	+.03																
54	+515	+529	+14	+78	+.05	+529	+14	+31	+.02																
56	+494	+510	+16	+64	+.04	+504	+10	+17	+.01																
58	+525	+542	+17	+48	+.03	+538	+13	+7	0																
60	+770	+779	+9	+31	+.02	+777	+7	-6	0																
62	+824	+828	+4	+22	+.01	+822	-2	-13	-.01																
64	+780	+775	-5	+18	+.01	+769	-11	-11	-.01																
66	+291	+290	-1	+23	+.01	+284	-7	0	0																
68	+275	+277	+2	+24	+.01	+273	-2	+7	0																
70	+207	+211	+4	+23	+.01	+205	-2	+9	+.01																
72	+282	+292	+10	+18	+.01	+288	+6	+11	+.01																
74	+254	+262	+8	+8	0	+259	+5	+5	0																
	+9431	+13188		-682		13124		-711																	

Fig. 4. Summary sheet for borehole inclinometer calculations (Cording, 1975).

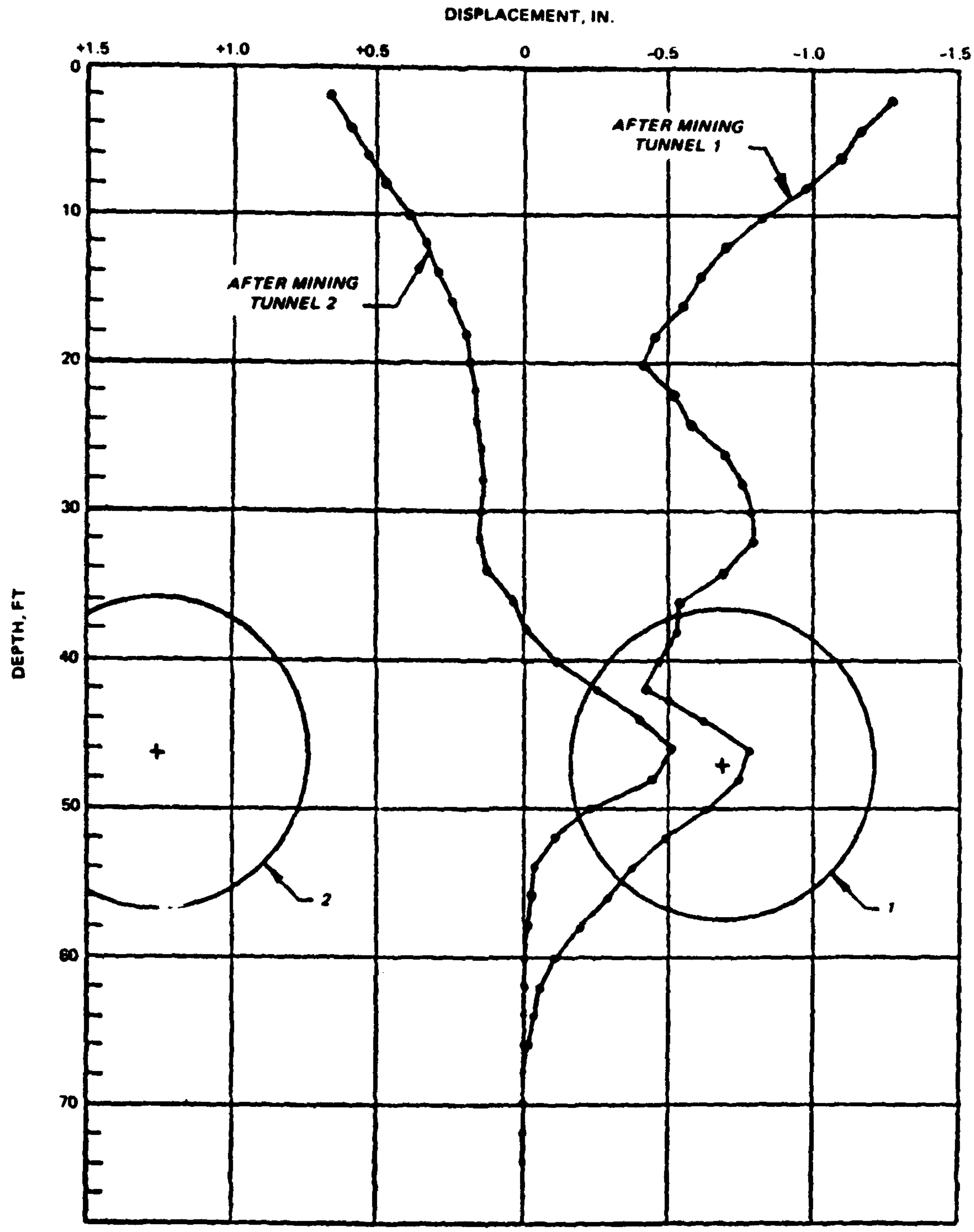


Fig. 5. Typical lateral movement profile from borehole inclinometer (Cording, 1975).

6.3 Department of the Army, Office, Chief of Engineers, "Instrumentation of Earth and Rock-Fill Dams, Part 2, Earth-Movement and Pressure Measuring Devices," Engineering Manual EM 1110-2-1908, Washington, D. C., 1975.

6.4 "Instruction Manual, Series 200-B Instrument," Seattle, Washington.

6.5 ______________________, "Instruction, Digitilt with Tally Tape Perforator Model 50303," Seattle, Washington.

6.6 Geo-Testing, Inc., "Instruction Manual, Hall Inclo-Meter System," San Rafael, California.

6.7 Soiltest, Inc., "Operating Instructions, Model C-350 Slope Meter," Evanston, Illinois.

6.8 Cording, E. J., et al., "Methods for Geotechnical Observations and Instrumentation of Tunneling," Vol I and II, NSF Research Grant GI-33644X, Department of Civil Engineering, University of Illinois, Urbana, Illinois, 1975.

SUGGESTED METHODS FOR MONITORING ROCK MOVEMENTS USING TILTMETERS

(International Society for Rock Mechanics)

1. (a) A tiltmeter consists of a housing containing a gravity operated sensor which detects static or dynamic angular movements at a point. Normally a portable tiltmeter is temporarily located on a reference plate (Fig. 1) cemented or bolted to intact rock at the ground surface or in a tunnel or adit[1]. Periodic measurement of the surface tilt of each plate enables determination of the magnitude and rate of angular deformation.

(b) Nonportable tiltmeters are also available which may be used for static or dynamic angular measurement and can provide continuous monitoring[2]. These sensors are enclosed in a waterproof housing and cemented or bolted and grouted directly to the rock surface. Local access for reading may be utilized or remote reading facilities may be installed.

(c) The tiltmeter, unlike the probe or fixed-in-place inclinometers, only measures the tilt at a discrete, normally accessible point. It does not operate at depth along a borehole although in certain situations it can be permanently buried.

(d) The reference plate must be anchored on a surface which properly reflects movements of the rock mass under investigation. The weathered initial few meters at the surface can often be avoided by locating the reference plate on a pipe or concrete pillar founded on intact rock 1 to 2 meters below the surface. The monument must be free from contact with the upper 1 to 2 meters of rock. Failure to protect the reference plate from the effects of surficial temperature and moisture variations may result in erroneous readings.

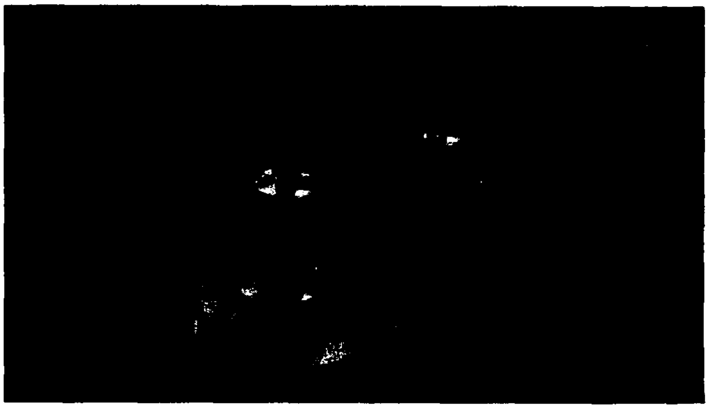


Fig. 1. Portable tiltmeter, reference plate and readout.

Apparatus

2. Surface preparation and drilling equipment including:

(a) Hand tools or bush hammer or jack hammer for surface cleanup, leveling and preparation of a fresh dust-free surface.

(b) A rock drill to give drillholes 15 to 40 mm in diameter, up to about 300 mm deep for fixing bolts.

(c) A rock drill to give a shallow drillhole of the required diameter for a steel anchor tube 70 to 150 mm in diameter, up to 3 m long. Core drilling should be used, but if no core is available, a borehole periscope may be used to inspect the hole.

(d) A rock drill to excavate a large diameter shallow hole for an isolated concrete pillar. Blasting should be avoided.

3. Surface reference plates and installation equipment:

(a) The surface reference plate on which the portable tiltmeter sensor is periodically located should be made of dimensionally stable metal, ceramic or rock. The reference surface should be corrosion-free, easy to wipe clean and not easily damaged. The surface of the reference plate should incorporate a precise positioning system compatible with the portable sensor and which locates the sensor in two mutually perpendicular directions. The positioning system must permit the sensor to be reversed through 180 deg to enable zero errors in the portable sensor to be eliminated. Inaccurate replacement may be the greatest source of error with a sensitive tiltmeter.

(b) Epoxy or polyester resin cement, Portland cement or similar grouts for fixing the reference plate directly to prepared rock surface or steel plate.

(c) Anchor bolts where additional anchorage is needed.

(d) A steel tube 70 to 150 mm in diameter, up to 3 m long to be grouted into a shallow hole and isolated from the weathered initial 1 to 2 meters of rock at the surface. The tube should have an integral reference plate welded in place.

(e) Materials for constructing a concrete pillar or monument founded below the rock surface and isolated from the weathered initial 1 to 2 meters of rock at the surface.

4. Tiltmeter sensor and readout (for example, Fig. 1) including:

(a) If a portable tiltmeter sensor is to be used this should comprise a housing containing a sensing device[3], and with a reference surface incorporating a precise positioning system compatible with the fixed reference plates. The reference surface on the portable sensor should be corrosion-free, easy to wipe clean and not easily damaged. The electrical sensing device is connected by a cable to a compatible portable readout box[4].

(b) If a nonportable tiltmeter is to be used, this should comprise an electrically operated sensor[3] enclosed in a waterproof corrosion resistant housing designed for direct and permanent fixing to the rock surface. The electrical sensing device is connected by a permanently installed cable to a compatible monitoring and readout system. The tiltmeter housing should preferably incorporate a reference surface suitable for a portable tiltmeter which is used periodically to check the permanently installed tiltmeter for zero drift or other malfunction.

(c) The measuring range, sensitivity and accuracy of the tiltmeter with its readout system should be specified according to the requirements of the project. The range and resolution of tiltmeters varies considerably; ±30 deg and 10 seconds, ±0.7 deg and 2 seconds are typical specifications.

(d) The equipment should be designed to ensure that the specified accuracy is maintained irrespective of normal mechanical handling, water pressures, and corrosive environments encountered in use.

5. Calibration equipment including:

(a) A calibrating device to enable initial checking of fixed tiltmeters or routine on-site checking of the portable sensor and readout unit. The device should allow the sensor to be set in its normal operating position and should be adjustable from horizontal to the maximum operating angle of the sensor, with at least one intermediate setting either side of the horizontal. The calibrator should ideally have an independent angle measuring accuracy better than the resolution of the portable tiltmeter sensor[5].

Procedure

6. Preparatory investigations:

(a) The site and project characteristics should be considered in detail in order to specify the performance requirements of equipment to be used.

(b) Tiltmeter locations should be selected on the basis of a study of the geotechnical features of the site, taking into consideration the directions and magnitudes of anticipated ground movements and the nature of other instrumentation to be installed.

(c) The amount and depth of weathering of the rock surface should be investigated so that the tiltmeter is located on or anchored in intact rock. Failure to eliminate the influence of localized surficial movements or inhomogeneities may entirely invalidate the tiltmeter measurements especially if the true subsurface movements are small. If extensive weathering exists, subsurface location in a tunnel or adit is desirable.

7. Installation:

(a) If suitable fresh, hard, unweathered, sound rock exists at the surface, minimal cleanup is required to form a dust-free, level surface.

(b) Holes should be drilled for fixing bolts when the grout to rock adhesion is unreliable. Alternatively, a shallow hole should be drilled or excavated to install a tubular steel anchor or to construct a concrete pillar, isolated from the weathered near-surface rock and founded on fresh rock.

(c) The underside of the reference plate or of the nonportable tiltmeter, thoroughly cleaned and abraded, is oriented to correspond with the required direction of measurement and is then cemented or grouted in place on the rock, anchor or pillar and leveled. The azimuth should be recorded to an accuracy of ±3 deg.

(d) A protective cap or cover should be installed over the exposed reference plate or tiltmeter to prevent damage.

8. Readings:

(a) The nonportable tiltmeter should be calibrated prior to installation.

(b) The portable tiltmeter should be checked on site both before and after each day's readings. Instrument errors should be promptly investigated and corrected and a diary of calibrations and adjustments should be kept. Unnecessary adjustment must be avoided.

(c) Several sets of initial readings should be taken immediately after the cement or grout has set. These readings are averaged to provide a baseline for all subsequent observations. Thereafter, readings should be taken at intervals specified by the project engineer on the basis of site requirements. A set of readings with the portable tiltmeter should comprise, as a minimum, steps 9(d) and 9(e) below.

(d) The reference plate and portable tiltmeter are wiped with a clean dry cloth and inspected for dirt or damage. The tiltmeter is accurately located on the reference plate and a reading taken. The tiltmeter is removed, the contact surfaces rewiped and replaced. This procedure is repeated three or four times until consistent readings are obtained. The tiltmeter is then rotated through 180 deg and the readings repeated, rewiping the contact surfaces each time.

(e) The procedure 9(d) is repeated with the tiltmeter located at 90 deg to the initial position.

(f) A permanently installed tiltmeter may be read manually or automatically at suitable time intervals. It should, where feasible, be periodically checked by a portable tiltmeter as in 9(d) and 9(e).

Calculations and Data Processing

9. (a) Unless otherwise specified all data should be processed within 24 hours of readings being taken[6].

(b) The field data are scrutinized and obvious errors marked on the field data sheet. If corrections are made, these should be clearly noted.

(c) Pairs of opposite face readings obtained with the portable tiltmeter are averaged to correct for face error. The direction of angular rotation must be carefully checked and recorded.

(d) Single face readings obtained with the permanently installed tiltmeter may be corrected for zero error, based on intermittent portable tiltmeter readings on both faces.

(e) Corrected readings are compared with initial readings at the same location to determine the incremental change in angle or displacement.

(f) Graphs of angular change or displacement versus time are plotted for each reference plate location (for example, Fig. 2).

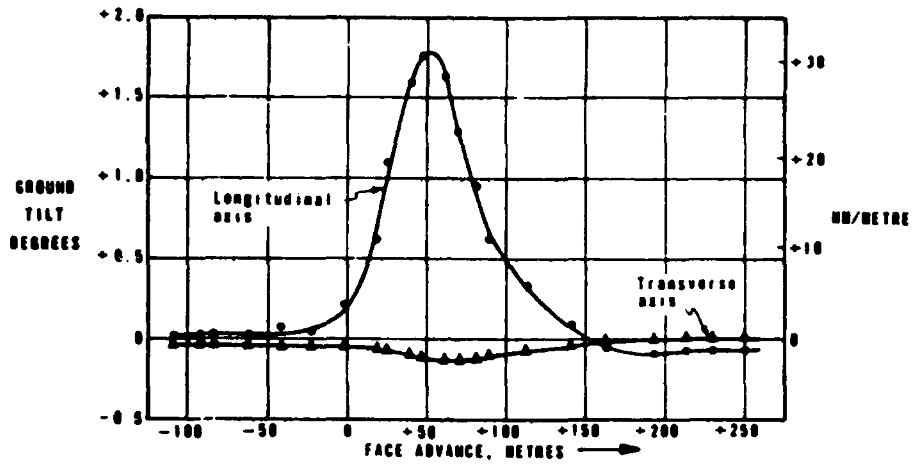


Fig. 2. Tilt at ground surface due to longwall mining face advance at depth.

Reporting of Results

10. Results should, unless otherwise specified, be presented in two forms of report: an Installation Report giving basic data on the instrumentation system at the time of installation; followed by Monitoring Reports presenting periodically the results of routine observations. The Monitoring Reports will generally be required at frequent intervals to minimize delay between the detection of adverse behavior and the implementation of any remedial measures that may be necessary.

11. The Installation Report should include the following:

(a) A description and diagrams of the monitoring equipment used including detailed performance specifications and manufacturers' literature.

(b) A station location plan with details of the reference plates, their surveyed positions and elevations.

(c) Details of methods used for tiltmeter installation, calibration, and monitoring; reference may be made to this ISRM Suggested Method stating only departures from the recommended procedures.

(d) For each station, a diagram showing the geotechnical characteristics of the ground and the position of the reference plate. The azimuth of reference plate guides should be reported clearly stating conventions adopted for the sign of movement and angle directions.

(e) For each station, a tabulated list of initial tiltmeter readings.

12. The Monitoring Reports should include the following:

(a) A set of field monitoring result tabulations; the set to cover all observations since the preceding report.

(b) Graphs of angular change or displacement versus time, sufficient to show clearly the magnitudes, rates, and directions of all significant movements.

(c) A brief commentary drawing attention to significant movements and to all instrument malfunctions occurring since the preceding report.

Notes

[1]Tiltmeters may also be installed on structures founded on rock, for example on concrete dams or turbine foundations.

[2]In addition, there exist very sensitive and narrow range tiltmeters designed to resolve angles as small as 1×10^{-8} radians, used mainly for detecting earthtides and other geodetic or seismic events.

[3]The sensing device may, for example, include an electrolytic spirit level, a pendulum actuated vibrating wire or closed loop servo-accelerometer or alternatively a precise spirit level with a manual mechanical-optical micrometer system, which is not suitable for automatic recording.

[4]The readout box may for example include a direct reading voltmeter, a manual null-balance bridge circuit, a digital voltmeter, or an automatic null balance bridge with digital display and an integrating circuit to sum incremental displacements. Units are available that allow recording on magnetic or paper tape.

[5]It is recognized that this may be difficult to achieve if a narrow range tiltmeter with a resolution of one or two seconds of arc is used, and some compromise will be necessary.

[6]Data processing may be manual or with the aid of a computer; however, the various stages of computation must in either case be fully supervised and checked for reading or transcribing errors and to ensure that the significance of any anomalous behavior is fully appreciated. For example, it is often difficult to distinguish between anomalies due to ground behavior and those due to instrument malfunction.

Designation: D 4403 – 84 RTH 303-89

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa. 19103
Reprinted from the Annual Book of ASTM Standards, Copyright ASTM
If not listed in the current combined index, will appear in the next edition.

Standard Practice for EXTENSOMETERS USED IN ROCK[1]

This standard is issued under the fixed designation D 4403; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ϵ) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This practice covers the description, application, selection, installation, data collecting, and data reduction of the various types of extensometers used in the field of rock mechanics.

1.2 Limitations of each type of extensometer system are covered in Section 3.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Significance and Use

2.1 Extensometers are widely used in the field of engineering and include most devices used to measure displacements, separation, settlements, convergence, and the like.

2.2 For tunnel instrumentation, extensometers are generally used to measure roof and sidewall movements and to locate the tension arch zone surrounding the tunnel opening.

2.3 Extensometers are also used extensively as safety monitoring devices in tunnels, in underground cavities, on potentially unstable slopes, and in monitoring the performance of rock support systems.

2.4 An extensometer should be selected on the basis of its intended use, the preciseness of the measurement required, the anticipated range of deformation, and the details accompanying installation. No single instrument is suitable for all applications.

3. Apparatus

3.1 *General*—Experience and engineering judgment are required to match the proper type of extensometer systems to the nature of investigation for a given project.

3.1.1 In applications for construction in rock, precise measurements will usually allow the identification of significant, possibly dangerous, trends in rock movement; however, precise measurement is much less important than the overall pattern of movement. Where measurements are used to determine rock properties (such as in plate-jack tests), accurate measurements involving a high degree of precision are required. For in-situ rock testing, instrument sensitivity better than 0.0012 in. (0.02 mm) is necessary for proper interpretation.

3.1.2 Most field measurements related to construction in rock do not require the precision of in-situ testing. Precision in the range of 0.001 to 0.01 in. (0.025 to 0.25 mm) is typically required and is readily obtainable by several instruments.

3.1.3 As the physical size of an underground structure or slope increases, the need for highly precise measurements diminishes. A precision of 0.01 to 0.04 in. (0.25 to 1.0 mm) is often sufficient. This range of precision is applicable to underground construction in soil or weak rock. In most hard rock applications, however, an instrument sensitivity on the order of 0.001 in (0.025 mm) is preferred.

3.1.4 The least precision is required for very large excavations, such as open pit mines and large moving landslides. In such cases, the deformations are large before failure and, thus, relatively coarse precision is required, on the order

[1] This practice is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

Current edition approved Aug 31 1984. Published November 1984.

of 1 % of the range where the range may be 3 ft. (1 m) or more.

3.1.5 For long-term monitoring, displacements are typically smaller than those that occur during construction. Therefore, greater precision may be required for the long-term measurements.

3.2 *Extensometers:*

3.2.1 *Rod Extensometers*—A large variety of rod extensometers are manufactured. They range from simple single-point units to complicated multipoint systems with electrical readout. The single-point extensometer is generally used to detect support system failures. The rod can also serve as a safety warning device in hazardous areas. Generally, the rod extensometer is read with a depth-measuring instrument such as a dial gage or depth micrometer, however, various electrical transducers such as LVDTs (linear variable differential transformers), linear potentiometers, and microswitches have been used where remote or continuous readings are required (as shown in Fig. 1). Another type of readout recently developed is a noncontact removable sonic probe digital readout system which is interchangeable with the depth micrometer type. Multipoint rod extensometers have up to eight measuring points. Reduced rod diameters are required for multipoint instruments and have been used effectively to depths of at least 150 ft (45 m). The rod acts as a rigid member and must react in both tension and compression. When used in deep applications, friction caused by drill hole misalignment and rod interference can cause erroneous readings.

3.2.2 *Bar Extensometers*—Bar extensometers are generally used to measure diametric changes in tunnels. Most bar extensometers consist of spring-loaded, telescopic tubes that have fixed adjustment points to cover a range of several feet. The fixed points are generally spaced at 1 to 4-in. (25 to 100-mm) increments. A dial gage is used to measure the displacements between the anchor points in the rock (as shown in Fig. 2). If the device is not constructed from invar steel, ambient temperature should be recorded and the necessary corrections applied to the results. Bar extensometers are primarily used for safety monitoring devices in mines and tunnels.

3.2.3 *Tape Extensometers*—Such devices are designed to be used in much the same manner as bar extensometers, however, tape extensometers allow the user to measure much greater distances, such as found in large tunnels or powerhouse openings. Tape extensometers consist of a steel tape (preferably invar steel), a tensioning device to maintain constant tension, and a readout head. Lengths of tape may be pulled out from the tape spool according to the need. The readout may be a dial gage or a vernier, and the tensioning mechanism may be a spring-loading device or a dead-weight (as shown in Figs. 3 and 4). The tape and readout head are fastened, or stretched in tension, between the points to be measured. Accuracies of 0.010 to 0.002 in. (0.25 to 0.05 mm) can be expected, depending on the length of the tape and the ability to tension the tape to the same value on subsequent readings, and provided that temperature corrections are made when necessary.

3.2.4 *Joint Meters*—Normally, joint meters consist of an extensometer fixed across the exposed surface of a joint (as demonstrated in Fig. 5), and are used to measure displacements along or across joints. The joint movements to be measured may be the opening or closing of the joint or slippage along the joint. Rod-type extensometers are generally used as joint meters with both ends fixed across the joint. Preset limit switches are often mounted on the joint meter to serve as a warning device in problem areas such as slopes and foundations.

3.2.5 *Wire Extensometers*—Such devices utilize a thin stainless steel wire to connect the reference point and the measuring point of the instrument (as shown in Fig. 6). This allows a greater number of measuring points to be placed in a single drill hole. The wire or wires are tensioned by springs or weights. The wire is extended over a roller shiv and connected to a hanging weight. Wire extensometers tensioned by springs have the advantage of variable spring tension caused by anchor movements. This error must be accounted for when reducing the data. Wire-tensioned extensometers have been used to measure large displacements at drill hole depths up to approximately 500 ft (150 m). The instruments used for deep measurements generally require much heavier wire and greater spring tensions. Although wire extensometers are often used in open drill holes for short-term measurements, in areas of poor ground or unstable holes it is necessary to run a protective sleeve or tube over the measuring wires between the anchors.

3.3 *Anchor Systems:*

3.3.1 *Groutable Anchors*—These were one of the first anchoring systems used to secure wire extensometer measuring points in the drill hole. Groutable anchors are also used for rod type extensometers. Initially PVC (poly(vinyl chloride)) pipes clamped between the anchor points were employed to isolate the measuring wires from the grout column (as shown in Fig. 7), however, this arrangement was unreliable at depths greater than 25 ft (7.5 m) because the hydrostatic head pressure of the grout column often collapsed the PVC tubing. To counteract this condition, oil-filled PVC tubes were tried. The use of oil enabled this method to be used to depths of over 50 ft (15 m). As an alternative to this system, liquid-tight flexible steel conduit is used to replace the PVC pipe. This alternative system seems to work well and can be used in most applications. Resin anchors fall in this category and are very successful.

3.3.2 *Wedge-Type Anchors*—These consist of a mechanical anchor that has been widely used for short-term anchoring applications in hard rock. Fig. 8 shows the two basic types of wedge anchors: (*1*) the self-locking spring-loaded anchor, and (*2*) the mechanical-locking anchor. Self-locking anchors, when used in areas subject to shock load vibrations caused by blasting or other construction disturbances, may tend to slip in the drill holes or become more deeply-seated, causing the center wedge to move. Another disadvantage of the wedge anchor is that no protection is offered, if using wires, to the measuring wires in the drill hole against damage that might be caused by water or loose rock.

3.3.3 *Hydraulic Anchors*—These anchors have proven to be successful in most types of rock and soil conditions. Fig. 9 shows the two basic types of hydraulic anchors manufactured for use with extensometer systems: (*1*) the uncoiling Bourdon tube anchor, and (*2*) the hydraulic piston of grappling hook anchor, which is limited to soft rock and soils. Both anchors have the disadvantage of being rather costly. The Bourdon tube anchor works well in most rock and soil conditions and the complete anchor system can be fabricated before installing it in the drill hole. There have been other specialized anchor systems developed, however, these systems have proven to be too costly and unsuccessful for most applications.

3.4 *Extensometer Transducers*—These extensometers convert displacements occurring in insitu materials between two anchored points to mechanical movements that can be measured with conventional measuring devices such as dial gages, LVDTs, strain gages, and the like.

3.4.1 *Depth-Measuring Instruments*—A dial gage, or a depth micrometer are the simplest and most commonly used mechanical measuring instruments. Used in conjunction with extensometers, they provide the cheapest and surest methods of making accurate measurements. When using the dial gage or depth micrometer, the operator is required to take readings at the instrument head, however, local readings may not be practical or possible due to the instrument location or area conditions.

3.4.2 *Electrical Transducers*—For remote or continuous readings, electrical transducers are used rather than dial gages. LVDTs are often used because of their accuracy, small size, and availability. LVDTs require electrical readout equipment consisting of an a-c regulated voltage source and an accurate voltmeter, such as a digital voltmeter or bridge circuit. The use of linear potentiometers or strain gages is often desirable because of the simplicity of the circuitry involved. The disadvantage of using linear potentiometers is their inherently poor linearity and resolution.

3.4.3 When very accurate measurements are dictated by certain excavations, for example, the determination of the tension arch zone around a tunnel opening, extensometers which can be calibrated in the field after installation shall be used. In all cases, the accuracy of extensometers, either determined through calibration or estimation, should be given in addition to the sensitivity of the transducers. The strain-gaged cantilever extensometer (shown in Fig. 10) has been used successfully for many years. The strain-gaged cantilever operates on the principles of the linear strain produced across a given area of a spring material when flexed. This type of extensometer readout is normally used when rock movements of 0.5 in. (12.5 mm) or less are expected. Strain gages produce a linear change in resistance of 1 to 3% of their initial resistance, over their total measurement range. Because of this small change in resistance, it is absolutely necessary to provide extremely good electrical connections and cable insulation when using this type of transducer. Standard strain-gage readout equipment can be used with this type of extensometer, however, care must be taken to protect this equipment from the hostile environments found in most field applications. Vibrating wire and sonic readouts are also reliable and are becoming more common than strain-gage readouts. Provision should always be made for mechanical readout capability.

4. Procedure

4.1 *Preparatory Investigations:*

4.1.1 Select the location, orientation, length, and number of anchors for each extensometer on the basis of a thorough review of both the construction and geotechnical features of the project. Among the items to be considered are: direction and magnitude of anticipated rock movements, location and nature of other instruments to be installed, and the procedures and timing of construction activities before, during, and after installation of the instrument. If the instrument is installed where rock bolts are used for support, the deepest extensometer anchor shall be located beyond the end of the rock bolt. The length of the extensometer shall depend upon the anticipated depth of rock influenced by excavation, expressed for example in terms of tunnel diameter or slope height. As a general rule, the deepest anchor (reference point for all subsequent anchors) shall be placed at least 2½ tunnel diameters beyond the perimeter of the tunnel.

4.1.2 Displacement measurements are most valuable when extensometers are installed at, or before, the beginning of excavation, and when measurements have been taken regularly throughout the entire excavation period at several locations so that a complete history of movements is recorded. Documentation of the geologic conditions and construction events in the vicinity of the measurements is essential to the proper interpretation of the field data.

4.2 *Drilling:*

4.2.1 The size of borehole required for extensometers depends on the type, character, and number of anchors. The borehole size shall conform to the recommendations of the extensometer manufacturer.

4.2.2 The method of drilling used depends upon the nature of the rock, the available equipment, the cost of each method, and the need for supplemental geologic data. Percussion drilling equipment of the type used for blast holes is usually available and is the least costly. Coring methods, like those used for subsurface exploration, are usually more expensive but provide important information on the presence and nature of rock discontinuities. On large projects, coring or close observation of the percussion hole is usually justified to better define the geology. In addition, coring affords the opportunity to position extensometers accurately in the vicinity of major discontinuties.

4.2.3 Immediately prior to drilling, verify the location and orientation of the drill hole.

4.2.4 For percussion-drilled holes, maintain visual inspection of the drilling operation from start to completion of the hole. At all times, the operation shall be under the direct supervision of an individual familiar with drilling and knowledgeable in the peculiarities and intended use of the extensometer. For later use in summarizing the installation, keep notes on drilling rates, use of casing, soft zones, hole caving, plugging of drilling equipment, and any other drilling difficulties.

4.2.5 For cored holes, similar inspection and observation as that for percussion-drilled holes shall be recorded, giving particular attention to drilling techniques that may affect the quality of the rock core obtained. The core shall be logged, including rock lithology, joint orientation, joint roughness, and degree of weathering. For both percussion-drilled and cored holes, note the location of water bearing seams or joints and water flows.

4.2.6 Immediately prior to installing the anchor assembly, thoroughly clean the completed borehole by washing with a pressure water hose. Holes in which instruments are not installed for a lengthy time after drilling (a day or more) shall be carefully cleaned immediately prior to installing the anchor. If hole caving or other blocking in zones of poor rock is suspected, verify the openness of the hole by inserting a pipe or wooden dowel the full length of the hole. In very poor ground conditions, special procedures involving grouting and temporary casing may be required to keep the borehole open sufficiently long to allow installation.

4.3 *Installation:*

4.3.1 Installation of the anchor assembly and connection of the displacement sensor to the anchor assembly shall be performed by a suitably qualified instrumentation specialist. This specialist may be the manufacturer's representative or an individual who, through previous experience and training, is qualified to perform the task.

4.3.2 Whenever possible, adjust the position of the anchors to maximize the information obtained by the extensometer. For instance, it is desirable to have one anchor to each side of a shear zone or filled joint. If not determined from rock cores, discontinuties can be located by borehole television or borehole periscope surveys (Fig. 11 illustrates a typical extensometer installation in a tunnel).

4.3.3 For grouted anchor assemblies, allow sufficient time for setting and hardening of the grout before installation of the extensometer sensor unit. During this time, keep notes on any blasting or other construction activities in the vicinity of the instrument. The strength and compressibility of the grout should somewhat match the surrounding soil or rock.

4.3.4 Install a protective cover if the instrument does not have an inherently rugged protective cover or is not recessed within a borehole. The protective cover must provide full protection from damage due to workmen, equipment, and fly rock from blasting. For installations in blast-damaged areas, it is preferable to initially install the instrument with a mechanical sensor only. After the risks of damage have been reduced, an electrical, remotely read sensor can be installed. If an instrument is an electrical, remotely read type, suitably protect the electrical cable (such as by armored cable or steel pipe) to prevent damage during the intended period of use. Instruments installed at the ground surface shall be installed below the depth of frost penetration. Manholes shall be watertight in cold climates to prevent icing.

4.3.5 Verify zero readings of each extensometer at least two times prior to the start of construction, or at the time of installation or resetting if construction is in progress. Instruments installed several weeks or months in advance of construction should be monitored to detect equipment malfunctions and reading variations due to temperature or operation. Two calibrations are required per extensometer, one following installation and one following the conclusion of the test series. Additional calibrations are required only where: (*1*) a transducer is replaced or rewired, (*2*) the power supply is changed, (*3*) the instrument head is damaged, or (*4*) a transducer is moved or reset to change its zero point.

4.3.6 Completion of installation of the sensing unit requires a thorough check for electrical or mechanical malfunctions. For future reference, keep notes of any measurements, in-situ calibrations, or settings performed during this final checkout.

4.4 *Readings:*

4.4.1 Readings shall be taken by a person familiar with the equipment and trained to recognize critical measurements and their relevance to the particular project.

4.4.2 The mechanical or electrical device used to read an extensometer shall be checked on site both before and after each day's use. For instance, verify zero settings on dial gages, and compare readouts for resistance or vibrating-wire gages to standards.

4.4.3 Electrical readout equipment shall undergo full-range calibration by the manufacturer, or an appropriately qualified commercial calibration service, on a routine basis. This calibration shall take place before and after times of critical measurement, or periodically for long-term measurements.

4.4.4 For those instruments having such a feature, a periodic in-situ calibration shall be performed to determine changes in the behavior of the instrument. The calibration may be done at times of critical measurements or during regular maintenance.

4.4.5 After reading the extensometer, record the data in a field notebook or data sheet that contains a record of previous readings. When a reading is taken, check it immediately with the previous reading to determine if any significant displacements have taken place since the last reading, or to determine if the reading is in error. If a reading is in question (that is, unanticipated displacements are indicated), the observer shall take additional readings. The observer shall also check to see if the extensometer is dirty or has been damaged, or if any construction events have taken place that would explain the change in the readings. For all readings, record construction conditions and temperature.

4.4.6 Several observations will aid the interpretation of displacement measured by borehole extensometers and shall be noted in a "remarks" column on the data sheet or field book. Examples of these observations are as follows:

4.4.6.1 Opening of joints or movement of rock blocks.

4.4.6.2 Mapping of joints, shear zones, and other geologic features that could be related to movement. Observations of overbreak and rock loosening along the joints and shear zones will aid in evaluating the significance of these features.

4.4.6.3 Crack surveys in shotcrete. The width, length, and relative movement of the crack shall be measured with time, and the thickness of the shotcrete in the vicinity of the crack determined.

4.4.6.4 In tunnels, evidence of distress or displacement of steel ribs and timber blocking.

4.4.6.5 Evidence of distress or loosening of rock bolts.

4.4.6.6 The increase of waterflow in the drainage system of dams that can reflect the opening of joints in the upstream part of a rock foundation. This is also helpful in tunnels to indicate loosening and opening of rock mass.

5. Calculation

5.1 Unless otherwise specified, process all data as soon as possible, but within 24 h of the reading.

5.2 Again scrutinize the field data in the office and clearly mark obvious errors in the field book. Supposedly erroneous readings shall be replaced by additional readings and shall not be discarded or obliterated from the field records.

5.3 If not entered on a special data sheet at the time of the reading, the field data shall be transferred to a computation and data summary sheet, such as shown in Fig. 12.

5.4 The method of calculating displacements from the field data depends on the particular instrument. The procedure recommended by the manufacturer shall be followed unless an alternative method is proven acceptable. Thermal displacement corrections to extensometer output are made by interpolating the change in measuring rod or wire temperature between the depths where thermocouples are able to directly measure temperature change, integrating the interpolation function (a cubic spine) over each length and multiplying this quantity by the thermal expansion coefficient of the wire or rod.

5.5 A plot of displacement versus time is the best means of summarizing current data and should be kept up to date. Interpretation of the measurements is facilitated by considering not only displacement, but the rate of displacement and the rate of change of displacement with time. Rate of displacement is equal to the slope of the displacement curve.

5.6 Periodically, prepare displacement-depth plots, as illustrated for a tunnel in Figs. 11 and 13. The deepest anchor (No. 6) has been assumed a fixed point of reference for all anchors. The rock movements can be correlated with the position of supports.

6. Report

6.1 *General*—Present results, unless otherwise specified, in two forms: (*1*) an installation report giving basic data on the instrumentation system at the time of installation, and (*2*) a monitoring report that presents periodically the results of routine observations. The monitoring reports will generally be required at frequent intervals to minimize delay between the detection of adverse behavior and the implementation of any remedial measures that may be necessary.

6.2 Installation reports shall include the following:

6.2.1 A description, with diagrams, of all components of the extensometer (anchor assembly, displacement-sensing unit, readout equipment), including detailed performance specifications.

6.2.2 Type and details of drilling equipment used.

6.2.3 Log of drilling—For cored holes, a summary log including the log of drilling and a log of the core. Also include summaries of borehole television or periscope investigations when undertaken.

6.2.4 Details and methods of installation, calibration, and monitoring; reference may be made to this practice, stating only departures from the recommended procedures.

6.2.5 A borehole location diagram that relates the specific instrument to the entire project and other instrumentation. This diagram shall include (*1*) the station or coordinates and elevation of the head of instrument, (*2*) depth, orientation, and diameter of borehole, (*3*) distances between anchors and the reference head, and (*4*) the relative position of the instrument to present and future structures and other construction.

6.2.6 A plan and section of the installation that illustrates present and anticipated construction and geology.

6.3 Monitoring reports shall include the following:

6.3.1 A set of tabulated field monitoring results (containing information in the manner shown in Fig. 12), including all observations since the preceding report.

6.3.2 Updated diagrams of displacement of all individual sensing points with respect to time.

6.3.3 For selected instruments and locations, a diagram of displacement versus depth for various times. The reading times shall be correlated to the construction activity and shall emphasize the development or progressive nature of displacements that might be taking place.

6.3.4 A brief summary of the most significant displacements and all instrument malfunctions since the preceding report.

7. Precision and Bias

7.1 The precision and bias of any extensometer system are limited to the type of extensometer, transducer, and data acquisition system. The situation dictates the degree of precision and bias required.

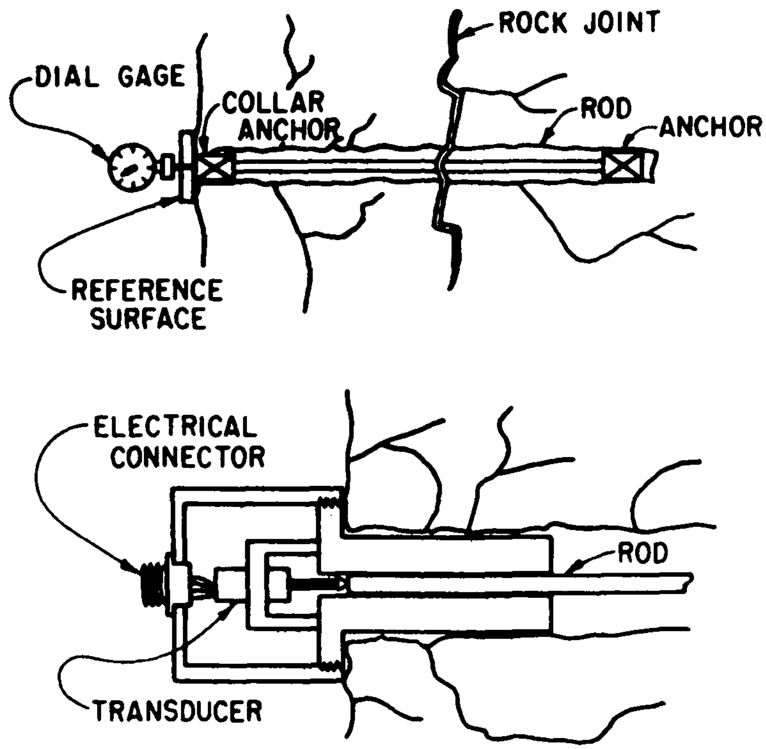


FIG. 1 Rod Extensometer

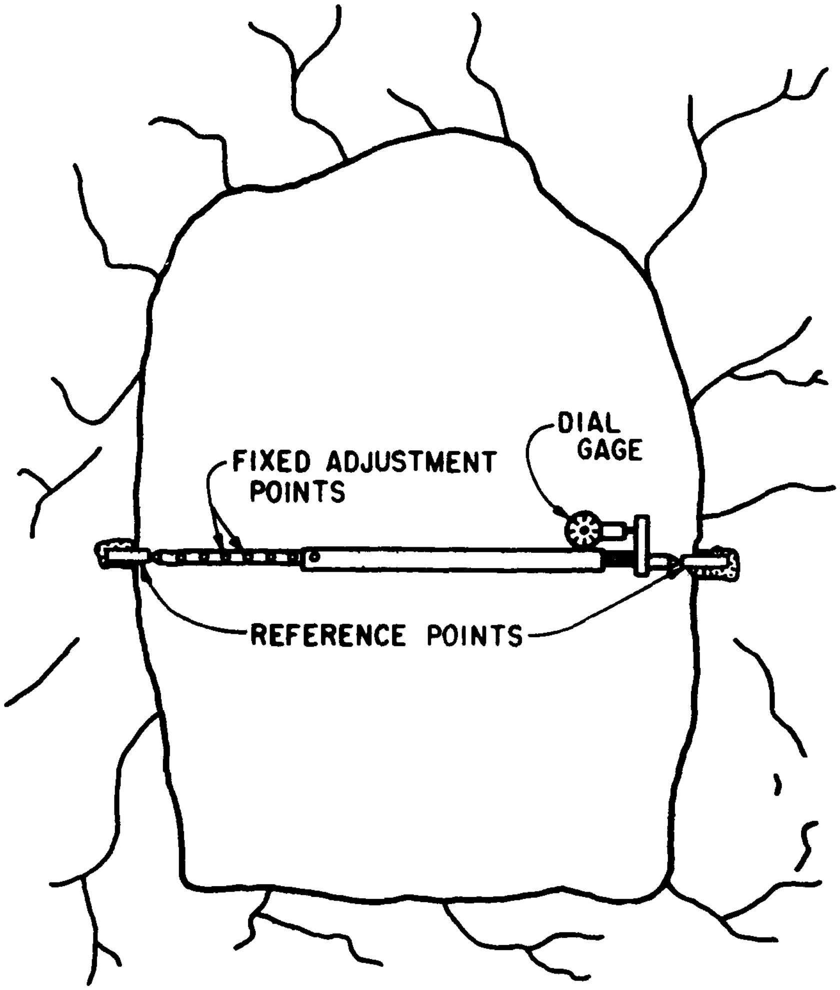


FIG. 2 Bar Extensometer

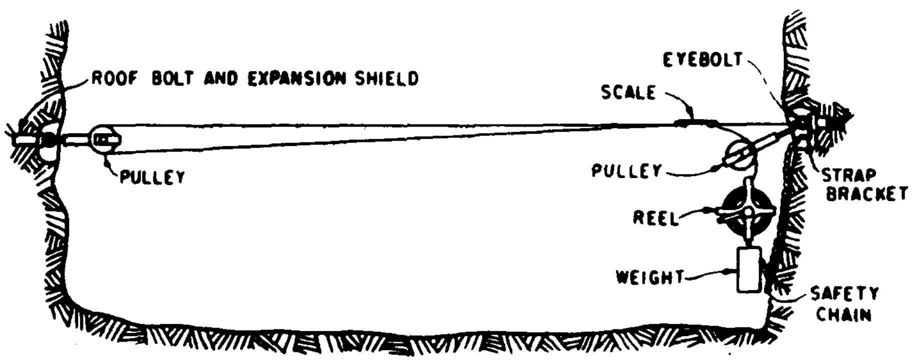


FIG. 3 Tape Extensometer with Vernier Readout and Deadweight

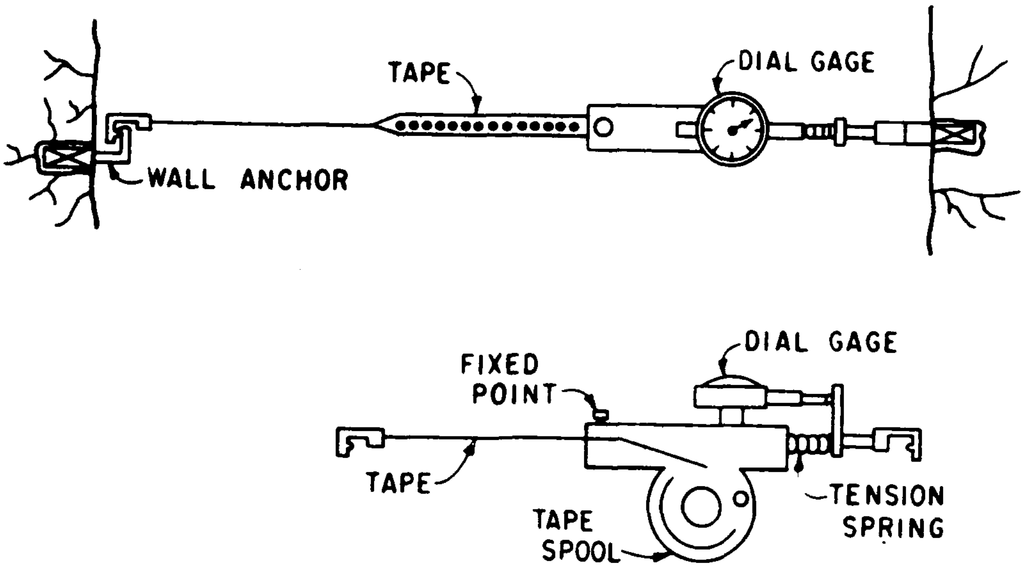


FIG. 4 Tape Extensometer with Dial Gage and Tension Spring

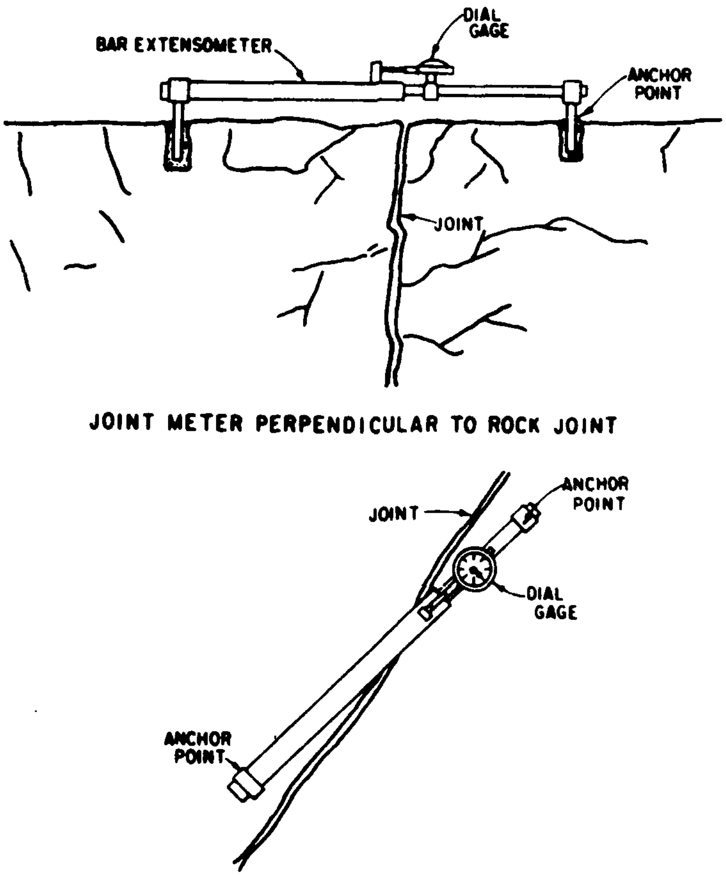
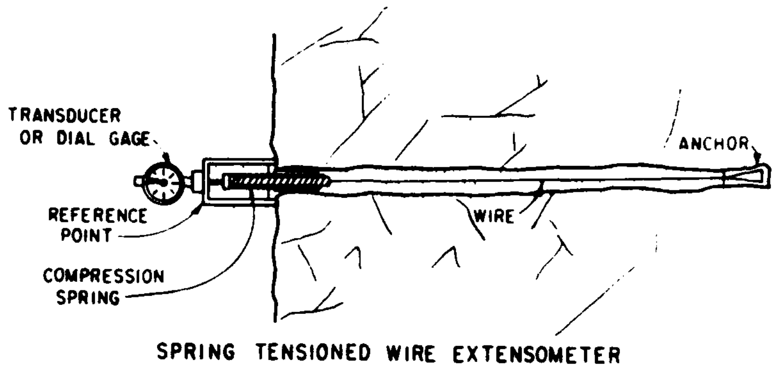


FIG. 5 Joint Meters



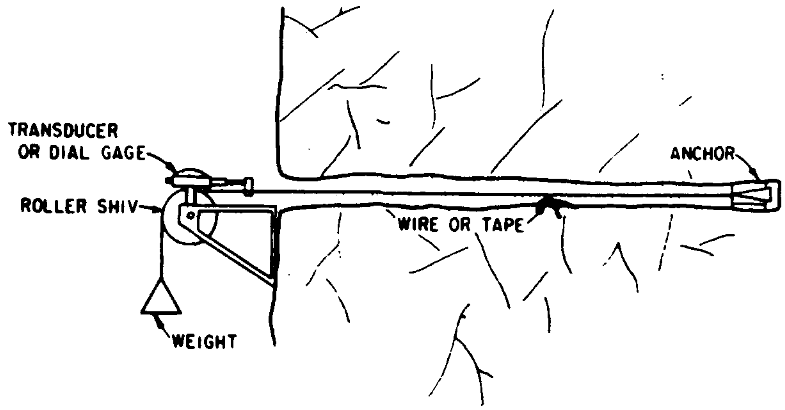


FIG. 6 Wire Extensometers

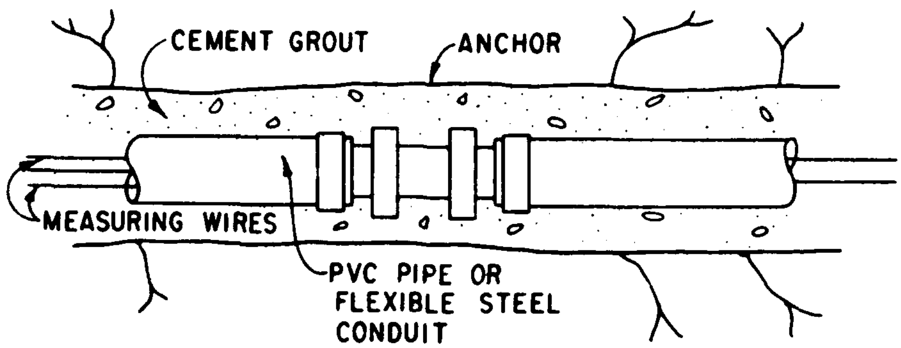


FIG. 7 Grouted Anchor System

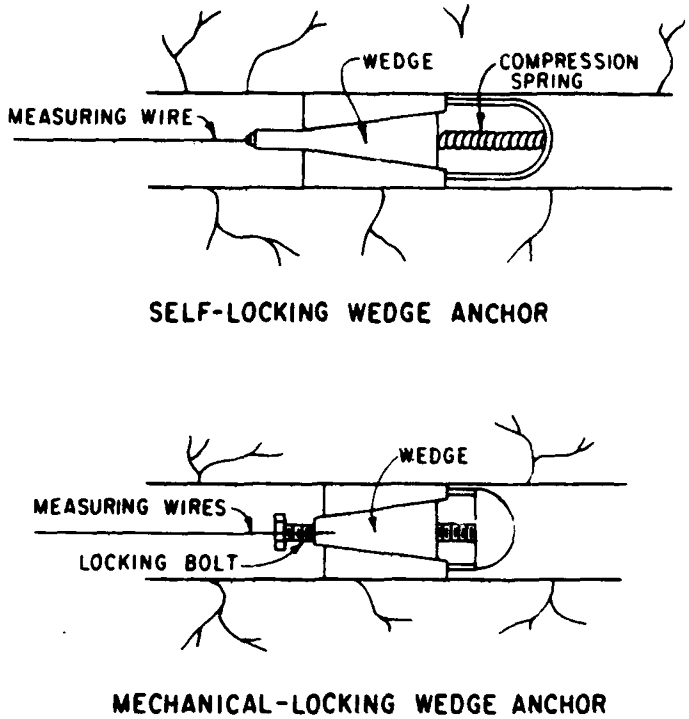


FIG. 8 Wedge Anchors

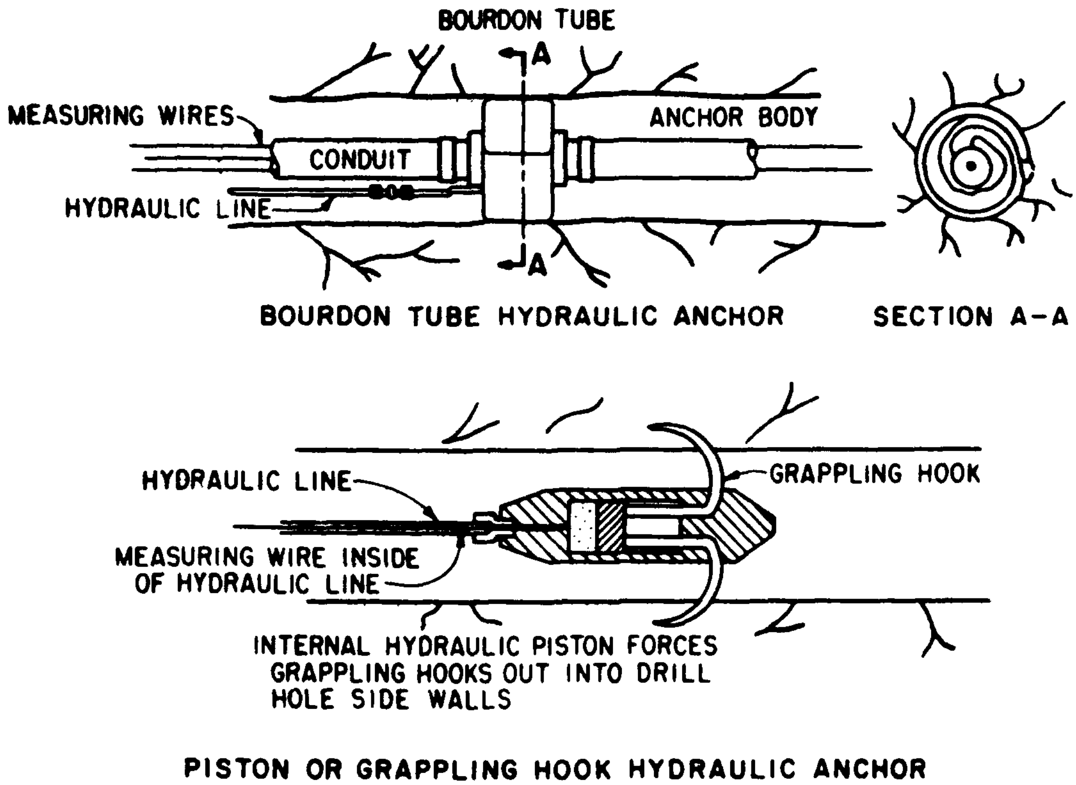


FIG. 9 Hydraulic Anchors

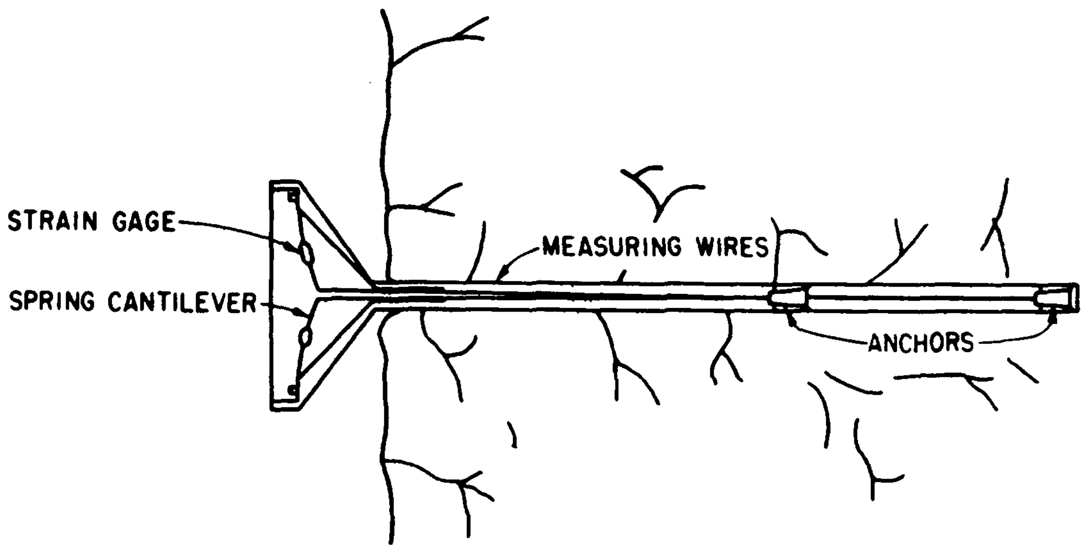
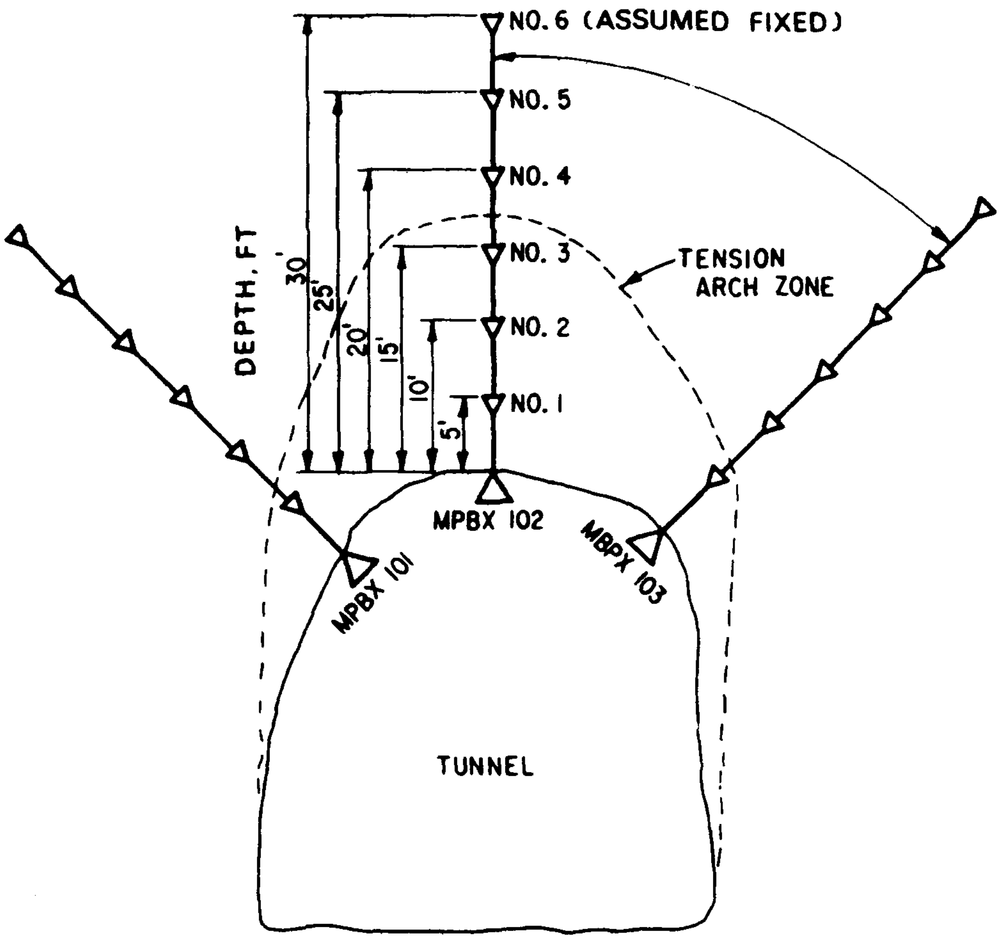


FIG. 10 Extensometer Using Strain-Gaged Spring Cantilevers



STATION 110

NOTE—Include such features as shear zones, rock bolts, and the like.

FIG. 11 Typical Extensometer Installation for a Tunnel Using Three 6-Point Extensometers with Anchors Spaced at 5-ft (1.5 m)

DOUBLE-POSITION MECHANICAL EXTENSOMETER

EXTENSOMETER NO. DX4 STATION 66+03 LOCATION CROWN

DATE	TIME	DEPTH 30 FT.		DEPTH 6 FT.		COMMENTS
		READING 0.001 INCHES	DISPLACEMENT INCHES	READING 0.001 INCHES	DISPLACEMENT INCHES	
7/11/72	10:30	2.252	–	2.460	–	INITIAL READING
10/24	11:00	2.264	+.012	2.459	-.001	STAGE I -65+66
10/27	15:30	2.270	+.018	2.464	+.004	STAGE I -65+76
10/31	21:30	2.283	+.031	2.471	+.011	STAGE I -65+86
11/3	18:20	2.240		2.374		NEW ZERO STAGE I - 66+06
11/6	13:52	2.281	+.072	2.392	+.029	STAGE I -66+11
11/6	22:00	2.353	+.094	2.398	+.035	STAGE I -66+16
11/7	22:00	2.323	+.114	2.400	+.037	STAGE I -66+21
11/8	10:00	2.329	+.120	2.400	+.037	STAGE I -66+21
11/9	9:45	2.349	+.140	2.402	+.039	STAGE I -66+26
11/10	10:30	2.366	+.157	2.406	+.043	STAGE I -66+31
11/13	17:00	–	–	2.408	+.045	STAGE I -66+36
11/14	10:00	2.381	+.172	2.409	+.046	STAGE I -66+36
11/14	18:00	2.388	+.179	2.409	+.046	STAGE I - 65+41 2b-65+32

FIG. 12 Sample Computation and Data Summary Sheet for a Double-Postion Mechanical Extensometer

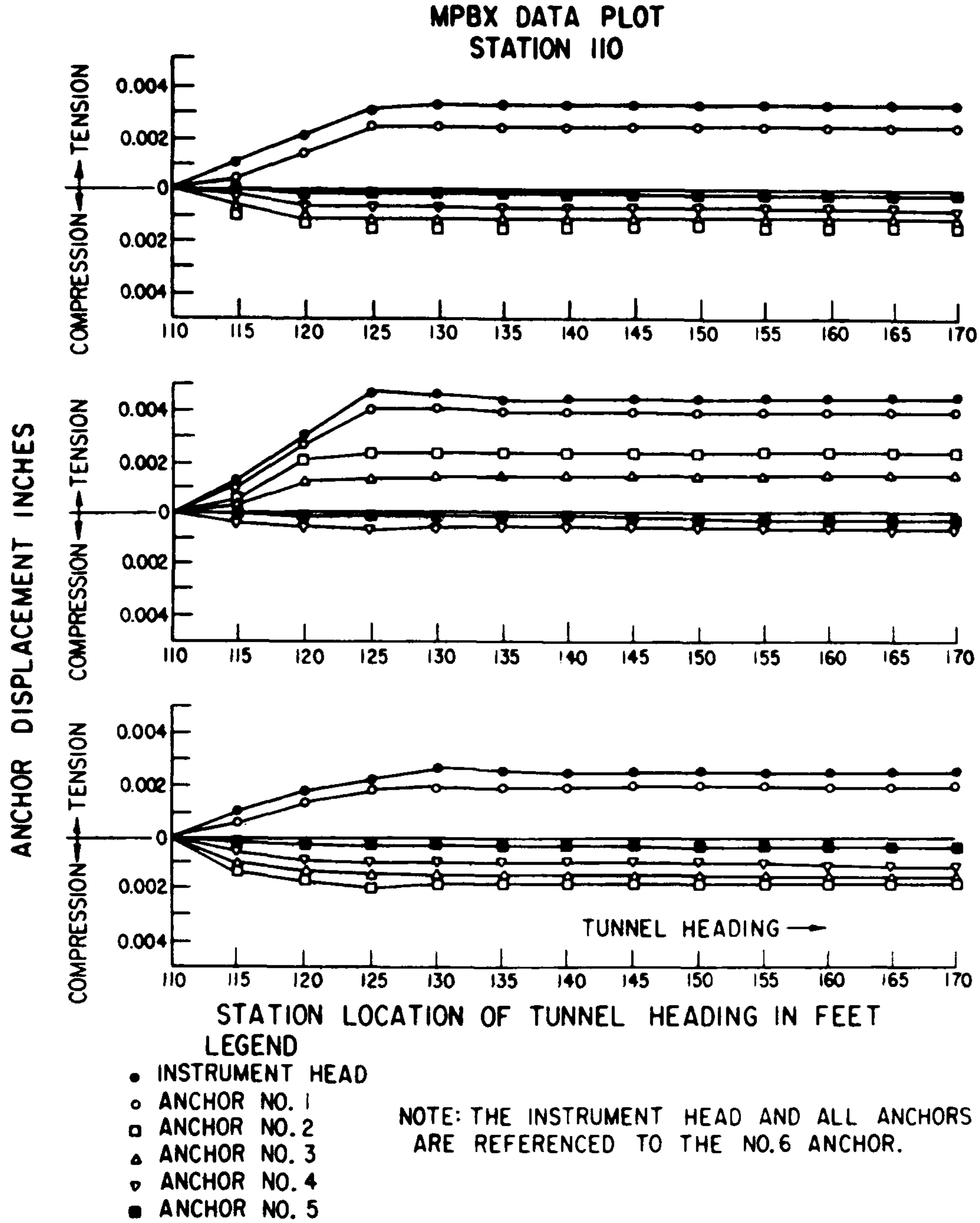


FIG. 13 Hypothetical Extensometer Data Plot

EXTENSOMETER SUPPLIERS AND MANUFACTURERS

Earth Science Research, Inc., 133 Mt. Auburn St., Cambridge, Massachusetts, 02138, USA.

Geonor, Norway, U. S. supplier, Slope Indicator Co.

Interfels International, Germany, North American supplier, Rocktest Ltd.

Irad Gage, 14 Parkhurst St., Lebanon, New Hampshire, 03766, USA.

Rocktest Ltd, 1485 Desaulneirs, Longeuil (Montreal), Quebec, Canada.

Peter Smith Instrumentation, England, North American supplier, Rocktest LTD.

Slope Indicator Co., 3668 Albion Place North, Seattle, Washington, 98103, USA.

Soil and Rock Instrumentation Inc., 30 Tower Road, Tower Office Park, Newton Upper Falls, Massachusetts 02164, USA.

Soil Test Inc., 2205 Lee St., Evanston, Illinois 60222, USA.

Terrametrics Inc., 16027 West 5th Avenue, Golden, Colorado 80401, USA.

RTH 305-80

LOAD CELLS

1. Scope

1.1 This method deals with load cells and their application in the field of rock mechanics and includes descriptions of various types, their construction, readout procedure, and data reduction.

2. Apparatus

2.1 Load cells have been utilized for a wide variety of engineering applications. Such uses include tunnel support, rock bolt, and tieback load monitoring. The data obtainable from load cells can be used for safety monitoring and as an engineering aid for support design. Some of the more common types of load cells used in rock mechanics applications utilize one of four basic types of measurement systems: (1) hydraulic, (2) mechanical, (3) strain gage (this includes bonded foil, vibrating wire, and unbonded wire gages), and (4) photoelastic. Although the strain gage type load cell is the most commonly used, all types of load cells have certain advantages depending on their application.

2.1.1 Hydraulic Load Cells - Hydraulic load cells are basically a fluid-filled deformable chamber connected to a pressure gage or an electric pressure transducer. The load is transferred to the fluid by means of a piston, or in the case of the flat jack, deformation of the fluid confinement chamber (Fig. 1). Hydraulic load cells allow the user to preload the load member, such as rock bolt tiebacks, by applying an initial pressure to the fluid. It is often desirable to posttension rock bolts and tiebacks due to anchor slippage or shifting load distributions. Although most hydraulic load cells are of rugged construction, their application has been limited due to their physical size, poor load resolution, and temperature sensitivity. Hydraulic flat jacks have had the greatest application as earth pressure cells and concrete stress cells. Flat jacks have also been used in conjunction with other apparatus on radial jacking tests and in situ stress measurements.

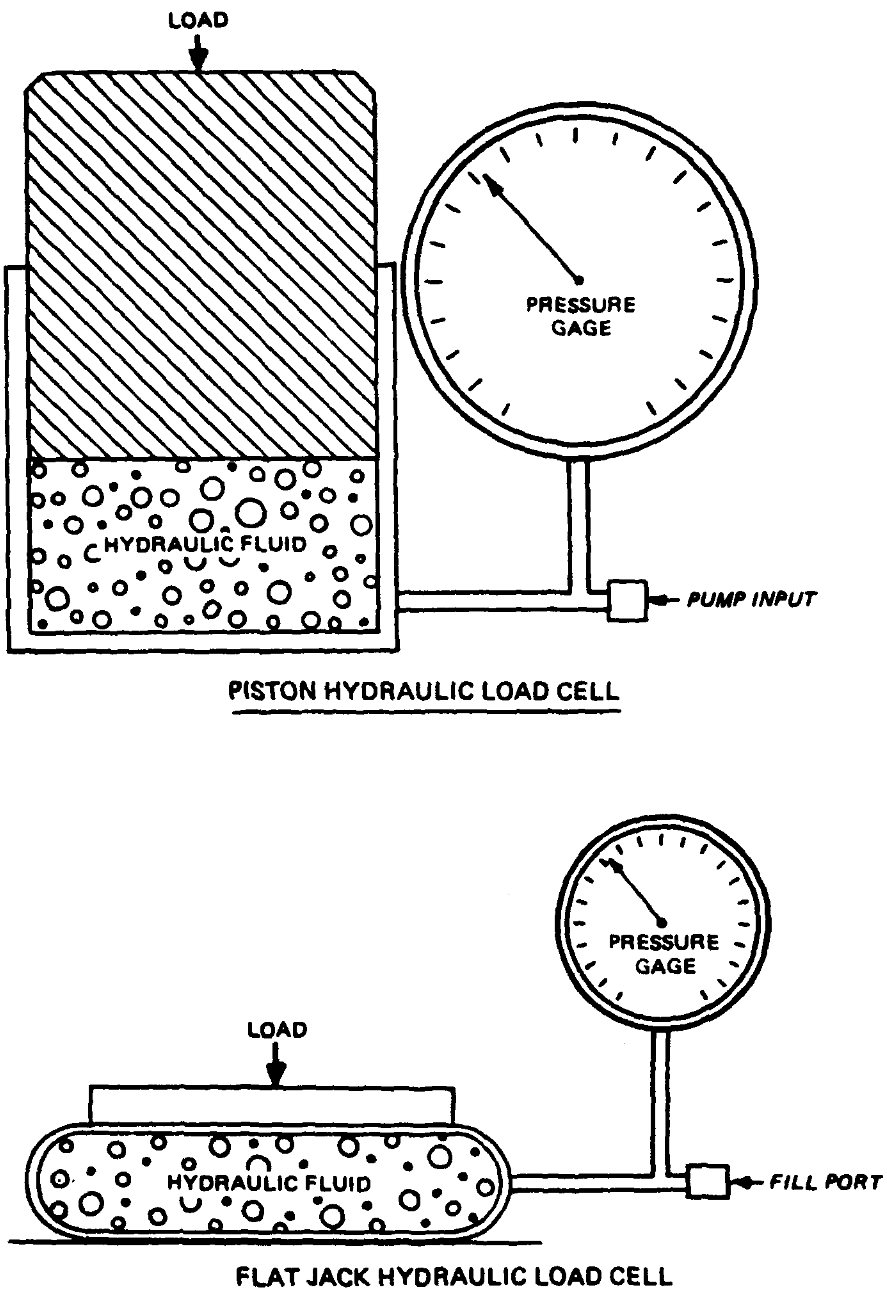


Fig. 1. Hydraulic load cells.

2.1.2 Mechanical Load Cells - The proving ring is the most common type of mechanical load cell but, due to its construction, has had very little application to field uses. The most commonly used mechanical load cells consist of an elastic disk element sandwiched between two plates. The disk deflects under load, changing the distance between the plates. The deflection is measured with a dial gage or suitable electronic transducer (Fig. 2). Although this type of load cell is relatively inexpensive to manufacture, it has had limited use because of its nonlinear calibration curve and restricted application. (This type of cell is generally designed to be used on rock bolts or tieback tendons.)

2.1.3 Strain-Gaged Load Cell

2.1.3.1 The strain-gaged load cell is by far the most commonly used for both field and laboratory applications. This type of cell is manufactured by a large number of geotechnical instrumentation suppliers. Most strain-gaged load cells consist of a metal cylindrical column. The column is loaded axially and the axial strain is measured with a suitable strain gage (Fig. 3). Bonded foil strain gages are used by most cell manufacturers because of their simplicity and availability, but the vibrating-wire and unbonded gages have also been proved to have distinct advantages. The bonded strain gage is a resistive element that undergoes a change in resistance when subjected to an axial strain. The gages are generally bonded to the load cell's cylindrical wall and connected together to form a Wheatstone bridge. The bonded gages are oriented in such a way as to cause a linear resistive imbalance proportional to the strain in the load cell. The unbalanced signal is amplified and observed on a galvanometer. Most strain gage readout equipment contain resistive balancing circuits that allow the operator to null the unbalanced signal with a potentiometer connected to a digital indicator. The load cells are calibrated to read load in pounds per readout digit (Fig. 4). The main advantages of the bonded strain-gaged load cells are simplicity of construction, relatively small physical size to load capacity ratio, direct reading requiring no summing, averaging or

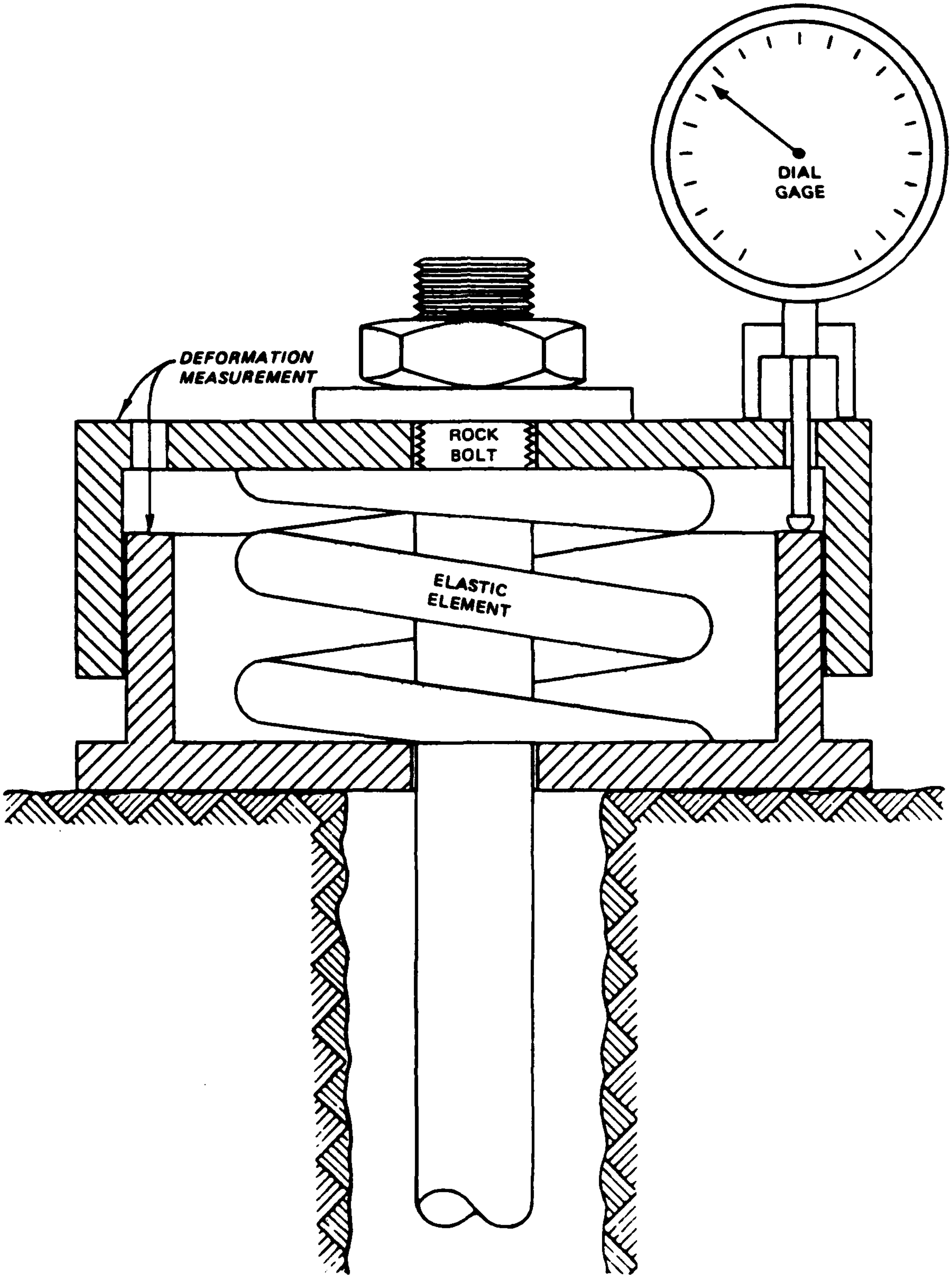
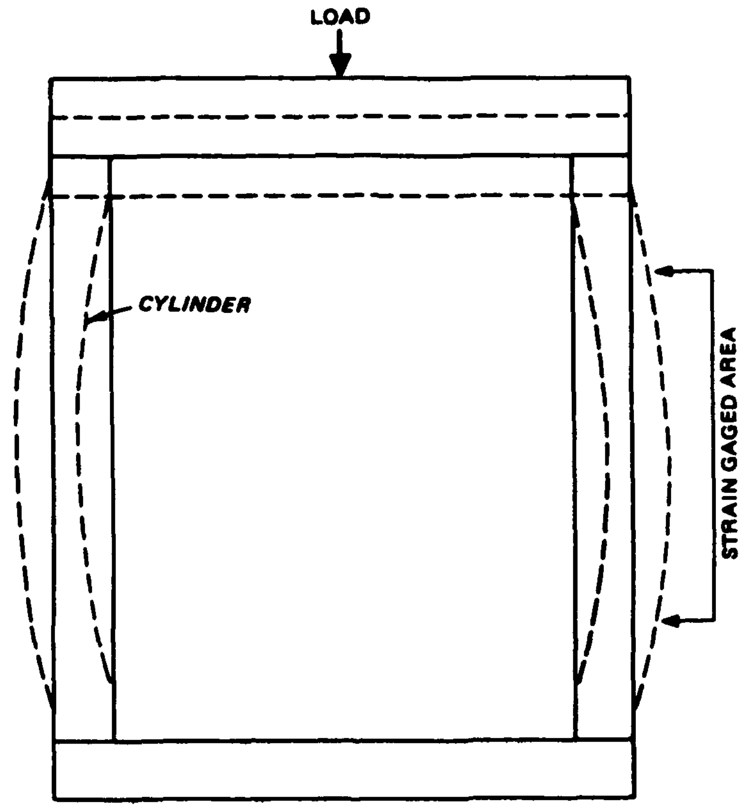


Fig. 2. Mechanical load cell.



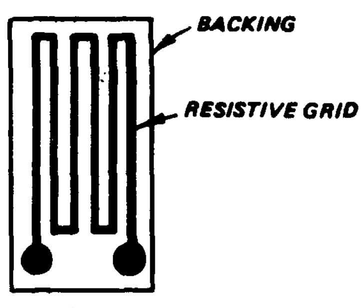


Fig. 3. Cylindrical column load cell.

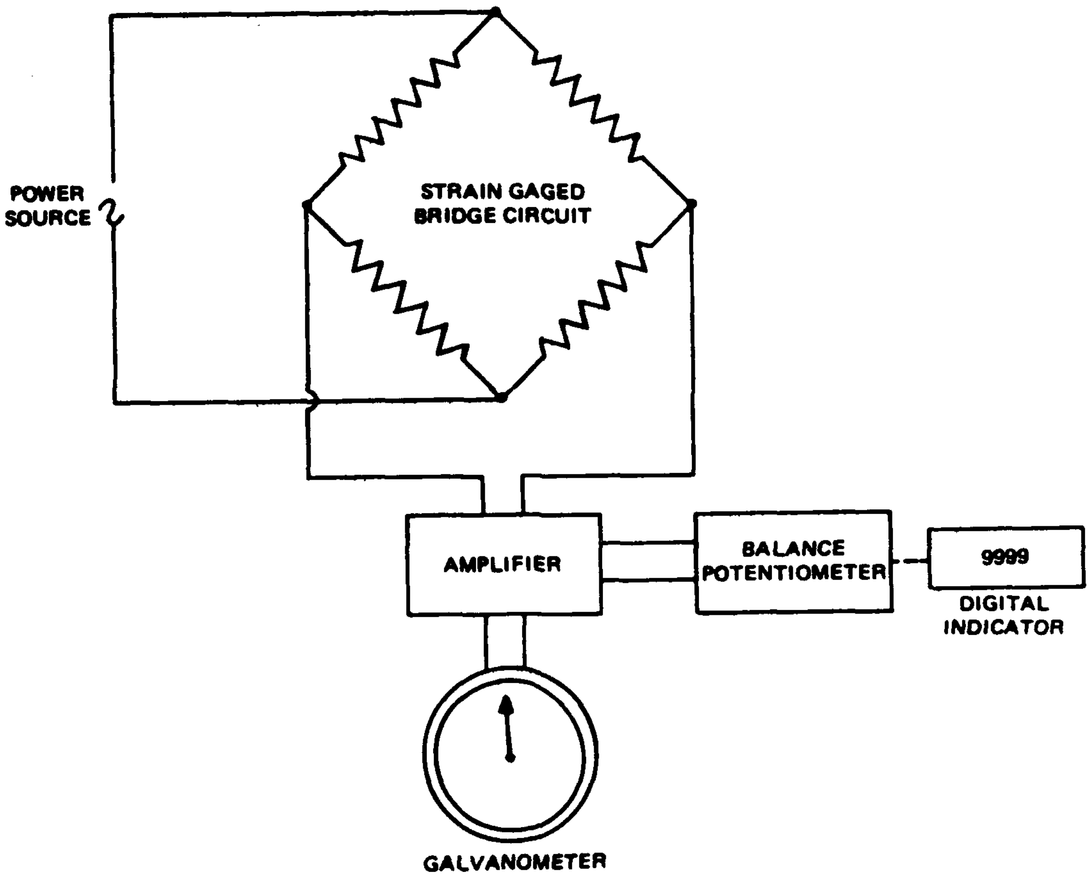


Fig. 4. Typical strain gage readout unit.

correction factors to obtain true load readings, and good temperature stability over a wide temperature range, thus allowing the load cells to be used in changing environmental conditions. The readout equipment is small, compact, and generally suitable for field use. The disadvantages of the bonded strain-gaged load cells are the extreme care required in waterproofing to prevent electrical leakage in the gaged circuit, the gage bonding technique required to assure long-term stability of the load cell, and the recalibration required when changing cable lengths due to lead wire load resistance changes and parasitic electrical signals.

2.1.3.2 The vibrating-wire load cell is generally constructed similar to the bonded strain gage cell. The load cells differ in the method of measuring the axial strain of the cell body. The vibrating-wire strain gage consists of a length of steel wire stretched between two posts extending from the cylinder wall (Fig. 5). The wire is pre-tensioned below its elastic limits when installed on the cell body. An electromagnet is placed near the wire, providing a method of plucking the wire when an electrical pulse excites the magnet. This pulse causes the wire to vibrate over the magnetic coil. The vibrating wire induces an electrical current in the electromagnet coil with a frequency equal to the frequency of the vibrating wire. The signal is then amplified by the readout unit and the frequency is determined by a frequency counter. As the load cell is subjected to load, the strain in the cylinder body reduces the tension on the vibrating wire, changing its frequency. This change in frequency per unit load is used to calibrate the load cell. Most load cells contain at least three vibrating-wire transducers placed at 120 deg around the periphery of the cell body. The vibrating wires are read separately and the readings averaged. This method of reading reduces errors caused by eccentric loading of the load cell. Some advantages of the vibrating-wire load cell are that the loads are read as a frequency, thus reducing the problems caused by ground leakage and poor signal cable condition. Long signal cables should not effect the frequency readings as long as the signal is not attenuated beyond the sensitivity of the readout equipment. The vibrating-wire load cell could also be read through radio telemetry systems, eliminating the need for an analog to frequency converter. Some of the disadvantages of the vibrating-wire load cells are their physical size, cost of manufacturing, poor temperature compensation, expensive and complicated readout equipment, and vulnerability to shock damage, causing zero shifts in load cell readings.

2.1.3.3 Load cells utilizing the unbonded strain gage have not found wide usage in load cell manufacturing. The unbonded strain gage employs the same principle as the bonded gage in that it consists of a wire made of a resistive material that, when strained, changes its resistance in proportion to the strain. The unbonded strain gage has more commonly been used in soil and concrete stress meters such as manufactured by the Carlson Company. The gages are mounted similar to the vibrating-wire gage and are generally employed in a bridge utilizing the same readout principles as the bonded gage (Fig. 6).

2.1.4 Photoelastic Load Cell - The photoelastic load cell consists of a cylindrical steel column with a hole drilled through its center diameter. A photoelastic element (optical glass) is inserted in this hole and locked in place. When polarized light passes through the optic glass, interference fringes can be observed if viewed through a polarizing filter. The number of fringes observed depends on the amount of stress in the optic glass. The load cell is calibrated by counting the interference fringes produced by a given load. Although this type of load cell is quite rugged and is a comparatively simple device, limited use has been made of it due to its coarse calibration and inability to be read from a remote location.

2.1.5 Manufacturers - Although there are a large number of companies manufacturing load cells for various applications, most of this equipment is not suitable for rock mechanics instrumentation.

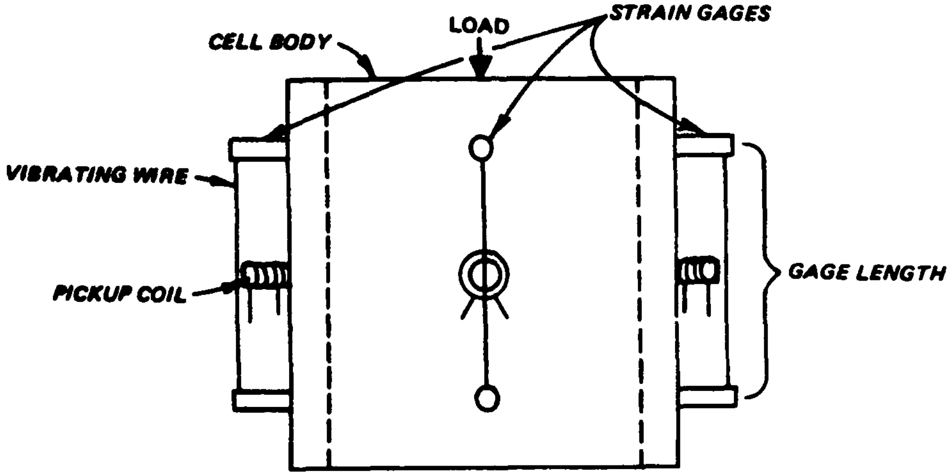


Fig. 5. Vibrating-wire load cell.

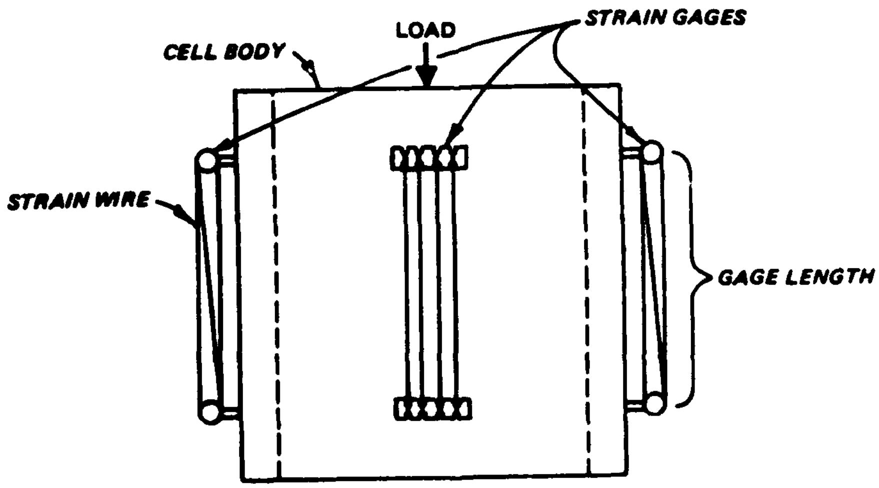


Fig. 6. Unbonded strain gage load cell.

The following is a partial list of geotechnical instrumentation suppliers that deal with the various types of load cells previously described:

Hydraulic load cells

Terrametrics, PJJ Machine Co., Interfels

Mechanical load cells

Terrametrics, Interfels, Norseman, Procep, Doboku Sokki, Strain sert

Strain-gaged load cells

Terrametrics, Soil Test, Telamac, Maihok, Geonor, Remote Systems

Photoelastic load cells

Terrametrics, Stress Engineering

3. Procedure

3.1 Although load cells have been employed for many diversified applications, the primary function in the field of rock mechanics has been for tunnel support load monitoring and rock bolt tension measurements. The following uses and methods of installation are typical for most applications.

3.1.1 Steel Arch Tunnel Support Instrumentation - The structural steel arch support set is the most commonly used tunnel support system. Measurements of the compressive loads actually being supported by the arch support provide a direct means of comparing actual loads with assumed support design loads. Load cells are generally installed under the base plates of arch-type sets; however, at times, it is desirable to include a crown load cell. In areas of squeezing or swelling ground invert struts may be used with load cells placed in them to measure the side loads. Close attention to the placement of blocking should be observed to assure proper load distribution on the arch supports (Fig. 7). The load cells should be installed on the set at the time of the set placement. It is desirable to place the instrumented sets as close to the blasting face as practical to measure the entire load history. Care must be taken to afford blast protection for the load

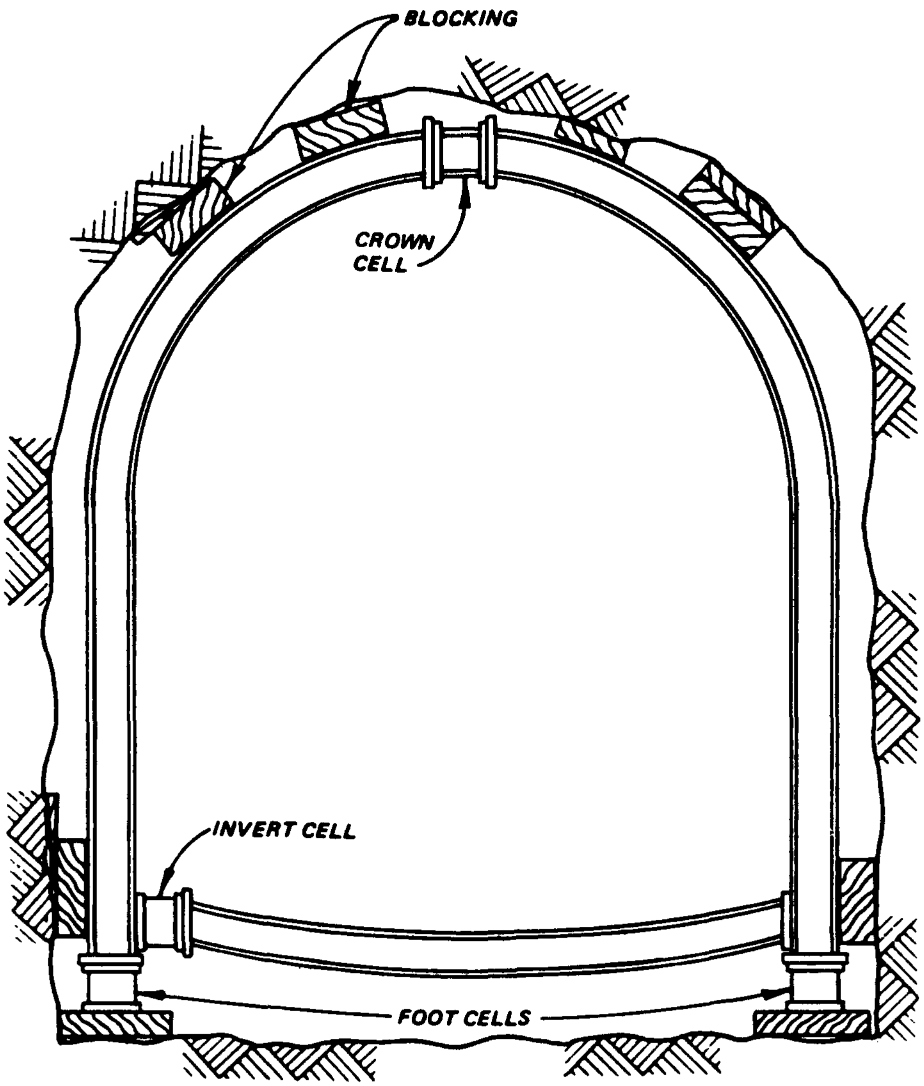
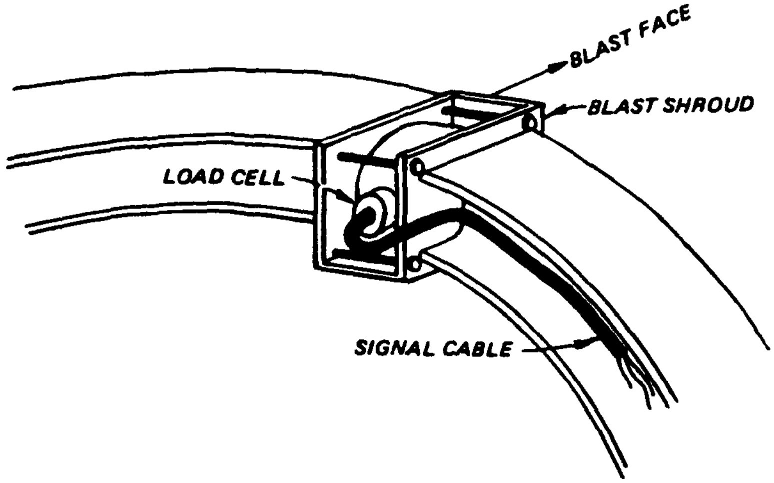


Fig. 7. Arch support load cell placement.

cells and signal cables when used near the blast face. This can be accomplished by running the signal cable inside the set flange facing away from the tunnel face. Steel shrouds can be welded to the sets to protect the load cells from fly rock (Fig. 8). The load cells selected should have a capacity greater than the yield strength of the arch supports. It is generally desirable to instrument at least three consecutive sets to reduce errors caused by anisotropy of the rock mass or nonuniform blocking on individual sets. Provisions should be made for the removal of the load cells after their portion of the tunnel has stabilized. This allows the load cells to be reused in a leap frog fashion, resulting in considerable instrumentation savings. Methods for removal of load cells on arch and circular sets are shown in Figs. 9 and 10. The load values obtained during a systematic instrumentation program provide a quantitative basis for reviewing the structural tunnel lining requirements for the final tunnel bore.

3.1.2 Rock Bolts and Tieback System Instrumentation - Rock bolts and tiebacks are often used to stabilize subsurface and surface excavations. In either case, the system usually incorporates steel rods or cables anchored at the base of a drill hole and tensioned to produce a compressive load along the axis of the drill hole. The actual loads acting on the bolt can be monitored (Fig. 11) by using a hollow core load cell acting as a washer at the collar of the bolt assembly. Calibrated torque wrenches have been widely used in rock bolting to produce the desired tension in the anchor tendon. Extensive tests have proven that such torque measurements can produce errors in the bolt tension as much as one to two times the indicated load. This variation can be caused by the condition of the threads on the bolt or anchor, dirt, rust, bending of the bolt, or anchor misalignment. Hydraulic jacks are often used in place of the torque wrench, especially in bolts or cables requiring high tensile loads. This system, where applicable, is far superior to the torque wrench but requires the use of specialized equipment for its adaptation. Actual loads can be monitored, using load cells, at the



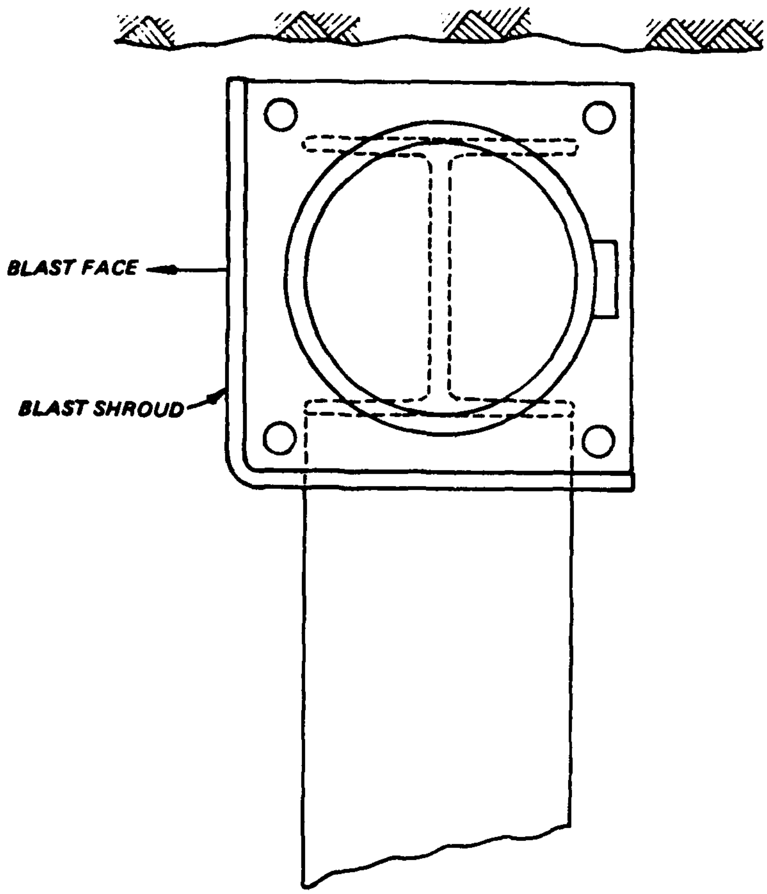


Fig. 8. Crown load cell blast protection

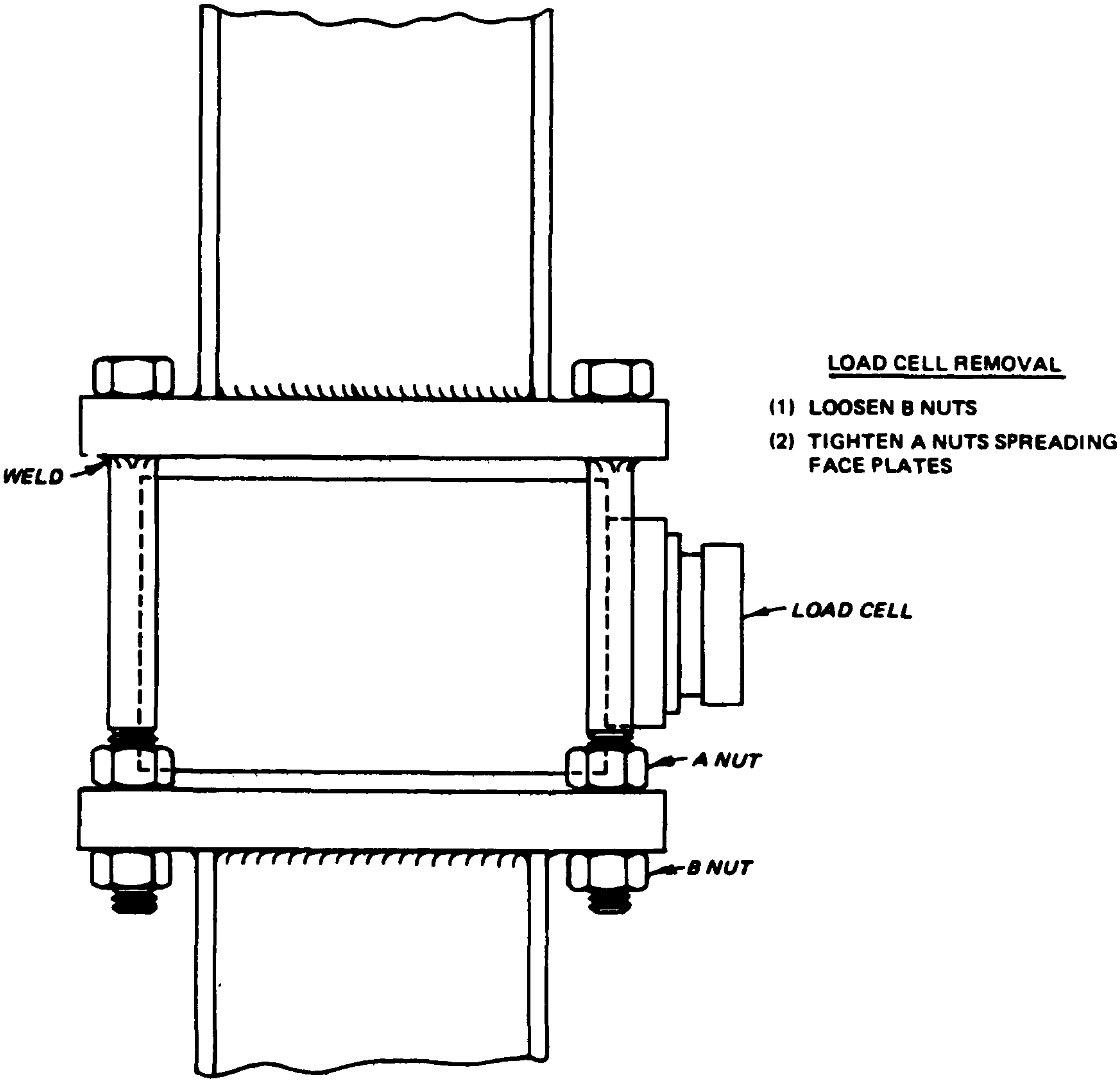


Fig. 9. Crown or springline load cell removal.

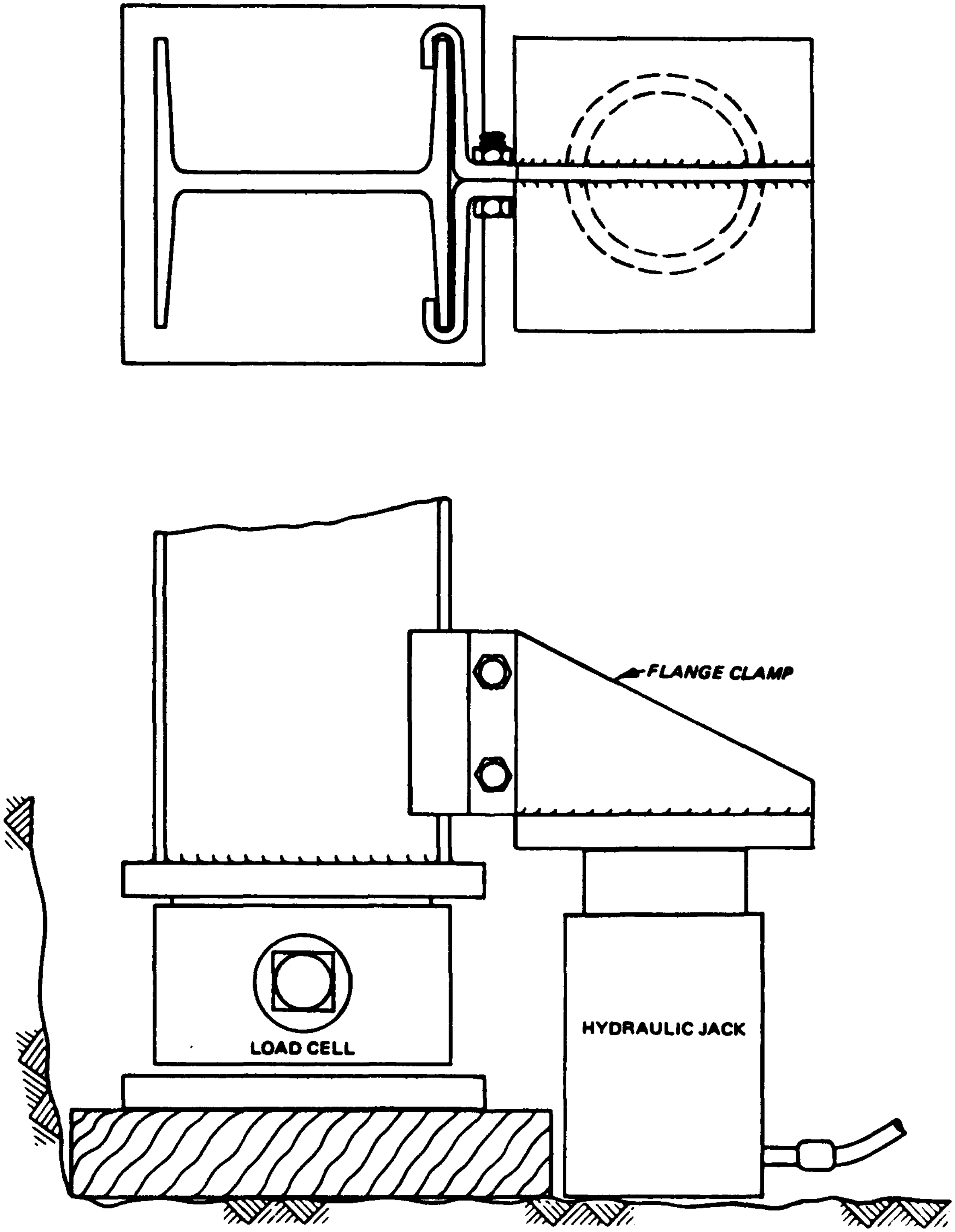


Fig. 10. Foot load cell removal.

time of installation and thereafter to plot the load history of the bolt system. It is desirable to use a number of load cells over a tunnel length equivalent to 2-1/2 tunnel diameters to obtain the average load being supported by the rock bolts. The load cell capacity should be greater than the yield strength of the rock bolts because the strain induced into the bolts at the time of blasting often exceeds their yield point. After tunnel stabilization the load cells can be removed and reused as the tunnel advances. The cells should be removed one at a time and the bolt retensioned to maintain stability of the tunnel section.

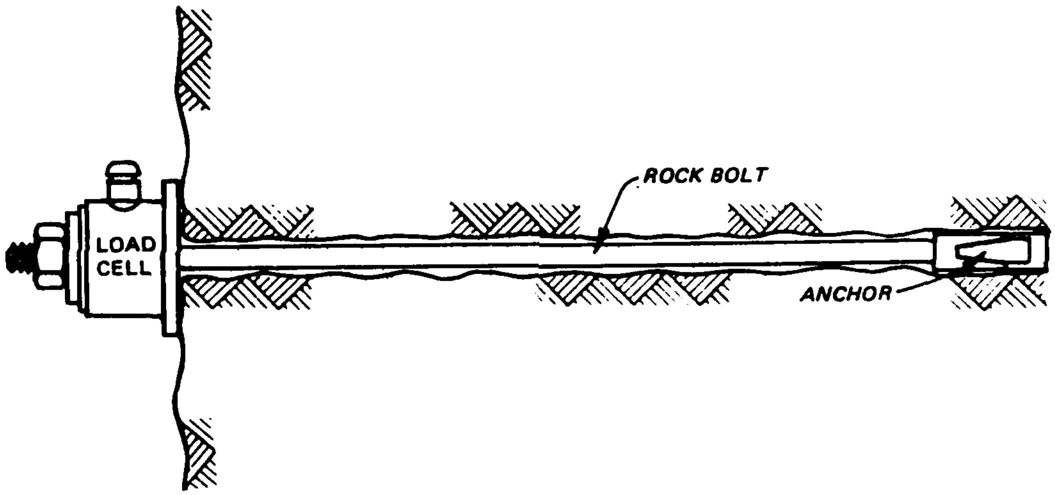


Fig. 11. Rock bolt load cell installation.

4. <u>Data Reduction</u>

4.1 <u>Steel Arch Supports</u> - As the tunnel excavation advances, the load normally being supported by the rock or soil removed in the excavated section is redistributed into the side walls and support system. At some point in time, depending on the tunnel advancement rate, material strength, and support system, the tunnel reaches a point of equilibrium. Support loads should be monitored during this stabilization period to determine support adequacy and future requirements.

Fig. 12 is a typical plot of load versus tunnel advancement. Note the decreasing frequency of readings as tunnel heading advances away from the load cells. The load is generally highest two to three diameters behind the tunnel face because the load has not stabilized or shifted to the side walls at this time. In this example the peak loading occurred when the tunnel heading was at station 130 (20 ft (6 m) or 2 tunnel diameters beyond the instrumental set) and equilibrium at station 110 was essentially achieved when the tunnel heading reached heading 148. A blocking diagram is often included with the data plot to help explain the stress distribution on the support member. A load versus time plot may also be used in some instances where varying time lags exist between tunnel heading advancements.

4.2 Rock Bolts - Rock bolt load data may be plotted in the same manner as the data for the arch support load cells; however, the indicated load should not decrease with time or tunnel advancement. By the use of the load plot the engineer can readily recognize anchor slippage, i.e., load loss or overload. A certain amount of anchor slippage is generally observed with bolts located near the blast face. When this occurs, the bolts should be retorqued to design specifications. The load plot will indicate stabilization versus tunnel advancement and can be used to determine a bolt torquing program for noninstrumented tunnel sections.

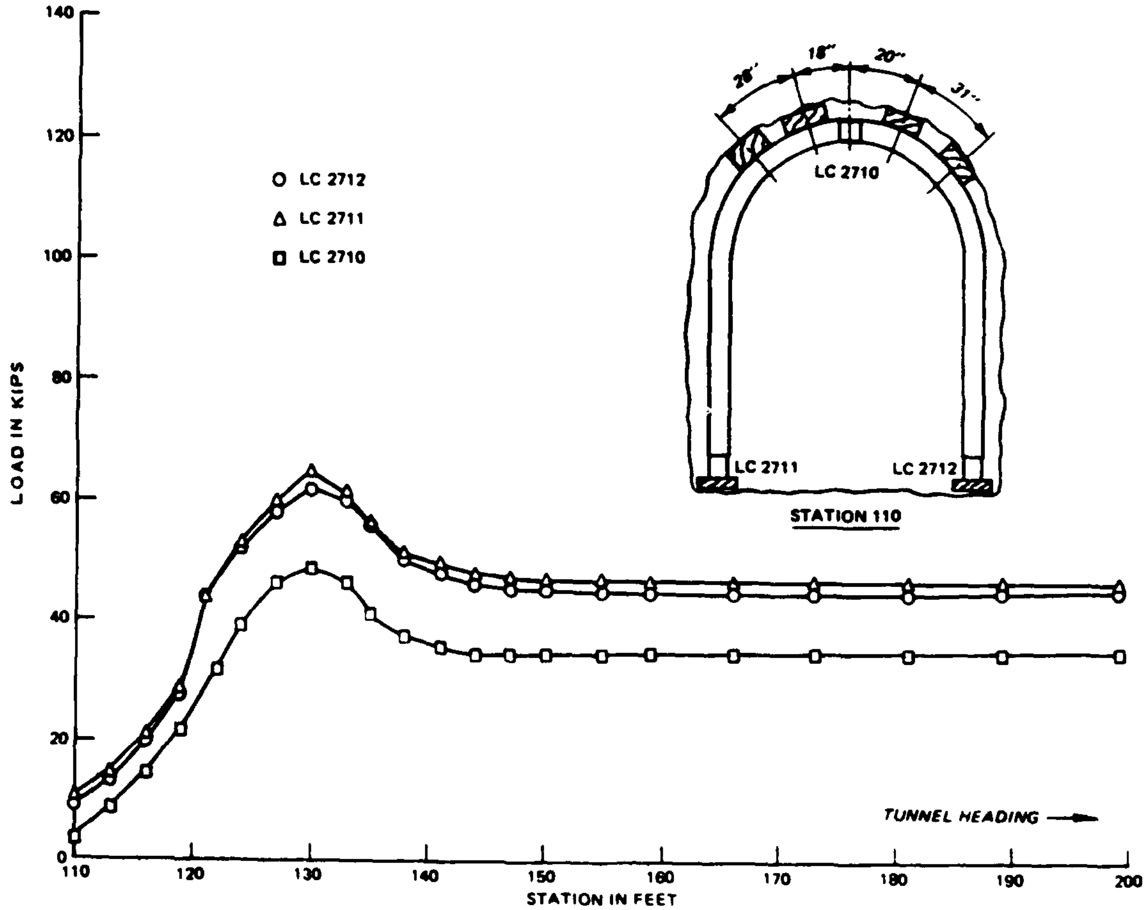


Fig. 12. Typical plot of load versus tunnel advancement.

BIBLIOGRAPHY

Able, J. F., "Tunnel Mechanics," Quarterly of the Colorado School of Mines, Vol 62, No. 2, 1967.

Cording, E. J., "Methods for Geotechnical Observations and Instrumentation in Tunneling," The National Science Foundation Research Grant GI-33644X, 1975.

Hartmann, B. E., "Rock Mechanics Instrumentation for Tunnel Construction," Terrametrics, Inc., Golden, Colorado, 1967.

Huie, J. S. and Lachel, D. J., "Warm Springs Project Instrumentation Program," U. S. Army Engineer Division, Missouri River, Omaha, Nebraska, 1973.

Lane, K. S., "Field Test Sections Save Cost in Tunnel Support," Underground Construction Research Council, Published by American Society of Civil Engineers, October 1975.

SUGGESTED METHOD OF DETERMINING ROCK BOLT TENSION USING A TORQUE WRENCH

1. Scope

1.1 This method can be used to apply a specified tension during rock bolt installation or to estimate loss of tension in a previously installed bolt. It can also be used to verify that anchor strength is greater than a specified value consistent with the maximum tension that can be applied with the wrench.

2. Apparatus

2.1 A torque wrench, preferably with a maximum applied torque indicator, capable of giving readings that are repeatable to 5 percent throughout the range of torques to be measured. It should be provided with sockets suitable for the nuts or bolt heads to be tested, should be used only for testing, and should be stored, together with its most recent calibration chart, in a dry place so as to preserve its accuracy of reading.

2.2 Equipment for calibrating the torque wrench (Fig. 1) including a rigidly fixed bolt head, a weight pan and weights, and a measuring tape.

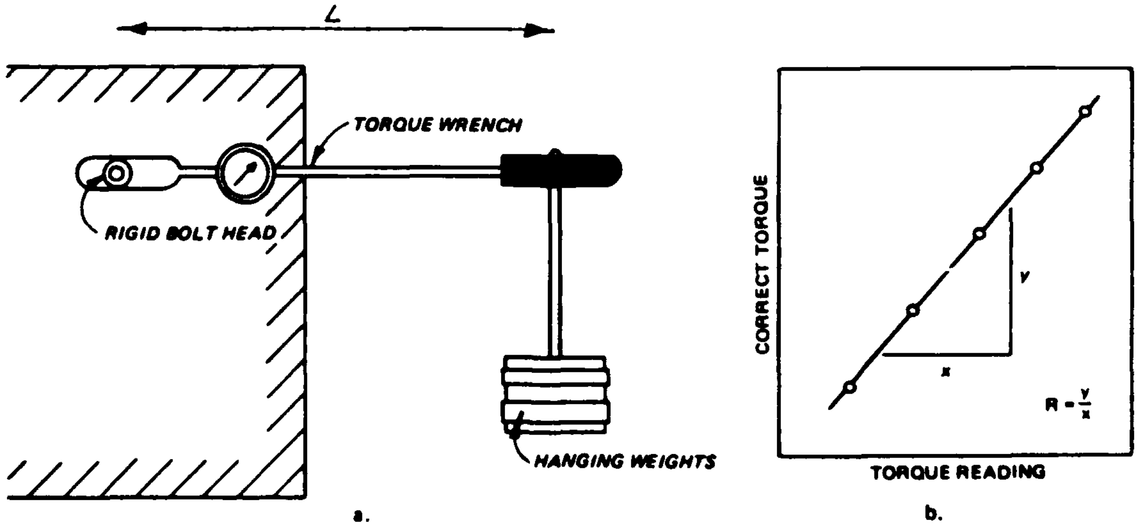


Fig. 1. Calibration of torque wrench.

2.3 Equipment for determining the relationship between tension and torque (Fig. 2), typically an installed rock bolt and faceplate assembly identical with that to be used in practice, and a hydraulic ram with handpump and pressure gage (to be used for tension measurement) or alternatively a rock bolt load cell. Tension should be measured with an accuracy better than 2 percent of the maximum reached in the test.

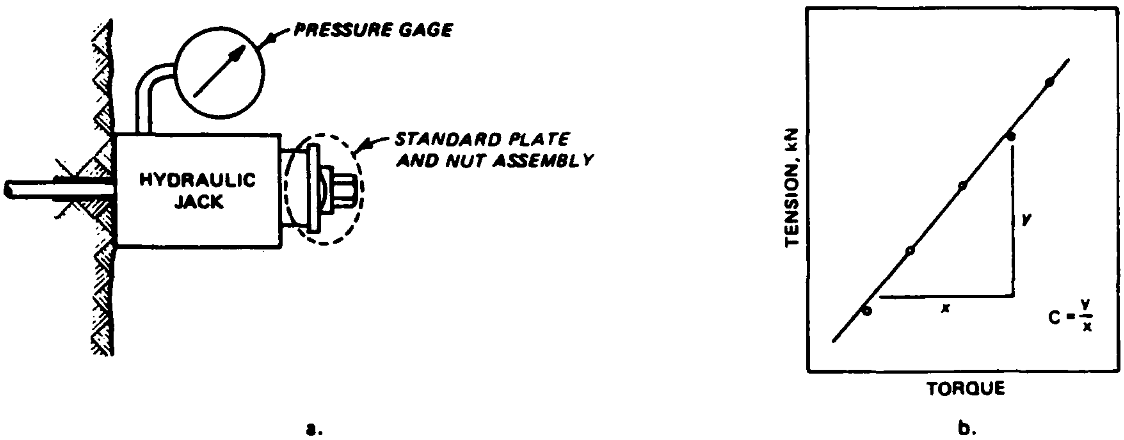


Fig. 2. Determination of ratio tension/torque.

3. Procedure

3.1 Calibration of the torque wrench should be accomplished as follows:

(a) With the wrench horizontal, the wrench socket is positioned on a rigid bolt head. A weight pan is suspended from the center of the wrench handle (Fig. 1) and weights are added. The torque reading is noted, also the weight of the pan together with the weights it contains. The procedure is repeated with increasing weights to obtain at least five torque readings covering the range of torques for which the wrench is to be used. The distance L between the center of the wrench handle and that of the bolt head is recorded.

(b) Correct torque values are calculated by multiplying the distance L by the applied weights. A graph is plotted of correct torque values against torque readings, and a straight line is fitted to the data points (Fig. 1b). The gradient of this line is measured, equal to the ratio R of correct torque divided by torque reading. Torque readings later obtained when using this wrench should be multiplied by the ratio R to obtain corrected values.

(c) Torque wrenches should be recalibrated at intervals not exceeding six months.

3.2 Determination of the ratio C of tension to torque is as follows:

(a) The load cell, or alternatively a hydraulic ram with the ram extended to 3/4 travel, is positioned concentrically and coaxially over the bolt to be tested, and the face nut is tightened to take up slack in the assembly (Fig. 2). Ram pressure should be increased to a nominally small value before the start of the test, and the pump valve firmly closed.

(b) The bolt diameter, state of lubrication, thread pitch, faceplate, and washers should be identical with the conditions expected in the actual rock bolt installation.

(c) Torque is applied in increasing increments to the nut, taking readings of torque and bolt tension. Torque application should be smooth and force should be applied through the center of the wrench handle only. At least five pairs of readings are required, covering the complete range of torques for which the wrench is to be used.

(d) A graph of tension versus torque is plotted, showing data points and a straight line fitted to these points. The gradient C of this line, the ratio of tension to torque, is measured (Fig. 2b).

(e) The ratio C is determined separately for each change in bolt diameter, thread pitch, and state of lubrication of for any other variation in the bolt/anchor/faceplace assembly that may result in a change in the tension/torque ratio.

3.3 Determination of bolt tension using the torque wrench is done as follows:

(a) If a torque wrench of the type that applies a preset torque is used, the torque setting should be increased in small increments until just sufficient to cause the face nut to rotate. The torque setting, the bolt identification, and the date are recorded.

(b) If a torque wrench with a maximum applied torque indicator is used, the torque may be applied steadily rather than in increments. Both types of torque wrench should be used with care to ensure that loading is smooth and that force is applied through the center of the wrench handle.

(c) Bolt tension is calculated using the correction R and a value of tension/torque ratio C determined for the identical bolt and faceplate assembly conditions using the method described in paragraph 3.2 above.

(d) An approximate check on minimum anchor strength may be obtained by applying an increasing torque, recording this torque as a function of number of rotations until no further torque can be applied, or until the anchor shows signs of failing.

4. Reporting of Results

4.1 The report should include diagrams and graphs as illustrated in Figs. 1 and 2, together with full details of:

(a) Torque wrench calibration; type of torque wrench, methods used for calibration, and results.

(b) Determination of the tension/torque ratio C; methods used and results obtained.

(c) The rock bolts tested; types, locations, dates installed, rock characteristics, methods used for drilling and installation, appearance, and condition of the faceplate assembly at time of testing.

(d) The method used for tension determination; tabulated values of bolt identification, applied torque to cause rotation of the nut, corresponding bolt tension, and any other observations pertinent to the test results.

4.2 If the method is used as a check on minimum anchor strengths, data should be included in the form of graphs of torque versus nut rotation, with scales converted to show bolt tension versus displacement. The report may compare these results with an arbitrary acceptable performance established by previous extensive testing. The complete bolt tension versus displacement curve should be considered when making such a comparison.

SUGGESTED METHOD FOR MONITORING ROCK BOLT TENSION USING LOAD CELLS

1. Scope

1.1 This method is for monitoring changes in tension that occur in a rock bolt over an extended period of time following installation (Note 1).

NOTE 1—Tension may fall below that applied at the time of rock bolt installation due to loosening and slip either of the anchor or of the faceplate assembly, for example as a result of rock creep, anchor corrosion, fretting of rock from beneath the faceplate, or blasting vibration. Tension may also either rise or fall as a result of bulk rock dilation or contraction associated with the progress of nearby excavation.

2. Apparatus

2.1 Rock bolt load cells should be used to monitor tension in approximately one bolt in ten of the support system to be studied (special considerations may require the instrumentation of a greater or lesser percentage of bolts). The load cells may, for example, be of mechanical, photoelastic, hydraulic, or electric type, depending on requirements of cost and accuracy. The cells should have reversible and preferably linear calibrations (see paragraph 3) and should incorporate a spherical seating or other provision to ensure that load transfer and measurement are reproducible. They should be capable of withstanding the effects of nearby blasting, water, and dust over long periods of time.

3. Procedure

3.1 Calibration of load cells should be accomplished as follows:

(a) Calibration is required when selecting a suitable type of load cell, and each load cell to be installed should be individually calibrated before use.

(b) Short-term calibration of each cell is performed in a testing laboratory by increasing the load in increments, taking readings of 'observed' and 'true' load values (Note 2). Tension is released and incremental readings are taken during unloading. A further cycle of loading and unloading is made, and a graph plotted showing data points and curves fitted to these points (Fig. 1).

NOTE 2—Details of the calibrating devices and their precision should be included in the report.

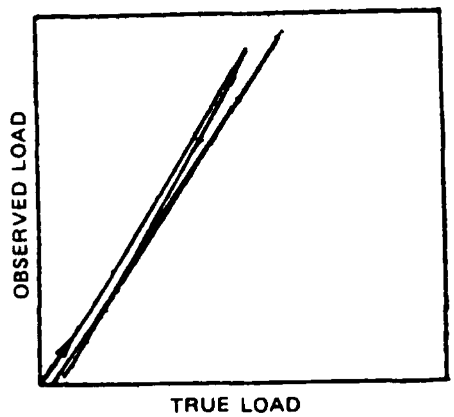


Fig. 1. Load cell calibration

(c) A check should also be made on the stability of readings over extended periods of time. Tension is increased to a value approximately equal to that to be measured on site, and is maintained at this value for as long a time as is practical. Any 'drift' in reading is noted. The effect of water on the cell should be observed, also the effect of coupling and uncoupling any electrical connections.

3.2 Installation and monitoring shall be as follows:

(a) Load cells are installed on selected rock bolts at the time of installation of the support system. Care should be taken to ensure that spherical seatings are correctly positioned and lubricated. Bolts that have been instrumented should be clearly and permanently numbered, and may be painted for ease of recognition. Bolt length, diameter, and type of anchor should also be noted.

(b) Tension readings should be taken immediately following installation and again a few hours later. Further readings may be taken at intervals depending on the rate at which readings are changing. In the vicinity of an advancing face, for example, intervals between readings should be of the order of hours, whereas if steady values are recorded for bolts in inactive areas the intervals between readings may be increased to days or weeks. Each reading should be accompanied by a record of bolt number, location, and the date and time of observation.

4. Calculations

4.1 Bolt tension readings are corrected using the calibration charts. Graphs of bolt tension versus time are plotted for each bolt (Fig. 2). For comparison, the loss or gain of tension may be reduced to a percentage of the initial installed value.

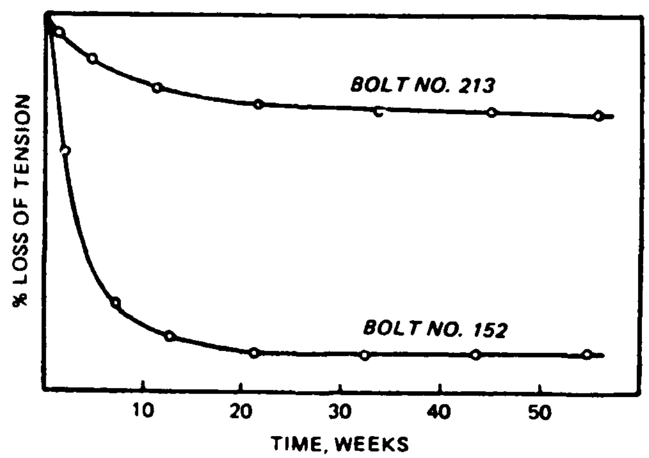


Fig. 2. Graph of bolt tension loss.

5. Reporting of Results

5.1 The report should include:

(a) Details of the load cells used, calibration methods, and results.

(b) Locations of the rock bolts monitored.

(c) Details of rock characteristics, bolt and anchor types and dimensions, and dates and methods of installation and grouting.

(d) Monitoring results in the form of tabulated values and a graph of bolt tension versus time for each bolt monitored.

PART II. IN SITU STRENGTH METHODS

B. In Situ Strength Tests

SUGGESTED METHOD FOR IN SITU DETERMINATION OF DIRECT SHEAR STRENGTH

(International Society for Rock Mechanics)

1. Scope

1.1 This test measures peak and residual direct shear strengths as a function of stress normal to the sheared plane. Results are usually employed in limiting equilibrium analysis of slope stability problems or for the stability analysis of dam foundations (Notes 1-3).

NOTE 1—Direct shear strength can be determined in the laboratory (using the method described in RTH 203) if the plane to be tested is smooth and flat in comparison with the size of specimen, and if the specimen can be cut and transported without disturbance.

NOTE 2—Definitions (clarified in Figs. 5 and 6):

Peak shear strength - the maximum shear stress in the complete shear stress displacement curve.

Residual shear strength - the shear stress at which no further rise or fall in shear strength is observed with increasing shear displacement. A true residual strength may only be reached after considerably greater shear displacement than can be achieved in testing. The test value should be regarded as approximate and should be assessed in relation to the complete shear stress-displacement curve.

Shear strength parameters c and ϕ - respectively, the intercept and angle to the normal stress axis of a tangent to the shear strength-normal stress curve at a normal stress that is relevant to design (see Fig. 6).

NOTE 3—The measured peak strength can be applied directly to full-scale stability calculations only if the same type and size of roughness irregularities are present on the tested plane as on a larger scale. If this is not the case, the true peak strength should be obtained from the test data using appropriate calculations (for example, Patton, F. D., 1966, Proc. 1st Int. Cong. Rock Mech. ISRM, Lisbon, Vol 1, pp. 509-512;

Ladanyi, B. and Archambault, G., 1970. In "Rock Mechanics - Theory and Practice," (W.II. Somerton, ed.), AIME, New York, pp. 105-125; Barton, N. R., 1971, Proc. Symp. ISRM, Nancy, Paper 1-8).

1.2 The inclination of the test block and system of applied loads are usually selected so that the sheared plane coincides with a plane of weakness in the rock (e.g., a joint, plane of bedding, schistosity, or cleavage), or with the interface between soil and rock or concrete and rock (Note 4).

NOTE 4—Tests on intact rock (free from planes of weakness) are usually accomplished using laboratory triaxial testing. Intact rock can, however, be tested in direct shear if the rock is weak and if the specimen block encapsulation is sufficiently strong.

1.3 A shear strength determination should preferably comprise at least five tests on the same test horizon with each specimen tested at a different but constant normal stress.

1.4 In applying the results of the test, the pore water pressure conditions and the possibility of progressive failure must be assessed for the design case as they may differ from the test conditions.

2. Apparatus

2.1 Equipment for cutting and encapsulating the test block, rock saws, drills, hammer and chisels, formwork of appropriate dimensions and rigidity, expanded polystyrene sheeting or weak filler, and materials for reinforced concrete encapsulation.

2.2 Equipment for applying the normal load (e.g., Fig. 1) including:

(a) Flat jacks, hydraulic rams, or dead load of sufficient capacity to apply the required normal loads (Note 5).

NOTE 5—If a dead load is used for normal loading, precautions are required to ensure accurate centering and stability. If two or more hydraulic rams are used for either normal or shear loading, care is needed to ensure that they are identically matched and are in exact parallel alignment. Each ram should be provided with a spherical seat. The travel of rams and particularly of flat jacks should be sufficient to accommodate the full anticipated specimen displacement. A normal displacement of ±5–10 mm may be expected, depending on the clay content and roughness of the shear surface.

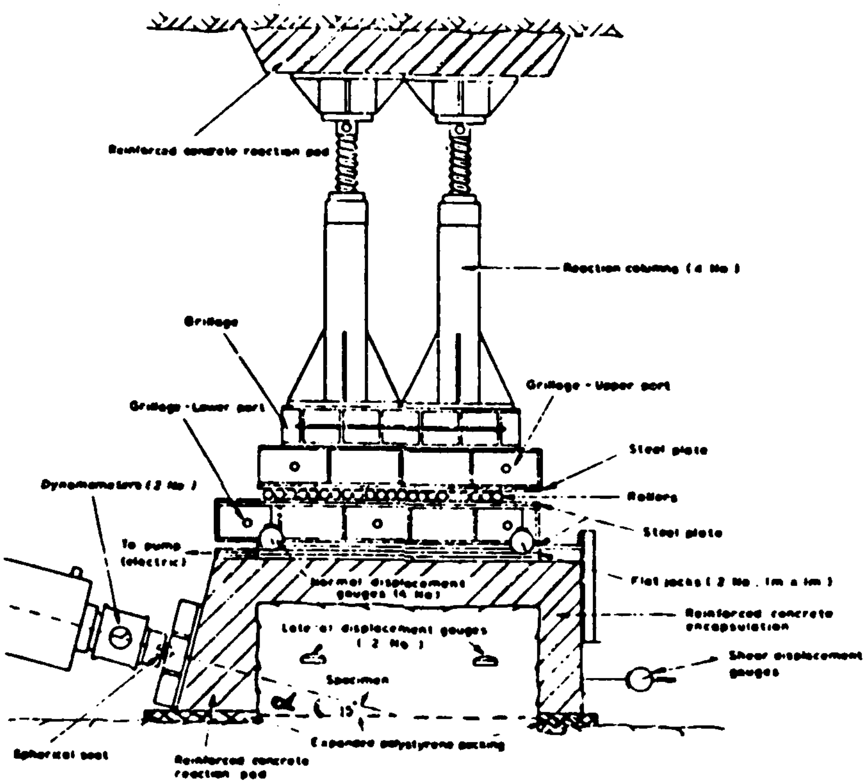


Fig. 1. Typical arrangement of equipment for in situ direct shear test.

(b) A hydraulic pump if used should be capable of maintaining normal load to within 2 percent of a selected value throughout the test.

(c) A reaction system to transfer normal loads uniformly to the test block, including rollers or a similar low friction device to ensure that at any given normal load, the resistance to shear displacement is less than 1 percent of the maximum shear force applied in the test. Rock anchors, wire ties, and turnbuckles are usually required to install and secure the equipment.

2.3 Equipment for applying the shear force (e.g. Fig. 1) including:

(a) One or more hydraulic rams (see Note 5) jacks of adequate total capacity with at least 70-mm travel.

(b) A hydraulic pump to pressurize the shear force system.

(c) A reaction system to transmit the shear force to the test block. The shear force should be distributed uniformly along one face of the specimen. The resultant line of applied shear forces should pass through the center of the base of the shear plane (Note 6) with an angular tolerance of ±5 deg.

NOTE 6—The applied shear force may act in the plane of shearing so that the angle α is 0 (Fig. 1). This requires a cantilever bearing member to carry the thrust from the shear jacks to the specimen. If a method is used where the shear force acts at some distance above the shear plane, the line of action of the shear jacks should be inclined to pass through the center of area of the shear plane. The angle for a specimen 700 by 700 by 350 mm approximates to 15 deg depending on the thickness of encapsulation. Tests where both shear and normal forces are provided by a single set of jacks inclined at greater angles to the shear plane are not recommended, as it is then impossible to control shear and normal stresses independently.

2.4 Equipment for measuring the applied forces including one system for measuring normal force and another for measuring applied shearing force with an accuracy better than ±2 percent of the maximum forces reached in the test. Load cells (dynamometers) or flat jack pressure measurements may be used. Recent calibration data applicable to the range of testing should be appended to the test report. If possible, the gages should be calibrated both before and after testing.

2.5 Equipment for measuring shear normal, and lateral displacements:

(a) Displacements should be measured (e.g. using micrometer dial gages (Note 7)) at eight locations on the specimen block or encapsulating material, as shown in Fig. 2.

NOTE 7—The surface of encapsulating material is usually insufficiently smooth and flat to provide adequate reference for displacement gages, and glass plates may be cemented to the specimen block for this purpose. These plates should be of adequate size to accommodate movement of the specimen. Alternatively, a tensioned wire and pulley system with gages remote from the specimen can be used. The system as a whole must be reliable and conform with specified accuracy requirements. Particular care is needed in this respect when employing electric transducers or automatic recording equipment.

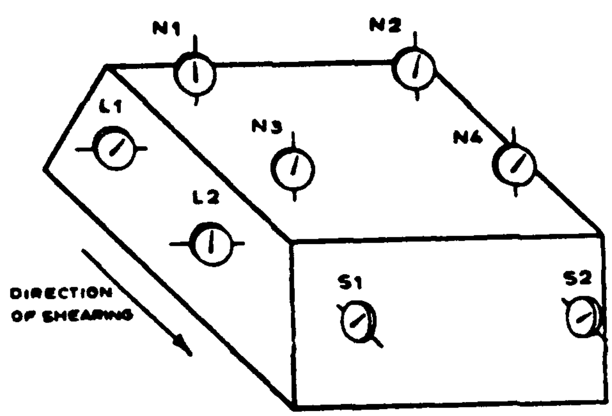


Fig. 2. Arrangement of displacement gages (S1 and S2 for shear displacement L1 and L2 for lateral displacement, N1-N4 for normal displacement.

(b) The shear displacement measuring system should have a travel of at least 70 mm and an accuracy better than 0.1 mm. The normal and lateral displacement measuring systems should have a travel of at least 20 mm and an accuracy better than 0.05 mm. The measuring reference system (beams, anchors, and clamps) should, when assembled, be sufficiently rigid to meet these requirements. Resetting of gages during the test should be avoided if possible.

3. Procedure

3.1 Preparation:

(a) The test block is cut to the required dimensions (usually 700 x 700 x 350 mm) using methods that avoid disturbance or loosening of the block (Notes 8 and 9). The base of the test block should coincide with the plane to be sheared and the direction of shearing should correspond if possible to the direction of anticipated shearing in the full-scale structure to be analyzed using the test results. The block and particularly the shear plane should, unless otherwise specified, be retained as close as possible to its natural in situ water content during preparation and testing, e.g., by covering with saturated cloth. A channel approximately 20 mm deep by 80 mm wide should be cut around the base of the block to allow freedom of shear and lateral displacements.

NOTE 8--A test block size of 700 x 700 x 350 mm is suggested as standard for in situ testing. Smaller blocks may be permissible, for example if the surface to be tested is relatively smooth; larger blocks may be needed when testing very irregular surfaces. The size and shape of test block may for convenience be adjusted so that faces of the block coincide with natural joints or fissures; this minimizes block disturbance during preparation. Irregularities that would limit the thickness or emplacement of encapsulation material or reinforcement should be removed.

NOTE 9--It is advisable, particularly if the test horizon is inclined at more than 10-20 deg to the horizontal, to apply a small normal load to the upper face of the test specimen while the sides are cut, to prevent premature sliding and also to inhibit relaxation and swelling. The load, approximately 5-10 percent of that to be applied in the test, may for example be provided by screw props or a system of rock bolts and crossbeams and should be maintained until the test equipment is in position.

(b) A layer at least 20 mm thick of weak material (e.g. foamed polystyrene) is applied around the base of the test block, and the remainder of the block is then encapsulated in reinforced concrete or similar material of sufficient strength and rigidity to prevent collapse or significant distortion of the block during testing. The encapsulation formwork should be designed to ensure that the load bearing faces of the encapsulated block are flat (tolerance ±1 mm) and at the correct inclination to the shear plane (tolerance ±2 deg).

(c) Reaction pads, anchors, etc., if required to carry the thrust from normal and shear load systems to adjacent sound rock, must be carefully positioned and aligned. All concrete must be allowed time to gain adequate strength prior to testing.

3.2 Consolidation:

(a) The consolidation stage of testing is to allow pore water pressures in the rock and filling material adjacent to the shear plane to dissipate under full normal stress before shearing. Behavior of the specimen during consolidation may also impose a limit on permissible rate of shearing (see paragraph 3.3(c)).

(b) All displacement gages are checked for rigidity, adequate travel, and freedom of movement, and a preliminary set of load and displacement readings is recorded.

(c) Normal load is then raised to the full value specified for the test, recording the consequent normal displacements (consolidation) of the test block as a function of time and applied loads (Figs. 3 and 4).

DIRECT SHEAR TEST DATA SHEET

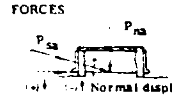
Client: Project. Concrete Dam Location: Alcântara Loc. No.: Block No.:

TEST BLOCK SPECIFICATION See drawings & photographs Nos..

General rock description, index properties and water conditions
Phyllite sound to moderately weathered Normal conditions

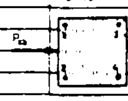
Description and index properties of surface to be sheared Type;
Dip, Dip direction; Roughness; Filling & alteration,
Persistance; Spacing of set; Surface dimensions; 0.70 x 0.70 Initial area A: 0.490 m^2

FORCES



$P_n = P_{na} + P_{sa} \sin \alpha$
$P_s = P_{sa} \cos \alpha$
$\sigma_n = P_n / A$
$\tau = P_s / A$
Normal displacement
$\alpha = 15^o$ (σ_n nominal)

1 Time elapsed (min)	2 Applied normal force P_n Reading	Force (kN)	3 Normal displacement Δ_n Reading 1	2	3	4	Average (mm)	4 Applied shear force P_s Reading	Force (kN)	5 Shear displacement Δ_s Reading 1	2	Average (mm)	6 Contact area A (corrected) m^2	7 P_{na} (kN)	8 σ_n (MPa)	9 P_{sa} (kN)	10 τ (MPa)
10		196	0.100	0.070	0.130	0.070			0	0	0	-	0.490	196		0	
35		233	0.130	0.065	0.140	0.090			137	0.05	0.05	0.05		•		142	
48		270	0.050	0.065	0.285	0.290			275	0.55	0.35	0.45		•		284	
64		306	-0.200	0.010	0.435	0.495			412	1.35	1.10	1.22		•		426	
87		343	-0.710	-0.205	0.600	0.720			549	2.55	2.30	2.42		•		568	
109		380	-1.165	-0.445	0.680	0.850			686	3.90	3.50	3.70		•		710	
131		417	-1.675	-0.615	0.710	0.970			824	5.15	4.60	4.88		•		853	
154		453	1.965	-0.745	0.720	1.050			961	6.10	5.50	5.80		•		995	
172		490	-2.245	-0.880	0.720	1.105			1098	7.20	6.50	6.85		•		1137	
189		527	-2.480	-1.055	0.695	1.165			1235	8.20	7.40	7.80		•		1279	
206		564	-2.750	-1.205	0.640	1.165			1373	9.45	8.45	8.95		•		1421	
234		601	-3.075	-1.505	0.465	1.100			1510	11.00	10.00	10.50		•		1563	
252		637	-3.350	-1.830	0.280	0.910			1647	12.45	11.40	11.92		•		1705	
264		674	-3.675	-2.185	0.050	0.720			1764	14.00	12.80	13.40		•		1847	
276		711	-4.005	-2.665	-0.290	0.360			1922	15.55	14.40	14.98		•		1989	
289		748	-4.585	-3.125	-0.890	-0.020			2059	17.00	16.45	17.02		•		2132	
298	Rupture	784	-4.975	-3.375	-1.250	-0.290			2196	20.00	19.55	19.78		•		2274	



Calibration data

Remarks

Tested by: Dates:
Checked by: Date:

Fig. 3. Example layout of direct shear test data sheet.

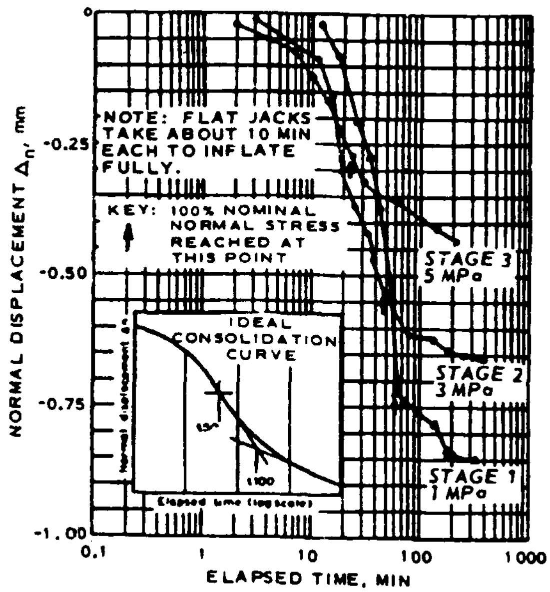


Fig. 4. Consolidation curves for a three-stage direct shear test, showing the construction used to estimate t_{100}.

(d) The consolidation stage may be considered complete when the rate of change of normal displacement recorded at each of the four gages is less than 0.05 mm in 10 minutes. Shear loading may then be applied.

3.3 Shearing:

(a) The purpose of shearing is to establish values for the peak and residual direct shear strengths of the test horizon. Corrections to the applied normal load may be required to hold the normal stress constant; these are defined in paragraph 4.5.

(b) The shear force is applied either in increments or continuously in such a way as to control the rate of shear displacement.

(c) Approximately 10 sets of readings should be taken before reaching peak strength (Figs. 3 and 5). The rate of shear displacement should be less than 0.1 mm/min in the 10-minute period before taking a set of readings. This rate may be increased to not more than 0.5 mm/min between sets of readings provided that the peak strength itself is adequately recorded. For a 'drained' test, particularly when testing clay-filled discontinuities, the total time to reach peak strength should exceed 6 t_{100} as determined from the consolidation curve (see paragraph 4.1 and Fig. 4) (Note 10). If necessary, the rate of shear should be reduced on the application of later shear force increments delayed to meet this requirement.

NOTE 10—The requirement that total time to reach peak strength should exceed 6 t_{100} is derived from conventional soil mechanics consolidation theory (for example Gibson and Henkel, Geotechnique 4, p 10-11, 1954) assuming a requirement of 90 percent pore water pressure dissipation. This requirement is most important when testing a clay-filled discontinuity. In other cases it may be difficult to define t_{100} with any precision because a significant proportion of the observed "consolidation" may be due to rock creep and other mechanisms unrelated to pore pressure dissipation. Provided the rates of shear specified in the text are followed, the shear strength parameters may be regarded as having been measured under conditions of effective stress ("drained conditions").

(d) After reaching peak strength, readings should be taken at increments of from 0.5-5 mm shear displacement as required to adequately define the force-displacement curves (Fig. 5). The rate of shear displacement should be 0.02-0.2 mm/min in the 10-minute period before a set of readings is taken and may be increased to not more than 1 mm/min between sets of readings.

(e) It may be possible to establish a residual strength value when the specimen is sheared at constant normal stress and at least four consecutive sets of readings are obtained which show not more than 5 percent variation in shear stress over a shear displacement of 1 cm (Note 11).

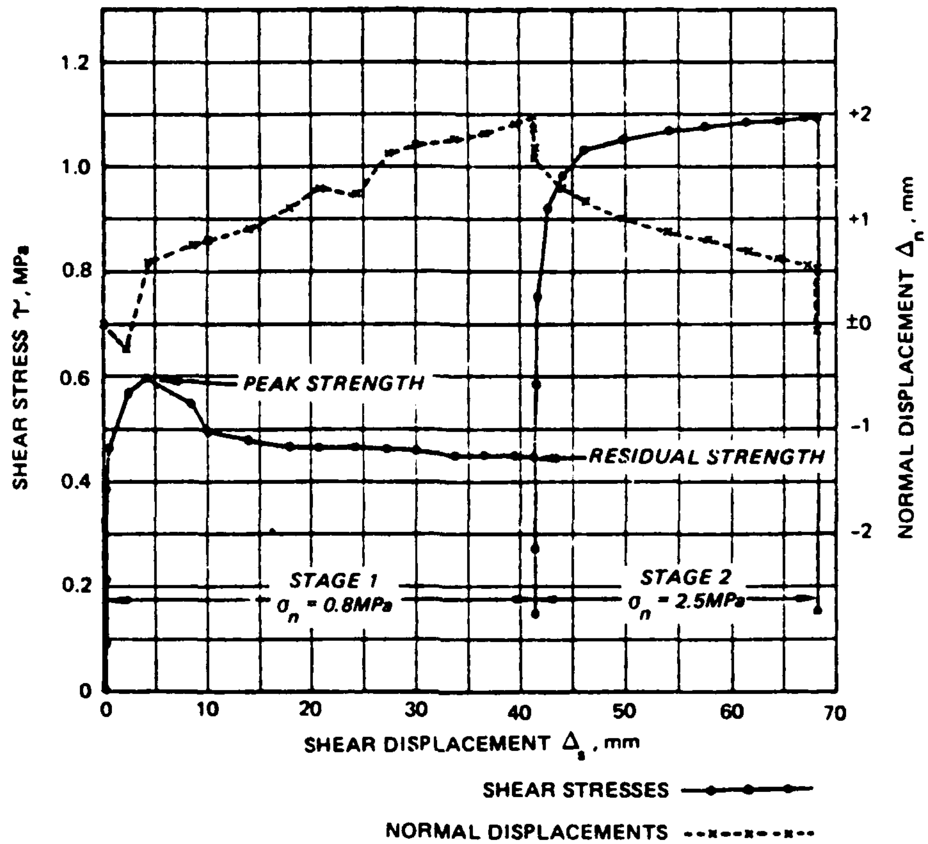


Fig. 5. Shear stress - displacement graphs.

NOTE 11—An independent check on the residual friction angle should be made by testing in the laboratory two prepared flat surfaces of the representative rock. The prepared surfaces should be saw-cut and then ground flat with No. 80 silicon carbile grit.

(f) Having established a residual strength, the normal stress may be increased or reduced (Note 12) and shearing continued to obtain additional residual strength values. The specimen should be reconsolidated under each new normal stress (see paragraph 3.2(d)) and shearing continued according to criteria given in 3.3(c) to 3.3(e).

NOTE 12--The normal load should when possible be applied in increasing rather than decreasing stages. Reversals of shear direction or resetting of the specimen block between normal load stages, sometimes used to allow a greater total shear displacement than would otherwise be possible, are not recommended because the shear surface is likely to be disturbed and subsequent results may be misleading. It is generally advisable, although more expensive, to use a different specimen block

(g) After the test, the block should be inverted, photographed in color, and fully described (see paragraph 5.1). Measurements of the area, roughness, dip, and dip direction of the sheared surface are required, and samples of rock, infilling, and shear debris should be taken for index testing.

4. Calculations

4.1 A consolidation curve (Fig. 4) is plotted during the consolidation stage of testing. The time t_{100} for completion of "primary consolidation" is determined by constructing tangents to the curve as shown. The time to reach peak strength from the start of shear loading should be greater than 6 t_{100} to allow pore pressure dissipation (see Note 10).

4.2 Displacement readings are averaged to obtain values of mean shear and normal displacements Δ_s and Δ_n. Lateral displacements are recorded only to evaluate specimen behavior during the test, although if appreciable they should be taken into account when computing corrected contact area.

4.3 Shear and normal stresses are computed as follows:

$$\text{Shear stress} \quad \tau = \frac{Ps}{A} = \frac{Psa\ \cos\alpha}{A}$$

$$\text{Normal stress} \quad \sigma_n = \frac{Pn}{A} = \frac{Pna + Psa\ \sin\alpha}{A}$$

where Ps = total shear force; Pn = total normal force

Psa = applied shear force; Pna = applied normal force

α = inclination of the applied shear force to the shear plane (if $\alpha = 0$, $\cos\alpha = 1$ and $\sin\alpha = 0$).

A = area of shear surface overlap (corrected to account for shear displacement)

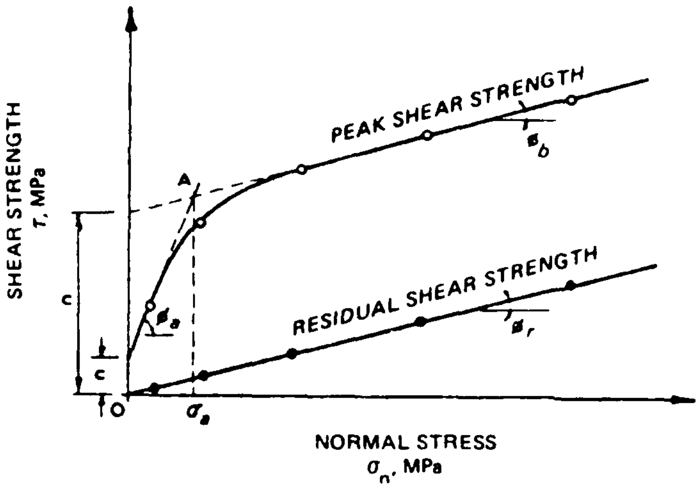
If α is greater than zero, the applied normal force should be reduced after each increase in shear force by an amount Psa sin α in order to maintain the normal stress approximately constant. The applied normal force may be further reduced during the test by an amount

$$\frac{\Delta_s\ (\text{mm}) \cdot Pn}{700}$$

to compensate for area changes.

4.4 For each test specimen graphs of shear stress (or shear force) and normal displacement versus shear displacement are plotted (Fig. 5), annotated to show the nominal normal stress and any changes in normal stress during shearing. Values of peak and residual shear strengths and the normal stresses and shear and normal displacements at which these occur are abstracted from these graphs (Note 2).

4.5 Graphs of peak and residual shear strengths versus normal stress are plotted from the combined results for all test specimens. Shear strength parameters ϕ_a, ϕ_b, ϕ_r, c', and c are abstracted from these graphs as shown in Fig. 6.



ϕ_r = residual friction angle

ϕ_a = apparent friction angle below stress σ_a; point A is a break in the peak shear strength curve resulting from the shearing off of major irregularities on the shear surface. Between points O and A, ϕ_a will vary somewhat; measure at stress level of interest. Note also that $\phi_a = \phi_u + i$, where ϕ_u is the friction angle obtained for smooth surfaces of rock on rock and angle i is the inclination of surface asperities.

Fig. 6. Shear strength - normal stress graph.

ϕ_b = apparent friction angle above stress level σ_a (Point A); note that ϕ_a will usually be equal to or slightly greater than ϕ_r and will vary somewhat with stress level; measure at the stress level of interest^r.

c' = cohesion intercept of peak shear strength curve; it may be zero.

c = apparent cohesion at a stress level corresponding to ϕ_b.

5. Reporting of Results

5.1 The report should include the following:

(a) A diagram, photograph, and detailed description of test equipment and a description of methods used for specimen preparation and testing. (Reference may be made to this "suggested method," stating only departures from the prescribed techniques).

(b) For each specimen, a full geological description of the intact rock, sheared surface, filling, and debris preferably accompanied by relevant index test data (e.g., roughness profiles and Atterberg limits, water content, and grain-size distribution of filling materials).

(c) Photographs of each sheared surface together with diagrams giving the location, dimensions, area, dip, and dip direction and showing the directions of shearing and any peculiarities of the blocks.

(d) For each test block, a set of data tables, a consolidation graph, and graphs of shear stress and normal displacement versus shear displacement (e.g. Figs. 3, 4, and 5). Abstracted values of peak and residual shear strengths should be tabulated with the corresponding values of normal stress and shear and normal displacement.

(e) For the shear strength determination as a whole, graphs and tabulated values of peak and residual shear strengths versus normal stress, together with derived values for the shear strength parameters (e.g. Fig. 6).

SUGGESTED METHOD FOR DETERMINING THE STRENGTH OF A ROCK BOLT ANCHOR (PULL TEST)

(International Society for Rock Mechanics)

1. Scope

1.1 This test is intended to measure the short-term strength of a rock bolt anchor installed under field conditions (Note 1). Strength is measured by a pull test in which bolt head displacement is measured as a function of the applied bolt load to give a load-displacement curve. The test is usually employed for selection of bolts and also for control on the quality of materials and installation methods (Note 2).

NOTE 1—It is essential to test anchors under realistic field conditions. It is, however, permissible to select safe and convenient test locations provided that the rock and the installation methods are identical with those encountered in full-scale utilization of the bolts. If the rock is schistose for example, test holes should be drilled at the same angle to the schistosity as anticipated for bolt utilization. If rock conditions are variable, the rock should be classified and tests conducted in rock of each class.

NOTE 2—The test is intended to measure anchor performance and this is possible only if the bolt, threads, nuts, and other components are stronger than the anchor. In some circumstances it may be desirable to reinforce the bolt or thread for purposes of anchor evaluation. Otherwise, if the bolt is consistently weaker than the anchor, it may be preferable to replace the field test with quality control of bolts and other components in a testing laboratory. Laboratory control testing may also be required as a supplement to field testing for evaluation of components, e.g. for their corrosion resistance, quality of materials, and consistency of dimensions.

1.2 At least five tests are required to evaluate an anchor in a given set of rock and installation conditions. The tests are destructive and should not in general be made on bolts that form part of the actual rock support system.

2. Apparatus

2.1 Equipment for installing the test anchors, including:

(a) Equipment for drilling and cleaning the drillhole, conforming to the manufacturers' specifications for optimum performance of the anchor provided that these are compatible with field conditions (Note 3).

NOTE 3--Manufacturers' specifications for hole dimensions and method of installation should be checked for compatibility with site operational limitations before testing and if compatible should be closely followed in the tests.

(b) Equipment for inspection and measurement of the drillhole, anchors, and bolts, e.g., a lamp, steel tape, internal and external calipers, and equipment for measuring the quantity of grout if used.

(c) Standard rock bolt assemblies as supplied by manufacturers of the bolts including anchors to be tested, grout and materials for grout injection if required, and equipment for installing the bolts in the manner recommended by the manufacturers (Note 3).

2.2 Equipment for applying the bolt load, e.g. as in Fig. 1, including:

(a) A hydraulic jack with hand pump and pressure hose capable of applying a load greater than the strength of both the anchor and the bolt to be tested and with travel of at least 50 mm.

(b) Equipment for transferring the load from the jack to the bolt (Note 4). A spherical seating, bevelled washers, and/or wedges under the jack are required to ensure that the applied load is coaxial with the bolt.

NOTE 4--Some types of anchors must essentially be tensioned during their installation, and these must be tested using a suitable coupling unit and bridging framework to carry load from the jack to the bolt (Fig. 1a). Whenever possible, however, anchors should be tested without pretensioning of the bolt, in which case a center-hole jack installed over the bolt may be used (Fig. 1b). The arrangement shown in Fig. 1a may also be used to test selected anchors in an operational support system at some time after their installation, provided this does not endanger the support as a whole. The percentage of initially applied bolt tension remaining at the time of test may be estimated from the load required to just loosen the faceplate and washers.

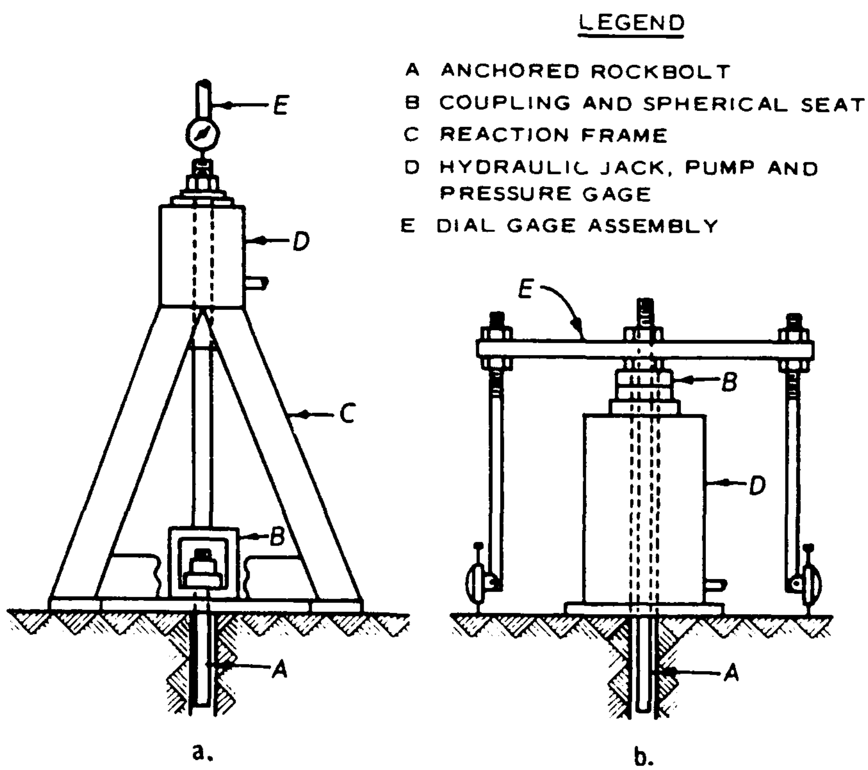


Fig. 1. Rock bolt testing equipment.

2.3 Equipment for measuring load and displacement, including:

(a) A load measuring device, e.g., a load cell or a hydraulic pressure gage connected to the pump and calibrated in load units. Measurement should be accurate to 2 percent of the maximum load reached in the test. The device should include a maximum load indicator.

(b) Equipment for measuring the axial displacement of the bolt head (travel at least 50 mm and accurate to 0.05 mm (Note 5)). For example, a single dial gage measuring directly onto the bolt head may be used; alternatively, the displacement may be obtained as an average from two or three gages spaced equidistant from the bolt as shown in Fig. 1b.

NOTE 5--When testing anchors that in operation are intended to provide a reaction for external loads (e.g. holding anchors for cranes, suspension cables), the test equipment should be designed so that no test reaction forces are applied closer than one bolt length from the anchor drillhole.

3. Procedure

3.1 Site Preparation:

(a) The test site or sites are selected to ensure that rock conditions are representative of those in which the bolts are to operate (see Note 1).

(b) Holes are drilled as specified and at locations convenient for testing (Notes 1 and 3). The rock face surrounding each hole should be firm and flat and the hole should be perpendicular to the face (±5 deg).

(c) Drillholes and anchor materials are inspected before installation to ensure that they conform to specifications. Preliminary data, e.g., the measured dimensions of the drillhole, bolt, and anchor and the type and condition of rock at the test location, are recorded on a data sheet (e.g. Fig. 2).

(d) Bolts are installed in the specified manner (Note 3), recording essential details such as the installation torque (if any) (Note 4) and the date and time of installation.

ROCK ANCHOR TEST RESULT SHEET TEST No.

Date of installation: ____________ Date of test ____________

PROJECT: ____________ Location: ____________

ANCHOR: Type ______ Length: ______ Installation Torque: ______

ROCK: Classification ______ Fracture spacing: ______ Strength ______

BOLT: Diameter: ______ Strength: ______ Length: ______ Untensioned length: ______

HOLE: Diameter: ______ Length: ______ Orientation & roughness: ______

Pump Pressure	Bolt tension	Displacement readings					REMARKS
		Reading	Displacement	Reading	Displacement	Average	

TEST RESULTS: Maximum pull force: ______

Displacement at maximum pull force: ______ Max. displacement in test: ______

Nature of failure or yield: ______

Other remarks: ______

TESTED BY: ______ CHECKED BY: ______

Fig. 2. Rock bolt test data sheet.

3.2 Testing:

(a) The loading equipment is assembled, taking care to ensure that the direction of pull is axial to the bolt, that the equipment sits firmly on the rock, and that no part of the bolt or grout column will interfere with the application or measurement of load during the test (Note 5).

(b) An initial arbitrary load not greater than 5 kN (500 kgf) is applied to take up slack in the equipment. The displacement equipment is assembled and checked (Note 6).

NOTE 6--The displacement measuring system should be securely mounted and dial gages should be located on firm flat rock; glass or metal plates can, if necessary, be cemented to the rock to provide smooth measuring surfaces perpendicular to the bolt. All measuring equipment must be checked and calibrated at regular intervals to ensure that the standards of accuracy required by this "suggested method" are maintained.

(c) The anchor is tested by increasing the load until a total displacement greater than 40 mm has been recorded, or until the bolt yields or fractures if this occurs first.

(d) Readings of load and displacement are taken at increments of approximately 5-kN (500-kgf) load or 5-mm displacement, whichever occurs first. The rate of load application should be in the range 5-10 kN/min. Readings are taken only after both load and displacement have stabilized. The times required for stabilization should be recorded.

4. Calculations

4.1 Total displacement values are computed as the test progresses by subtracting initial readings from the incremental readings, taking averages if more than one gage is used.

4.2 The test data are plotted graphically as shown in Fig. 3. Anchor strength, defined as the maximum load reached in the test provided that the bolt itself does not yield or fail, is recorded on this graph. If the bolt yields or fails, the load 'X' at which this occurs is recorded, and the anchor strength is specified as "unknown but greater than 'X'".

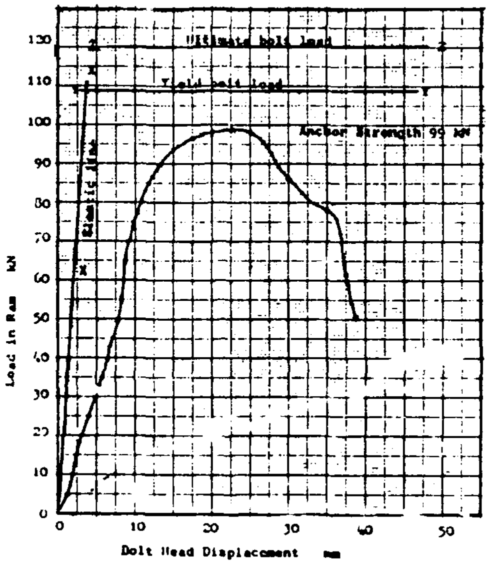


Fig. 3. Example of anchor test results graph.

4.3 The elastic elongation of the bolt at a given applied load may be calculated as

$$\text{Elongation at load P is equal to } \frac{P \cdot L}{A \cdot E}$$

where L is the tensioned ungrouted length of bolt + 1/3 the grouted length + length of extension bar used; A is the cross-sectional area of the bolt; and E is the modulus of elasticity of the bolt steel.

A straight line X-X is constructed to pass through this point and the origin of the load-displacement graph (Fig. 3). Straight lines Y-Y and Z-Z are constructed at the specified yield and ultimate loads of the bolt. Comparison of the actual test curve with these three lines allows independent assessment of anchor and bolt behavior.

4.4 For the evaluation of grouted anchors, the results of several tests should be abstracted and presented graphically to show the influence of grout cure time and bonded length on anchor strength (e.g. Fig. 4).

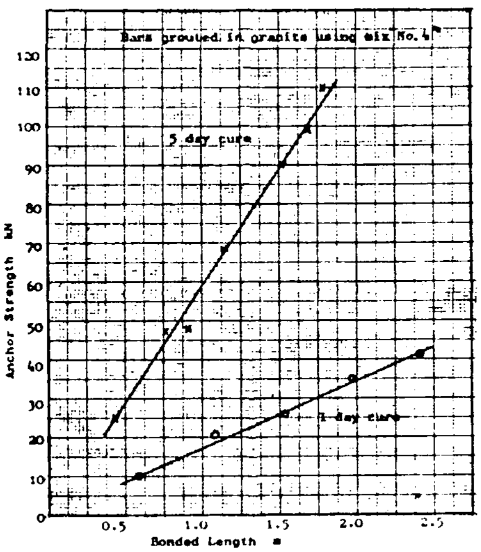


Fig. 4. Graph showing influence of bond length and cure time on the strength of anchors.

5. Reporting of Results

5.1 The report should include the data sheets and graphs illustrated in Figs. 2-4 together with full details of:

(a) Rock in which the anchors were tested.

(b) The anchors and associated equipment.

(c) The drillholes, including length, diameter, method of drilling, straightness, cleanness and dryness, and orientation.

(d) The method and time of installation.

(e) The method and time of testing.

(f) The nature of failure and other observations pertinent to the test results.

5.2 If required, the report may also compare performance of the anchors tested with an arbitrary acceptable performance established by previous extensive testing. Anchor strength, total displacements, and displacement per increment of load should be considered when making this comparison.

SUGGESTED METHOD FOR DEFORMABILITY AND STRENGTH DETERMINATION USING AN IN SITU UNIAXIAL COMPRESSIVE TEST

1. Scope

1.1 This method of test is intended to measure the strength and deformability of large in situ specimens of weak rock such as coal. The test results take into account the effect of both intact material behavior and the behavior of discontinuities contained within the specimen block.

1.2 Since the strength of rock is dependent on the size of the test specimen, it is necessary to test specimens by increasing size until an asymptotically constant strength value is found (Note 1). This value is taken to represent the strength of the rock mass. It can, for example, be applied to the design of mine pillars provided that the constraining effect of the roof and floor is taken into consideration.

NOTE 1—Bieniawski, Z. T. and Van Heerden, W. L., "The Significance of Large-Scale In Situ Tests," Int. J. Rock Mech. Min. Sci., Vol 1, 1975.

2. Apparatus

2.1 Preparation equipment, including:

(a) Equipment for cutting rectangular specimen blocks from existing underground mine pillars or exposed faces, e.g., a coal cutting machine, pneumatic chisel, and other hand tools. No explosives are permitted.

2.2 A loading system consisting of:

(a) Hydraulic jacks or flat jacks to apply a uniformly distributed load to the complete upper face of the specimen. The loading system should be of sufficient capacity and travel to load the specimen to failure.

(b) A hydraulic pumping system to supply oil at the required pressure to the jacks, the pressure being controlled to give a constant rate of displacement or strain rather than a constant rate of stress increase (Note 2).

NOTE 2—Experience has shown that deformation-controlled loading is preferable to stress-controlled loading because it results in a more stable and thus safer test. One way to achieve uniform deformation of the specimen is to use a separate pump for each jack and to set the oil delivery rate of each pump to the same value. Standard diesel fuel injection pumps have been found suitable and are capable of supplying pressures up to 100 MPa. The delivery rate of these pumps can be set very accurately.

2.3 Equipment to measure applied load and strain in the specimen, including:

(a) Load measuring equipment, e.g., electric, hydraulic, or mechanical load cells, to permit the applied load to be measured with an accuracy better than ±5 percent of the maximum in the test.

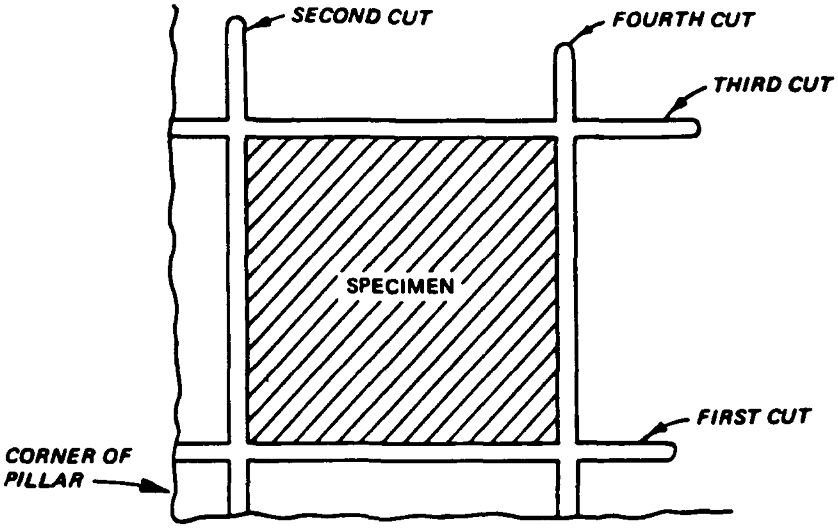
(b) Dial gage or similar displacement measuring devices with robust fittings to enable the instruments to be mounted so that the strain in the central third of each specimen face is measured with an accuracy better than $\pm 10^{-5}$. Strain is to be measured in the direction of applied load, also in a perpendicular direction if Poisson's ratio values are to be determined.

2.4 Equipment to calibrate the loading and displacement measuring systems, the accuracy of calibration to be better than the accuracies of test measurement specified above.

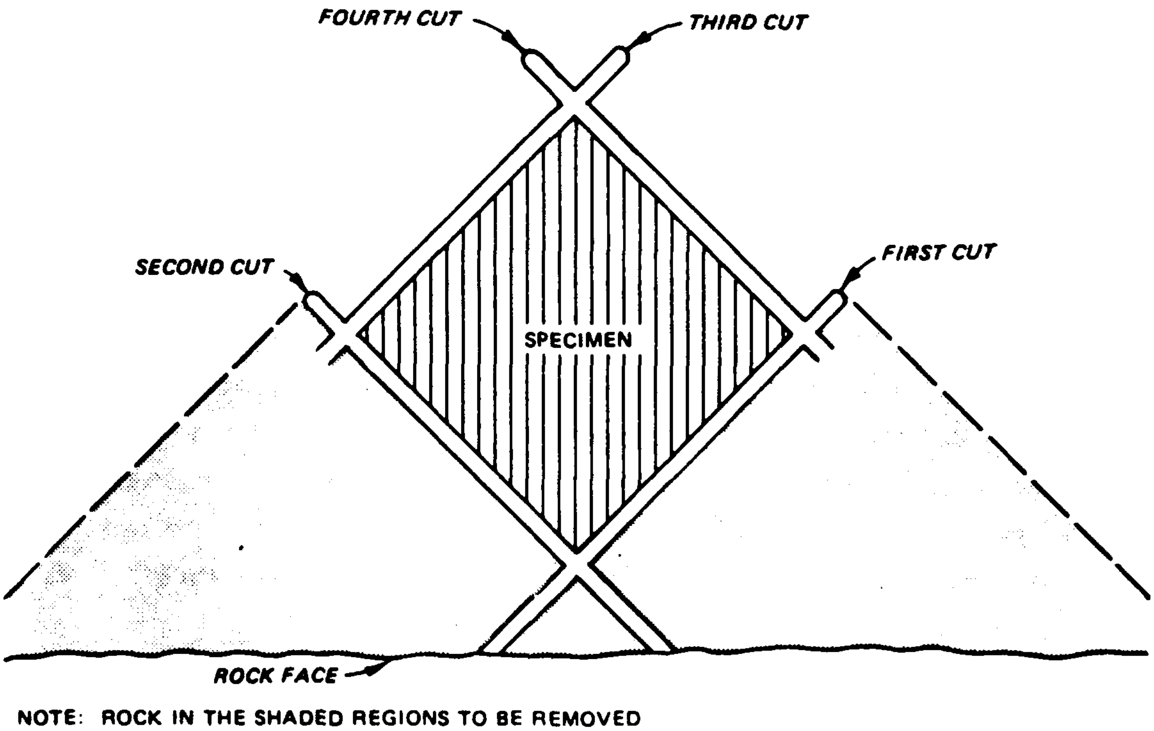
3. Procedure

3.1 Preparation:

(a) Specimens of the required dimensions (Note 3) are cut either from the corners of existing pillars or from exposed rock faces (Fig. 1). Loose and damaged rock is first removed. Vertical cuts are then made, e.g. as shown in Fig. 1, to form the vertical faces of



a. SEQUENCE OF VERTICAL CUTS TO SEPARATE A SPECIMEN FROM THE CORNER OF A PILLAR



b. SEQUENCE OF VERTICAL CUTS TO SEPARATE A SPECIMEN FROM A ROCK FACE

Fig. 1. Vertical cuts to separate specimen from the corner of a pillar and from a rock face.

the specimen. A horizontal cut is made to form the top face of the specimen. Loose rock is removed and the specimen trimmed to final size using hand tools.

NOTE 3—Specimen dimensions cannot be specified because these depend very much on the rock properties, e.g., the thickness of strata and the ease with which specimens can be prepared. It is recommended that a number of tests should be done with a specimen size of about 0.5 m and that the size of subsequent specimens should be increased until an asymptotically constant strength value is reached.

(b) The specimen is cleaned and inspected, recording in detail the geological structure of the block and of the reaction faces above and below. Specimen geometry, including the geometry of defects in the block, should be measured and recorded with an accuracy better than 5 mm. Photographs and drawings should be prepared to illustrate both geological and geometric characteristics.

(c) A concrete block, suitably reinforced, is cast to cover the top face of the specimen (Fig. 2). The thickness of this block should be sufficient to give adequate strength under the full applied load. The top face of the block should be flat to within ±5 mm and parallel to within ±5 deg with the basal plane of the block.

(d) Rock is removed from above the specimen to make space for the loading jacks, the rock being cut back to a stratum of sufficient strength to provide safe reaction. Generally, a concrete reaction pad must be cast to distribute the load on the roof and to prevent undue deformation and movement of the jacks during the test (Note 4). The lower face of the reaction block should be flat to within ±5 mm and parallel to within ±5 deg with the upper face of the specimen block. All concrete should be left to harden for a period of not less than 7 days.

NOTE 4--If a suitably designed concrete cap to the specimen is not employed, the corners and sides of the specimen will often fail before the central portion. The corner jacks will then cease to operate, and the test results will be suspect. The concrete cap should if possible be designed to ensure that the stress distributions in the top and bottom thirds of the specimen are nearly identical.

(e) The loading jacks and pumps are installed and checked to ensure that they operate as intended. Load and displacement measuring equipment is installed and checked. All measuring instruments should be calibrated both before and after each test series.

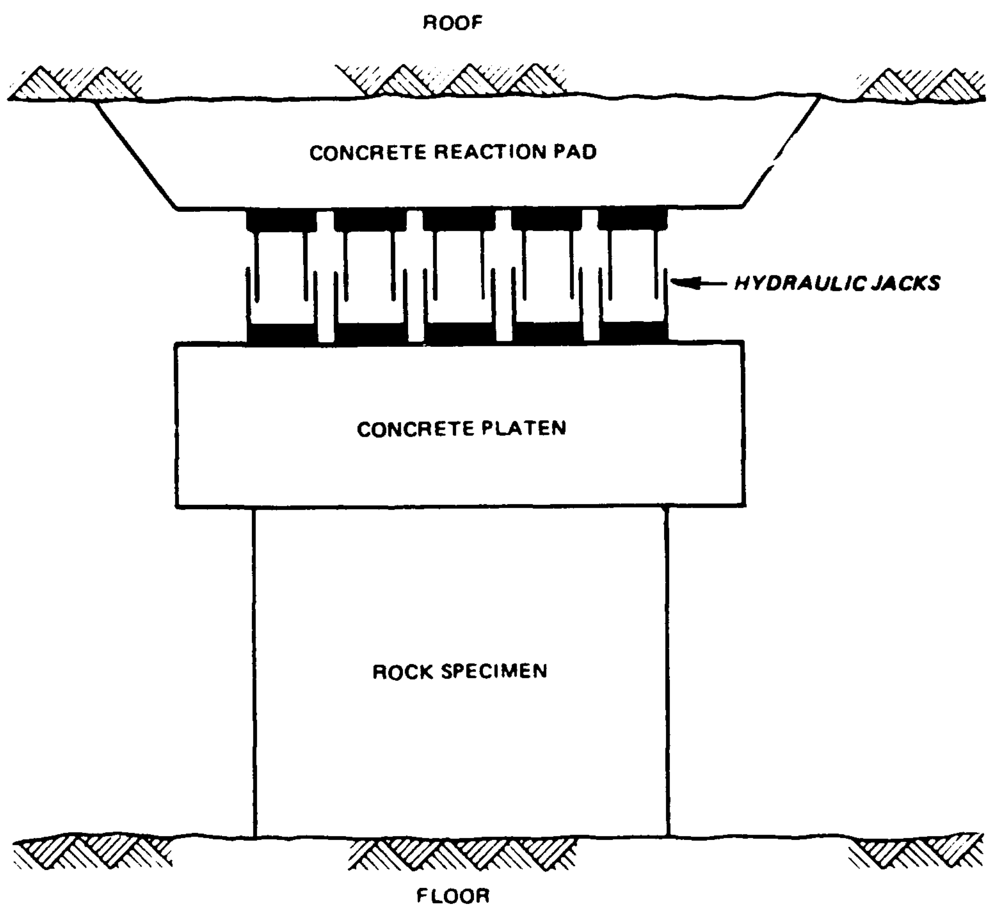


Fig. 2. Testing arrangement.

3.2 Testing:

(a) An initial load of approximately one-tenth of the estimated full test load is applied, and the jacks are checked to ensure that each is in firm contact with the specimen block. Displacement measuring equipment is again checked to ensure that it is rigidly mounted and is functioning correctly. Zero readings of load and displacement are taken.

(b) The specimen load is then increased by applying the same slow and constant oil delivery to each jack. The rate of specimen strain should be such that a displacement rate of between 5 and 15 mm per hour is recorded at each of the four faces of the specimen block.

(c) Readings of applied load and displacements are recorded at intervals such that the load-displacement or stress-strain curve can be adequately defined. There should be not less than ten points on this curve, evenly spaced from zero to the failure load.

(d) Unless otherwise specified, the test is to be terminated when the specimen fails. Specimen failure is indicated by a drop in hydraulic pressure to less than one-half the maximum applied, or by disintegration of the specimen to an extent that the loading system becomes inoperative or the test dangerous to continue. The mode of specimen failure is recorded, and a sketch is made of all failure cracks.

4. Calculations

4.1 The uniaxial compressive strength of the specimen shall be calculated by dividing the maximum load carried by the specimen during the test by the original cross-sectional area of the specimen.

4.2 Young's modulus for the specimen shall, unless otherwise specified, be calculated as the tangent modulus E_{t50} at one-half the uniaxial compressive strength. This modulus is found by drawing a tangent to the stress-strain curve at 50 percent maximum load, the gradient of this tangent being measured as E_{t50}. The construction and calculations used in deriving this and any other modulus values should be shown on the stress-strain curve.

4.3 If a number of specimens of different shape and/or size are tested, the trends in strength values due to shape and size effects should be plotted graphically, e.g., as shown in Fig. 3.

5. Reporting of Results

5.1 The report should include the following information:

(a) A diagram showing details of the locations of specimens tested, the specimen numbering system used, and the situation of each specimen with respect to the geology and geometry of the site.

(b) Photographs, drawings, and tabulations giving full details of the geological and geometrical characteristics of each specimen, preferably including index test data to characterize the rock. Particular attention should be given to a detailed description of the pattern of joints, bedding planes, and other discontinuities in the specimen block.

(c) A description, with diagrams, of the test equipment and method used. Reference may be made to this "suggested method," noting only the departures from recommended procedures.

(d) Tabulated test results, including recorded values of load and displacements together with all derived data, calibration results, and details of all corrections applied.

(e) Graphs showing load versus displacement or stress versus strain, including points representing all recorded data and a curve fitted to these points. The uniaxial compressive strength value should be shown, together with all constructions used in determining Young's modulus and other elastic parameters. The mode of specimen failure should be shown diagrammatically and described.

(f) Summary tables and graphs giving the values of uniaxial compressive strength and Young's modulus, and showing how these values vary as a function of specimen shape and size and the character of the rock tested.

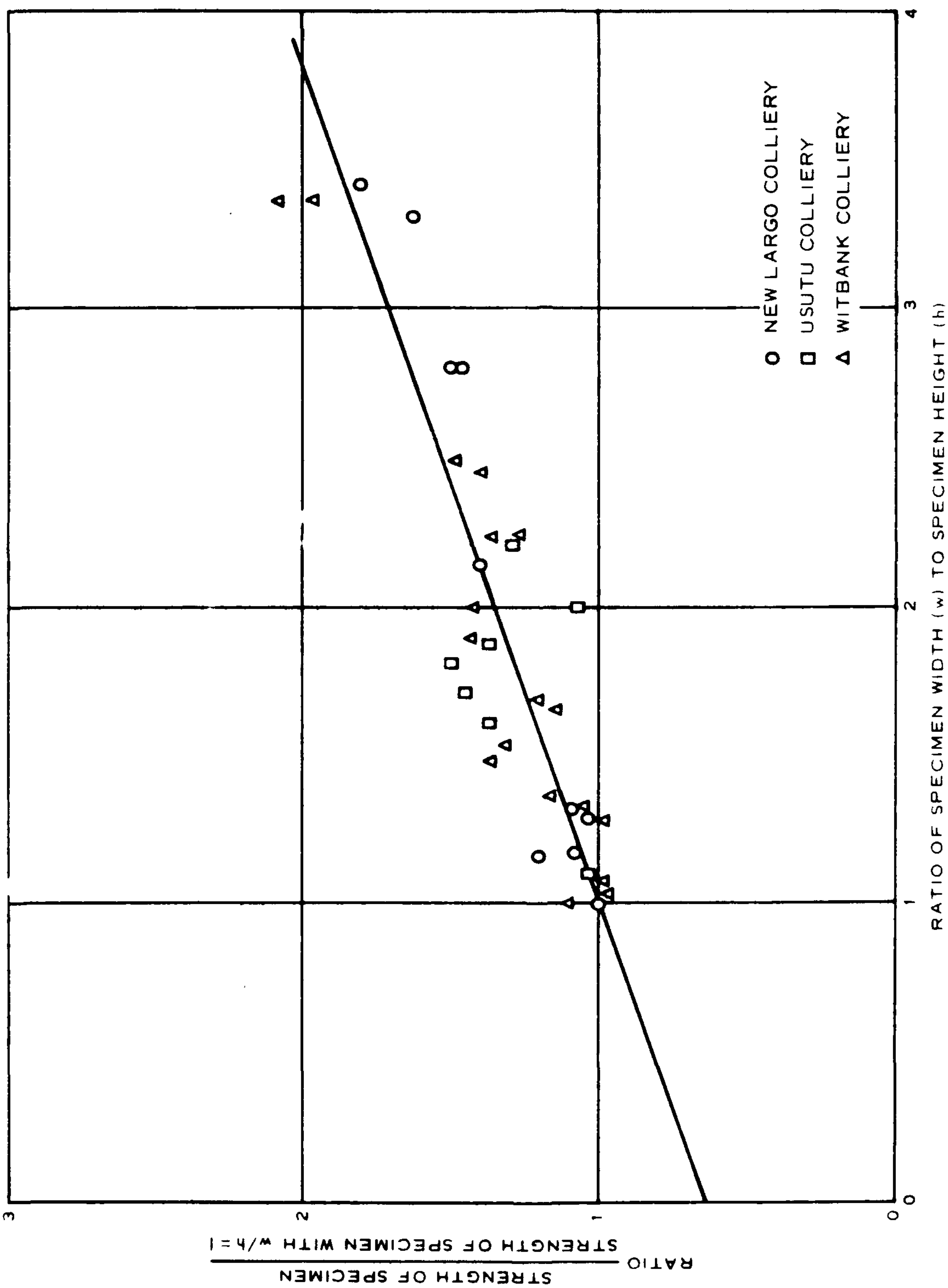


Fig. 3. Example showing the representation of strength data in dimensionless form.

RTH 325-89

Suggested Method for Determining Point Load Strength

SCOPE

1.(a) The Point Load Strength test is intended as an index test for the strength classification of rock materials. It may also be used to predict other strength parameters with which it is correlated, for example uniaxial tensile and compressive strength.[1]*

(b) The test measures the Point Load Strength Index ($I_{s(50)}$) of rock specimens, and their Strength Anisotropy Index ($I_{a(50)}$) which is the ratio of Point Load Strengths in directions which give the greatest and least values.

(c) Rock specimens in the form of either core (the *diametral* and *axial* tests), cut blocks (the *block* test), or irregular lumps (the *irregular lump* test) are broken by application of concentrated load through a pair of spherically truncated, conical platens.[2] Little or no specimen preparation is needed.

(d) The test can be performed with portable equipment or using a laboratory testing machine, and so may be conducted either in the field or the laboratory.

APPARATUS

2. The testing machine (Fig. 1) consists of a loading system (for the portable version typically comprising a loading frame, pump, ram and platens), a system for measuring the load P required to break the specimen, and a system for measuring the distance D between the two platen contact points (but see 5(e) below).

Loading system

3.(a) The loading system should have a platen-to-platen clearance that allows testing of rock specimens in the required size range. Typically this range is 15–100 mm so that an adjustable clearance is needed to accommodate both small and large specimens.

(b) The loading capacity should be sufficient to break the largest and strongest specimens to be tested.[3]

(c) The test machine should be designed and constructed so that it does not permanently distort during repeated applications of the maximum test load, and so that the platens remain co-axial within ± 0.2 mm throughout the testing. No spherical seat or other non-rigid component is permitted in the loading system. Loading system rigidity is essential to avoid problems of slippage when specimens of irregular geometry are tested.

(d) Spherically-truncated, conical platens of the standard geometry shown in Fig. 2 are to be used. The 60° cone and 5 mm radius spherical platen tip should meet tangentially.[4] The platens should be of hard material such as tungsten carbide or hardened steel so that they remain undamaged during testing.

Fig. 1. Photograph of portable point load test machine.

Load measuring system

4.(a) The load measuring system, for example a load cell or a hydraulic pressure gauge or transducer connected to the ram, should permit determination of the failure load P required to break the specimen and should conform to the requirements (b) through (d) below.

(b) Measurements of P should be to an accuracy of $\pm 5\% P$ or better, irrespective of the size and strength of specimen that is tested.[3,5]

(c) The system is to be resistant to hydraulic shock and vibration so that the accuracy of readings is not adversely affected by repeated testing.

(d) Failure is often sudden and a maximum load indicating device is essential so that the failure load is retained and can be recorded after each test.

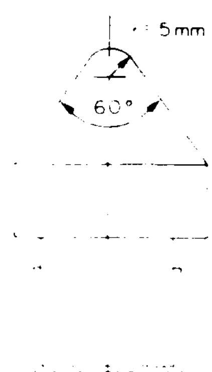


Fig. 2. Platen shape and tip radius.

* Superscript numbers refer to Notes at the end of the text.

Distance measuring system

5.(a) The distance measuring system, for example a direct reading scale or displacement transducer, is to permit measurement of the distance D between specimen-platen contact points and should conform with requirements (b) through (d) below.[6]

(b) Measurements of D should be to an accuracy of $\pm 2\%$ D or better irrespective of the size of specimen tested.

(c) The system is to be resistant to hydraulic shock and vibration so that the accuracy of readings is not adversely affected by repeated testing.

(d) The measuring system should allow a check of the "zero displacement" value when the two platens are in contact, and should preferably include a zero adjustment.

(e) An instrument such as calipers or a steel rule is required, to measure the width W of specimens for all but the diametral test.

PROCEDURE

Specimen selection and preparation

6.(a) A test sample is defined as a set of rock specimens of similar strength for which a single Point Load Strength value is to be determined.

(b) The test sample of rock core or fragments is to contain sufficient specimens conforming with the size and shape requirements for diametral, axial, block or irregular lump testing as specified below.[7]

(c) For routine testing and classification, specimens should be tested either fully water-saturated or at their natural water content.[5]

Calibration

7. The test equipment should be periodically calibrated using an independently certified load cell and set of displacement blocks, checking the P and D readings over the full range of loads and displacements pertinent to testing.

The diametral test[2]

8.(a) Core specimens with length/diameter ratio greater than 1.0 are suitable for diametral testing.

(b) There should preferably be at least 10 tests per sample, more if the sample is heterogeneous or anisotropic.[7]

(c) The specimen is inserted in the test machine and the platens closed to make contact along a core diameter, ensuring that the distance L between the contact points and the nearest free end is at least 0.5 times the core diameter (Fig. 3a).

(d) The distance D is recorded $\pm 2\%$.[6]

(e) The load is steadily increased such that failure occurs within 10–60 sec, and the failure load P is recorded. The test should be rejected as invalid if the

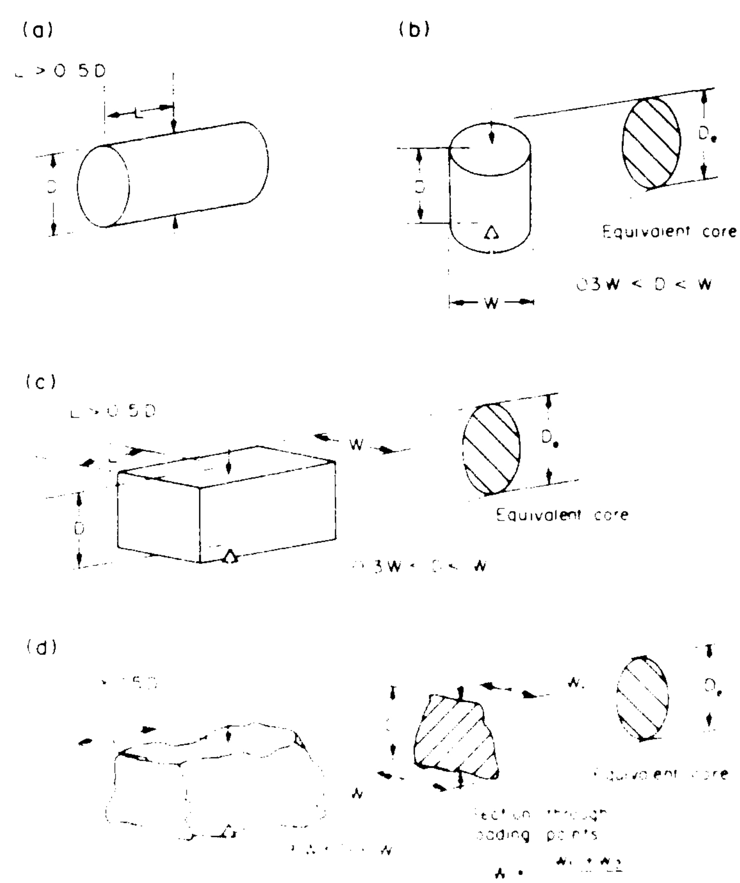


Fig. 3. Specimen shape requirements for (a) the diametral test, (b) the axial test, (c) the block test and (d) the irregular lump test

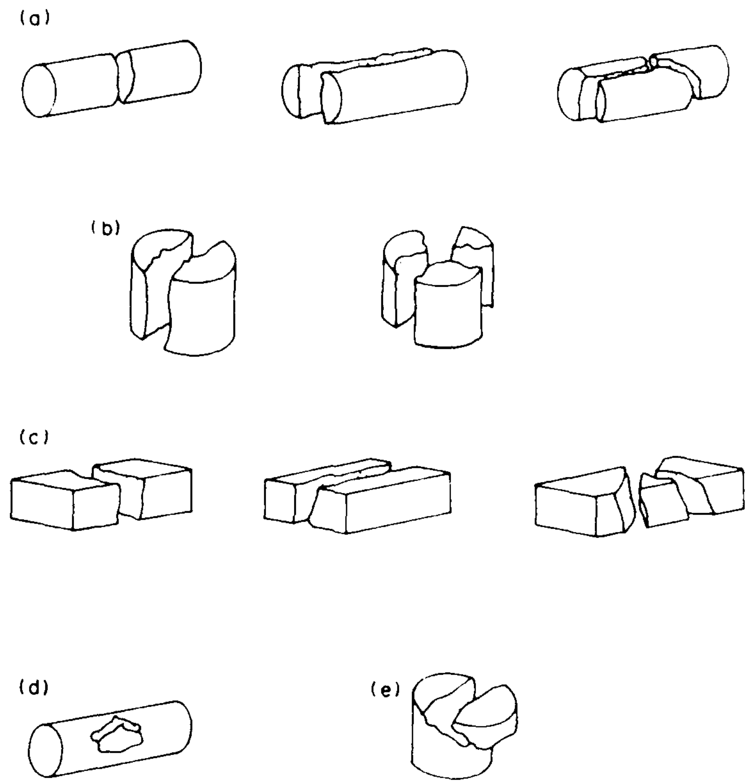


Fig. 4. Typical modes of failure for valid and invalid tests. (a) Valid diametral tests, (b) valid axial tests, (c) valid block tests, (d) invalid core test, (e) invalid axial test.

fracture surface passes through only one loading point (Fig. 4d).

(f) The procedure (c) through (e) above is repeated for the remaining specimens in the sample.

The axial test[2]

9.(a) Core specimens with length/diameter ratio of 0.3–1.0 are suitable for axial testing (Fig. 3b). Long pieces of core can be tested diametrally to produce suitable lengths for subsequent axial testing (provided that they are not weakend by this initial testing); alternatively, suitable specimens can be obtained by saw-cutting or chisel-splitting.

(b) There should preferably be at least 10 tests per sample, more if the sample is heterogeneous or anisotropic.[7]

(c) The specimen is inserted in the test machine and the platens closed to make contact along a line perpendicular to the core end faces (in the case of isotropic rock, the core axis, but see paragraph 11 and Fig. 5).

(d) The distance D between platen contact points is recorded $\pm 2\%$.[6] The specimen width W perpendicular to the loading direction is recorded $\pm 5\%$.[9]

(e) The load is steadily increased such that failure occurs within 10–60 sec, and the failure load P is recorded. The test should be rejected as invalid if the fracture surface passes through only one loading point (Fig. 4e).

(f) The procedures (c) through (e) above are repeated for the remaining tests in the sample.

The block and irregular lump tests

10.(a) Rock blocks or lumps of size 50 ± 35 mm and of the shape shown in Fig. 3(c) and (d) are suitable for the block and the irregular lump tests. The ratio D/W should be between 0.3 and 1.0, preferably close to 1.0.

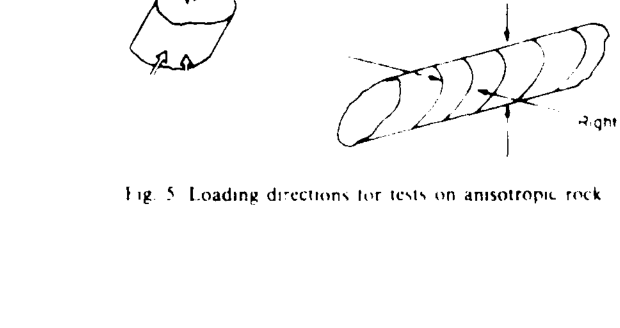


Fig. 5. Loading directions for tests on anisotropic rock

The distance L (Fig. 3c and d) should be at least $0.5W$. Specimens of this size and shape may be selected if available or may be prepared by trimming larger pieces by saw-or chisel-cutting.

(b) There should preferably be at least 10 tests per sample, more if the rock is heterogeneous or anisotropic.

(c) The specimen is inserted in the testing machine and the platens closed to make contact with the smallest dimension of the lump or block, away from edges and corners (Fig. 3c and d).

(d) The distance D between platen contact points is recorded $\pm 2\%$. The smallest specimen width W perpendicular to the loading direction is recorded $\pm 5\%$. If the sides are not parallel, then W is calculated as $(W_1 + W_2)/2$ as shown in Fig. 3d.[6] This smallest width W is used irrespective of the actual mode of failure (Figs 3 and 4).

(e) The load is steadily increased such that failure occurs within 10–60 sec, and the failure load P is recorded. The test should be rejected as invalid if the fracture surface passes through only one loading point (see examples for other shapes in Fig. 4d or e).

(f) The procedure (c) through (e) above is repeated for the remaining tests in the sample.

Anisotropic rock

11. (a) When a rock sample is shaly, bedded, schistose or otherwise observably anisotropic it should be tested in directions which give the greatest and least strength values, which are in general parallel and normal to the planes of anisotropy.

(b) If the sample consists of core drilled through the weakness planes, a set of diametral tests may be completed first, spaced at intervals which will yield pieces which can then be tested axially.

(c) Best results are obtained when the core axis is perpendicular to the planes of weakness, so that when possible the core should be drilled in this direction. The angle between the core axis and the normal to the weakness planes should preferably not exceed 30°.

(d) For measurement of the I_s value in the directions of least strength, care should be taken to ensure that load is applied along a single weakness plane. Similarly when testing for the I_s value in the direction of greatest strength, care should be taken to ensure that the load is applied perpendicularly to the weakness planes (Fig. 5).

(e) If the sample consists of blocks or irregular lumps, it should be tested as two sub-samples, with load applied firstly perpendicular to, then along the observable planes of weakness.[10] Again, the required minimum strength value is obtained when the platens make contact along a single plane of weakness.

CALCULATIONS

Uncorrected point load strength

12. The Uncorrected Point Load Strength I_s is calculated as P/D_e^2 where D_e, the "equivalent core diameter", is given by:

$D_e^2 = D^2$ for diametral tests;

$= 4A/\pi$ for axial, block and lump tests;

and

$A = WD$ = minimum cross sectional area of a plane through the platen contact points.[6]

Size correction

13.(a) I_s varies as a function of D in the diametral test, and as a function of D_e in axial, block and irregular lump tests, so that a size correction must be applied to obtain a unique Point Load Strength value for the rock sample, and one that can be used for purposes of rock strength classification.

(b) The size-corrected Point Load Strength Index $I_{s(50)}$ of a rock specimen or sample is defined as the value of I_s that would have been measured by a diametral test with $D = 50$ mm.

(c) The most reliable method of obtaining $I_{s(50)}$, preferred when a precise rock classification is essential, is to conduct diametral tests at or close to $D = 50$ mm. Size correction is then either unnecessary ($D = 50$ mm) or introduces a minimum of error. The latter is the case, for example, for diametral tests on NX core, $D = 54$ mm. This procedure is not mandatory. Most point load strength testing is in fact done using other sizes or shapes of specimen. In such cases, the size correction (d) or (e) below must be applied.

(d) The most reliable method of size correction is to test the sample over a range of D or D_e values and to plot graphically the relation between P and D_e^2. If a log–log plot is used the relation is generally a straight line (Fig. 6). Points that deviate substantially from the straight line may be disregarded (although they should not be deleted). The value of P_{50} corresponding to $D_e^2 = 2500$ mm^2 ($D_e = 50$ mm) can then be obtained by interpolation, if

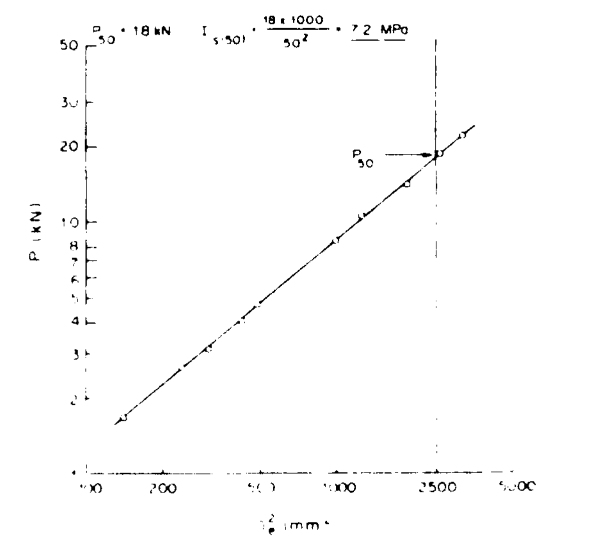


Fig. 6. Procedure for graphical determination of $I_{s(50)}$ from a set of results at D_e values other than 50 mm

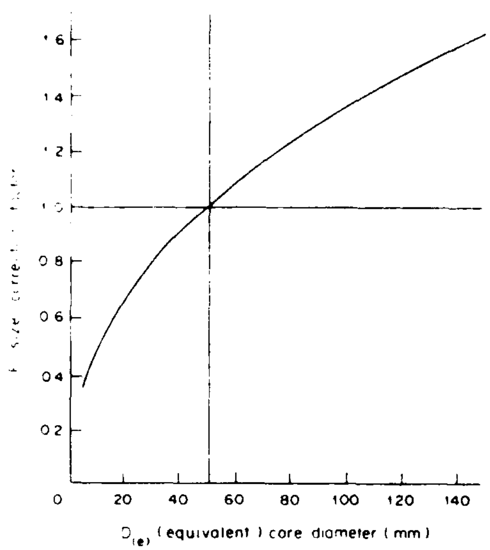


Fig. 7. Size correction factor chart.

necessary by extrapolation, and the size-corrected Point Load Strength Index calculated as $P_{50}/50^2$.

(e) When neither (c) nor (d) is practical, for example when testing single sized core at a diameter other than 50 mm or if only a few small pieces are available, size correction may be accomplished by using the formula:

$$I_{s(50)} = F \times I_s$$

The "Size Correction Factor F" can be obtained from the chart in Fig. 7,[11] or from the expression:

$$F = (D_e/50)^{0.45}$$

For tests near the standard 50 mm size, very little error is introduced by using the approximate expression:

$$F = \sqrt{(D_e/50)}$$

(f) The size correction procedures specified in this paragraph have been found to be applicable irrespective of the degree of anisotropy I_a and the direction of loading with respect to planes of weakness, a result that greatly enhances the usefulness of this test.

Mean value calculation

14.(a) Mean values of $I_{s(50)}$ as defined in (b) below are to be used when classifying samples with regard to their Point Load Strength and Point Load Strength Anisotropy Indices.

(b) The mean value of $I_{s(50)}$ is to be calculated by deleting the two highest and lowest values from the 10 or more valid tests, and calculating the mean of the remaining values. If significantly fewer specimens are tested, only the highest and lowest values are to be deleted and the mean calculated from those remaining.[12]

Point load strength anisotropy index

15. The Strength Anistropy Index $I_{a(50)}$ is defined as the ratio of mean $I_{s(50)}$ values measured perpendicular and parallel to planes of weakness, i.e. the ratio of greatest to least Point Load Strength Indices $I_{a(50)}$ assumes values close to 1.0 for quasi-isotropic rocks and higher values when the rock is anisotropic.

REPORTING OF RESULTS

16. Results for diametral tests, axial tests, block tests and irregular lump tests, and for tests perpendicular and parallel to planes of weakness should be tabulated separately (see typical results form, Fig. 8). The report should contain calibration data for the test machine and at least the following information for each sample tested:

(a) The sample number, source location and rock type, and the nature and *in situ* orientation of any planes of anistropy or weakness.

(b) Information on the water content of the rock at the time of testing.

(c) Information on which specimens were loaded parallel (//), perpendicular (⊥), or at unknown or random directions with respect to planes of weakness.

(d) A tabulation of the values of P, D, (W, D_e^2 and D_e if required), I_s, (F if required) and $I_{s(50)}$ for each specimen in the sample.

(e) For all isotropic samples, a summary tabulation of mean $I_{s(50)}$ values.

(f) For all anisotropic samples, a summary tabulation of mean $I_{s(50)}$ values for sub-samples tested perpendicular and parallel to the planes of weakness, and of the corresponding $I_{a(50)}$ values.

NOTES

1. When first introduced, the point load strength test was used mainly to predict uniaxial compressive strength which was then the established test for general-purpose rock strength classification. Point load strength now often replaces uniaxial compressive strength in this role since when properly conducted it is as reliable and much quicker to measure. $I_{s(50)}$ should be used directly for rock classification, since correlations with uniaxial compressive strength are only approximate. On average, uniaxial compressive strength is 20–25 times point load strength, as shown in Fig. 9. However, in tests on many different rock types the ratio can vary between 15 and 50 especially for anisotropic rocks, so that errors of up to 100% are possible in using an arbitrary ratio value to predict compressive strength from point load strength. The point load strength test is a form of "indirect tensile" test, but this is largely irrelevant to its primary role in rock classification and strength characterization. $I_{s(50)}$ is approximately 0.80 times the uniaxial tensile or Brazilian tensile strength.

2. Of the four alternative forms of this test, the diametral test and the axial test with saw-cut faces are the most accurate if performed near the standard 50 mm size, and are preferred for strength classification when core is available. Axial test specimens with saw-cut faces can easily be obtained from large block samples by coring in the laboratory. Specimens in this form are

Sample Details | Point Load Test | Date 17/11/83

1 block sample from Gamblethorpe Opencast site.

Fine grained pale grey Coal Measures sandstone with numerous coaly streaks along horizontal bedding planes.

Specimens 1-6 chisel cut blocks, air-dried 2 weeks;
7-10 sawn blocks, air-dried 2 weeks;
11-15 cores, air-dried 2 weeks;
16-20 cores, air-dried 2 weeks;
- tested in laboratory.

No.	Type	W (mm)	D (mm)	P (kN)	D_e^2 (mm^2)	D_e (mm)	I_s	F	$I_{s(50)}$
1	i ⊥	30.4	17.2	2.687	666	25.8	4.03	0.75	~~3.03~~
2	i ⊥	16	8	0.977	163	12.8	5.99	0.54	3.24
3	i ⊥	19.7	15.6	1.962	391	19.8	5.02	0.66	3.31
4	i ⊥	35.8	18.1	3.641	825	28.7	4.41	0.765	3.46
5	i ⊥	42.5	29	6.119	1569	39.6	3.90	0.875	3.49
6	i ⊥	42	35	7.391	1872	43.3	3.95	0.935	~~3.69~~
7	b ⊥	44	21	4.600	1176	34.3	3.91	0.84	3.29
8	b ⊥	40	30	5.940	1528	39.1	3.88	0.89	3.46
9	b ⊥	19.5	15	2.040	372	19.3	5.48	0.655	~~3.59~~
10	b ⊥	33	16	2.87	672	25.9	4.27	0.75	~~3.20~~
11	d //	-	49.93	5.107	-	-	-	-	2.05
12	d //	-	49.88	4.615	-	-	-	-	1.85
13	d //	-	49.82	5.682	-	-	-	-	~~2.29~~
14	d //	-	49.82	4.139	-	-	-	-	~~1.67~~
15	d //	-	49.86	4.546	-	-	-	-	1.83
16	d //	-	25.23	1.837	-	-	2.89	0.74	2.14
17	d //	-	25.00	1.891	-	-	3.02	0.735	2.22
18	d //	-	25.07	2.118	-	-	3.37	0.735	~~2.48~~
19	d //	-	25.06	1.454	-	-	2.32	0.735	~~1.70~~
20	d //	-	25.04	1.540			2.46	0.735	1.81

d = diametral;
a = axial;
b = block;
i = irregular lump test;
⊥ = perpendicular;
// = parallel to planes of weakness.

Mean $I_{s(50)}$ ⊥	3.38
Mean $I_{s(50)}$ //	1.98
$I_{a(50)}$	1.71

Fig. 8. Typical results form.

particularly suitable when the rock is anisotropic and the direction of weakness planes must be noted.

3. Loads of up to 50 kN are commonly required for the larger hard rock specimens. The maximum specimen size that can be tested by a given machine is determined by the machine's load capacity, and the smallest by the machine's load and distance measuring sensitivity. Tests on specimens smaller than $D = 25$ mm require particular precautions to ensure that the measuring sensitivity is sufficient. The range of required test loads should be estimated before testing, from approximate assumed strength values, to ensure that the load capacity and sensitivity of the equipment are adequate. It may be necessary to change the load measuring gauge or load cell, or to test smaller or larger specimens to conform with the capacity of available equipment or with the accuracy specifications for this test.

4. The conical platen design is intended to give standardized penetration of softer specimens. When testing is confined to hard rocks and small (less than 2 mm) penetrations the conical design is unimportant provided that the tip radius remains at the standard 5 mm. For such testing the platen can be manufactured by embedding a hard steel or tungsten carbide ball in a

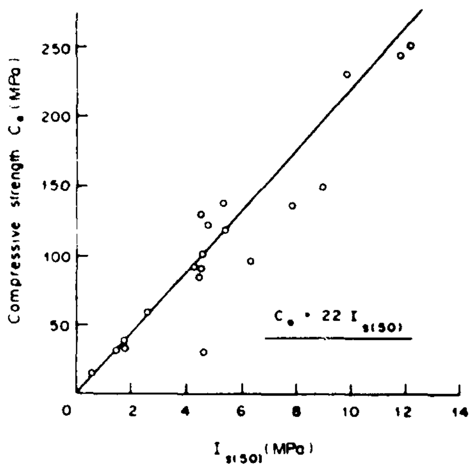


Fig. 9 Example of correlation between point load and uniaxial compressive strength results.

softer metal base of any geometry that will ensure that only the platen tip is in contact with the rock.

5. If a quick-retracting ram is used to reduce the delay between tests, either the ram return spring force and ram friction should together be less than about 5% of the smallest load to be measured during testing, or an independent load cell rather than an oil pressure gauge should be used for load determination. These forces can be significant when testing weaker and smaller specimens.

6. If significant platen penetration occurs, the dimension D to be used in calculating point load strength should be the value D' measured at the instant of failure, which will be smaller than the initial value suggested in paragraphs 8(d), 9(d) and 10(d). The error in assuming D to be its initial value is negligible when the specimen is large or strong. The failure value may always be used as an alternative to the initial value and is preferred if the equipment allows it to be measured (for example by electrical maximum-indicating load and displacement measurement). When testing specimens that are smaller than 25 mm, such as rock aggregate particles, equipment with electrical readout is usually necessary to obtain the required measuring accuracy, and should be designed to record D' at failure. Measurements of W or D made perpendicular to the line joining the platens are not affected and are retained at their original values. The value of D_e for strength calculation can then be found from:

$$D_e^2 = D \times D' \quad \text{for cores}$$

$$D_e^2 = \frac{4}{\pi}(W \times D') \quad \text{for other shapes}$$

7. Because this test is intended primarily as a simple and practical one for field classification of rock materials, the requirements relating to sample size, shape, numbers of tests etc. can when necessary be relaxed to overcome practical limitations. Such modifications to procedure should however be clearly stated in the report. It is often better to obtain strength values of limited reliability than none at all. For example, rock is often too broken or slabby to provide specimens of the ideal sizes and shapes, or may be available in limited quantities such as when the test is used to log the strength of drill core. In core logging applications, the concept of a "sample" has little meaning and tests are often conducted at an arbitrary depth interval, say one test every 1 m or 3 m depending on the apparent variability or uniformity of strength in the core and on the total length of core to be strength-logged.

8. As for all strength tests on rocks, point load strength varies with the water content of the specimens. The variations are particularly pronounced for water saturations below 25%. Oven dried specimens, for example, are usually very much stronger than moist ones. At water saturations above 50% the strength is less influenced by small changes in water content, so that tests in this water content range are recommended unless tests on dry rock are specifically required.

All specimens in a sample should be tested at a similar and well-defined water content, and one that is appropriate to the project for which the test data are required. Field testing of chisel-cut samples, not affected by drilling fluids, offers a method for testing at the *in situ* water content. If possible, numerical values should be given for both water content and degree of saturation at the time of testing. The ISRM Suggested Method for Water Content Determination should be employed. Whether or not water content measurements can be made, the sample storage conditions and delay between sampling and testing should be reported.

9. Some researchers argue in favour of measuring W as the minimum dimension of the failure surface after testing rather than of the specimen before failure (the German standard for this test is an example). Point load strengths computed using the two alternative W definitions may differ slightly. The minimum specimen dimension alternative has been adopted in this Suggested Method mainly because it is quicker and easier to measure, particularly in the field when fragments of broken specimens are easily lost.

10. Commonly the shortest dimension of naturally occurring anisotropic rock lumps is perpendicular to the weakness planes.

11. The size correction factor chart (Fig. 7) is derived from data on cores tested diametrally and axially and from tests on blocks and irregular lumps, for rocks of various strengths, and gives an averaged factor. Some rocks do not conform to this behaviour, and size correction should therefore be considered an approximate method, although sufficient for most practical rock classification applications. When a large number of tests are to be run on the same type of rock it may be advantageous to first perform a series of tests at different sizes to obtain a graph of load vs D_e^2 as in Fig. 6. If the slope of such a log-log graph is determined as "n", the size correction factor is then $(D_e/50)^m$ where $m = 2(1 - n)$. This can either be calculated directly or a chart constructed.

12. Mean results for small populations are generally better measures when the extreme values are not included in the calculation.

Received 10 *June* 1984.

BIBLIOGRAPHY

1. Franklin J. A., Broch E. and Walton G. Logging the mechanical character rock. *Trans. Instn Min. Metall.* **80**, A1-A9 (1971), and Discussion **81**, A43-A51 (1972).
2. Broch E. and Franklin J. A. The point-load strength test. *Int. J. Rock Mech. Min. Sci.* **9**, 669-697 (1972).
3. Bieniawski Z. T. The point-load test in geotechnical practice. *Engng Geol.* **9**, 1-11 (1975).
4. Boisen B. P. A hand portable point load tester for field measurements. *Proc. 18th U.S. Symp. on Rock Mechanics*, pp. 1-4 Keystone, Colorado (1977).
5. Broch E. Estimation of strength anisotropy using the point load test. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* **20**, 181-187 (1983).
6. Brook N. A method of overcoming both shape and size effects in point load testing. *Proc. Conf. on Rock Engineering*, pp. 53-70. Univ. of Newcastle, England (1977).
7. Brook N. Size correction for point load testing. Technical Note. *Int. J. Rock Mech. Sci. & Geomech. Abstr.* **17**, 231-235 (1980).
8. Fitzhardinge C. F. R. Note on point load strength test. *Aust. Geomech. J.* **G8**, 53 (1978).
9. Forster I. R. Influence of core sample geometry on the axial point load test. Technical Note. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* **20**, 291-295 (1983).
10. Gartung E. Empfehlung Nr. 5 des Arbeitskreises 19—Versuchstechnik Fels—der Deutschen Gesellschaft fur Erd- und Grundbau e.V. Punktlastversuche an Gesteinsproben. *Die Bautechnik* **59(1)**, 13-15 (1982).
11. Greminger M. Experimental studies of the influence of rock anisotropy on size and shape effects in point load strength testing. Technical Note. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* **19**, 241-246 (1982).
12. Guidicini G., Nieble C. M. and Cornides A. T. Analysis of point load test as a method for preliminary geotechnical classification of rocks. *Bull. Int. Ass. Engng Geol.* **7**, 37-52 (1973).
13. Haramy K. Y., Morgan T. A. and DeWaele R. E. A method for estimating coal strengths from point load tests on irregular lumps. USBM, Denver Research Center, Progress Rept 10028, 31pp (1981).
14. Hassani F. P., Scoble M. J. and Whittaker B. N. Application of the point load index test to strength of rock, and proposals for a new size correction chart. *Proc. 21st U.S. Symp. on Rock Mechanics*, pp. 543-556. Rolla, Missouri (1980).
15. International Society for Rock Mechanics. Suggested method for determining the point load strength index. ISRM (Lisbon, Portugal), Committee on Field Tests, Document No. 1, pp. 8-12 (1972).
16. Lajtai E. Z. Tensile strength measurement and its anisotropy measured by point-and line-loading of sandstone. *Engng Geol.* **15**, 163-171 (1980).
17. Pells P. J. N. The use of the point load test in predicting the compressive strength of rock materials. *Aust. Geomech. J.* **G5**, 54-56 (1975).
18. Peng S. S. Stress analysis of cylindrical rock discs subjected to axial double point-load. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* **13**, 97-101 (1976).
19. Read J. R. L., Thornton P. N. and Regan W. M. A rational approach to the point load test. *Proc. 3rd Aust. N.Z. Conf. on Geomechanics*, Vol. 2, pp. 35-39. Wellington (1980).
20. Reichmuth D. R. Point load testing of brittle materials to determine tensile strength and relative brittleness. *Proc. 9th U.S. Symp. on Rock Mechanics*, Colorado (1968).
21. Robins P. J. The point load test for concrete cores. *Mag. Concr. Res.* **32**, 101-111 (1980).
22. Wijk G. Some new theoretical aspects of indirect measurements of the tensile strength of rocks. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* **15**, 149-160 (1978).
23. Wijk G. The point load test for the tensile strength of rock. *Geotech. Testing J.*, pp. 49-54 (June, 1980).

PART II. IN SITU STRENGTH METHODS

C. Determination of In Situ Stress

DETERMINATION OF IN SITU STRESS BY THE OVERCORING TECHNIQUE

1. Scope

1.1 The equipment required to perform in situ stress tests using the U. S. Bureau of Mines' three-component borehole deformation gage is described.

1.2 The test procedure and method of data reduction are described, including the theoretical basis and assumptions involved in the calculations. A section on troubleshooting equipment malfunctions is included.

1.3 The procedure herein described was taken from the Bureau of Mines Information Circular No. 8618 dated 1974 [8.1] as were many of the figures. Some modifications based on field experience are incorporated. This test method can be used from the surface or from underground openings. Good results can be expected using this technique in massive or competent rock. Difficulties will be encountered if tests are attempted in fissile or fractured rock.

2. Test Equipment

2.1 Instrumentation

2.1.1 The three-component borehole deformation gage (BDG) is shown in Fig. 1. It is designed to measure diametral deformations during overcoring along three diameters 60 deg apart in a plane perpendicular to the walls of a 1-1/2-in.-diameter borehole. The measurements are made along axes referred to as the U_1, U_2, and U_3 axes. Accessories required with the gage are special pliers, 0.005- and 0.015-in.-thick brass piston washers, and silicone grease (Fig. 1).

2.1.2 Three Vishay P350A or equal strain indicators are required. (Alternatively, one indicator with a switching unit may be used or one unit may be used in conjunction with manual wire changing to obtain readings from the three axes.) These units have a full range digital readout limit of 40,000 indicator units. A calibration factor must be obtained for each axis to relate indicator units to microinches deflection. The calibration factor for each axis will change proportionally with the gage factor used. Normally, a gage factor of 0.40 gives a good balance between range and sensitivity. Figure 2 shows a strain indicator, calibration jig, and a switching unit.

2.1.3 The shielded eight-wire conductor cable transmits the strain measurements from the gage to the strain indicators. The length of cable required is the depth to the test position from the surface plus about 30 ft to reach the strain indicators.

2.1.4 The orientation and placement tools consist of:

(a) The placement tool or "J slot" shown in Fig. 3.

(b) Placement rod extensions as shown in Fig. 3.

(c) The orientation tool or "T handle," also shown in Fig. 3.

(d) The scribing tool is used to orient the core for later biaxial testing. It consists of a bullet-shaped stainless steel head attached to a 3-ft rod extension. Projecting perpendicular from the stainless steel head is a diamond stud. The stud is adjusted outward until a snug fit is achieved in the EX hole so that a line is scratched along the borehole wall as the scribing tool is pushed in.

(e) The Pajari alignment device is inserted into the hole to determine the inclination. It consists of a floating compass and an automatic locking device which locks the compass in position before retrieving it.

2.1.5 The calibration jig (Fig. 2) is used to calibrate the BDG before and after each test.

2.1.6 The biaxial chamber is used to determine Young's Modulus of the retrieved rock core. A schematic of the apparatus is shown in Fig. 4.[8.2]

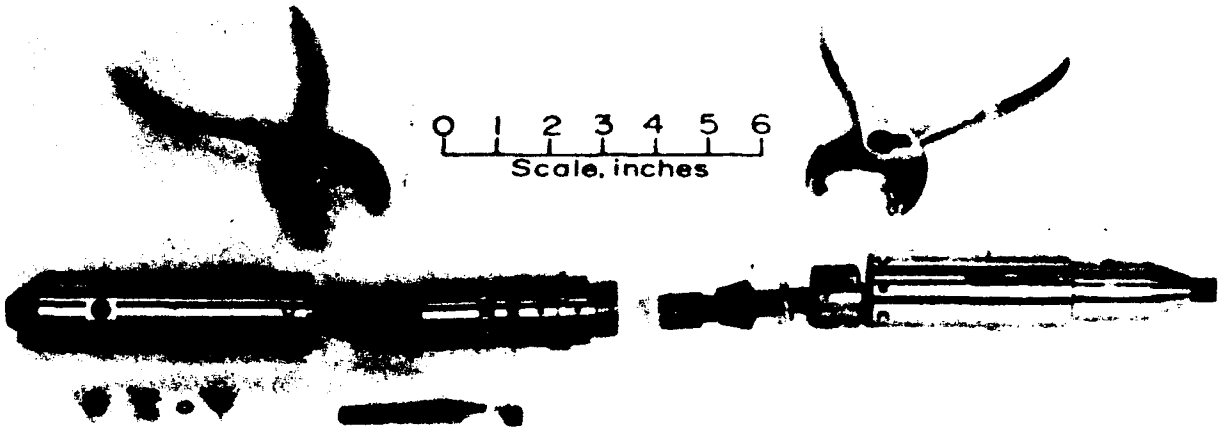


Fig. 1. Special pliers, the Bureau of Mines' three-component borehole gage, a piston, disassembled piston and washer, and a transducer with nut.

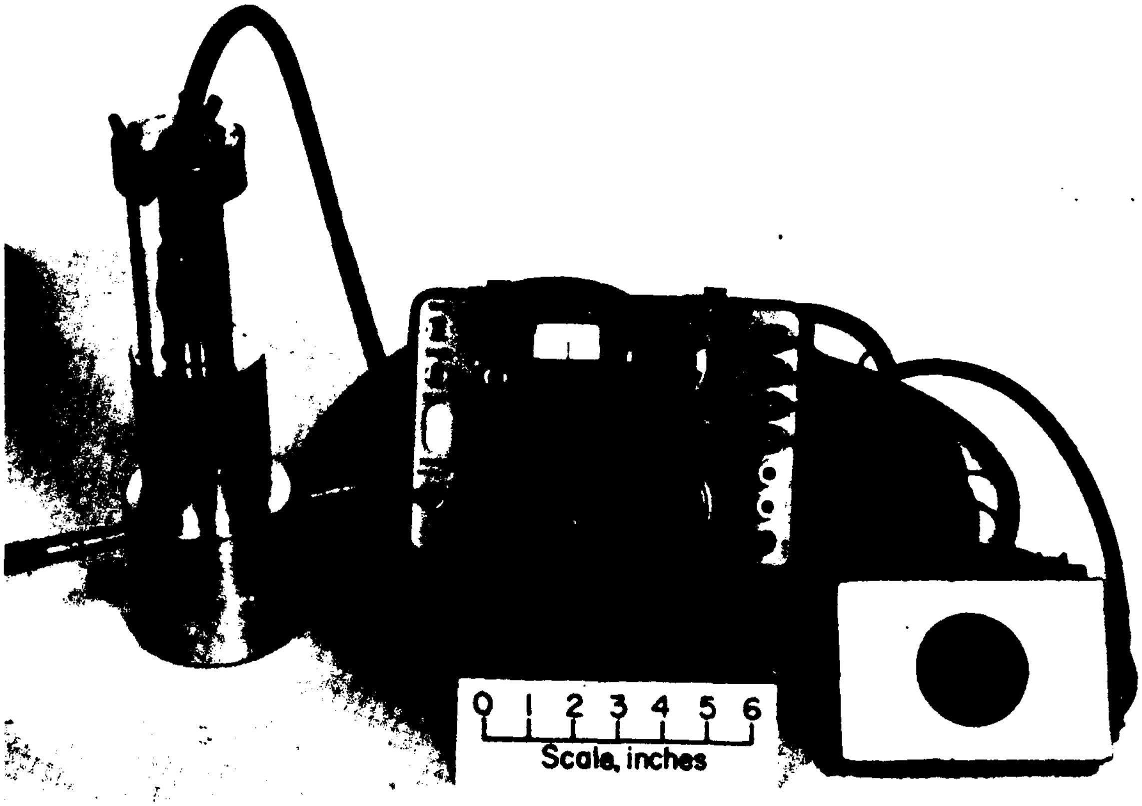


Fig. 2. The calibration device (left side) and a switching unit (right side).

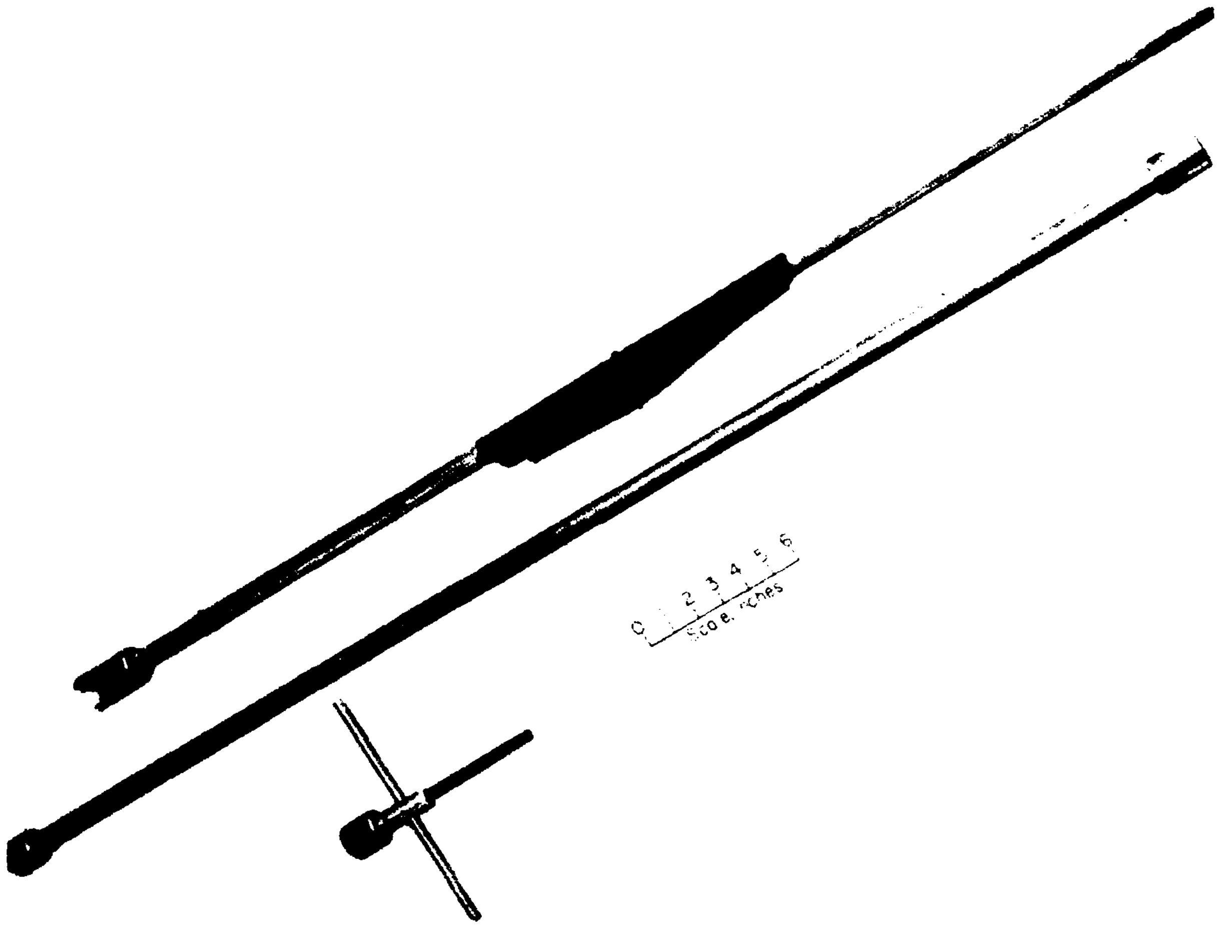
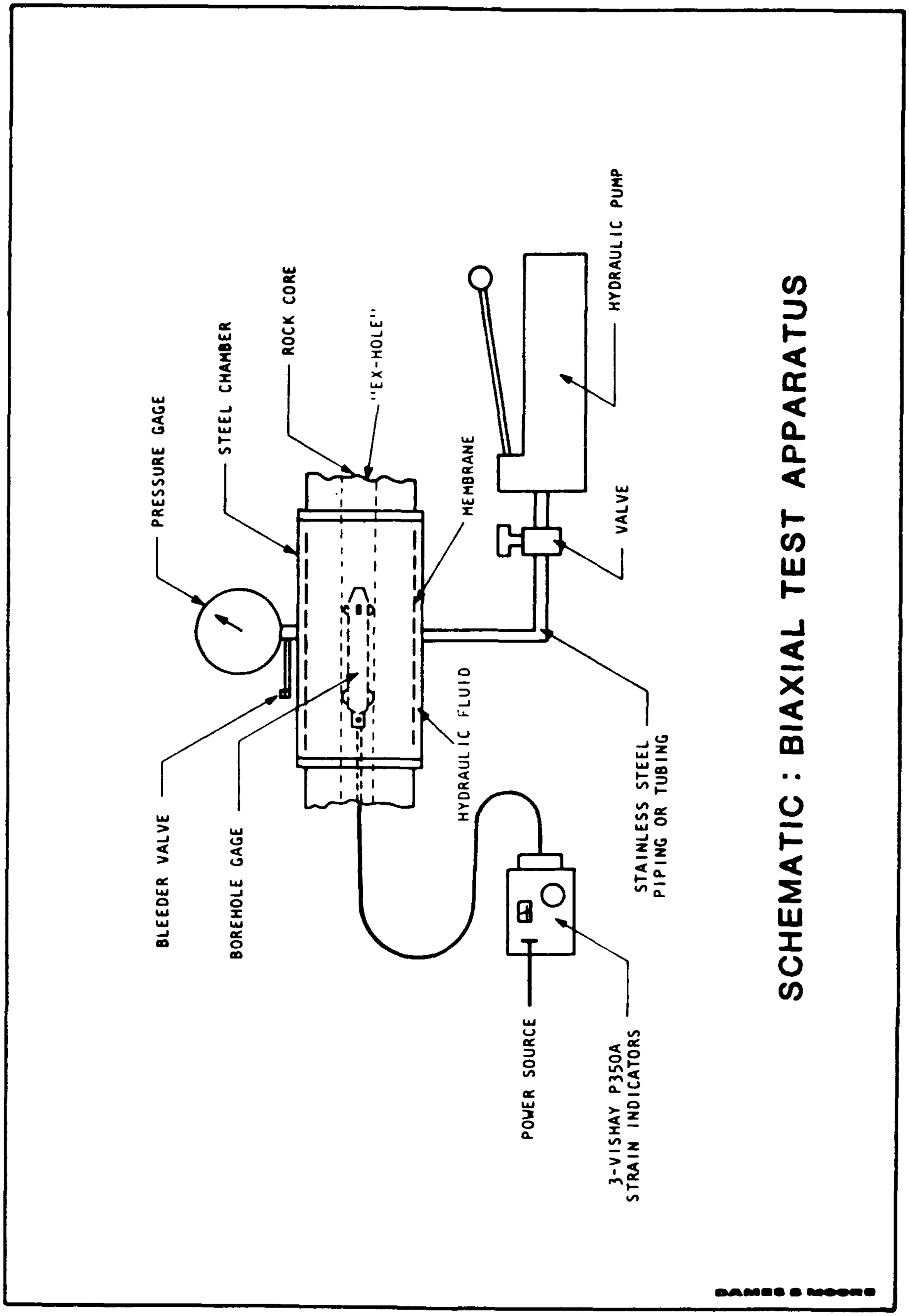


Fig. 3. Placement and retrieval tool.



SCHEMATIC : BIAXIAL TEST APPARATUS

Fig. 4

2.2 Drilling Equipment

2.2.1 A drill with a chuck speed ranging down to 50 rpm should be used. Achievement of this lower end speed will usually require a gear reduction.

2.2.2 An EWX single-tube core barrel, 2 ft long, is required (Fig. 5).

2.2.3 An EX diamond bit is required (Fig. 5).

2.2.4 A reamer is used with the EX bit for the 1-1/2-in.-diameter pilot gage hole (Fig. 5).

2.2.5 Two stabilizers are required. One should be 5-1/2 in. OD by 4 in. long and one should be 2-3/4 in. OD by 4 in. long. Each should have an inner concentric hole slightly larger than the OD of AX casing. Grooves should be cut into the stabilizers to allow water to pass through. The stabilizers are slipped over opposite ends of a 6 ft length of AX flush joint casing and secured with set screws. The stabilizer can be made from hard plastic stock. This assembly fits inside the 5-ft-long, 6-in.-diameter core barrel with the larger stabilizer at the bottom and the smaller stabilizer projecting into the NW casing. As overcoring proceeds, the stabilizer assembly is pushed upward into the NW casing.

2.2.6 An EWX rod stabilizer should be made that will slide over the EWX rod and fit inside the NW casing to align the EX bit with the hole in the center of the stabilizer in the 6-in. core barrel. Cut grooves along the outer edges of the stabilizer to allow water to flow through.

2.2.7 EW drill rods are used with the EX bit. The required length is dependent on the test depth.

2.2.8 NW casing is used with the 6-in. core barrel and bit. The required length will depend on test depth. An adapter is required to couple the NW casing to the 6-in. core barrel.

2.2.9 A water swivel (Fig. 6) is required with a 1/2-in. hole for the conductor cable to pass through and a plug is required to fit the hole when the gage is not being used.

2.2.10 A 6-in.-diameter starter barrel 1 ft long with a detachable 1-1/2-in.-diameter pilot shaft in the center (Fig. 7) is required. The pilot shaft should extend about 5 in. beyond the diamonds of the starter barrel. This barrel is used to center the 6-in.-diameter hole over an initial 1-1/2-in.-diameter hole at the face. The barrel and pilot shaft are not needed for vertical or near vertical holes.

2.2.11 An EW core barrel to replace the pilot shaft should be cut to extend 1 in. beyond the starter barrel. When the bit and reamer are attached, the unit is used to drill a 1-1/2-in.-diameter starter hole 4 in. deep at the end of a 6-in. horizontal hole. This piece of equipment is not needed in vertical holes.

2.2.12 A standard 6-in. diamond drill bit or a 6-in. thin wall masonry bit is used for overcoring.

2.2.13 A core breaker, at least 2-1/2 in. wide and hardened, to fit the EW rod (Fig. 8) is required.

2.2.14 A 6-in. core shovel (Fig. 8) to fit an EW rod is needed for retrieving core from horizontal holes.

2.2.15 A 6-in. core puller (Fig. 8) approximately 18 in. long to fit an EW drill rod is sometimes needed for retrieving core from vertical holes. The core puller is made from a used 6-in. core barrel. A 5/8-in.-thick steel plate is welded to the end of the barrel with an EW rod welded in the center. Three 1-1/2-in.-diameter holes on 120-deg centers are drilled into the plate to allow water to pass through. Four U cuts 90 deg apart are made on the front of the barrel. The rectangular pieces of metal inside the U cuts are pushed in slightly to grip the core. This is an optional piece of equipment. Normally, the core can be retrieved inside the 5-ft barrel.

2.2.16 A 6-in.-diameter core barrel 5 ft long is required.

2.2.17 A high-capacity water pump and hose are required.

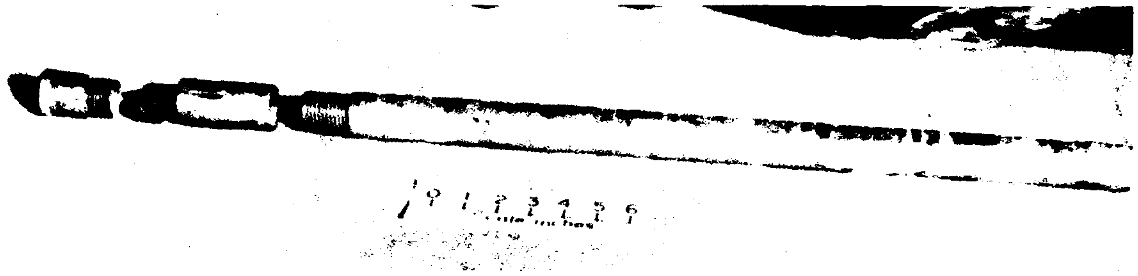


Fig. 5. EX size bit with core spring, reamer, and 2-ft EWX core barrel.

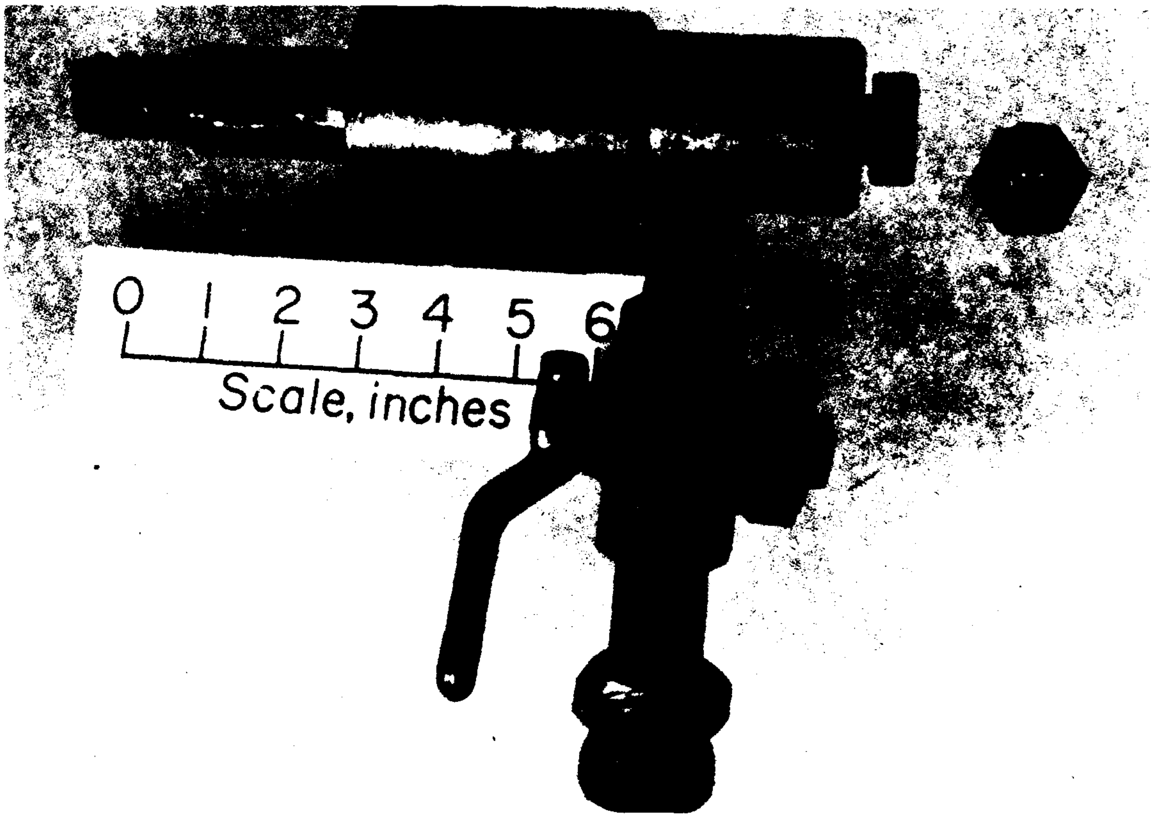


Fig. 6. Water swivel with solid plug. Plug at right used during overcoring.

Fig. 7. Six-inch-diameter starter barrel with pilot and expander head adapted for EW drill rod.

Fig. 8. Core breaker (lower right), core shovel (lower left), and core puller (center).

2.2.18 One clothesline pulley is required.

2.3 Miscellaneous Equipment - This field operation requires a good set of assorted hand tools which should include a soldering iron, solder and flux, pliers, pipe wrenches, adjustable wrenches, end wrenches, screwdrivers, allen wrenches, a hammer, electrical tape, a yardstick and carpenter's rule, chalk, and a stopwatch.

3. Overcoring Test Procedure

3.1 The procedure for determining in situ stress can be divided into three phases: (a) strain relief measurements in situ, (b) determination of Young's Modulus of the rock by recompression in a biaxial chamber, and (c) computation of stress.

3.2 Reliable information on subsurface rock conditions is essential for good test results. For this reason it is advisable to drill an exploratory NX size hole within a few feet of the desired test location.

3.3 Horizontal holes should be started 5 deg upward from horizontal to facilitate removal of water and cuttings. An EX pilot hole is first drilled about 4 in. into the rock. Attach the 6-in. starter barrel and pilot shaft and start the 6-in. hole. Remove the starter barrel, attach the regular 6-in. bit and barrel, and extend the 6-in. hole to within 12 in. of the desired test depth. Proceed to step 3.6.

3.4 When testing in vertical holes, first wash bore through the overburden and core about 5 ft into bedrock. Case the hole with 8-in. casing and grout in place.

3.5 Drill a 6-in.-diameter hole to within 12 in. of the desired test depth. Retrieve the core and go back down the hole with the stabilizers in place in the core barrel.

3.6 Insert the EX bit and reamer coupled to the 2-ft EX core barrel and EWX rods with the EWX stabilizer in place. Drill 2 ft of EX hole.

3.7 Retrieve the EX core and inspect. Insert the scribing tool coupled to the rod extensions. When the scribing tool reaches the stabilizers, it will have to be shoved through. It will then be at the top of the EX hole. Attach the orientation handle and orient the scribe mark as desired. Shove the scribe straight down the hole. (Note: If the scribe cannot be pushed down the hole, the diamond stud is projecting too far; adjust it inward. If the scribe feels loose the stud must be adjusted to project further.) When the scribe hits the bottom of the EX hole, slowly pull it back up along the same scribe mark. If joints or fractures intersect the borehole walls, they can often be detected by a subtle vibration as the diamond stud crosses them. If joints or fractures are detected, extend the hole and try again.

3.8 When the EX hole has been scribed, remove the scribing tool.

3.9 The BDG must now be calibrated. It should be calibrated before and after each test.

3.9.1 Grease all gage pistons with a light coat of silicone grease and install them in gage.

3.9.2 Place the gage in the calibration jig as shown in Fig. 2 with the pistons of the U_1 axis visible through the micrometer holes of the jig. Tighten the wing nuts.

3.9.3 Install the two micrometer heads and lightly tighten the set screws.

3.9.4 Set the strain indicators on "Full Bridge", center the balance knob, and set the gage factor to correspond to the anticipated in situ range and sensitivity requirements. (If high stress conditions are anticipated, sensitivity will have to be compromised to gain the required range.) A lower gage factor results in higher sensitivity. The gage factor used should be the same for calibration, in situ testing, and modulus tests.

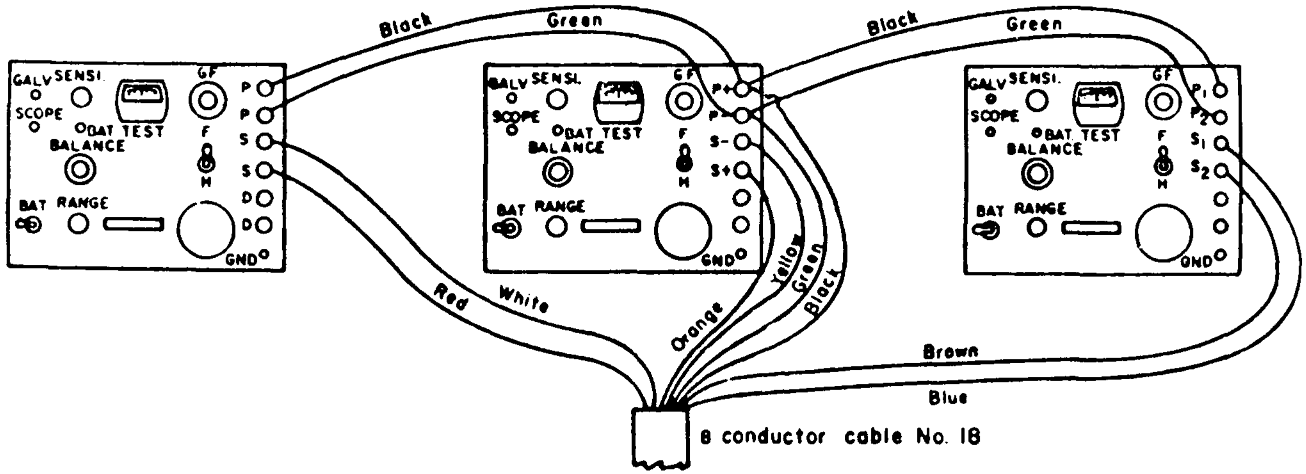
3.9.5 Wire the gage to the indicators as shown in Fig. 9 or to a switching and balance unit and one indicator.

3.9.6 Balance the indicator using the "Balance" knob.

3.9.7 Turn one micrometer in until the needle of the indicator just starts to move. The micrometer is now in contact with the piston. Repeat with the other micrometer.

3.9.8 Rebalance the indicator.

Sensitivity knob: turn full clockwise.
Balance knob: put mid-range (5 turns of the 10-turn potentiometer).
Bridge switch: Switch to full (F).



Note. Hook black and green wires to indicator No. 2 and use two other wires (No. 18 or No. 20) to common P+ and P- (or P_1 and P_2) of all three indicators.

Fig. 9. Wire hookup to model P-350 strain indicators.

3.9.9 Record this no load indicator reading for the U_1 axis.

3.9.10 Turn in each micrometer 0.0160 in. (a total of 0.0320-in. displacement).

3.9.11 Balance the indicator and record the reading and the deflection.

3.9.12 Wait two minutes to check the combined creep of the two transducers. Creep should not exceed 20 μ in./in. in two minutes.

3.9.13 Record the new reading.

3.9.14 Back out each micrometer 0.0040 in. (a total of 0.0080 in.).

3.9.15 Balance and record.

3.9.16 Continue this procedure with the same increments until the initial point on the micrometer is reached. This zero displacement reading will be the zero displacement reading for the second run.

3.9.17 Repeat steps 3.9.10 through 3.9.16.

3.9.18 Loosen the wing nuts and rotate the gage to align the pistons of the U_2 axis with the micrometer holes.

3.9.19 Retighten the wing nuts.

3.9.20 Repeat steps 3.9.6 through 3.9.17.

3.9.21 Loosen wing nuts and align pistons of U_3 axis with micrometer holes. Repeat the calibration procedure followed for the U_1 and U_2 axes.

3.9.22 To determine the calibration factor for each axis:

(a) Subtract the zero displacement strain indicator readings (last reading of each run) from the indicator readings for each deflection to establish the differences.

(b) Subtract the difference in indicator units at 0.0080-in. deflection from the difference in indicator units at 0.0320-in. deflection.

(c) Divide the difference in deflections (0.0240 in.) by the corresponding difference in indicator units just calculated to obtain the calibration factor for that axis.

(d) Repeat for the second cycle and take the average as the calibration factor.

(e) The following is an example of the calibration for one axis, calibrated at a gage factor of 0.40.

CALIBRATION OF AXIS U_1

	Displacement in.	Indicator Reading	Difference μ in./in.
Run 1	0	-693	---
	0.0320	+30,140	Wait 2 minutes
	0.0320	30,055	30,535
	0.0240	21,920	22,400
	0.0160	14,040	14,520
	0.0080	6,380	6,860
	0	-480	---
Run 2	0	-480	---
	0.0320	+30,034	Wait 2 minutes
	0.0320	29,980	30,430
	0.0240	21,914	22,364
	0.0160	13,975	14,425
	0.0080	6,335	6,785
	0	-450	---

$$\text{Calibration Factor} = K_1 = \frac{\text{Displacement}}{\text{Indicator Units}}$$

$$\text{For Run 1, } K_1 = \frac{32{,}000 - 8{,}000}{30{,}535 - 6{,}860} = \frac{24{,}000}{23{,}675} = 1.014$$

$$\text{For Run 2, K1} = \frac{32{,}000 - 8{,}000}{30{,}430 - 6{,}785} = \frac{24{,}000}{23{,}645} = 1.015$$

Use $K_1 = 1.01$ μ inches per indicator unit

3.10 Remove the BDG from the calibration jig and disconnect the wires from the strain indicators. Tape the ends of the wires together.

3.11 Thread the conductor cable through the chuck and water swivel and over the clothesline pulley attached to the top of the derrick if testing from ground surface. Reconnect the wires to the strain indicators exactly as during calibration.

3.12 Take zero deformation readings for each axis and record on the Field Data Sheet (Fig. 10) in the row labeled "zero" and in the three columns labeled U_1, U_2, and U_3. If just one indicator is being used, use a switching unit. If a switching unit is not available, the wires must be changed for each axis. Check each axis by applying slight finger pressure to opposing pistons and releasing. The balance needle should deflect, then return to the balanced position. Make sure the correct axis is connected to the correct strain indicator.

3.13 Engage the orientation pins of the BDG with the placement tool using a clockwise motion. Secure the conductor cable with the wire retainer clip on the placement tool. Make sure the orientation pins of the BDG are aligned with the U_1 axis. Push the gage through the stabilizer tube and about 9 in. into the EX hole. With the gage at test depth, orient the U_1 axis along the scribe mark by turning clockwise. If the BDG feels too loose or too tight in the EX hole, it must be removed. If too tight, remove one washer from one piston of each axis and try again. If too loose, add one washer to one piston of each axis. To add or remove washers, pull the piston out with the special pliers, unscrew the two piston halves with two pairs of special pliers, add or remove washers, and screw piston halves back together. Be careful not to damage the O ring. Regrease O ring and reinstall piston in gage. Do this to only one piston in each diametral pair initially. If gage is still too tight or loose, repeat with remaining pistons.

3.14 With the gage installed at test depth and correctly oriented, check the bias of the gage on the strain indicators. The bias set on each component should be between 10,000 and 15,000 indicator units

Hole No. ____________ Date ________ Orientation: U_1 ____________

Gage No. ____________ Calibration Factor U_1 ____________

Gage factor ____________ U_2 ____________

True Bearing of Hole ____________ U_3 ____________

DEPTH			DEFORMATION			TIME			TEMP.		REMARKS
			INDICATOR READING								
Gage	Hole (+)		U_1	U_2	U_3	Gage Set	Overcore Start	Deformation Read	Rock	Water	
		Zero									
9"	Face	Bias									
	1/2"										
	1"										
	1 1/2"										
	2"										
	2 1/2"										
	7"										
	7 1/2"										
	8"										
	8 1/2"										
Pistons	9"										
	9 1/2"										
	10"										
	10 1/2"										
	11"										
	13"										
	13 1/2"										
	14"										
	14 1/2"										
	15"										
	15 1/2"										
	16"										
	16 1/2"										
	17										
	17 1/2"										
	18										

Note. Next relief would start at 18 inches and go to 36 inches and gage would be orientated at a depth of 27 inches.

Fig. 10. Field data sheet.

with a gage factor of 0.40 for overcoring strain relief tests. For recompression tests in the biaxial chamber, the bias should be between 4,000 and 8,000 indicator units with a gage factor of 0.40. With a gage factor of 1.50, the bias should be between 2300 and 3400 indicator units for overcoring tests and between 900 and 1800 indicator units for recompression tests. Care must be taken to avoid overloading the transducers. Maximum load on any component should not exceed 20,000 indicator units with a gage factor of 0.40 and 4560 units with a gage factor of 1.50.

3.15 Turn the placement tool counterclockwise approximately 60 deg to disengage it from the BDG and remove the tool. (When retrieving the BDG, the tool is lowered onto the orientation pins and turned 60 deg clockwise.)

3.16 Pull the slack conductor cable through the chuck and over the clothesline pulley. Avoid excess tension in the cable or the gage may be pulled out of the EX hole. Tie off the cable and close the drill. Couple the NW casing to the chuck adaptor.

3.17 Turn on water. Allow approximately 10 minutes for gage, water, and rock to reach temperature equilibrium. Obtain new zero readings for each axis.

3.18 With the 6-in. bit resting on the bottom of the hole, tape a yardstick to the drill stand with the end flush with the bottom of the truck bed. As overcoring proceeds, check the advance rate by timing the descent of the yardstick with a stopwatch. Alternatively, the exposed casing may be marked at 1/2-in. increments to regulate the advance rate.

3.19 Start overcoring at a penetration rate of 1/2 in. per 40 seconds and a chuck speed of 50 rpm. The stopwatch is used to calibrate the drill to this rate. Each 1/2-in. penetration should be signaled to the recorder who records the indicator readings for each axis on the field data sheet. Overcore approximately 12 to 18 in. at this rate. If the core breaks during overcoring, the needles on the strain indicators will fluctuate erratically or the cable will twist. If either happens, stop overcoring immediately and retrieve the gage and core. If overcoring is successfully completed, stop the drill and continue to take periodic readings, with the water still running, until no appreciable changes in readings are occurring. This may take only a few minutes or it may take 2 or 3 hours depending on the rock.

3.20 Disconnect the wires from the strain indicators and tape the end of the cable so the drill can be uncoupled and raised without applying excess tension to the cable.

3.21 Pull the cable end back through the water swivel and chuck.

3.22 Secure the cable to the placement tool with the retainer clip and insert the tool over the BDG. When the placement tool engages the pins on the BDG, turn the tool 60 deg clockwise to secure the BDG. Pull the BDG and cable out of the hole.

3.23 Remove the core barrel and NW casing.

3.24 Retrieve the core (if it was not brought up inside the barrel) using the core breaker and core puller (or shovel in horizontal holes).

3.25 Plot the change in indicator units versus inches overcored for each test as shown in Fig. 11.[8.2] Compare this plot with the plot of an idealized overcore test (Fig. 12[8.2]) to determine if the test was successful.

3.26 Repeat this procedure for each additional test.

4. Procedure for Determining Young's Modulus of Elasticity of the Rock Core

4.1 The retrieved rock core should be tested in a biaxial chamber (Fig. 4) as soon as conveniently possible after recovery to determine the modulus of elasticity.

4.2 Place the calibrated BDG in the EX hole in the core at the same point and orientation where the in situ test was performed (align the U_1 axis of the BDG with the scribe mark).

4.3 Slide the rubber membrane over the rock core and place in the biaxial chamber.

4.4 Record initial or zero readings for all axes.

4.5 Increase hydraulic pressure in increments up to the measured in situ strain level and unload in identical increments.

4.6 Record deformation readings for each axis at each loading and unloading increment.

4.7 Repeat steps 4.4 through 4.6 for a second cycle.

4.8 Plot the applied pressure versus diametral deformations for each axis as shown in Fig. 13.[8.2] To calculate the average modulus value, E, obtain the differences in deflections corresponding to the differences in applied pressures on the second unloading cycle and use Equation 5 in the calculations section (see paragraph 6).

4.9 This test procedure requires an intact piece of core at least 10-1/2 in. long.

4.10 Alternatively, the modulus may be obtained by testing the NX core from the same depth in the nearby exploratory NX hole in uniaxial compression using standard procedures described in ASTM Standard Method of Test for Elastic Moduli of Rock Core Specimens in Uniaxial Compression, ASTM Designation D3148-72. This test method is described in Section 201-80 of this handbook.

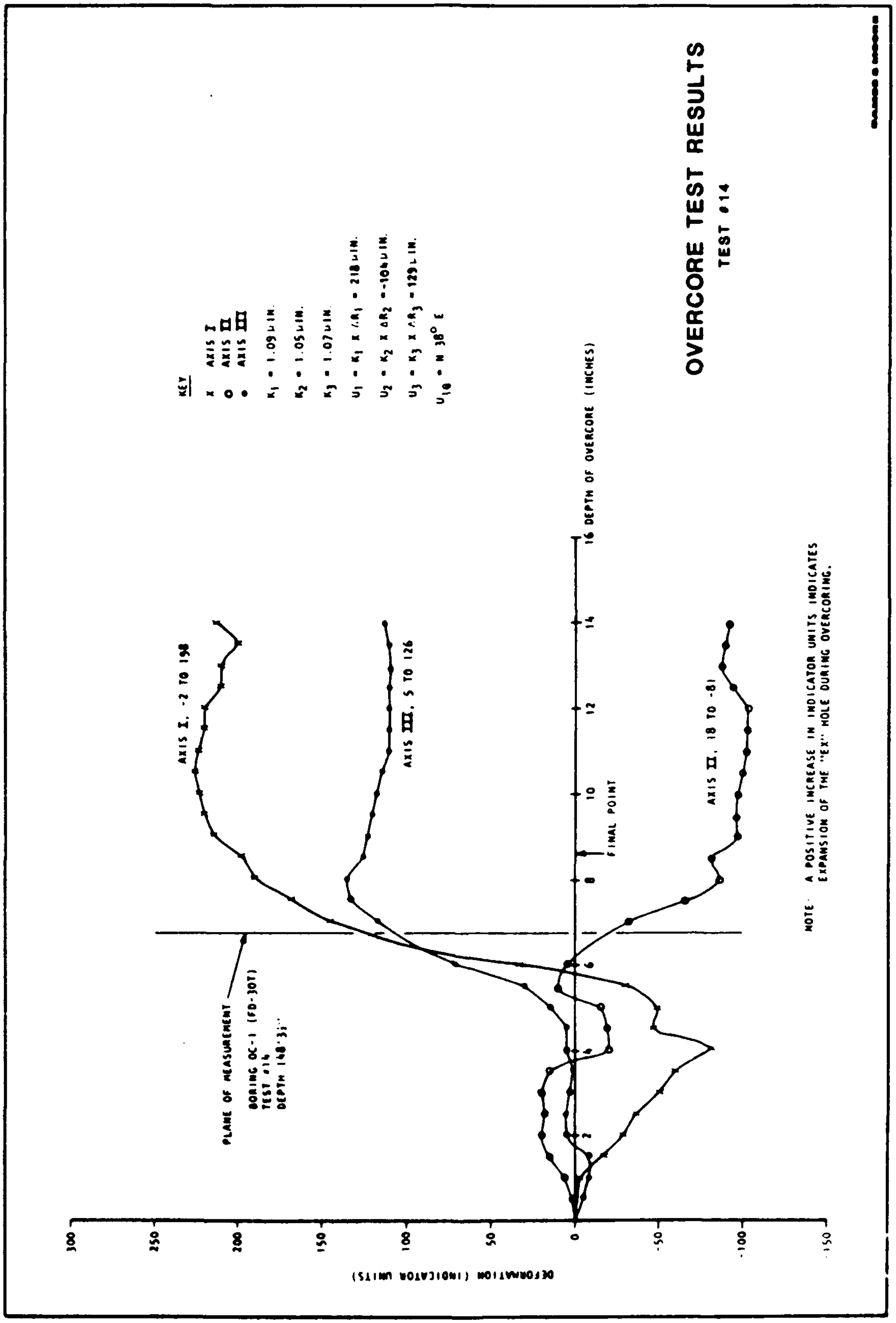
5. <u>Troubleshooting Equipment Malfunctions</u>

5.1 If balance on one or more indicators cannot be achieved.

5.1.1 Check wiring hookup against wiring diagram (Fig. 9). Make sure all connections are tight.

5.1.2 Check cable connector plug in BDG. Remove screws from placement end of gage, slide the end off, unscrew the knurled retaining cap, and check the plug connection. Push in firmly if loose.

5.1.3 Nonbalance may occur when too much tension has been applied to the conductor cable during gage retrieval. Sometimes in a vertical hole, cuttings or rock fragments drop into the 1-1/2-in. hole on top of the gage, making it impossible to hook the placement and retrieval tool onto the gage pins. A tendency always exists to try to retrieve the gage by pulling on the cable. Do this only as a last resort. Instead, snap the core off and bring it up with the gage inside if overcoring has been successfully completed.



OVERCORE TEST RESULTS
TEST #14
DEFORMATION (INDICATOR UNITS)
300
250
200
150
100
50
0
-50
-100
-150
2
4
6
8
10
12
14
16 DEPTH OF OVERCORE (INCHES)
FINAL POINT
PLANE OF MEASUREMENT
BORING OC-1 (FD-307)
TEST #14
DEPTH 148'3½"
AXIS I, -2 TO 198
AXIS III, 5 TO 126
AXIS II, 18 TO -81
KEY
x AXIS I
o AXIS II
• AXIS III
K1 = 1.09 μIN.
K2 = 1.05 μIN.
K3 = 1.07 μIN.
U1 = K1 X ΔR1 = 218 μIN.
U2 = K2 X ΔR2 = -104 μIN.
U3 = K3 X ΔR3 = 129 μIN.
U1θ = N 38° E
NOTE: A POSITIVE INCREASE IN INDICATOR UNITS INDICATES EXPANSION OF THE "EX" HOLE DURING OVERCORING.

Fig. 11

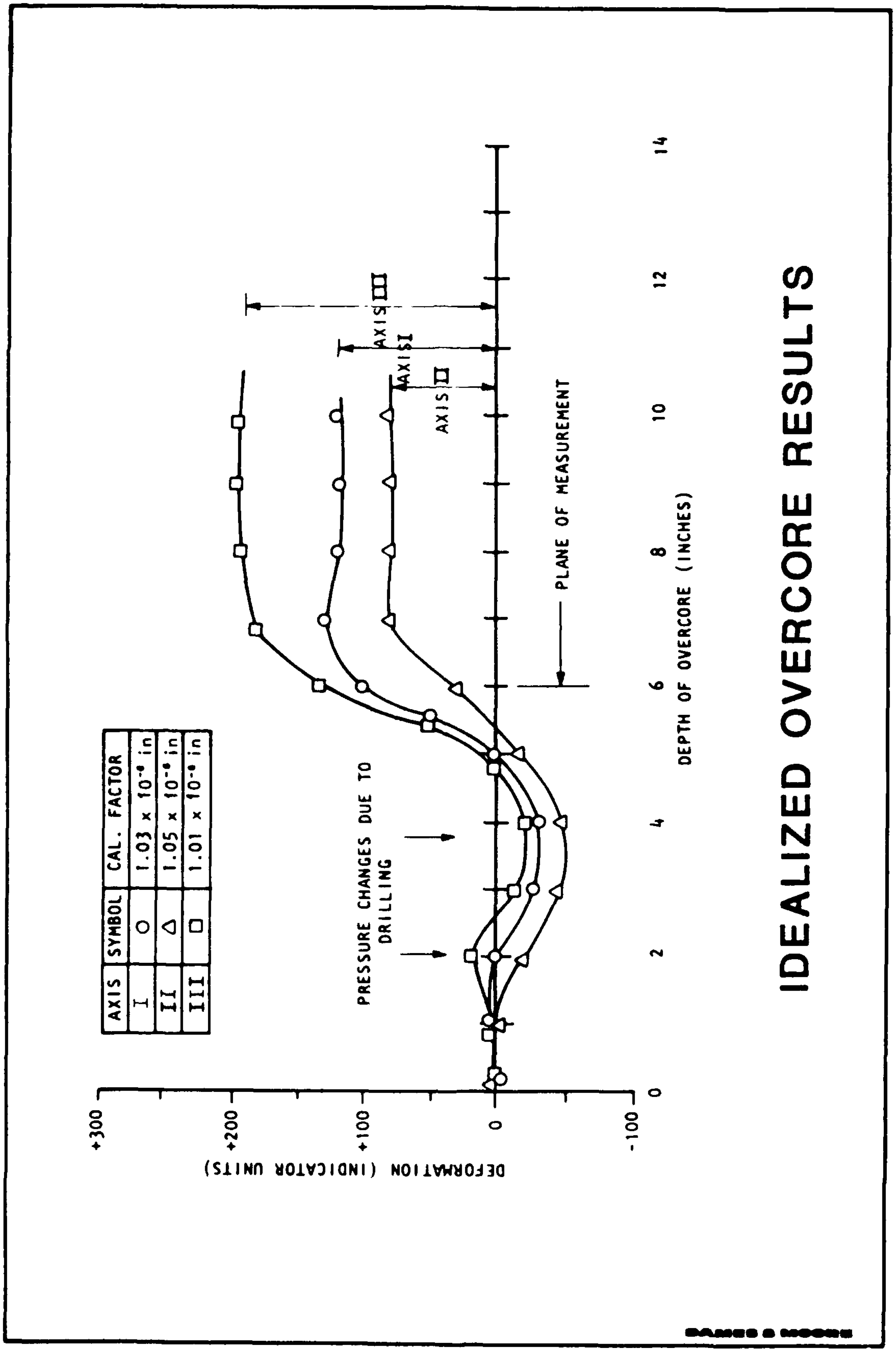
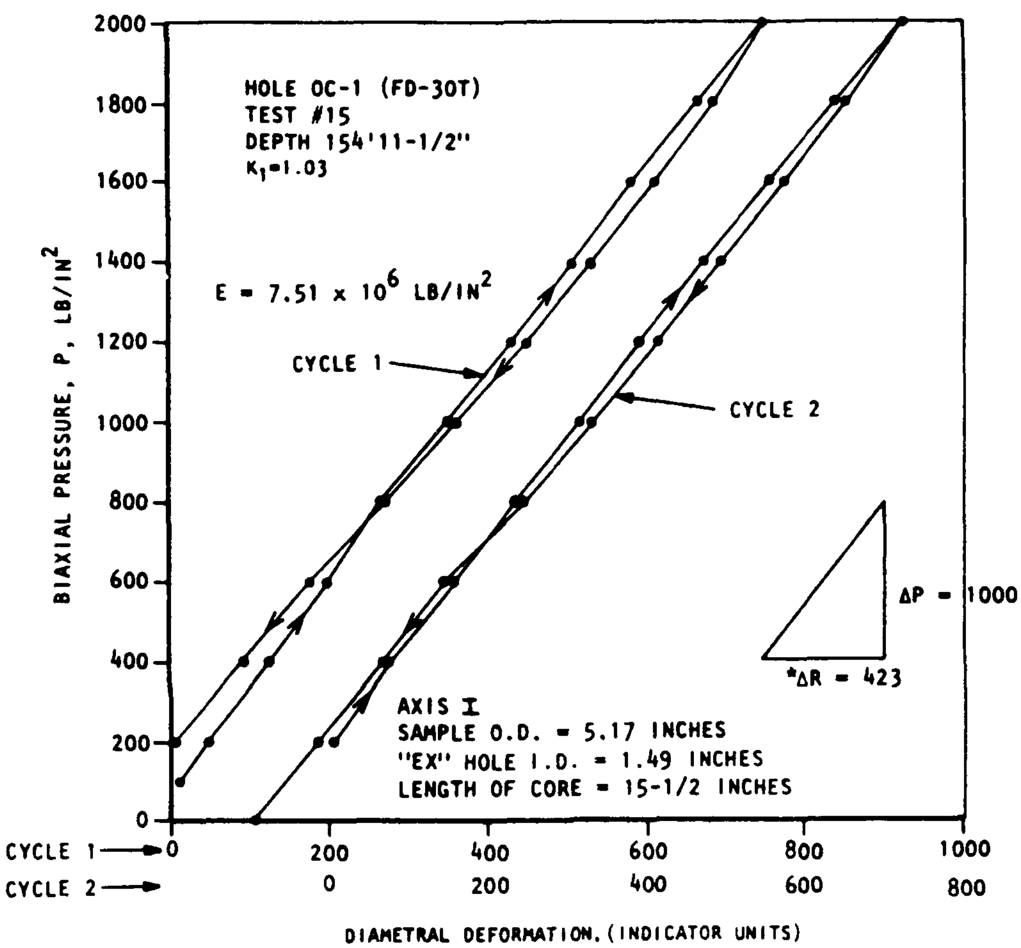

IDEALIZED OVERCORE RESULTS
AXIS | SYMBOL | CAL. FACTOR
I | ○ | 1.03 x 10⁻⁶ in
II | △ | 1.05 x 10⁻⁶ in
III | □ | 1.01 x 10⁻⁶ in
PRESSURE CHANGES DUE TO DRILLING
PLANE OF MEASUREMENT
AXIS III
AXIS I
AXIS II
DEFORMATION (INDICATOR UNITS)
+300
+200
+100
0
-100
DEPTH OF OVERCORE (INCHES)
0
2
4
6
8
10
12
14

Fig. 12



BIAXIAL TEST RESULTS

TEST #15

AXIS I

*ΔR = CHANGE IN INDICATOR UNITS CORRESPONDING TO CHANGE IN APPLIED PRESSURE ΔP.

5.2 If core breaks during overcoring:

5.2.1 If the indicators suddenly start to fluctuate erratically or the needles indicate maximum deflection, the core has probably broken. This situation may also be indicated by twisting of the conductor cable. If this situation occurs, stop the test immediately.

5.2.2 Disconnect the NW casing from the chuck and insert the retrieval tool, but leave the 6-in. bit on the bottom. If retrieval tool will slide over BDG pins, retrieve the BDG. If the retrieval tool will not slide over the pins a piece of rock has probably fallen in on top of it.

5.2.3 If overcoring has proceeded past the end of the gage, snap the core off below the gage and bring core and gage up inside the core barrel. Keep light tension on conductor cable as gage is brought up. If the core does not come up with the barrel, use the core puller.

5.2.4 If overcoring has not proceeded past the end of the gage, try gently tugging on the cable to free the BDG. If this fails, tug sharply on the cable. It will snap off, usually at the connector plug. Retrieve the cable and resume overcoring several inches past the gage (Note 3).

NOTE 3—Snapping the cable off is a drastic measure but is a necessary trade-off to retrieve the gage in working condition. Never raise the 6-in. bit until you are certain that the bit has overcored past the gage. Then snap off core and bring the bit, core, and gage up.

5.3 If one or more elements become insensitive on indicators:

5.3.1 If elements become insensitive to deflection of the pistons or unresponsive to turning of the indicator dial, or the needles drift, water has probably caused a short at the strain gage connections or at the cable connector plug.

5.3.2 Remove the pistons with the special pliers and check for moisture. If moisture is present, dry area thoroughly, check O rings for damage, and replace them if necessary. Apply thin coat of silicone grease to O rings before reinserting pistons.

5.3.3 Check the cable connector plug by removing the upper gage case. Unscrew the retaining cap and check for moisture on plug. Dry plug and surrounding area and grease cable where it passes through retainer cap and rubber grommet. Reassemble gage.

5.4 If one component does not balance anywhere on the indicator dial or balances intermittently:

5.4.1 This situation indicates a disconnected wire or a cold solder joint. Remove the borehole gage case and check all wires and connections, including the cable connector plug. Solder where needed and reassemble gage.

5.5 If indicators are sensitive to touch:

5.5.1 If indicator needles deflect when the units are touched, it is usually a result of prolonged use in a damp environment. Use plastic or other insulating material underneath the indicators as a moisture barrier. Store indicators in a dry place when not in use to allow them to dry out.

6. Calculations

6.1 To determine the secondary principal stress magnitudes and directions it is usually convenient to assume the rock is a linearly elastic isotropic material.

6.2 The diametric deflections are obtained for each of the three axes of measurement by multiplying the indicator reading differences by the appropriate calibration factor. These values, U_1, U_2, and U_3, along with the calculated modulus of elasticity, form the basis for evaluating the maximum and minimum stresses acting in a plane perpendicular to the borehole walls. These stresses are principal stresses only when the borehole is parallel to the third principal stress, which is not always the case for vertical boreholes. However, under the conditions of more or less homogeneous, gently dipping rocks of low relief, it can be assumed that the third principal stress is vertical and equal to the overburden stress and that the maximum and minimum stresses perpendicular to the borehole walls are in fact principal stresses. For other conditions and different borehole orientations this assumption would be invalid, so consideration must be given to the special features and conditions appropriate to the individual site. Fairhurst[8.3] presents a solution for the state of stress in a transversely isotropic medium, that is, one for which the elastic properties are constant in any direction in a given plane but change in directions that intersect the plane. Fairhurst states that the assumption of elastic isotropy can result in errors in the computed stresses of as much as 50 percent in cases where the rock is anisotropic or "transversely isotropic" (typical of rocks such as shale and gneiss).

6.3 The overburden stress, σ_v, is equal to the average density of the overlying material, γ, multiplied by the depth of overburden, H, or

$$\sigma_v = \gamma H \tag{1}$$

6.4 The secondary principal stresses are calculated as shown below, using equations based on a plane stress analysis of an elastic, isotropic thick-walled cylinder as discussed by Obert and Duvall.[8.4]

$$P_c = \frac{E}{6d} \left[U_1 + U_2 + U_3 + \frac{\sqrt{2}}{2} \left\{ (U_1 - U_2)^2 + (U_2 - U_3)^2 + (U_3 - U_1)^2 \right\}^{1/2} \right] \tag{2}$$

$$Q_c = \frac{E}{6d} \left[(U_1 + U_2 + U_3) - \frac{\sqrt{2}}{2} \left\{ (U_1 - U_2)^2 + (U_2 - U_3)^2 + (U_3 - U_1)^2 \right\}^{1/2} \right] \tag{3}$$

where U_1, U_2, U_3 are measurements of diametral deformations along three axes 60 deg apart. Deformation is positive for increasing diameter during overcoring.

P_c is the maximum normal stress, psi.*

Q_c is the minimum normal stress, psi.*

E is the modulus of elasticity, psi, and d is the "EX" hole diameter.

6.5 The orientation of the principal stress axis is given by

$$\theta_p = 1/2 \text{ arc tan } \frac{\sqrt{3}\ (U_2 - U_3)}{2U_1 - U_2 - U_3.} \qquad (4)$$

where θ_p is the angle from the U_1 axis (positive in a counterclockwise direction) to the major principal stress.

U_1, U_2, U_3 are the diametral deformations.

The angle θ_p could have two values 90 deg apart. The correct angle can be determined using the following rules. For a 60 deg rosette (angular measurements positive in the counterclockwise direction and all angles measured from U_1 to P_c):

6.5.1 If $U_2 > U_3$, θ_p lies between 0^o and $+90^o$ or -90^o and -180^o.

6.5.2 If $U_2 < U_3$, θ_p lies between 0^o and -90^o.

6.5.3 If $U_2 = U_3$, and if: (a) $U_1 > U_2 = U_3$, $\theta_p = 0°$

(b) $U_1 < U_2 = U_3$, $\theta_p = \pm 90°$.

6.6 The modulus of elasticity is calculated using the biaxial test results in the equation

$$E_i = \frac{(4ab^2)\ (\Delta Pi)}{(b^2 - a^2)\ \Delta Ui} \qquad (5)$$

* Positive values of P_c and Q_c indicate compressive stresses.

where E_i is the modulus of elasticity, psi

a is the diameter of the "EX" hole, inches

b is the radius of the core, inches

ΔPi is the change in applied pressure, psi

ΔUi is the diametral deformation, in inches corresponding to the change in applied pressure

i is the direction of the axis

7. Reporting Results

7.1 The report shall include:

7.1.1 A description of the test site, including a general area map, characteristic features, and the type and depth of rock at which tests were performed.

7.1.2 The test procedure and apparatus used in the field should be described, including any innovations in procedure or apparatus. The assumptions and equations used in calculating stresses should be presented.

7.1.3 For each test the sample should be described, including depth and rock type and description of joints present. Young's Modulus should be tabulated and a plot of the biaxial test should be presented, as in Fig. 13, along with a plot of the overcoring test results, as in Fig. 11. Calibration factors should be tabulated for each test. The maximum and minimum secondary principal stresses and the orientation should be tabulated for each test.

7.1.4 Any difficulties or unusual circumstances encountered should be noted.

8. References

8.1 Hooker, Verne E., and Bickel, David L., "Overcoring Equipment and Techniques Used in Rock Stress Determination," U. S. Bureau of Mines Information Circular 8618, Denver Mining Research Center, Denver, Colorado, 1974.

8.2 Nataraja, Mysore, "In Situ Stress Measurements, Park River Project," Miscellaneous Paper No. S-77-22, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi, 1977.

8.3 Fairhurst, Charles, "Methods of Determining In Situ Rock Stresses at Great Depths," Technical Report No. 1-68, U. S. Army Corps of Engineers, Missouri River Division, Omaha, Nebraska, 1968.

8.4 Chert, L., and Duvall, W. I., "Rock Mechanics and the Design of Structures in Rock," John Wiley and Sons, Inc., New York, 1967.

RTH-342-89

SUGGESTED METHOD FOR DETERMINING STRESS BY OVERCORING A PHOTOELASTIC INCLUSION

Scope

1. (a) This test determines in situ stress by sensing strain transmitted during overcoring from the rock to a stiff inclusion coupled in a drill hole.[1] The translation of strain to stress is accomplished by calibration of the inclusion at known stresses. The hard inclusion is sometimes described as a stressmeter.

(b) Maximum and minimum stresses are measured in the plane normal to the drill hole axis. At least three tests in separate drill holes will generally be needed to estimate directions and magnitudes of principal stresses.

(c) The method is relatively inexpensive.

(d) Rock anisotropy and fine dependent deformations complicate interpretation, and may lead to serious errors if ignored.

(e) Another overcoring method measures strain of the drillhole rather than the response of a hard inclusion, See RTH-341.

Apparatus

2. Equipment for drilling a 3-in. emplacement hole in any direction.[2]

3. Equipment for overcoring in any direction at the diameter of 9-in. Included is an axial stabilizer or other device for maintaining concentricity with the smaller center hole.

4. Optical transducer assembly (Figure 1) consisting of:

(a) Hard disk inclusion of glass or other photoelastic material, with axial hole.

(b) Waterproof light source positioned beyond the inclusion, powered by an outside source. Leads from the outside source pass axially through the inclusion.

(c) Means of mechanically or chemically bonding the inclusion to the hole wall.

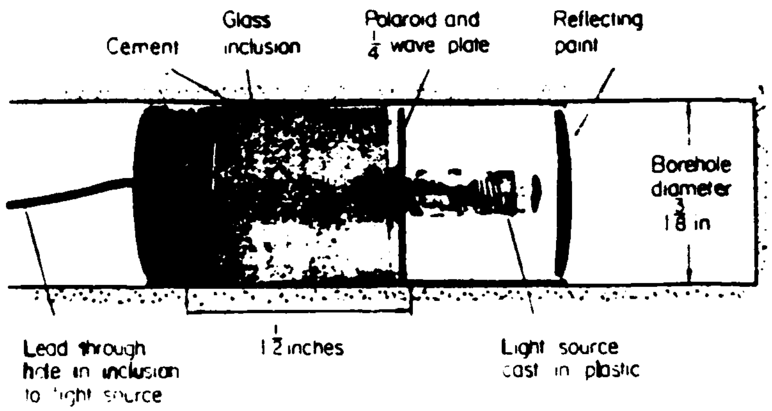


Figure 1. Example of photoelastic stressmeter.

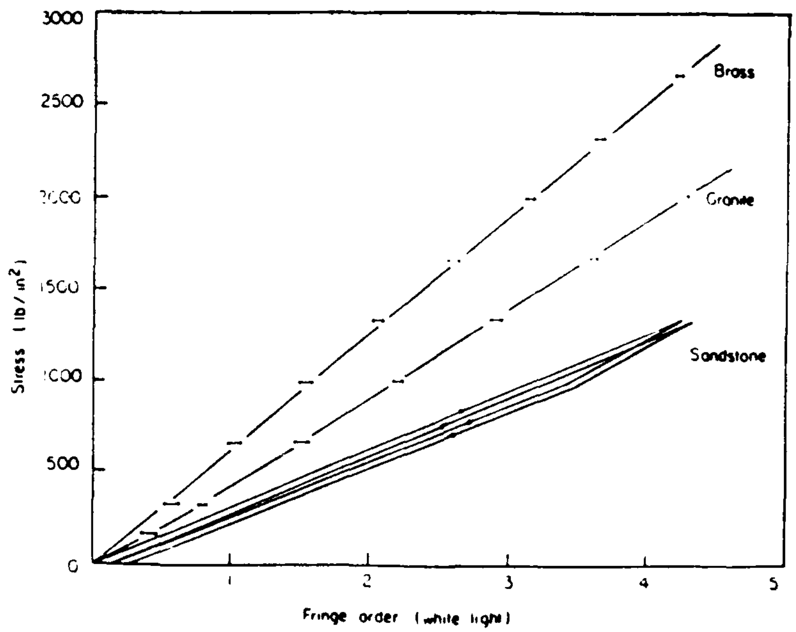


Figure 2. Example calibration curves for photoelastic glass inclusion in three materials.

(d) Analyzing and polarizing plates supplemented with telescope for distant viewing.

5. Tools for positioning and wedging or bonding the inclusion.

6. Calibrating equipment consisting of:

(a) Block of material having modulus like rock of interest and containing a 3-in. hole.

(b) Loading machine and recorder.

Procedure

7. Preparation

(a) The inclusion is calibrated by mounting the complete assembly in the test block and subjecting the block to known loads. Calibration curves are prepared (Figure 2).

(b) The site is selected emphasizing hard, homogeneous rock with no fractures in the volume to be overcored.

(c) If testing will be below the water table, the test volume should be dewatered and kept dry during testing.[3]

(d) The 3-in. hole should be cored to the full depth for all testing and the core should be logged and inspected to confirm suitability of the test intervals.[4]

(e) The optical transducer assembly is emplaced in the 3-in. hole and the hard inclusion is bonded to the side wall. There should be at least 9-in. of open hole between collar and inclusion and inclusion and far end.

8. Testing:

(a) The assembly is checked, a pretest observation of birefringence is made and documented, and lead lines are temporarily disconnected from the outside.[5]

(b) The 9-in. overcore hole is drilled precisely along the same axis to a depth 9-in. beyond the hard inclusion. The drill is withdrawn.

(c) Lead lines are reconnected.[5]

(d) Birefringence is observed (Figure 3 and 4) and is recorded photographically and/or by accurate drawing. A vertical reference should be established. Retrievable portions of the assembly are withdrawn.

(e) The inclusion and polarizing plate are usually not retrieved.

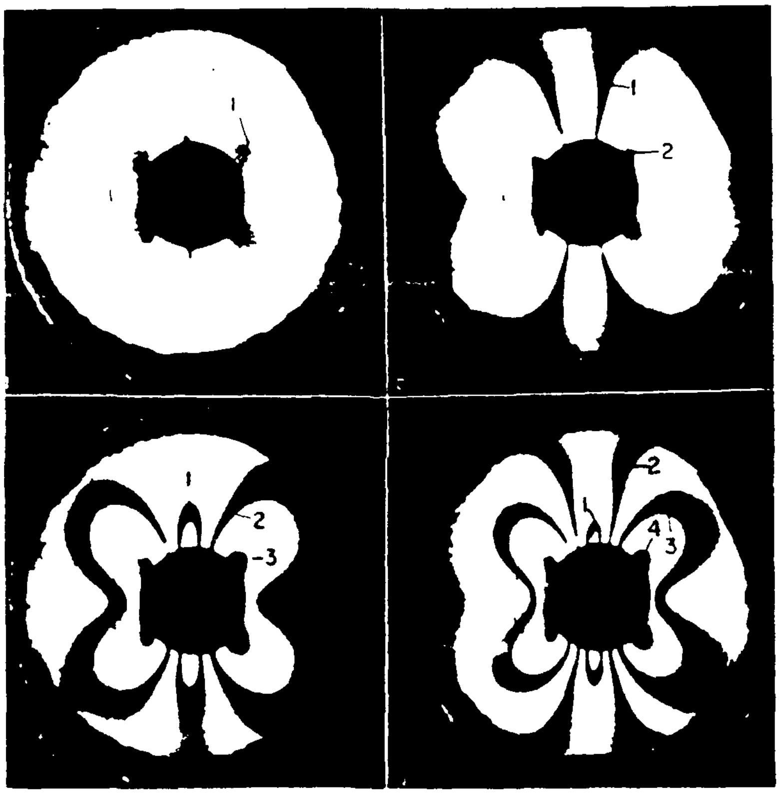


Figure 3. Patterns displayed by the photoelastic disk under increasing uniaxial loading

(f) Where another test is needed at greater depth, the overcored rock stub is broken and removed to reopen the 3-in. hole.

Calculations

9. The glass inclusion with hole along its axis forms a biaxial gage displaying birefringence patterns which identify strain in the glass and, by calibration, stress in the rock. The following characteristics are diagnostic (Figures 3 and 4):

(a) The axes of symmetry of the signal identify the principal stress directions.

(b) The direction in which the signal moves with increase of stress, and the presence of isotropic points in biaxial fields, identifies the major principal stress direction.

(c) The fringe order at a selected point of reference on the pattern is measured to give the major principal stress directly in terms of a calibration factor for any particular principal stress ratio (Figure 1).

(d) The ratio between major and minor principal stresses is indicated approximately by the shape of the signal and precisely by the measured distance between two isotropic points on the major axis.

(e) The manner in which the optical pattern changes, when the analyzer of the polariscope is operated in the process of taking the measurement, identifies whether the measured stress is tensile or compressive.

10. Calculations based on elastic theory are unnecessary in following the procedure presented in paragraph 9. However, the calibration must be well founded in theory. Fortunately, the effects are similar for all materials of great stiffness[1] and even metal may be substituted for site rock during calibration.

Reporting

11. The report should include the following:

(a) Position and orientation of test.

(b) Logs and other geological descriptions of rock near the test. Geological structure and elastic properties are particularly important.

(c) Record of test activities.

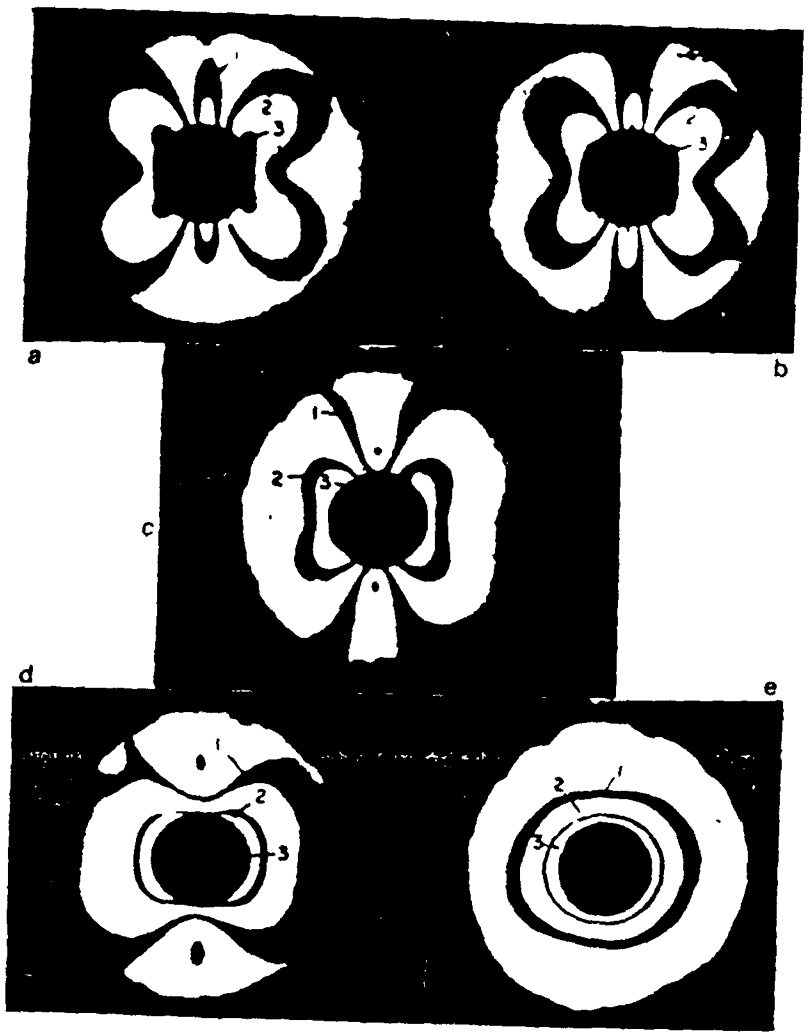


Figure 4. Patterns displaved by photoelastic disk in various stress states, all at third fringe order.

(d) Accurate drawings or photographs of birefringence patterns including that prior to overcoring.

(e) Biaxial stresses indicated from calibration curves.

(f) Description of calibration investigations and resultant curves. Where several inclusions are calibrated at once, a reference to description elsewhere may be sufficient.

Notes

[1]The stress induced in a stiff inclusion will be about 1.5 times the comparable stress in the host rock provided the elastic modulus of the inclusion exceeds that of host by a factor of 5 or more (Roberts 1968).

[2]Diamond core drilling is recommended for obtaining the necessary close tolerance between wall and inclusion.

[3]The test is most commonly conducted in tunnel walls where water is usually not a major problem.

[4]The 9-in. hole is sometime started first and advanced beyond any disturbed zone along the excavated surface.

[5]If the light is retrievable, it is withdrawn temporarily for overcoring.

References

1. Roberts, A., "The Measurement of Strain and Stress in Rock Masses," Chapter 6 in Rock Mechanics in Engineering Practice, John Wiley & Sons, New York, 1968.

PART II: IN SITU STRENGTH METHODS

D. Determination of Rock Mass Deformability

RTH-361-89

SUGGESTED METHOD FOR DETERMINING ROCK MASS DEFORMABILITY USING A PRESSURE CHAMBER

Scope

1. (a) This test determines the deformability of a rock mass by subjecting the cylindrical wall of a tunnel or chamber to hydraulic pressure and measuring the resultant rock displacements. Elastic or deformation moduli are calculated in turn.

(b) The test loads a large volume of rock so that the results may be used to represent the true properties of the rock mass, taking into account the influence of joints and fissures. The anisotropic deformability of the rock can also be measured.

(c) The results are usually employed in the design of dam foundations and for the proportioning of pressure shaft and tunnel linings.

(d) Two other methods are available for tunnel-scale deformability. See RTH-366 and RTH-367 to compare details. Potentially large impacts, especially in terms of cost, of variations at this scale justify the separation of methods.

(e) This method reflects practice described in the references listed at the end.

Apparatus

2. Equipment for excavating and lining the test chamber including:

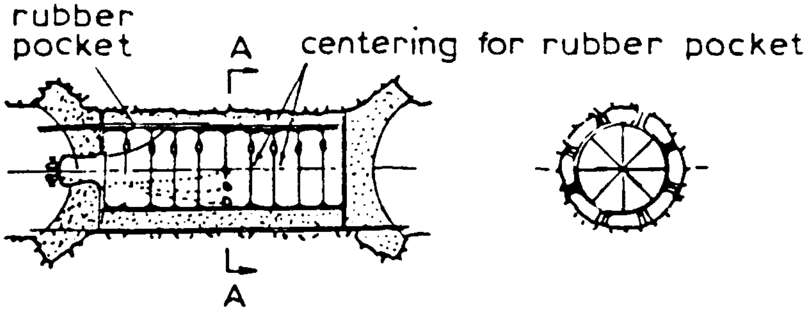
(a) Drilling and blasting materials or mechanical excavation equipment.[1*]

(b) Materials and equipment for lining the tunnel with concrete or flexible membrane.

3. A reaction block (Figure 1) usually comprising a tunnel bulkhead of sufficient strength and rigidity to resist the force applied by the pressurizing fluid. The bulkhead must also be waterproof. Reusable bulkheads are possible where several sites are to be investigated.

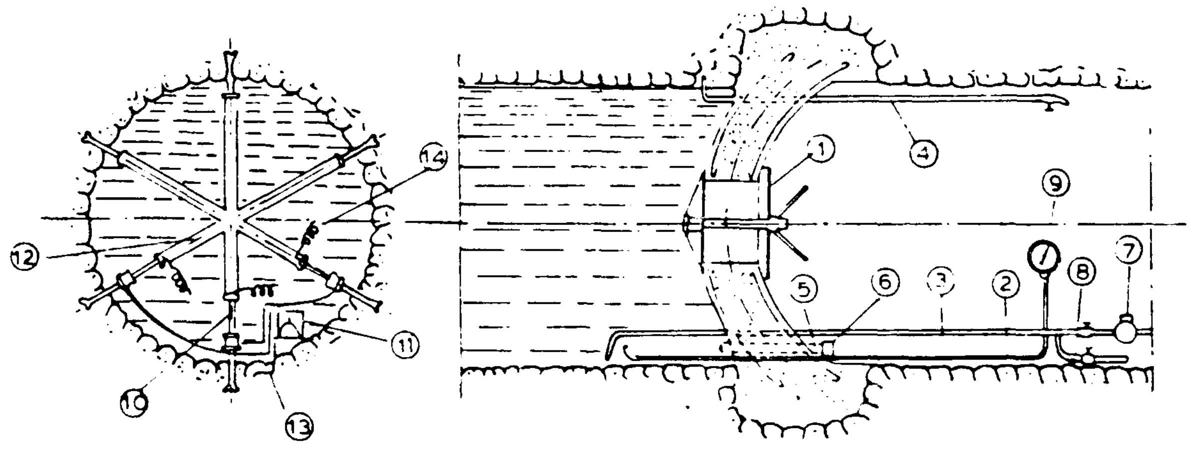
4. Loading equipment to apply the hydraulic pressure to the inner face of the lining including:

*Numbers refer to NOTES at end of the text.



(a) (b)

(A)



1. manhole
2. pressure gauge tube.
3. water inlet.
4. air outlet
5. cable admission tube
6. cable tube seal
7. water meter.
8. water outlet
9. pressure gauge.
10. vibrating meter
11. air pressure equalising chamber.
12. invar rods
13. air pressure equalising tube.
14. cable

(B)

Figure 1. Example Pressure Chamber.

(a) A hydraulic pump capable of applying the required pressure and of holding this pressure constant within 5% over a period of at least 24 hr. together with all necessary hoses, connectors and fluid.[2]

(b) Water seals to contain the pressurized water behind the bulkhead and along any vulnerable seams and boundaries of the membrane. Special water seals are also required to allow the passage of extensometer rods through the lining and instrumentation lines through the bulkhead. Pressurized water should not be allowed to escape into the rock since this may greatly affect the test results.

5. Load measuring equipment comprising one or more hydraulic pressure gages or transducers of suitable range and capable of measuring the applied pressure with an accuracy better than ±2%.

6. (a) Displacement measuring equipment to monitor rock movements radial to the tunnel with a precision better than 0.01 mm. Equipment may be extensometers, vibrating wire deflectometers, joint meters, or multipurpose strain meters. Changes in tunnel diameter have been measured with spring-loaded invar wires using LVDT's to sense. All measuring systems must be waterproof and designed to stand the pressure to which they are to be subjected.

Procedure

7. Preparation

(a) The test chamber location is selected taking into account the rock conditions, particularly the orientation of the rock fabric elements such as joints, bedding, and foliation in relation to the orientation of the proposed tunnel or opening for which results are required.

(b) The test chamber is excavated to the required dimensions.[1,3] Where possible, the dead end of the tunnel serves as one end of the test chamber, thus eliminating the need for a second bulkhead.

(c) The geology of the chamber is recorded and specimens taken for index testing as required.

(d) Extensometer holes are accurately marked and drilled, ensuring no interference between loading and measuring systems. Directions of measurement should be chosen with regard to the rock fabric and any other anisotropy.

(e) The bulkhead is installed.

(f) The chamber is lined with flexible, impermeable membrane. To protect this lining from sharp irregularities, the wallrock is first covered with thin reinforced concrete walling, gypsum board, or plaster. Rubber membrane may be stapled and vacuumed into place and the whole sprayed with rubber.

(g) Measuring equipment is installed and the equipment is checked. For multiple position extensometers the deepest anchor may be used as a reference provided it is situated at least 2 chamber diameters from the lining. Alternatively the measurements may be related to a rigid reference beam passing along the axis of the chamber and anchored of not less than 1 chamber diameter from either end of the chamber.

(h) Check water seals for leakage, if necessary by filling and pressurizing the chamber. Leaks are manifested as anomalous pressure decay and visible seepage through the bulkhead.

8. Testing

(a) The test is carried out in at least three loading and unloading cycles, a higher maximum pressure being applied at each cycle.[2]

(b) For each cycle the pressure is increased at an average rate of 0.05 MPa/min to the maximum for the cycle, taking not less than 3 intermediate sets of load-displacement readings in order to define a set of pressure-displacement curves.

(c) On reaching the maximum pressure for the cycle the pressure is held constant (±2% of maximum test pressure) recording displacements as a function of time until approximately 80% of the estimated long-term displacement has been recorded. Each cycle is completed by reducing the pressure to near-zero at the same average rate, taking a further three sets of pressure-displacement readings.

(d) For the final cycle the maximum pressure is held constant until no further displacements are observed. The cycle is completed by unloading in stages taking readings of pressure and corresponding displacements.

(e) The test equipment is then dismantled.[3]

Calculations

9. (a) The value of E is calculated from

$$E = \frac{P_i a^2}{r(U_r)}(1 + \nu)$$

where:

E = modulus of elasticity,

P_i = internal pressure,

a = radius to rock face - assuming circular chamber,

r = radius to point here deflection is measured,

U_r = change in radius due to pressure, and

ν = Poisson's ratio

and rock is regarded as linearly elastic.

(b) Where only surface deflections are measured, the equation is reduced to:

$$E = \frac{P_i a}{U_r} (1 + \nu)$$

Reporting

10. The report should include the following:

(a) Drawings, photographs, and detailed description of the test equipment, chamber preparation, lining, and testing.

(b) Geological plans and section of the test chamber showing features that may affect the test results.

(c) Tabulated test observations together with graphs of displacement versus applied pressure and displacement versus time at constant pressure for each of the displacement measuring locations.

(d) Transverse section of the test chamber showing the total and plastic displacements resulting from the maximum pressure. The orientations of significant geological fabrics should be shown on this figure for comparison with any anisotropy of test results. Calculated moduli should be shown also.

NOTES

[1]The recommended diameter is 2 m., with a loaded length about 5 m. The chamber should be excavated with as little disturbance as possible. Material disturbed by blasting may need to be removed since it tends to produce moduli lower than found at depth. However, blast effects are representative if the test results are applied directly as a "model" test to the case of a blasted full-scale tunnel.

[2]Maximum test pressure varies from 1.5 to 4.0 MPa.

[3]To assess the effectiveness of grouting, a second, adjacent chamber may be prepared. Grouting is carried out after completion of testing in the ungrouted chamber, and the equipment is then transferred to the grout chamber.

References

1. Clark. G. B., "Deformation Moduli of Rocks," in Testing Techniques for Rock Mechanics, Special Technical Publication 402, American Society for Testing and Materials, 1966, pp. 133-172.

2. Stagg, K. G., "In Situ Tests on the Rock Mass," in Rock Mechanics in Engineering Practice, John Wiley & Sons, New York, 1968, pp. 125-156.

3. Lama, R. D., and Vutukuri, V. S., Handbook on Mechanical Properties of Rock Vol. III, Trans Tech Publications, 1978, 406 pp.

4. 'nternational Society for Rock Mechanics, "Suggested Method for Measuring Rock Mass Deformability Using a Radial Jacking Test," International Journal of Rock Mechanics and Mining Sciences, V. 16, 1979, pp. 208-214.

RTH 362-89

PRESSUREMETER TESTS IN SOFT ROCK

1. Scope

The pressuremeter test consists of lowering an inflatable cylindrical probe into a predrilled borehole, expanding the probe laterally against the borehole wall, and recording the increase in size of the probe and associated pressure within the probe. This method covers the procedure for testing in soft rocks.

2. Principle of the Method

The pressuremeter probe is placed in the ground by lowering it into a predrilled hole. Once the probe is placed at the desired depth, the pressure in the probe is increased in equal increments and the associated increase on probe volume is recorded. The test is terminated if yielding in the rock becomes large. This procedure is repeated at the desired depth intervals but not closer than the length of the probe to the previously tested zone. A pressure-volume curve is plotted and a pressuremeter modulus is calculated.

3. Apparatus

3.1 The pressuremeter consists of two basic components: the probe and the pressure regulator-volumeter. Various sizes of probes are available to accommodate different borehole diameters. The probe consists of a light, flexible inner sheath and heavy, durable outer sheath, as shown schematically in Fig. 1. The inner sheath is pressurized with a liquid (water) through ports in the brass cylinder. The outer sheath is pressurized with a gas (normally dry nitrogen) through ports at each end of the cylinder. During testing, the outer sheath is kept at a pressure slightly less than that within the inner sheath. Normally, a pressure differential of 30 to 45 psi is maintained between sheaths since differences greater than 45 psi could possibly cause a rupture of the inner sheath. The bursting strength of the outer sheath depends on the deformability of the material being tested, but pressures in the range of 1000 to 1500 psi can often be obtained unless the surrounding medium has deformed excessively.

3.2 The purpose of the double-sheath arrangement is to simulate plane strain conditions. All volume change measurements are made within the inner sheath, although pressure is distributed along the entire length of the outer sheath; thus, end effects are greatly reduced.

3.3 The probe pressure is controlled by the pressure regulator-volumeter. The change of volume of the probe caused by the applied probe pressure is also monitored by this device. The probe is connected to the pressure regulator-volumeter by means of a coaxial tube, the inner tube being filled with liquid and the outer tube with gas.

4. Calibration

4.1 The probe must be calibrated to correct for its compressibility and inertia (Fig. 2). The compressibility of the sheaths, the fluid, and the co-axial tubing is determined by placing the probe into a rigid container, such as a pipe, and measuring the pressure-volumeter relationship. During a field test, the volume increase caused by the compressibility (V_p) of the probe system is deducted from that recorded by the volumeter at the corresponding field test pressure.

4.2 The inertia of the system is determined by inflating the instrument with no confining pressure and again determining the pressure-volume relationship. The pressure (P_p) required to inflate the probe to a given volume under no confinement can then be deducted from the (field) recorded pressure (plus pressure due to the head of water), which results in the true pressure exerted on the borehole wall during a field test.

4.3 Corrections for temperature changes and head losses due to circulating fluid are usually small and may be disregarded in routine tests.

4.4 Hydrostatic pressure (P_y) existing in the probe due to the column of fluid in the testing equipment must be determined before each test. This is accomplished by measuring the test depth (H) and multiplying the unit weight of the test fluid by the distance from the probe to the pressure gauge. This pressure must be added to the pressure readings obtained on the readout device.

5. Procedure

5.1 Drilling of the borehole must be performed in such a manner as to cause the least possible disturbance to the walls of the borehole and produce an adequate hole diameter for testing. The hole is advanced to the test level and cleaned of any debris or cuttings.

5.2 With the probe still at the surface, the fluid circuit valve open, and without applying pressure, an accurate setting of the zero volume reading (V_o) is accomplished by adjusting the water level in the instrument to zero. The volume circuit is then closed to prevent any further change in the volume of the measuring circuit. The probe is lowered to the test depth in this condition. Failure to close the valve will result in probe expansion as it is lowered into the hole. The test depth is determined as the depth to the midpoint of the probe.

5.3 Once the probe is positioned, the volume circuit is opened and the probe allowed to equalize under the hydrostatic head. Since the probe and inner coaxial tube are initially water-filled, a pressure equal to the head of water is exerted on the borehole walls at the beginning of each test. During loading, the pressure is increased in approximately 30-psi increments by controlling the pressure regulator valve. Volume measurements are recorded at lapsed times of 15, 30, and 60 sec after each pressure increase. The 60-sec readings are used for the modulus calculations. Typically, relatively large volume changes occur at low pressures as the probe is seated against the borehole wall; thus, the test usually indicates hardening response until the seating pressure p_o is reached (Fig. 3). This is followed by an essentially linear response range up to some pressure p_e. Above p_e, the curve again becomes nonlinear, but softening, as the material around the borehole begins to fail.

5.4 Once the maximum loading has been reached, or upon reaching the maximum expanded volume of the probe, the test is terminated; the probe is deflated to its original volume and withdrawn or repositioned in the hole at the next test depth. Cyclic testing may be performed when required by alternately inflating and deflating the probe.

6. Calculation

6.1 Calculate the pressure transmitted to the rock by the probe from the pressure readings as follows:

$$P = P_g + P_\gamma - P_p$$

where

P = pressure exerted by the probe on the rock (psi)

P_g = pressure reading on control unit (psi)

P_γ = hydrostatic pressure between control unit and probe (psi)

P_p = pressure correction due to inertia of instrument (psi)

For determination of P_p see paragraph 4.2. The pressure P_γ shall be the hydrostatic pressure as follows:

$$P = H \times \gamma_t$$

where

H = vertical distance from probe to pressure gauge (ft)

γ_t = unit weight of measuring fluid in instrument (lb/ft^3)

6.2 Calculate the increase in volume of the probe from the volume readings. The corrected increase in volume of the probe is calculated as follows:

$$V = V_R - V_P$$

V = Corrected volume increase of probe (cm^3)

V_R = Volume reading on readout device (cm^3)

V_P = Volume correction (cm^3)

The volume correction, V_P, shall be determined as outlined in paragraph 4.1.

6.3 Plot the pressure-volume increase curve by entering the corrected volume of the ordinate and the corrected pressure on the abscissa. Connect the points by a smooth curve. This curve is the corrected pressuremeter test curve and is used in the determination of the test results (Fig. 3).

7. Modulus Interpretations

If it is assumed that the material surrounding the pressuremeter probe behaves in a linear elastic manner and that the theory of thick-walled cylinders is applicable, then the applied internal pressure increment p and tangential and radial stress increments $\Delta\sigma_\theta$ and $\Delta\sigma_r$, respectively, are related by

$$-\Delta\sigma_\theta = \Delta\sigma_r = \Delta p$$

for compression positive. The volumetric strain $\Delta v/v$ of a unit length of the borehole probe may be related to the radial strain ϵ_r in the material by

$$\frac{\Delta v}{v} = \frac{\pi (r + \Delta r)^2 - \pi r^2}{\pi r^2} \approx \frac{2\Delta r}{r} \approx 2\epsilon_r$$

where v is the average total volume per unit length of the deformed borehole. From Hooke's law

$$\varepsilon_r = \frac{1}{E} (\sigma_r - \nu\sigma_\theta)$$

where ν is Poisson's ratio and E is Young's modulus. Thus

$$E = \frac{v\Delta p}{\Delta v} 2(1 + \nu)$$

or

$$G = \frac{E}{2(1 + \nu)} = \frac{v\Delta p}{\Delta v}$$

The above equations may be used to interpret the Young's modulus, E, from the pressuremeter tests only if Poisson's ratio is independently determined or assumed. The last equation indicates that the pressuremeter results can be used as a direct measure of shear modulus, G.

8. Limitations

The accuracy of the pressuremeter test is dependent in part upon the stiffness of the material being tested. For stiffer materials, the determination of the instrument compressibility (e.g., apparent change in volume per unit pressure when the probe is completely restrained from external volume change) is important since an increasing proportion of the measured volume response results from the instrument and not the material. The effect of any uncertainties in the instrument stiffness S_I of the pressuremeter (S_I is the reciprocal of the compressibility) on the predicted shear modulus is shown on Fig. 4. As seen in the figure, shear modulus calculations for materials in which the ratio of the instrument stiffness to shear modulus (S_I/G) is small will be less accurate than when the ratio is large, if the instrument compressibility at the time of the test is not accurately known.

9. Report

For each pressuremeter test a data form similar to Fig 5 shall be used.

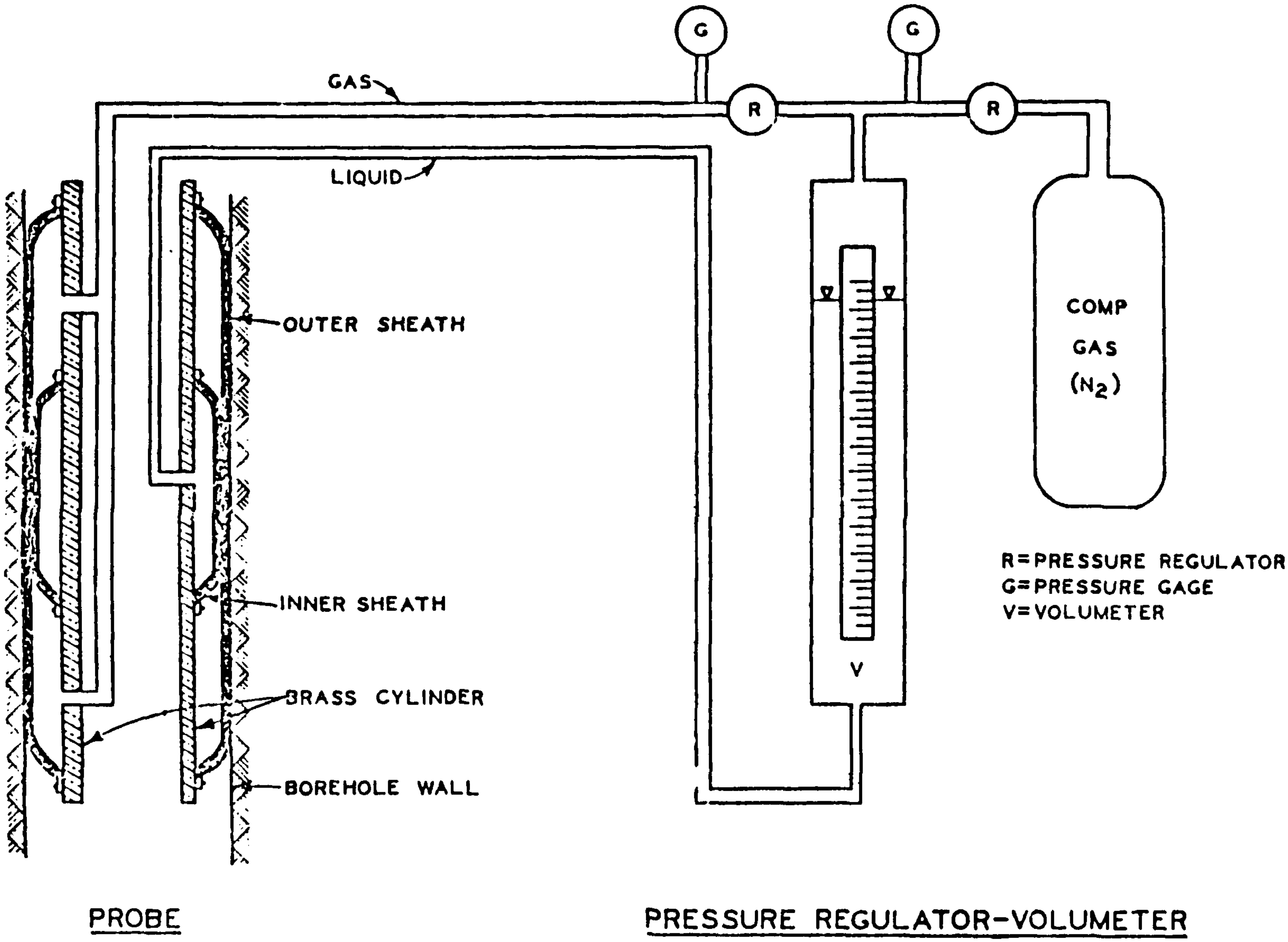


Fig. 1. Pressuremeter Schematic

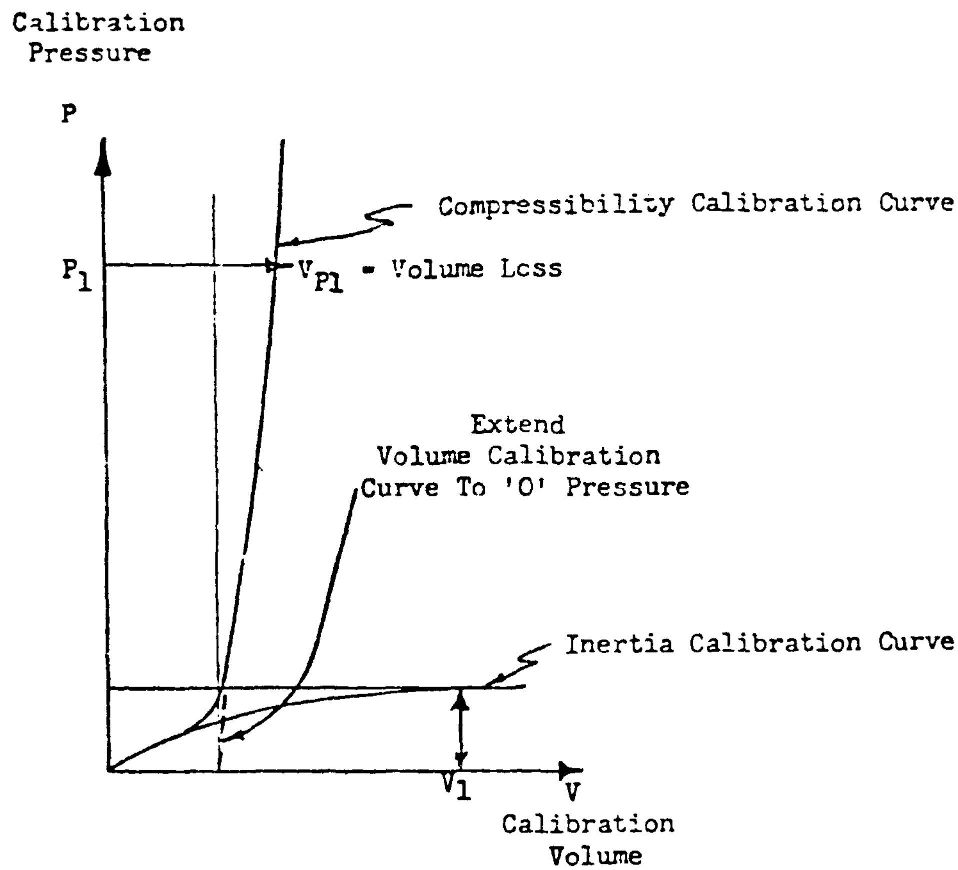


Figure 2. Calibration for Volume & Pressure Corrections.

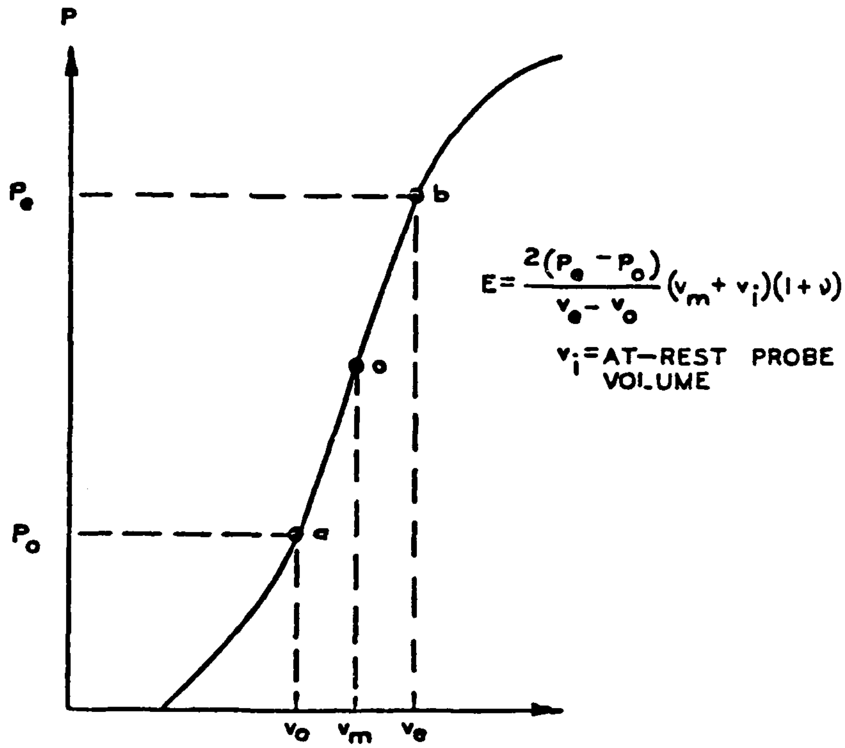


Fig. 3 Idealized pressure-volume relationship

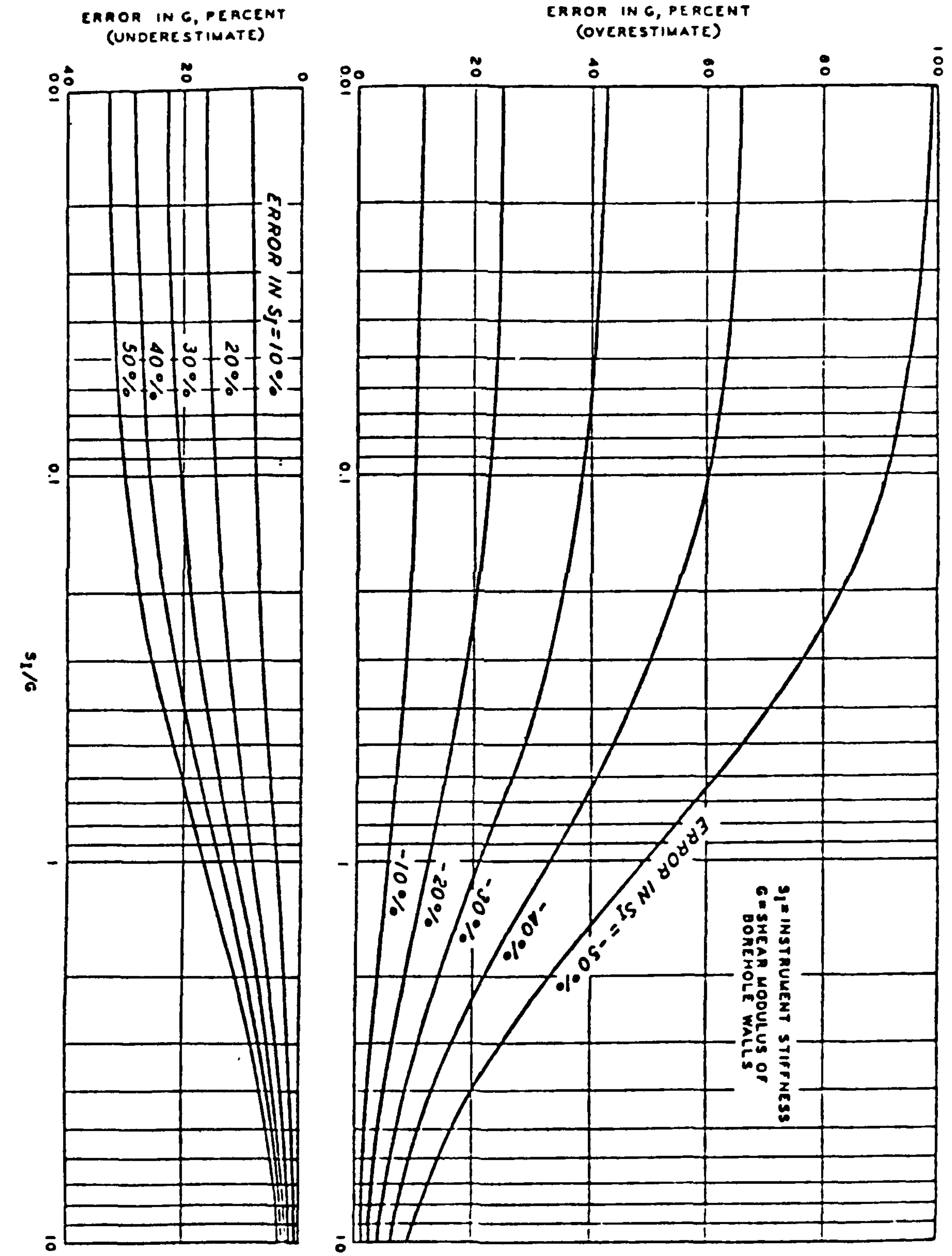


Fig. 4 Uncertainty in predicted shear modulus versus instrument stiffness

SITE	DATE	BORING NO.	DEPTH	OPERATOR	PAGE ___ of ___
PROBE SIZE & TYPE	REMARKS				

TIME	LOAD PSI	VOLUME - CM3			CREEP (30-60) SEC	DIFF (60 SEC)	REMARKS
		15 SEC	30 SEC	60 SEC			

Fig 5: Data Sheet

SUGGESTED METHOD FOR DETERMINING ROCK MASS DEFORMABILITY USING A HYDRAULIC DRILLHOLE DILATOMETER

Scope

1. (a) This test determines the deformability of a rock mass by subjecting a section of drillhole to hydraulic pressure and measuring the resultant wall displacements.[1] Elastic moduli and deformation moduli are calculated in turn.

(b) The results are employed in design of foundations and underground construction.

(c) The dilatometer is self-contained and tests are relatively inexpensive compared to similar tests at a larger scale.[2] Also, the wall is damaged only minimally by the drilling of the hole and usually remains representative of the undisturbed rock condition. These advantages, however, come at a sacrifice of representation of the effects of joints and fissures which are usually spaced too widely to be fully represented in the loaded volume around the drillhole.

(d) This method reflects practice described in the references at the end.

(e) Another type of dilatometer for drillholes transmits pressure to the rock through mechanical jacks: See RTH-368.

Apparatus

2. Drilling equipment to develop access hole in a given orientation without disturbing the wallrock.[3]

3. A drillhole dilatometer similar to that in Figure 1, which consists of:

(a) stainless steel cylinder

(b) Rubber (neoprene) jacket surrounding the steel cylinder and sealed at both ends to confine pressurized fluid between the jacket and the steel cylinder

(c) Two end plugs containing pipes, electric wires, and relief valves.

(d) Linear differential transformers oriented along different diameters of the drillhole (commonly four at 45-degree sectors). Deflections as large as 5 mm are measured, commonly with accuracy of about 0.001 mm.

4. Hydraulic pump capable of applying the required pressure and of holding this

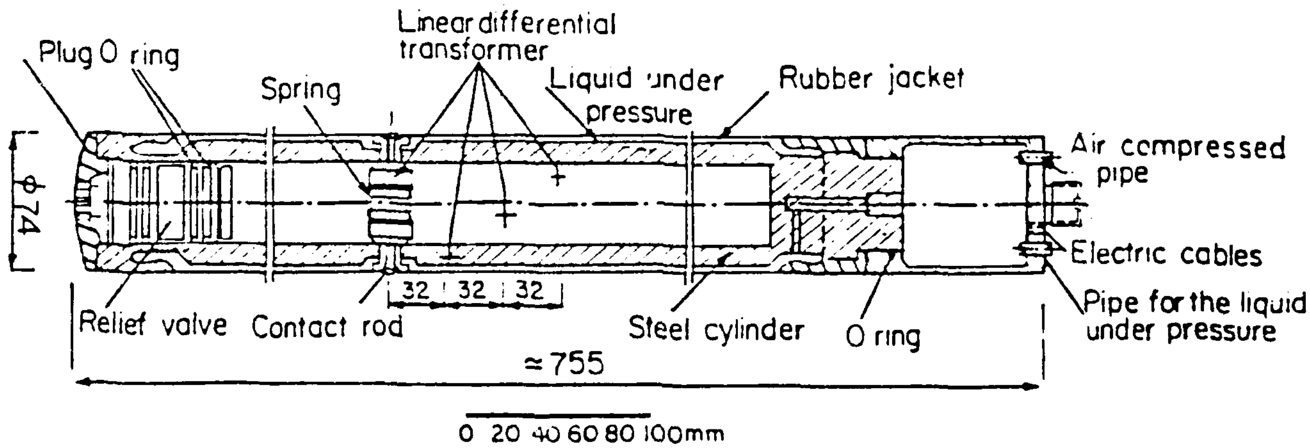


Figure 1. Example hydraulic drillhole dilatometer.

pressure constant within 5% for a period of at least 2 hr. together with all necessary hoses, connectors, and fluid.

5. Hydraulic pressure gages or transducers of suitable range and capable of measuring the applied pressure with accuracy better than 2%.

Procedure

6. Preparation

(a) The positions for testing are planned with due regard to the location of drilling station and the rock conditions to be investigated. The effects of geological structure and fabric are particularly important.

(b) The hole is drilled and logged. The log is studied for possible modifications in positions for testing. Multiple testing positions in one hole should be separated by at least 0.5 m.

(c) The dilatometer is assembled and inserted into the hole, commonly using an attachable pole to position and rotate and taking special care with trailing lines.

(d) The rods of the linear differential transformers are seated against the wall.

7. Testing

(a) The dilatometer is pressurized in increased stages, with pressure released between stages. Typically, the stage pressures are 25, 50, 75, and 100 percent of the planned maximum of the complete test.[4]

(b) Pressure is increased at a rate of 0.5 MPa/min. or less.

(c) On reaching the planned pressure for the stage, the pressure is held constant for at least 1 min. to detect and define nonelastic deformation. Each stage is completed by releasing pressure at a prescribed rate up to 0.5 MPa/min.

(d) The test history is documented with no less than four sets of measurements during pressure increase and two during pressure decrease. Supplementary notes are necessary to describe any complexities not otherwise revealed (such as nonelastic deformation).

(e) The pressure is released and fluid withdrawn. The rods of the linear differential transformers are retracted away from the wall and the dilatometer is removed from the hole.

Calculations

8. For presumed quasi elastic conditions, an elastic modulus is calculated from

$$E = \frac{p\ a}{U_r}(1 + \nu)$$

where

p = fluid pressure

a = hole radius

U_r = change in radius

ν = Poisson's ratio

Where permanent deformation (nonelastic) occurs also, that portion of U_r should be excluded from the equation.

Reporting

9. The report should include for each test or all tests together the following:

(a) Position and orientation of the test, presented numerically, graphically, or both ways.

(b) Logs and other geological descriptions of rock near the test. The structural details are particularly important.

(c) Tabulated test observations together with graphs of displacement versus applied pressure and displacement versus time at constant pressure for each of the displacement measuring devices (e.g. linear differential transformers).

(d) Transverse section of hole showing the displacements resulting from the pressure in all orientations tested. Calculated moduli are indicated also.

Notes

[1]In very deformable rocks, the diametral strain can also be determined indirectly from changes in volume of pressurizing fluid. See RTH-362 for that procedure.

[2]See RTH-361, -366, and -367 for similar test at tunnel scale.

[3]Diamond core drilling is recommended for obtaining the necessary close tolerance when using dilatometer only slightly smaller than the hole and displacement measuring devices with very limited stroke.

[4]Typically, the maximum pressure is about 15 MPa.

References

Lama, R. D., and Vutukuri, V. S., Handbook on Mechanical Properties of Rock, Vol.III, TransTech Publications, 1978, 406 pp.

Stagg, K. G., "In Situ Tests on the Rock Mass," in Rock Mechanics in Engineering Practice, John Wiley & Sons, New York, 1968, pp. 125-156.

RTH-364-89

SUGGESTED METHOD FOR DETERMINING ROCK MASS DEFORMABILITY BY LOADING A RECESSED CIRCULAR PLATE

Scope

1. (a) This test determines the deformability characteristics of a rock mass by loading a flat surface at the end of a drill hole or other recess and measuring the resultant displacement of that surface. Elastic or deformation moduli are calculated as well as time dependent (creep) properties.

(b) Several depth horizons may be tested from the same setup using a large-diameter drill to advance between tests. The direction of loading necessarily coincides with the drill hole axis, usually near-vertical, so that no information can be obtained regarding rock anisotropy.

(c) Plate bearing tests are commonly used to provide information for the design of foundations.

(d) This method is a modification of practice suggested by the International Society for Rock Mechanics. See Reference. That previous version provides details not included here.

(e) Another method, differing in the loading system, is available for comparison and consideration in RTH-365.

Apparatus

2. Equipment for drilling, cleaning, and preparing a test hole at least 500 mm[1*] in diameter. The hole may need casing. A reamer and other special tools are useful in flattening the bearing surface (± 5 mm) perpendicular to the hole axis ($\pm 3^{o}$) and for removing water and debris.

3. Core drill for taking samples to at least 3 m below the bearing surface, the diameter to be less than 10% that of the bearing plate.

* Numbers refer to NOTES at the end of the text.

4. Circular bearing plate of diameter at least 500 mm and sufficiently rigid to distort by not more than 1 mm under the test conditions (Figures 1 and 2)[2].

5. Hollow loading column to transmit the applied force centrally to the test plate without detrimental buckling.

6. Hydraulic jack and reaction anchors such that:

(a) Loads can be varied throughout the required range and can be held constant to within 2% of a selected value for a period of at least 24 hr.

(b) The travel of the jack is greater than the sum of anticipated displacements of the plate and reaction beam.

(c) Reaction anchors are located further than 10 test hole diameters from the bearing plate.

7. Equipment (load cell or proving ring) to measure the applied load with an accuracy better than ± 2% of the maximum reached in the test.

8. Equipment to measure axial displacement of the plate[3] with accuracy better than 0.05 mm. The reference anchors should be at least 10 test hold diameters from the loading plate and reaction anchors.

Procedure

9. Preparation

(a) The site is selected to allow testing at the actual foundation level with loading in the direction of foundation loading. Alternatively rock considered typical of anticipated conditions may be tested. Attention should be given to locations for reaction and reference anchors and to ground water and other influential conditions.

(b) Test hole and anchor holes are drilled and logged. The test hole is cased if necessary for stability throughout the test.

(c) Exploratory core is taken to a depth of at least 3 m below the proposed test horizon, and the choice of horizon confirmed or modified.

(d) Ground water encountered in the test hole should be lowered by pumping from well points surrounding the test area or otherwise during installation of the bearing plate.

(e) The bearing surface is trimmed, one or more layers of mortar or plaster are placed, and the bearing plate installed before the last layer has set. The delay between excavation of the surface and installation of the plate should not exceed 12 hr.[4]

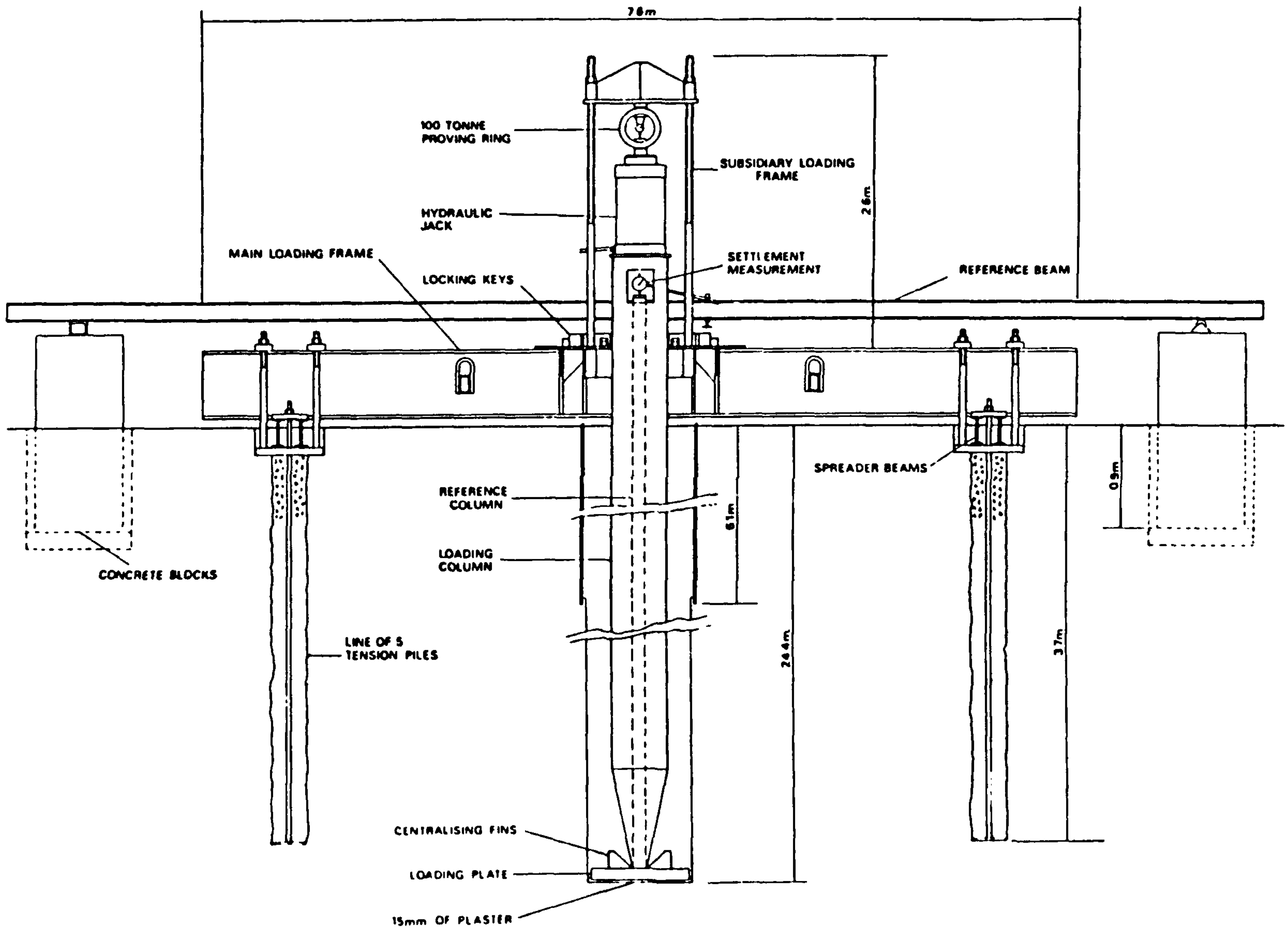


Figure 1. Example plate-loading equipment.

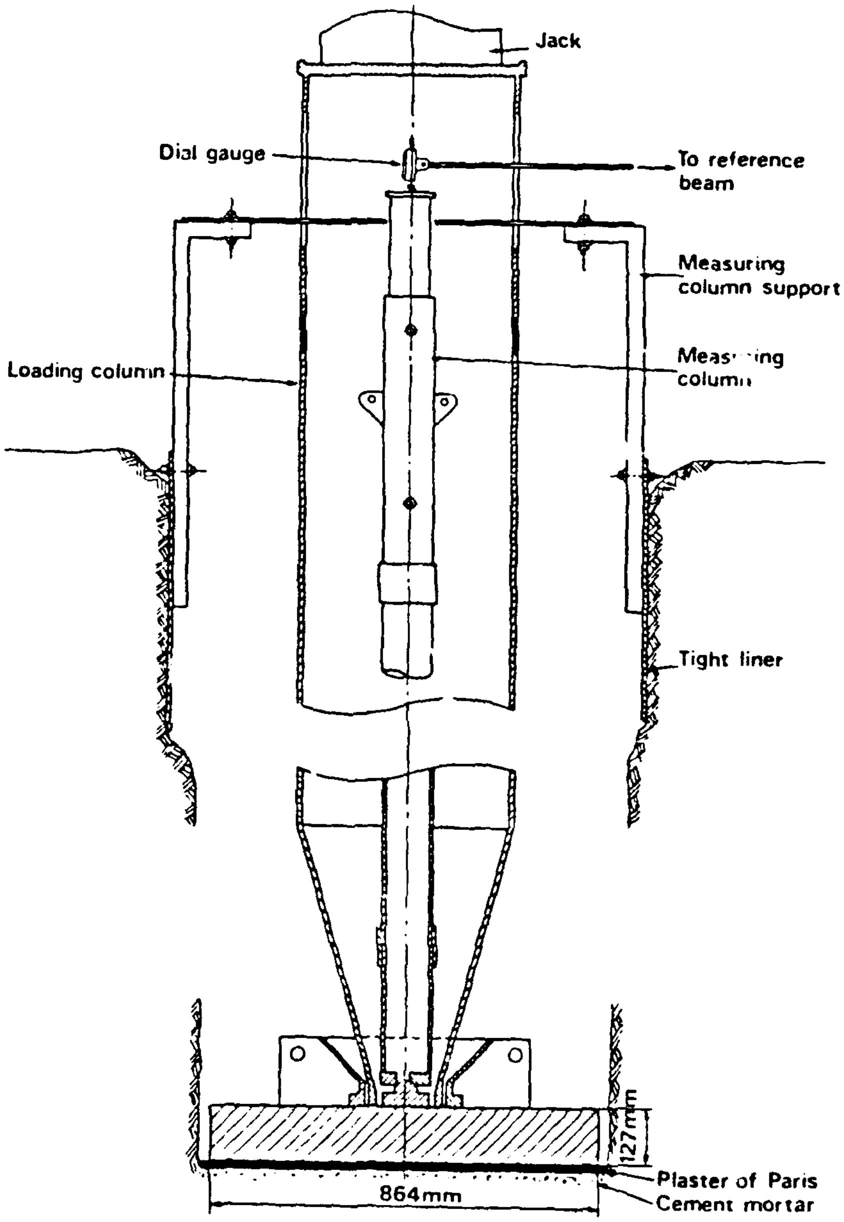


Figure 2. Details of plate test equipment.

(f) Reaction and reference anchors are installed 10 or more diameters away and the equipment assembled and checked. A small seating load (approximately 5% of the maximum test value) is applied and held until the start of testing.

(g) The water table should be allowing to return to its normal elevation before the start of testing.

10. Testing

(a) With the seating load applied, load and displacement should be observed and recorded over a period not less than 48 hours to establish datum values and to assess variations due to ambient conditions.[5]

(b) Loads and load increments to be applied during the test should be selected to cover a range 0.3-1.5 q_o, where q_o is the stress intensity produced by the proposed structure.[6]

(c) Load is increased in not less than five approximately equal increments to a maximum of approximately 1/3 the maximum for the test. At each increment the load is held constant (±3%) and plate displacement recorded intermittently until it stabilizes.[6] The procedure is continued for decreasing load increments until the seating load is again reached.

(d) Procedure (c) is repeated for maximum cycle loads of approximately 2/3 and 3/3 the maximum for the test.

(e) The equipment is removed from the test hole and further tests may be carried out on deeper horizons by re-drilling in the same hole.

Calculations

11. (a) Graphs are plotted of incremental settlement (or uplift in the case of unloading) against the logarithm of time (Figure 3).

(b) Bearing pressure versus settlement curves are plotted for each test (Figure 4).

(c) Deformqtion modulae may be determined using tangents to the pressure-settlement curves as follows:

$$E = \frac{dq}{d\rho} \frac{\pi}{4} D(1 - \nu^2).I_c$$

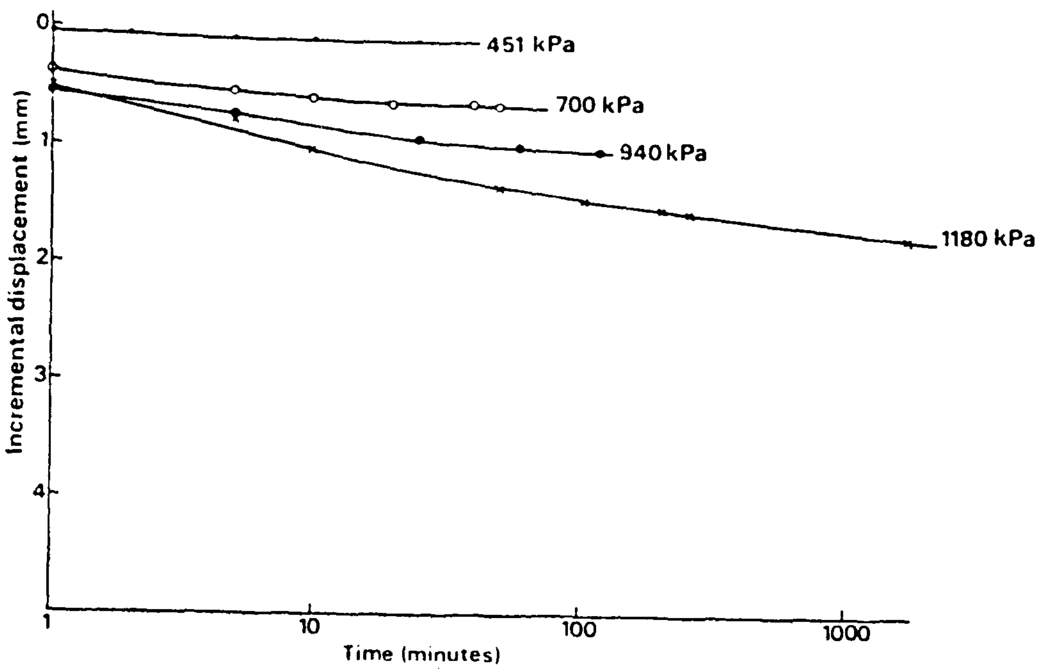


Figure 3. Typical relations between small displacement and time for various load intensities.

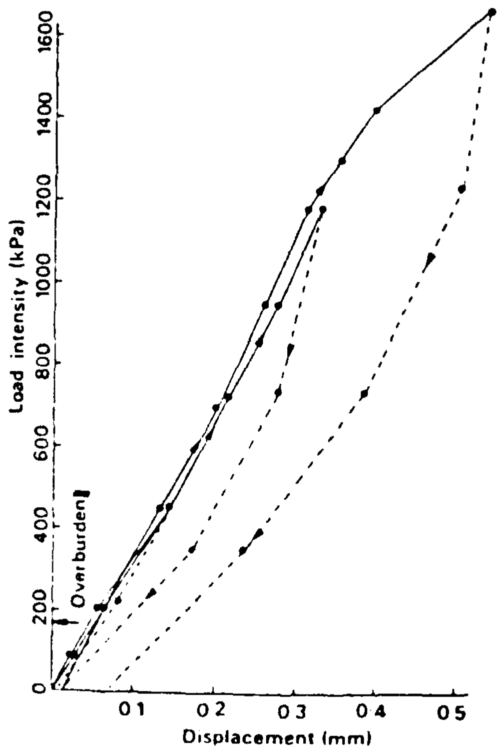


Figure 4. Example plate test results.

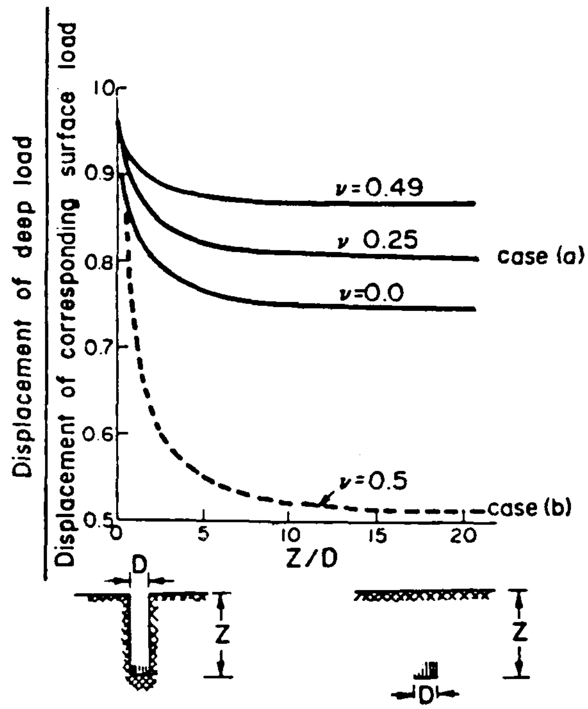


Figure 5. I_c factors for deep loading.

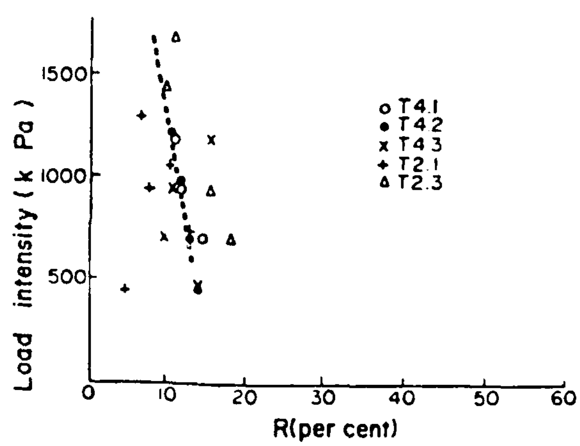


Figure 6. Relation between load intensity and creep ratio R.

where

q is applied pressure

p is settlement

D is plate diameter

ν is Poisson's ratio

I_c is a depth correction factor from Figure 5

(d) A time-dependent parameter R (known as the creep ratio) is determined for each load increment. The parameter R is defined as the incremental settlement per cycle of log time divided by the total overall settlement due to the applied pressure. The relationship between R and applied pressure may be presented graphically (Figure 6).

Reporting

12. The report should include the following

(a) Diagrams and detailed descriptions of the test equipment and methods used for drilling, preparation, and testing.

(b) Plans and sections showing the location of tests in relation to the generalized topography, geology, and ground-water regime.

(c) Detailed logs and descriptions of rock at least 3 m above and below each tested horizon.

(d) Tabulated test results, graphs of displacement versus time for each load increment, and graphs of load versus displacement for the test as a whole.

(e) Derived values of deformability parameters, together with details of methods and assumptions used in their derivation. Variations with depth in the ground should also be shown graphically as 'deformability profiles' superimposed on the log of the test hole.

Notes

[1] The test hole should preferably be of sufficient diameter to allow manual inspection, and preparation of the bearing surface. Otherwise preliminary coring is needed to provide adequate samples of ground conditions.

[2] Steel plate unreinforced by webs, should be at least 20 mm thick for a diameter of 500 mm.

[3]If required, the displacement of rock at any level below the bearing plate may be monitored, using rods passing through a hole in the center of the plate and rigidly anchored in the exploratory drillhole.

[4]Particularly when testing weaker rocks there will be rebound, loosening, and possibly swelling associated with excavation of the bearing surface and changes in ground water conditions.

[5]Small fluctuations in displacement are likely to result from changes in the ground water regime, temperature, and other environmental effects.

[6]At higher applied loads the displacement may not completely stabilize in a reasonable time; a criterion that readings should continue until the rate of displacement is less than 2% of the incremental displacement per hour may be used. This criterion may be modified to suit the purpose of the test. The final increment in any one cycle should be held for as long as practical if the displacement is still increasing.

Reference

International Society of Rock Mechanics, "Suggested Method for Field Deformability Determination Using a Plate Test Down a Borehole," International Journal of Rock Mechanics and Mining Sciences, v. 16, 1979, pp. 202 - 208

RTH 365-80

Authorized Reprint from
Special Technical Publication 554

Bureau of Reclamation Procedures for Conducting Uniaxial Jacking Tests

D. L. Misterek,[1] *E. J. Slebir,*[1] *and J. S. Montgomery*[1]

REFERENCE: Misterek, D. L., Slebir, E. J., and Montgomery, J. S., **"Bureau of Reclamation Procedures for Conducting Uniaxial Jacking Tests,"** *Field Testing and Instrumentation of Rock, ASTM STP 554,* American Society for Testing and Materials, 1974, pp. 35–51.

ABSTRACT: The Bureau of Reclamation's present method of conducting uniaxial jacking tests incorporates the desirable features and techniques developed over many years of testing. Successful tests yielding usable data are attributed to thorough pre-test geologic exploration; careful site preparation; special drilling and explosive excavation techniques; and well-planned procedures for installation and removal of specialized equipment. Non-engineering factors, such as good contractor-test team relationship, also play a critical role in a successful test program. The process where hydraulic flatjacks are used to apply desired loads to prepared rock surfaces within a tunnel is described. Instrumentation to measure resulting displacement is referenced within the rock mass and is independent of the loading system. Details of all phases of conducting the tests are presented. In addition, a discussion is given of some of the problems, solutions, and philosophies that evolved while tests were being planned or conducted. This discussion is of value to those conducting *in situ* jacking tests or those preparing standards for the tests.

KEY WORDS: rocks, uniaxial tests, geologic investigations, logging (recording), mapping, drilling equipment, instruments, installing, evaluation, tests, jacking.

The Bureau of Reclamation has performed a variety of *in situ* tests in recent years. The most frequently utilized, because of economy and usable data, is the uniaxial jacking test. This test determines how foundation rock reacts to controlled loading and unloading cycles and provides data on deformation moduli, creep, rebound, and set.

[1] Engineers and civil engineering technician, Rock Mechanics Section, Division of General Research, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colo. 80225.

Previously published articles [2,3] provide a general discussion of the uniaxial jacking test procedures together with some typical graphs and explanations of data obtained. In consonance with the purpose of this symposium, this paper updates that information and explains in more detail the sequential procedures utilized to accomplish the test.

Test Site Selection

Prior to selecting a test site location, all available surface and subsurface geologic data are compiled and analyzed. A three-dimensional portrayal of these data is prepared, either in plotted form or by construction of a plastic model.

Exploratory tunnels are generally driven into the rock mass to further investigate geologic conditions and to provide access to test site locations. Ideally, these tunnels are oriented parallel to the horizontal projection of the resultant thrust that the proposed structure will exert on the foundation. Test adits are driven from the tunnels so that test loads applied to surfaces of the adit will be oriented in the same direction as loads from the proposed structure. To provide additional data on possible variations in properties of the rock mass, test loads may also be applied normal and parallel to geologic structure. Therefore, sufficient mapping and probe drilling are performed to delineate the geology in the test adit. After reviewing all available information, the precise location of a test site is designated by a team of geologists and engineers who must determine that the test location is representative of conditions to be found in a significant portion of the rock mass. In this case, significance is determined by the relative influence on the deformability of total rock mass. That is, a small volume of relatively deformable material may be as significant to overall rock mass deformation as a much larger volume of more competent material. In making this evaluation, a number of factors must be considered. These include: (1) spatial orientation and stress intensity of the loads to be transmitted to the rock mass by the proposed structure, (2) the various types of material found in the rock mass and the relative volume and location of each, (3) spatial orientation of rock structure, that is, bedding, foliation, jointing, etc., and its relationship to applied loads from the structure, and (4) fracture density of the rock mass.

[2] Wallace, G. B., Slebir, E. J., and Anderson, F. A. in *Determination of the* In Situ *Modulus of Deformation of Rock, ASTM STP 477*, American Society for Testing and Materials, 1970, pp. 3–26.

[3] Wallace, G. B., Slebir, E. J., and Anderson, F. A. in *Eleventh Symposium on Rock Mechanics*, University of California, Berkeley, Calif., 16–19 June 1969, American Institute of Mining, Metallurgical, and Petroleum Engineers, 1970, pp. 461–498.

Preparation of the Test Site

After site selection, the area is prepared for testing by removing blast-damaged material using pneumatic chipping hammers and drills. Although only a 34.2 in. (87 cm) diameter area of rock is actually loaded during a test, an area approximately 5 ft (1.52 m) in diameter is prepared to reduce the restraining influence of the surrounding rock. If the rock cannot be removed with pneumatic tools and blasting is required, the wire-lath system, shown schematically in Fig. 1, has proved to be successful. Four wooden laths are installed in drill holes located approximately 4.5 ft (1.37 m) from the center of the area to be loaded. Wires are strung from lath to lath forming a square reference plane over the test area. Percussion-drilled blasting holes, 1⅝ in. (4.13 cm) in diameter, are driven to terminate in a common plane at a predetermined depth parallel to the wire-lath reference plane. The holes are spaced on 4 to 5 in. (10.16 to 12.7 cm) centers to ensure shearing of rock along a flat plane during blasting. Starting with the shallowest blasting holes, the test area is excavated in three to five separate shots using either 4 in. (10.16 cm) of 70 percent dynamite tamped at the bottom of each hole or several windings of detonating cord (18 in.) (44.7 cm) placed at the bottom of each hole and detonated with electric blasting caps. This procedure produced a test area with minimum blast damage and minor final cleanup.

Drilling for Instrumentation

After two diametrically opposite test areas have been prepared, a 20-ft (6.10-m) deep Nx (3-in. (7.6-cm) diameter) hole is drilled into the center of each area. Care must be exercised to ensure the two holes are aligned coaxially. This is essential for proper alignment of the test equipment and to allow accurate measurements to be made between the centroids of the loaded surfaces. The upper hole is drilled first to minimize debris entering the lower hole. After a hole is completed, a special bit is used to produce a 5-in. (12.7-cm) diameter counterbored recess at the collar of the hole. This step ensures a good coupling between the rock and the extensometer installed in the hole to measure deformation. Additional details are presented in the section on instrument installation. Figure 2 shows a test area with the instrument hole and counterbored recess complete.

Maximum core recovery with ensured correct orientation is important for the geologic and structural evaluation of the test site. To accomplish these objectives, two procedures are involved.

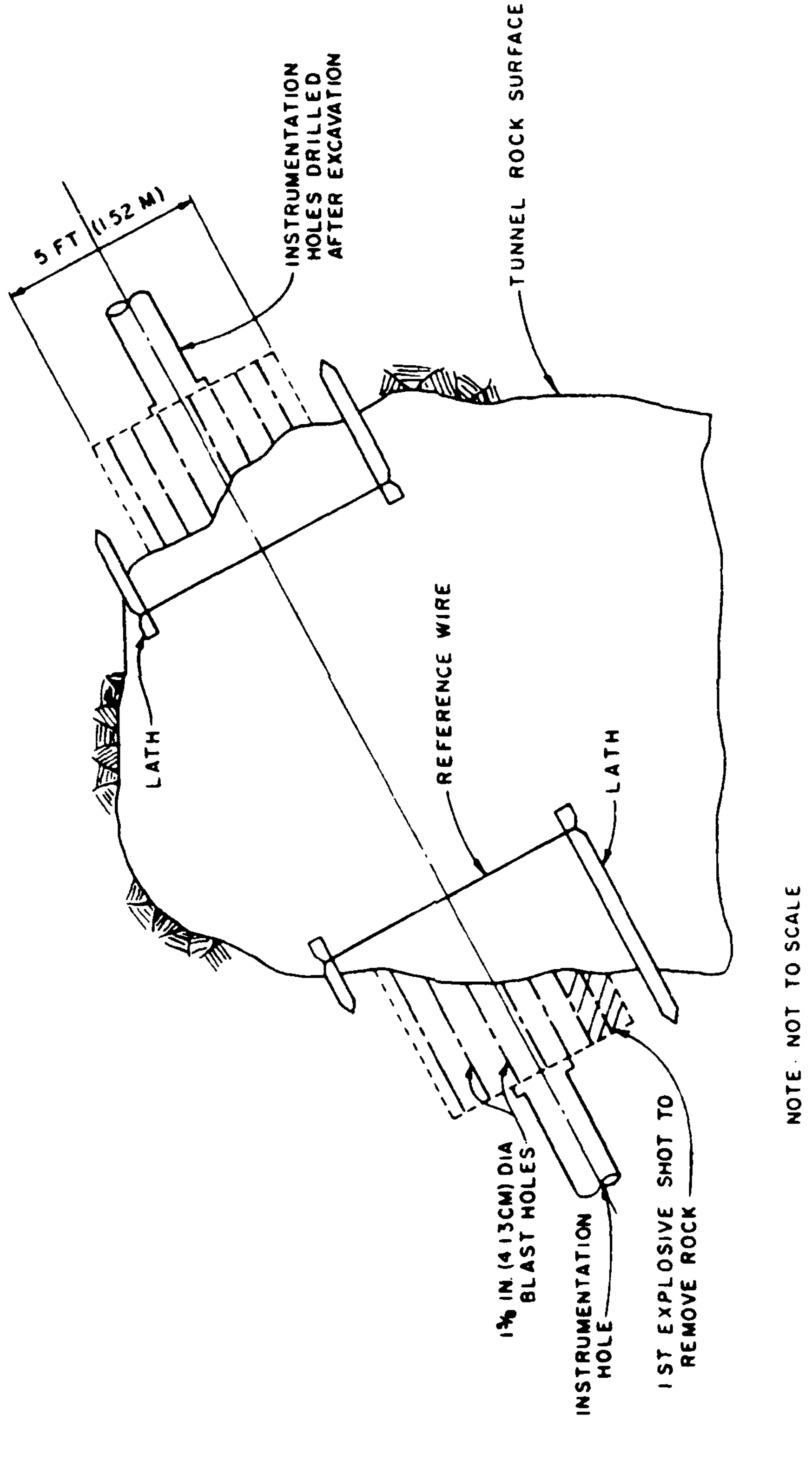


FIG. 1—*Wire lath system.*

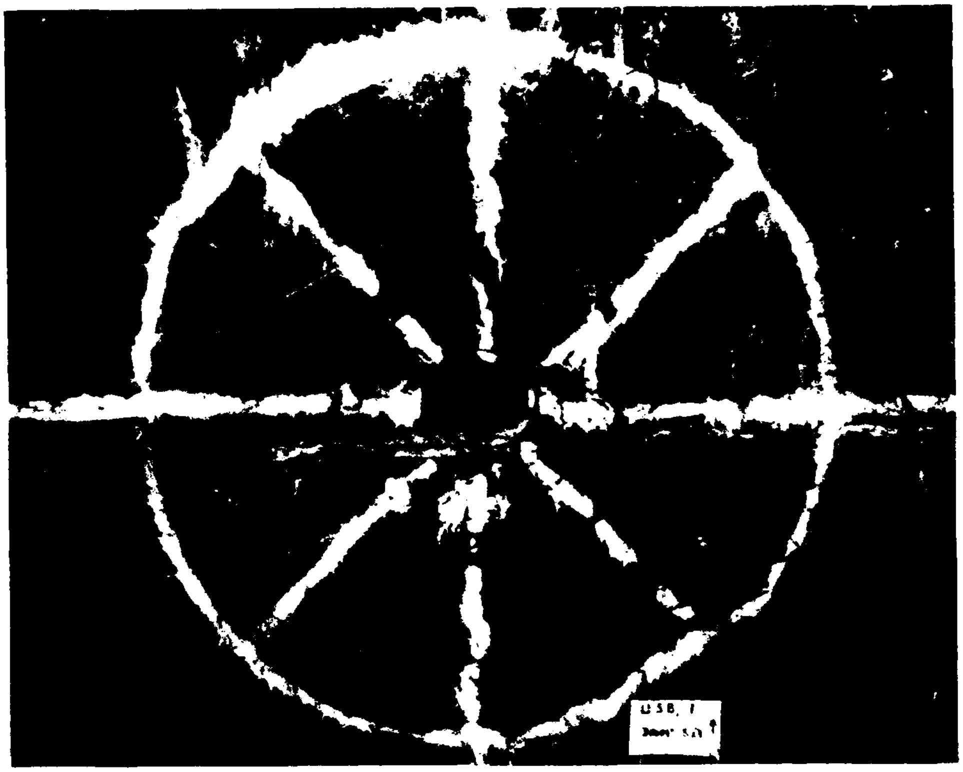


FIG. 2—*Instrumentation hole with counterbored recess. (Note the termination of blasting holes on the final rock surface.)*

The first procedure utilizes a split tube inner core barrel to obtain better quality core specimens. This inner barrel keeps the core from rotating while the outer barrel transfers torque and thrust to the diamond cutting bit. After a drill run, the split tube barrel is retrieved and placed in a horizontal position, the end fastening rings are removed, and half of the split inner tube is replaced by a cardboard split tube. The assembly is then rotated 180 deg so the cardboard half cylinder is on the bottom supporting the core. The remaining piece of the inner barrel is then removed.

The second procedure involves the use of a clay pot impression for orienting recovered core. At the end of a drill run, a special pot made of drill casing and loading poles is filled with oil base clay, oriented, and lowered to the bottom of the hole. An impression of the hole bottom is matched with the next core run.

Logging and Mapping at the Test Site

The core from the two instrumentation holes is logged to determine rock type, strike and dip of foliation, bedding and joints, whether breaks (joints) are mechanical or natural, weathering and alteration, plus any other pertinent features. After the core has been inspected, the drill holes are logged using a TV borehole camera. Prior to logging, the holes are washed and, if possible, filled with water to give a clearer picture. Borehole conditions observed during the TV examination are compared with core logs to verify the frequency, location and magnitude of significant geologic features. (*Note:* The Bureau of Reclamation already had the TV camera and appurtenant equipment for use in other work. A borescope-type device can be utilized for this phase of the work.) Next the prepared test areas are measured and mapped, and a geologic cross section through the test axis is prepared. Finally, the test site location in relation to the established survey of the tunnel is determined.

Measurement Locations

After comparing TV and rock core logs to verify existing geologic conditions, a composite drill hole log is assembled and used to select locations for instrumentation. Locations within each hole are selected to receive mechanically expandable anchors to serve as reference points for displacement measurements. Figure 3 shows some typical anchor locations for various geologic conditions. Some points are located to indicate strain in solid rock, others to span joint or fracture systems, and some to span lithology changes. Results of early tests [2,3] show that most of the significant portion of deformation occurs in the first several feet of rock nearest the applied load. It was also noted that displacement of the loaded surface is not always symmetrical with respect to the center of the loaded area; that is, displacement on one side of the center might be greater than on the other side. These two conditions led to the development of a semi-standard arrangement of anchor locations. Instead of utilizing seven anchors, the bottom three anchors have been replaced by a single anchor located within a few inches of the bottom of the hole and fitted with three metering rods. The reason for this procedure is presented in the section on testing. The remaining four anchors are normally located within the upper half of the drill hole. In general, careful logging of the borehole allows anchors to be placed in competent rock rather than within undesirable fractured or jointed zones.

Instrumentation Installation

After the anchor depths in each instrument hole are established, a 3-in. (7.6-cm) diameter stainless steel borehole collar sleeve is inserted into each hole. The outside flange of the sleeve fits into the counterbored

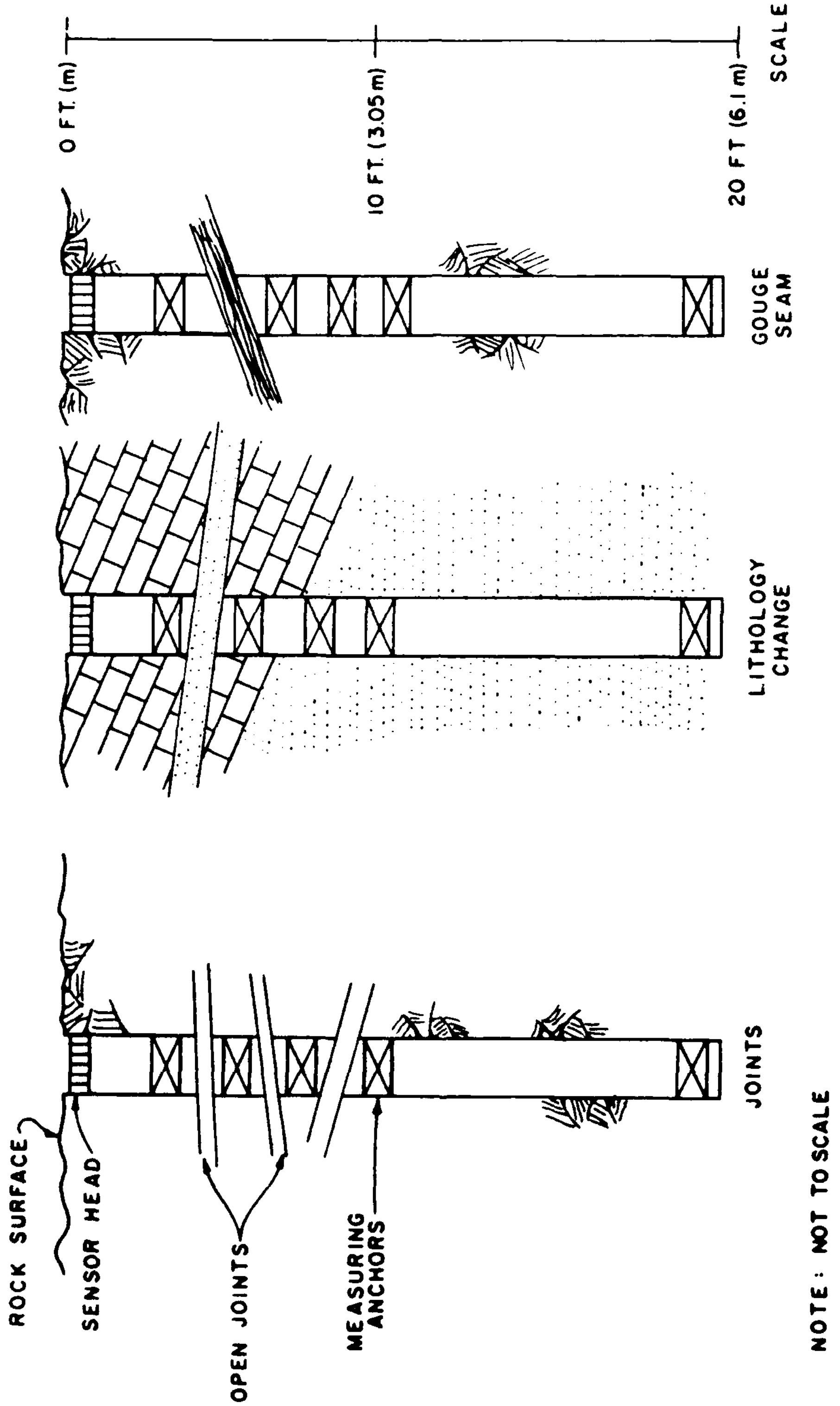


FIG. 3—*Typical anchor locations.*

recess previously described and is covered with mortar to secure it to the rock. Figure 4 shows a test area with the sleeve installed. After the mortar has set, a final inspection is made to ensure that the hole is clear of all obstructions. At this point, the Retrievable Borehole Extensometer (REX-7P) can be installed. The REX-7P is a seven-position extensometer, which, except for lead wires to the readout device, is installed entirely within the instrumentation hole. The instrument, developed by the Bureau of Reclamation, has been granted U.S. Patent No. 3,562,916. Detailed drawings, installation, and operating instructions are presented in the patent documents and will be summarized only briefly in this paper.

The instrument consists of anchors installed at depth within the borehole, a sensor head including seven linear variable differential transformers (LVDT) located at the collar of the borehole, and thin-wall stainless steel metering rods relating the movement of the anchors to that of the sensor head. The device has a sensitivity and repeatability of 100 $\pm$ μin. (0.00025 cm) and reliably measures rock movement to 200 $\pm$ μin.

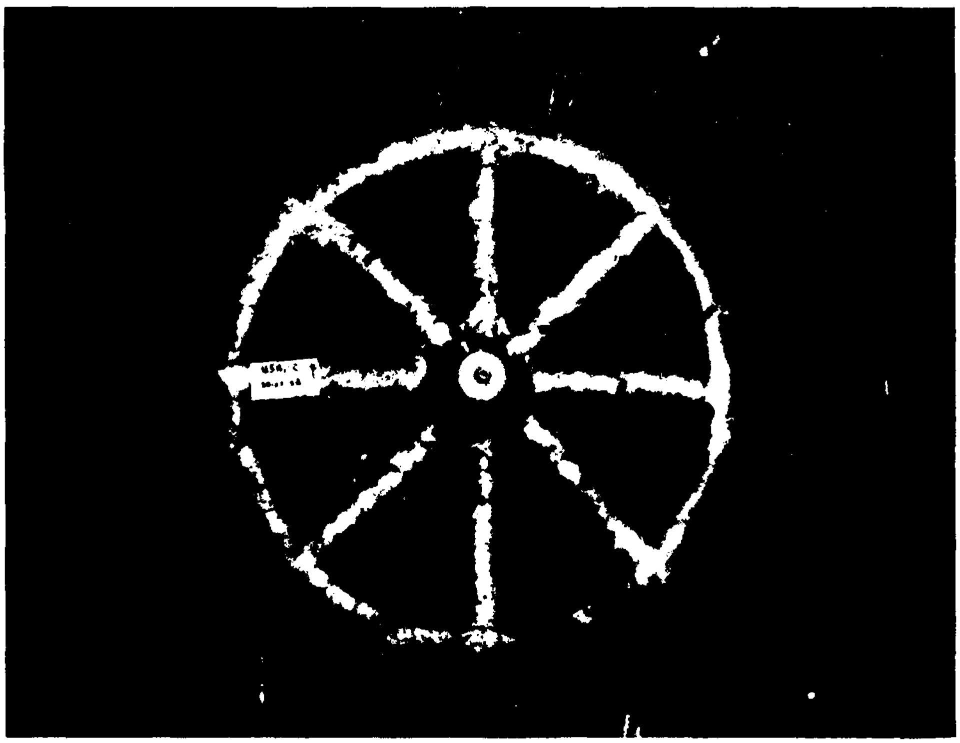


FIG. 4—*Instrumentation hole with borehole collar sleeve installed and cover cap in place. (Note fitting for tunnel diameter gage at center of cover cap and exit tube for lead wires on upper left of sleeve.)*

(0.0005 cm). Each anchor, with its metering rod attached, is installed by expanding it radially against the drill hole with a placing tool. Each subsequent anchor is installed at a lesser depth than the previous one and has holes which allows metering rods from previously set anchors to pass through. The sensor head containing the LVDT's is attached to the borehole collar sleeve after all anchors and metering rods are in place. After all LVDT's are set, the lead wires are covered with plastic tubing to protect them during placement of the concrete bearing pad.

Finally, a cover cap is screwed over the borehole collar sleeve. The cap contains fittings which serve as reference points for the tunnel diameter gage which is inserted between the two diametrically opposite REX-7P's to provide a redundant measurement of total displacement. All measurements with the REX-7P are referenced within the borehole and, therefore, within the rock mass. This arrangement of referencing all measurements within the rock mass eliminates the necessity of monitoring the deformation of various other components of equipment during load applications and release.

Equipment Installation

The complete setup of the test equipment is shown schematically in Fig. 5. To handle the various components of equipment, a hoist supported by steel pins inserted in holes drilled in the vicinity of the test site is used. A timber platform is used to assure proper alignment of test equipment. Installation of equipment is greatly facilitated if the platform surface is accurately located. It should be just far enough away from the center line axis of the instrumentation holes so that minimal amounts of wedges and shims are sufficient to align the equipment into final position. This final placement will ensure that the center line of the test equipment coincides with the axis of the instrumentation holes. With the platform in place, wood blocking is positioned against the lower rock test area. The test equipment is then assembled upward from the blocking, starting with the baseplate with four concave bearing shoes attached. Four screws with convex ends are then positioned against the bearing shoes. Four sets of columns with their connecting plates are placed on the screws. The top plate is then installed, and the total unit is forced together by chain comealongs attached to the base and top plates. The blocking on the lower end is then removed as the unit is supported by the platform. Flatjacks 34.2 in. (87 cm) in diameter with a 2-in. (5.08-cm) diameter hole in the center and particle board separators are fitted on both base and top plates. Although only one flatjack is required on each end of the test setup for load

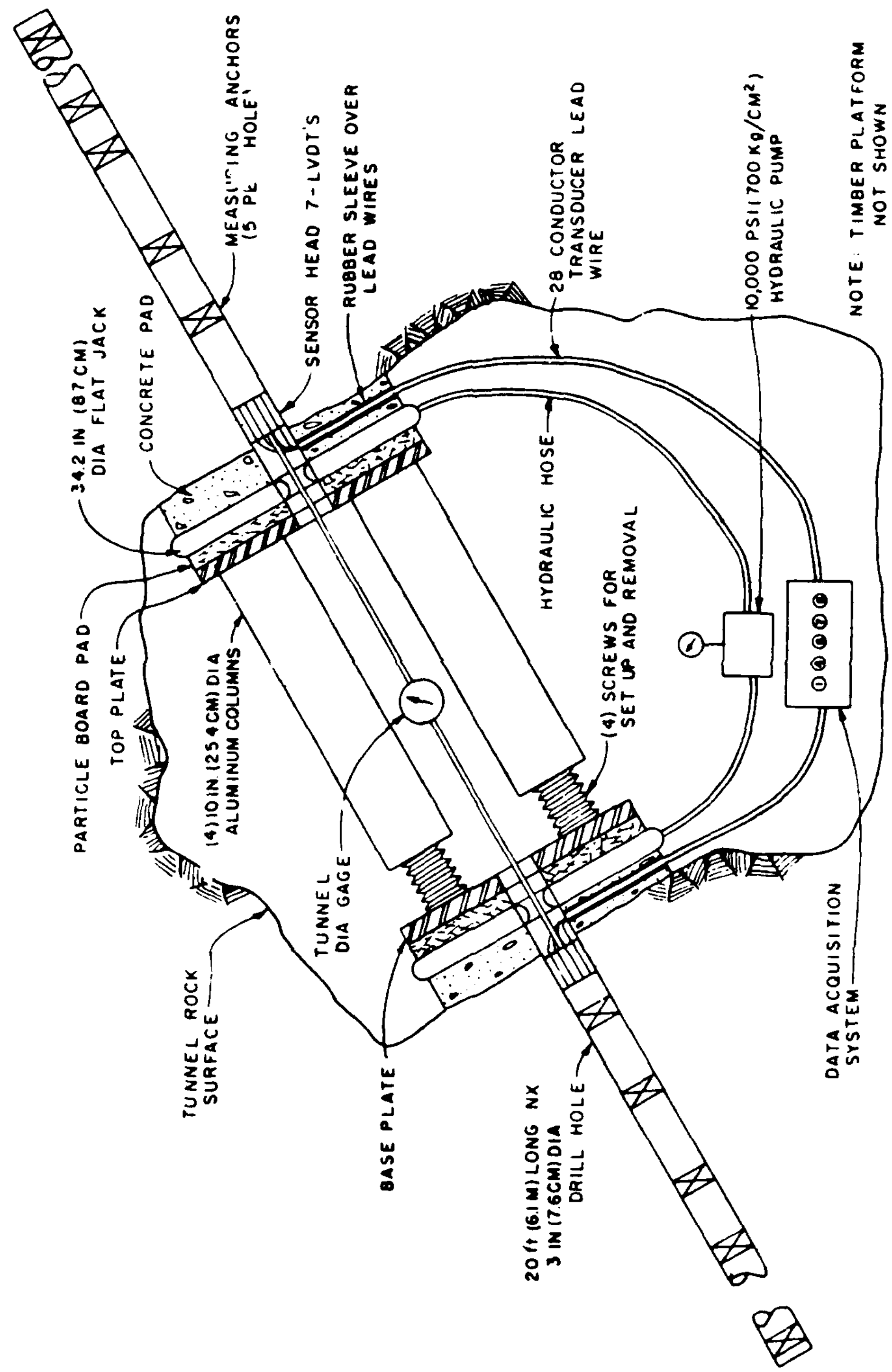


FIG. 5—*Uniaxial jacking test.*

application, an additional jack may be inserted to guard against loss of the entire test setup in case one flatjack was to rupture or if large deformations are anticipated. The flatjack and particle board assembly is held against the base and top plates by special "C" clamps to take up any slack and keep jacks in alignment. A bolt with a hemispherical gage point for the tunnel diameter gage is inserted through the center hole in the base and top plates, flatjacks, and particle board and screwed into each borehole collar cap. The bolt is isolated from the concrete bearing pad placement by a rubber sleeve.

The space between the flatjack asembly and the rock surface is formed and filled with small aggregate concrete (¼ to ⅜ in.) (0.63 to 0.95 cm). Placing the bottom pad first and then securing the test setup from moving, prevents misaligning the equipment while placing the top concrete pad. After 12 h, the forms are stripped so that the rock surface interface with the concrete pad can be observed. A concrete pad with forming material removed is shown in Fig. 6.

The hydraulic system is composed of a 1½-hp electric pump (10 000-psi (700-kg/cm^2) maximum output pressure), high-pressure hose with miscellaneous fittings, and a 2000-psi (140-kg/cm^2) laboratory test gage. This system provides pressure to the hydraulic flatjacks, which, because the jacks have been calibrated previously in the laboratory, results in a known load being applied to the rock.

Testing

The uniaxial jacking test loading and unloading procedures have been presented in detail in previous publications [2,3] and are only briefly reiterated here. Loads are applied in 200-psi (14-kg/cm^2) increments from 200 to 1000 psi (14 to 70 kg/cm^2) with periods of no load between each incremental loading. Frequently, loads are maintained for a 48-h period followed by a 24-h period of no load. However, this should not be considered a standard, for in some rock, very little deformation occurs after 12 to 18 h of loading. In these cases, considerable time can be saved if deformation characteristics of the rock mass are monitored to determine the point in time at which the creep rate becomes insignificant and that time is then used as a terminal point for a load or unload cycle.

While the test is in progress, deformation as indicated by the REX-7P transducers and by the tunnel diameter gage are recorded at intervals ranging from 15 min to 2 h. The REX-7P measures displacements between the bottom anchor and three points 120 deg apart at the collar of the hole. This provides a method of measuring tilting that might occur

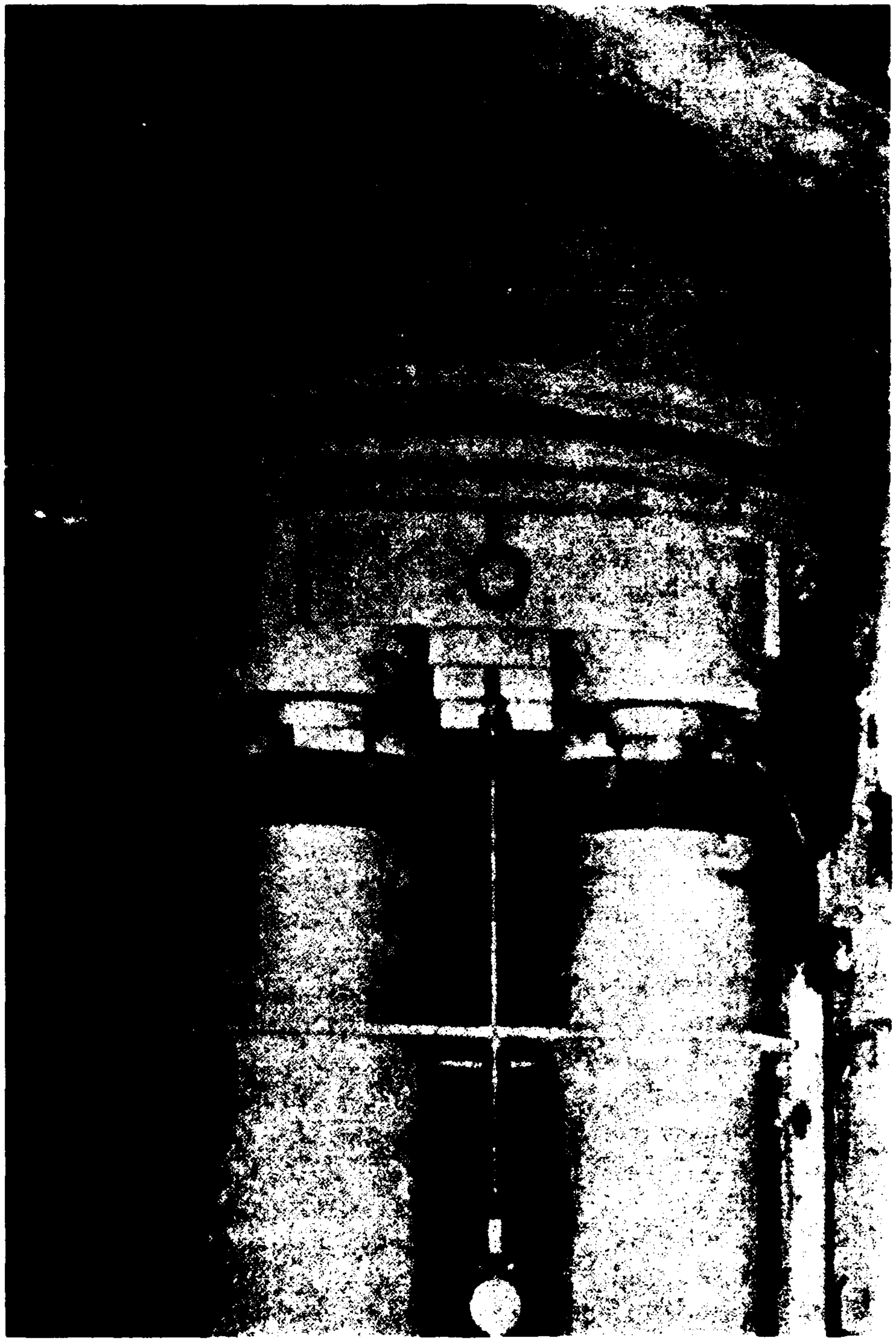


FIG. 6—*Concrete bearing pad for horizontal uniaxial jacking test.*

at the collar of the hole due to differential surface rock deformation. The surface deformation and the remaining measurements are then adjusted for any tilt. The tunnel diameter gage provides a measurement of total displacements between the two loaded rock surfaces. The policy of recording frequent readings and of making redundant measurements (the tunnel diameter gage) has contributed to our confidence in the reliability of measured displacements.

Removal of Equipment and Instrumentation

After the test is completed, the installation procedure is reversed and the entire test setup is retrieved for subsequent use. The centers of the two concrete bearing pads are then partially excavated with pneumatic chipping hammers to allow access for the removal of the REX-7P's. Finally, the concrete pads are shot down with a small explosive charge so as not to create a safety hazard by having them released by the action of gravity at some later date.

Philosophies, Problems, and Solutions

The procedures and comments presented are a result of approximately 30 tests performed over a 4-year period. Almost all tests were conducted by the same individuals; therefore, the overall experience, comments, and conclusions expressed in this paper are those of a limited number of individuals.

In situ jacking tests are nearly always conducted in tunnels where the test environment is anything but ideal. Frequently, access to the test site is difficult, lighting is less than desired, the source of power is unstable or unreliable, and, most serious, dampness in the tunnel can play havoc with the electronic instrumentation used to gather data. These conditions place an extra burden on the test team. Thorough advance planning, careful selection of instrumentation, adherence to the best instrumentation procedures, and built-in redundancy in the loading and data gathering systems become essential features of an *in situ* test program.

The complete sequence of events previously discussed involves the help and cooperation of a contractor and his miners, drillers, and other personnel. Bureau of Reclamation engineers and technicians have found that a general explanation to all involved contractor personnel of what, how, and why certain sequences of the site preparation and equipment installation are to be performed prior to starting the work will result in a more successful test.

Although precautions are taken to minimize the damaging effects of blasting and surface preparation of the test area, the damage cannot be eliminated completely. This condition leads to the question of how to evaluate the results to arrive at a deformation modulus which best represents the behavior of the rock mass when subjected to loads from the structure to be built. That is, should an effort be made to determine a modulus in which the effect of the damaged surface material is eliminated—either by the type of displacement measurements made or by a mathematical treatment of the data? In the case of a concrete dam (which is the usual structure for which the Bureau of Reclamation conducts *in situ* jacking tests), a similar blast-damaged or somewhat disturbed zone is created by the excavation for the keyways and foundation. This zone is undoubtedly much deeper than the disturbed zone at a test site. However, the load from the structure is also much greater, spread over a larger area and, therefore, affects the rock mass to a much greater depth. (So although there is no assurance that the relative effects of the two disturbed zones are equal or even comparable, they have been at least considered.) In the absence of specific information indicating the desirability of a different approach, the effect of the disturbed zone is included in the calculation of deformation modulus for the jacking tests.

Although *in situ* jacking tests are usually conducted in tunnels, data from the tests are converted to deformation moduli by utilizing the mathematical theory for a load applied to the boundary of a semi-infinite solid, as shown on Fig. 7. As there is little physical resemblance between a tunnel and a semi-infinite solid, a potential source of significant error may exist. Unless open cracks in the tunnel arch, invert, or walls allow free movement, deformation of the loaded area will be resisted because a portion of the force will be required to deform the adjacent tunnel surfaces. An analytical study has been made to evaluate the magnitude of the possible error. It was concluded that the distance between the loaded area and a restraining tunnel surface should be at least equal to the radius of the loaded area so that any restraint would not affect calculated moduli significantly.

Either of two philosophies can be embraced in regard to load distribution and location of displacement measurements for *in situ* jacking tests. In one case, displacements are measured at or beneath the surface of the loaded area. At this location, elastic theory indicates that the magnitude of the displacement is highly dependent on the distribution of the applied load. In the second case, displacements are measured at or beneath the rock surface at a point some distance from the loaded area. For these locations, displacement is dependent on the total force applied and the distance from the resultant of the force to the point of measurement, but

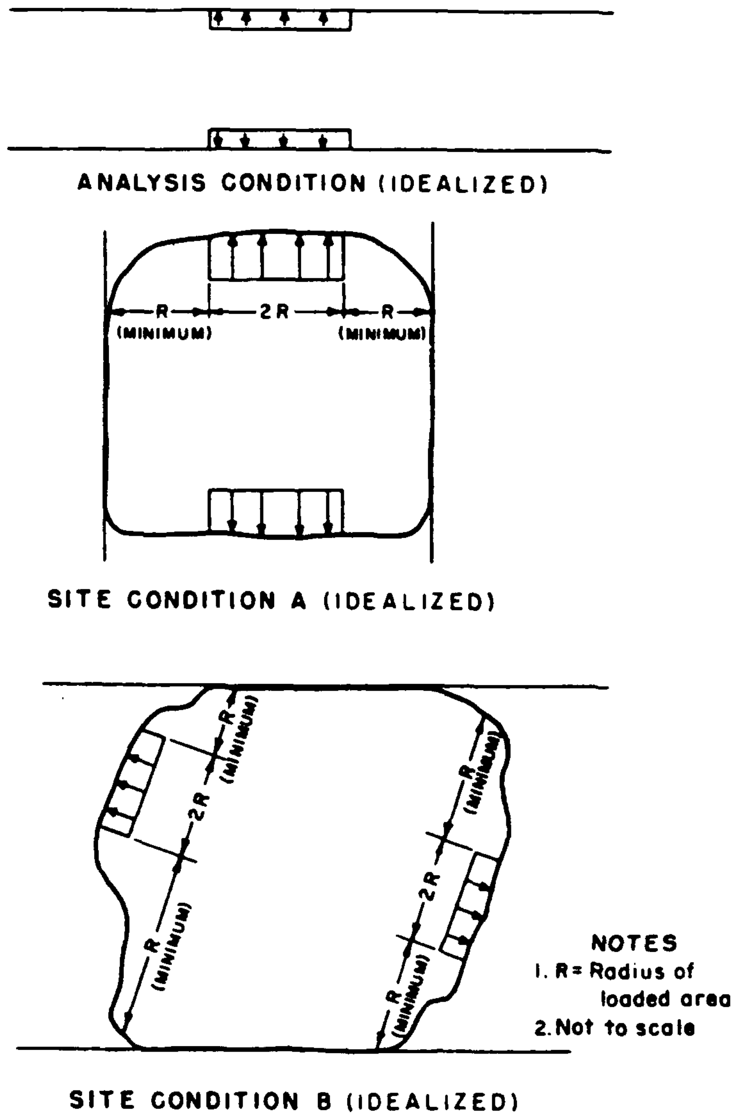


FIG. 7—*Analysis versus site conditions.*

is affected little by the load distribution. For the second case, however, the displacements will be much smaller, requiring much more sensitive measuring devices or resulting in a serious loss of accuracy in calculated moduli. For the Bureau of Reclamation tests, the first philosophy is used, and every effort is made to ensure that the applied load is uniformly distributed.

Following are some of the specific problems encountered during Bureau of Reclamation tests; some have explicit solutions, others lend themselves only to recommendations or suggestions:

1. *Preparing test site*—Obtaining an understanding between contractor's crew and test engineers on the importance of careful treatment of rock during all excavation phases. Define the work carefully in specifications so that the contractor is aware that test site excavation will be more time consuming and more costly than when merely advancing a tunnel. Maintain constant liaison with the shift foreman who supervises the excavation crew.

2. *Drilling of instrumentation holes and retrieving maximum undisturbed core*—Because of his past experience with unit price contracts, a contractor frequently has a tendency to have his crew drill for footage against time. To ensure the necessary high quality workmanship, define work carefully in specifications and also state that an owner's representative must be present during drilling operations. Develop liaison with drillers and emphasize that good "undisturbed" core recovery is important for instrumentation location. (A compliment to individuals involved in a good site preparation or drill hole and core recovery will usually result in a repeat performance.)

3. *Line power*—During a test, a separate source of line power is desirable for electronic instrumentation. Even though voltage regulators are utilized, periodic high power consumption by the contractor can cause problems.

4. *Metal to concrete contact*—Any component of the measuring system coming in contact with concrete should not be made of aluminum. Experience has shown that when fresh concrete is placed in contact with aluminum, a chemical reaction can occur. Under conditions at a test site, this reaction can have an adverse effect on the accuracy of deformation measurements.

5. *Traffic through test area*—Avoid letting traffic pass the test setup. Additional probe drilling, heading advance, or other activities beyond test site location should be scheduled to take place before or after the jacking tests. The test equipment occupies the majority of the adit opening, and hauling other equipment through the test site area containing all the required instrumentation and electronics may result in the loss of valuable data.

Conclusions

1. The equipment and procedures used by the Bureau of Reclamation to conduct *in situ* jacking tests produce satisfactory results.

2. Thorough pre-test geologic exploration is essential to ensure that the site selected is representative of the rock mass.

3. The position of the loaded area within the tunnel should be selected to prevent tunnel side walls from significantly affecting deflections.

4. Careful site preparation is essential for good test results.

5. Since *in situ* jacking tests are nearly always conducted in areas of adverse environmental conditions, the best of instrumentation procedures and a redundancy in the loading and measuring systems are important.

6. Although standardization of *in situ* jacking tests is a desirable goal, it is a goal that may be difficult to achieve. Variations in the needs, available resources, and design philosophy of the user may preclude a universally acceptable standard.

SUGGESTED METHOD FOR DETERMINING ROCK MASS DEFORMABILITY USING A MODIFIED PRESSURE CHAMBER

Scope

1. (a) This test determines the deformability of a rock mass by subjecting the cylindrical wall of a tunnel or chamber to hydraulic pressure and measuring the resultant displacements. Elastic moduli or deformation moduli are calculated in turn.

(b) The test loads a large volume of rock so that the results may be used to represent the true properties of the rock mass, taking into account the influence of joints and fissures. The anisotropic deformability of the rock can also be measured.

(c) The results are usually employed in the design of dam foundations and for the proportioning of pressure shaft and tunnel linings.

(d) Two other methods are available for tunnel-scale deformability. See RTH-361 and RTH-367 to compare details. Potentially large impacts, especially in terms of cost, of variations at this scale, justify the detailing of each method separately.

(e) This method reflects practice described in the reference at the end.

Apparatus

2. Equipment for excavating and lining the test chamber including:

(a) Drilling and blasting materials or mechanical excavation equipment.[1]

(b) Materials and equipment for lining the tunnel with concrete or flexible membrane.[2]

3. A reaction frame (Figure 1) composed of a set of steel rings of sufficient strength and rigidity to resist the force applied by pressurizing fluid must also act as a waterproof membrane.

4. Loading equipment to apply a uniformly distributed radial pressure to the lining including:

(a) A hydraulic pump capable of applying the required pressure and of holding this pressure constant within 5% over a period of at least 24 hr. together with all necessary hoses, connectors, and fluid.[3]

*Numbers refer to NOTES at the end of the text.

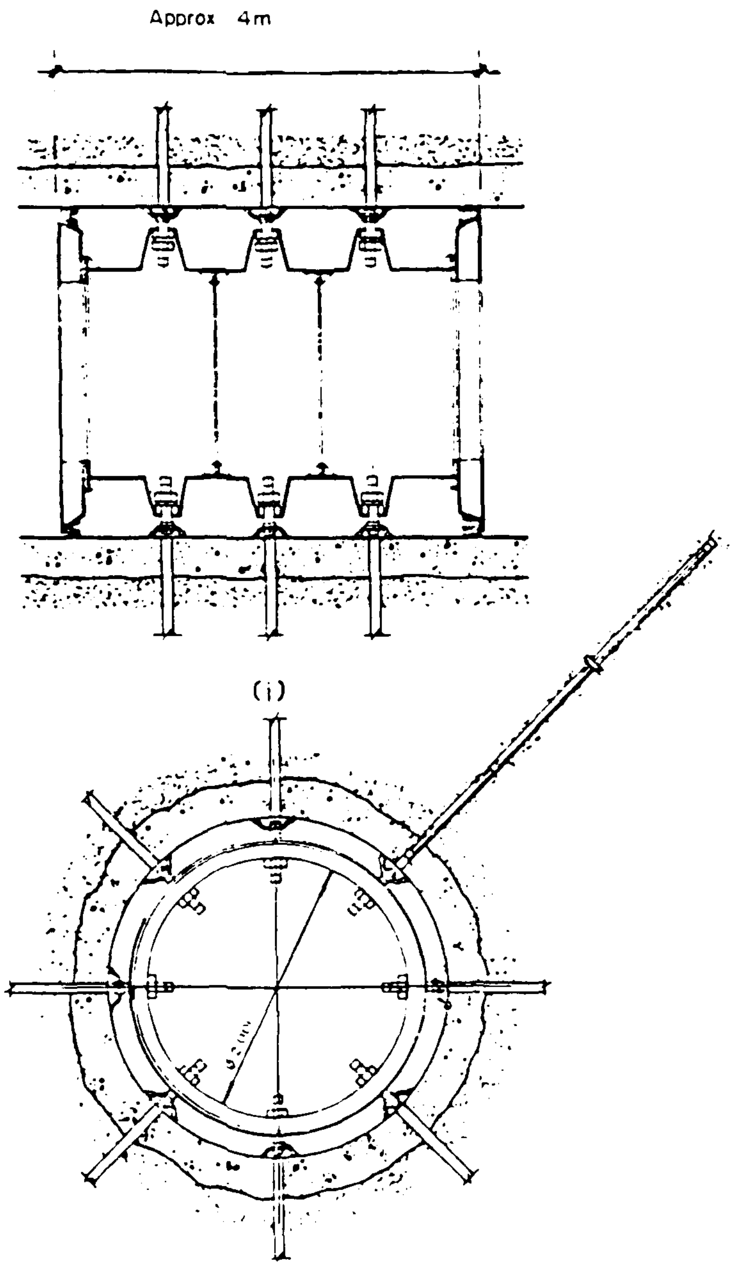


Figure 1. Example partial pressure chamber loading system.

(b) Water seals to contain the pressurized water between the lining and the reaction frame. Special water seals are also required for extensometer rods passing through the lining and reaction frame: pressurized water should not be allowed to escape into the rock since this will greatly affect the test results.

5. Load measuring equipment comprising one or more hydraulic pressure gages or transducers of suitable range and capable of measuring the applied pressure with an accuracy better than $\pm 2\%$.

6. (a) Displacement measuring equipment to monitor rock movements radial to the tunnel with a precision better than 0.01 mm. Single or multiple position extensometers are suggested but joint meters and other measuring devices are also available.

Procedure

7. Preparation

(a) The test chamber location is selected taking into account the rock conditions, particularly the orientation of the rock fabric elements such as joints, bedding, and foliation in relation to the orientation of the proposed tunnel or opening for which results are required.

(b) The test chamber is excavated to the required dimensions.[1]

(c) The geology of the chamber is recorded and specimens taken for index testing as required.

(d) The test section is lined with concrete.[2]

(e) The reaction frame and loading equipment are assembled.

(f) Holes for extensometers or other measuring devices are accurately marked and drilled, ensuring no interference between loading and measuring systems. Directions of measurement should be chosen with regard to the rock fabric and any other anisotropy.

(g) Measuring equipment is installed and checked. For multiple position extensometers, the deepest anchor may be used as a reference provided it is situated at least 2 chamber diameters from the lining. Alternatively the measurements may be related to a rigid reference beam passing along the axis of the chamber and anchored not less than 1 diameter from either end of the test section.

(h) Check water seals for leakage, if necessary by filling and pressurizing the hydraulic chamber. Leaks are manifested as anomalous pressure decay and visible seepage through the reaction frame.

8. Testing

(a) The test is carried out in at least three loading and unloading cycles, a higher maximum pressure being applied at each cycle.

(b) For each cycle the pressure is increased at an average rate of 0.05 MPa/min to the maximum for the cycle, taking not less than 3 intermediate sets of load-displacement readings in order to define a set of pressure-displacement curves.

(c) On reaching the maximum pressure for the cycle the pressure is held constant (± 2% of maximum test pressure) recording displacements as a function of time until approximately 80% of the estimated long-term displacement has been recorded. Each cycle is completed by reducing the pressure to near-zero at the same average rate, taking a further three sets of pressure-displacement readings.

(d) For the final cycle the maximum pressure is held constant until no further displacements are observed. The cycle is completed by unloading in stages taking readings of pressure and corresponding displacements.

Calculations

9. (a) The value of deformation modulus is calculated as follows

$$E = \frac{P_i \ a^2}{r(U_r)}(1 + \nu)$$

where:

E = modulus of deformation

P_i = internal pressure

a = radius to rock face - assuming circular chamber,

r = radius to point here deflection is measured,

U_r = change in radius due to pressure, and

ν = Poisson's ratio.

(b) The elastic modulus is sometimes represented by using only the portion of U_r which is recovered upon unloading.

Reporting

10. The report should include the following:

(a) Drawings, photographs, and detailed description of the test equipment, chamber preparation, lining, and testing.

(b) Geological plans and section of the test chamber showing features that may affect the test results.

(c) Tabulated test observations together with graphs of displacement versus applied pressure and displacement versus time at constant pressure for each of the displacement measuring locations.

(d) Transverse section of the test chamber showing the total and plastic displacements resulting from the maximum pressure. The orientations of significant geological fabrics should be shown on this figure for comparison with any anisotropy of test results. Calculated moduli should be shown also.

NOTES

[1] The recommended diameter is 2.5 m, with a loaded length equal to this diameter. The chamber should be excavated with as little disturbance as possible. Material disturbed by blasting may need to be removed since it tends to produce moduli lower than found at depth. However blast effects are representative if the test results are applied directly as a "model" test to the case of a blasted full-scale tunnel.

[2] When testing only the rock, the lining should be segmented so that it has negligible resistance to radial expansion; in this case the composition of the lining is relatively unimportant, and it may be of either shotcrete or concrete. Alternatively when it is required to test the lining together with the rock, the lining should not be segmented and its properties should be modeled according to those of the prototype.

[3] Maximum hydraulic pressure varies from 5 to 10 MPa.

Reference

International Society for Rock Mechanics, "Suggested Method for Measuring Rock Mass Deformability Using a Radial Jacking Test," International Journal of Rock Mechanics and Mining Sciences, v. 16, 1979, pp. 208-214.

SUGGESTED METHOD FOR DETERMINING ROCK MASS DEFORMABILITY USING A RADIAL JACK CONFIGURATION

Scope

1. (a) This test determines the deformability of a rock mass by subjecting the cylindrical wall of a tunnel or chamber to uniformly distributed radial jack loading and measuring the resultant rock displacements. Elastic or deformation moduli are calculated in turn.

(b) The test loads a large volume of rock so that the results may be used to represent the true properties of the rock mass, taking into account the influence of joints and fissures. The anisotropic deformability of the rock can also be measured.

(c) The results are usually employed in the design of dam foundations and for the proportioning of pressure shaft and tunnel linings.

(d) Two other methods are available for tunnel-scale deformability. See RTH-361 and RTH-366 to compare details. The large impacts, especially in terms of cost, of variations at this scale justify the separation of methods.

(e) This test is expensive to perform, and therefore, should only be used in cases where the information to be gained is of critical importance to the success or failure of the project. Laboratory tests, together with plate bearing and borehole jack tests and seismic surveys may provide adequate estimates of deformability at less cost.

(f) This method largely follows the method in the ISRM and ASTM references listed at the end of this method.

Apparatus

2. Equipment for excavating and lining the test chamber including:

(a) Drilling and blasting materials or mechanical excavation equipment.[1*]

*Numbers refer to NOTES at then end of the text.

(b) Concreting materials and equipment for lining the tunnel, together with strips of weak jointing material for segmenting the lining.[2]

3. A reaction frame (Figure 1) composed of a set of steel rings of sufficient strength and rigidity to resist the force applied by flat jacks. The frame must be provided with smooth surfaces; hardwood planks are usually inserted between the flat jacks and the steel rings.

4. Loading equipment to apply a uniformly distributed radial pressure to the inner face of the concrete lining, including:

(a) A hydraulic pump capable of applying the required pressure and of holding this pressure constant within 5% over a period of at least 24 hr together with all necessary hoses, connectors, and fluid.

(b) Flat jacks, designed to load the maximum of the full circumference of the lining, with sufficient separation to allow displacement measurements, and with a bursting pressure and travel consistent with the anticipated loads and displacements.

5. Load measuring equipment comprising one or more hydraulic pressure gages or transducers[3] of suitable range and capable of measuring the applied pressure with an accuracy better than ±2%.

6. Displacement measuring equipment to monitor rock movements radial to the tunnel with a precision better than 0.01 mm. Single or multiple position extensometers are suggested but joint meters and other measuring devices are also available. Measured movements must be related to fixed reference points, outside the zone of influence of the test section.

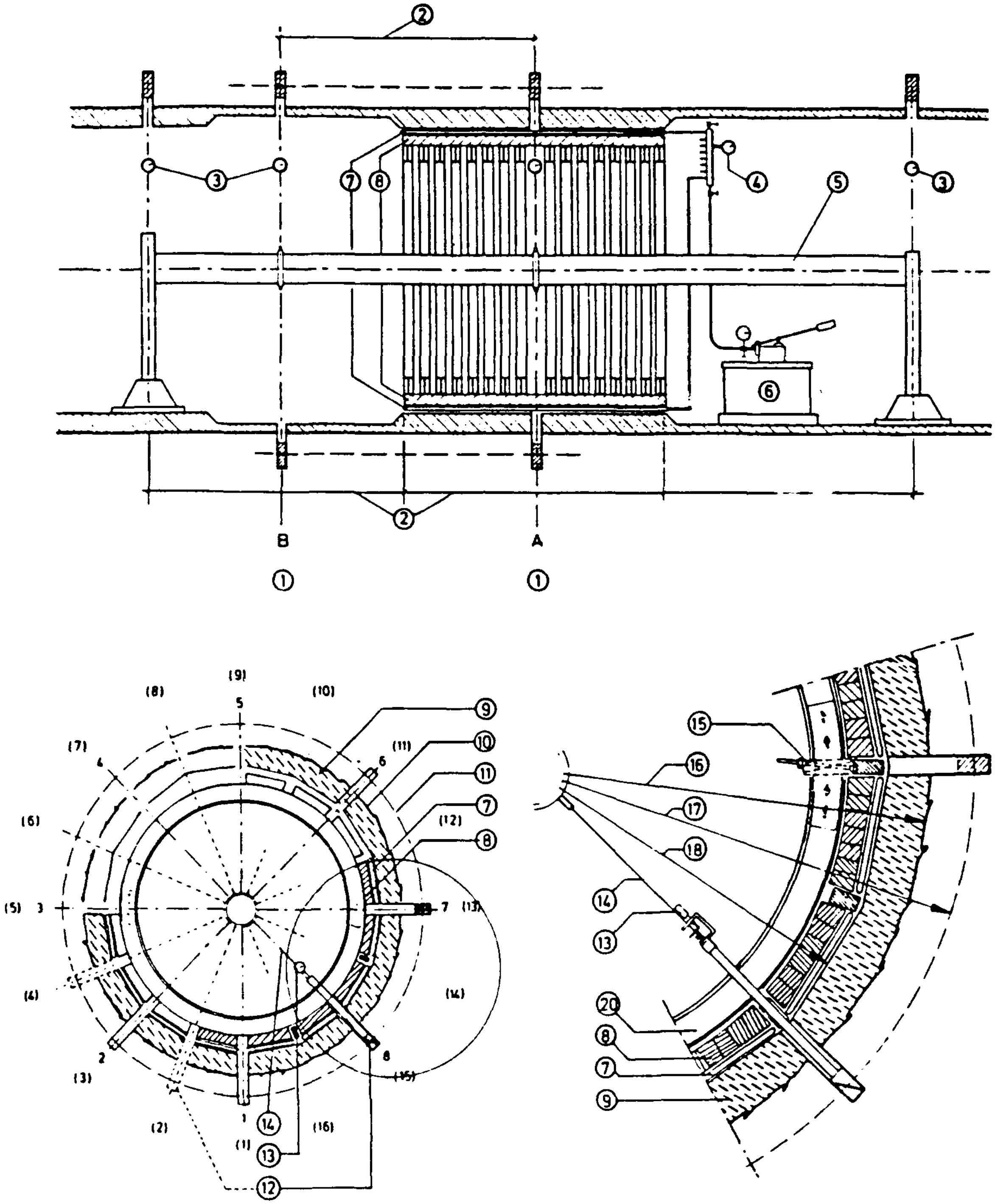
Procedure

7. Preparation

(a) The test chamber location is selected taking into account the rock conditions, particularly the orientation of the rock fabric elements such as joints, bedding and foliation in relation to the orientation of the proposed tunnel or opening for which results are required.

(b) The test chamber is excavated to the required dimensions.[1,4] Generally, the test chamber is about 3 m diam and 9 m long or longer.

(c) The geology of the chamber is mapped and recorded and specimens taken for testing as required, e.g. laboratory strain gaged uniaxial and triaxial testing.



1. Measuring profile. 2. Distance equal to the length of active loading. 3. Control extensometer. 4. Pressure gage. 5. Reference beam. 6. Hydraulic pump. 7. Flat jack. *. Hardwood lagging. 9. Shotcrete. 10. Excavation diameter. 11. Measuring diameter. 12. Extensometer drill holes. 13. Dial gage extensometer. 14. Steel rod. 15. Expansion wedges. 16. Excavation radius. 17. Measuring reference circle. 18. Inscribed Circle. 19. Rockbolt anchor. 20. Steel ring.

Figure 1. Radial jacking test schematic.

(d) The chamber is lined with concrete.[2]

(e) The reaction frame and loading equipment are assembled.

(f) Holes for extensometers or other measuring devices are accurately marked and drilled, ensuring no interference between loading and measuring system. Locations of radial measurement should be chosen with regard to the rock fabric and any other anisotropy. These holes should be continuously cored and all core carefully logged. These holes may be drilled before the test section is lined, but this complicates the lining formwork.

(g) Measuring equipment is installed and checked. With multiple position extensometers the deepest anchor may be used as a reference provided it is situated at least 2 chamber diameters from the lining. Alternatively the measurements may be related to a rigid reference beam passing along the axis of the chamber and anchored not less than 1 diameter from either end of the test section (Figure 1).

8. Testing

(a) The test is carried out in at least three loading and unloading cycles, a higher maximum pressure being applied at each cycle.[5] Maximum test pressures are typically about 1000 psi (7 MPa), but should correspond to expected actual loads.

(b) For each cycle the pressure is increased at an average rate of 0.05 to 0.7 MPa/min to the maximum for the cycle. Three to 10 or more intermediate sets of load-displacement readings are taken for each load increment to define a set of pressure-displacement curves (e.g. Figure 2). Data acquisition may be automated.

(c) On reaching the maximum pressure for the cycle the pressure is held constant (±2% of maximum test pressure) while recording displacements as a function of time until approximately 80% of the estimated long-term displacement has been recorded (Figure 3).[6] Each cycle is completed by reducing the pressure to near-zero at the same average rate, taking a further three sets of pressure-displacement readings.

(d) For the final cycle the maximum pressure is held constant for 24 hr of displacement are observed to evaluate creep.[6] The cycle is completed by unloading in stages while taking readings of pressure and corresponding displacements.

(e) The test equipment is then dismantled and moved.[4]

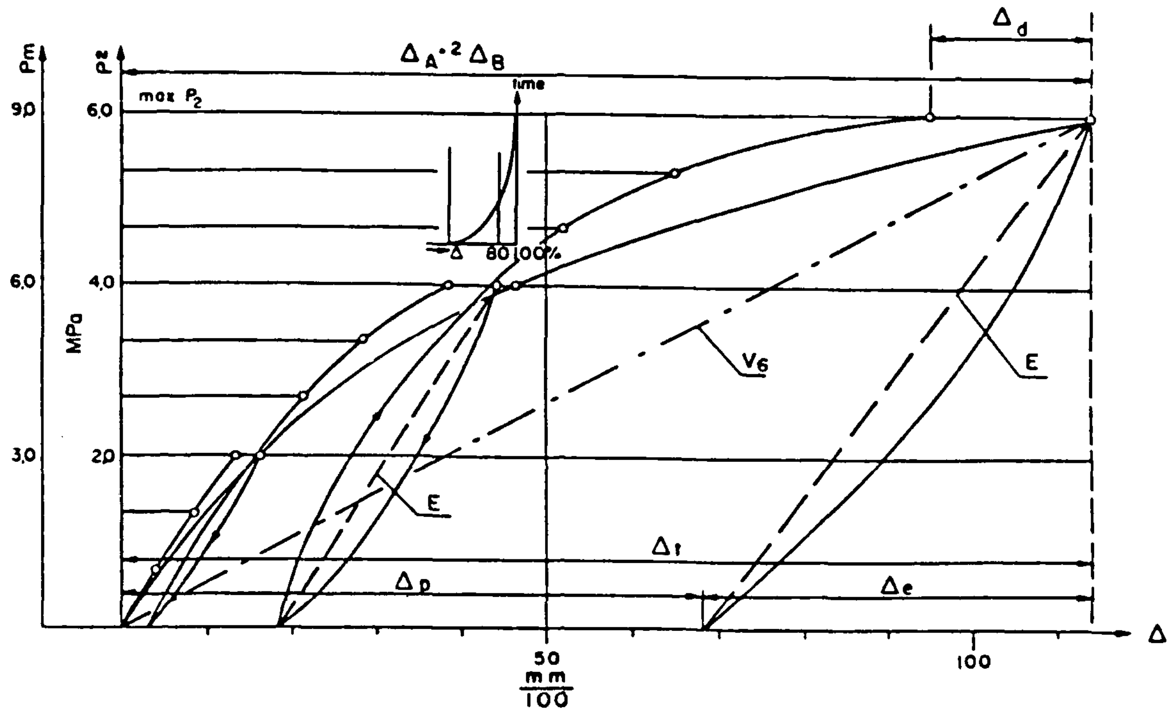


Figure 2. Typical pressure-displacement curves.

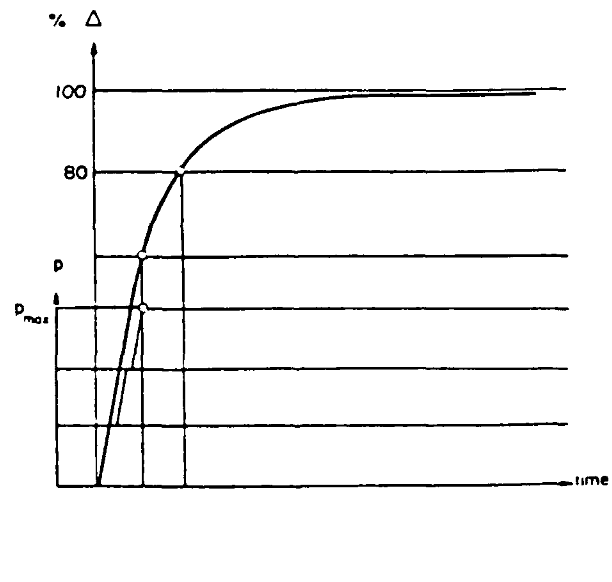


Figure 3. Typical displacement-time curves
at constant applied pressure.

Calculations

9. (a) A solution is given only for the case of a single measuring circle with extensometer anchors immediately behind the lining. This solution, which also assumes linear-elastic behavior for the rock, is usually adequate in practice although it is possible to analyze more complex and realistic test configurations using for example finite element analysis.

(b) The load applied through the flat jacks are first corrected to given an equivalent distributed pressure p_1 on the test chamber lining.

$$p_1 = \frac{\Sigma b}{2 \cdot \pi \cdot r_1} p_m$$

p_1 = distributed pressure on the lining at radius r_1

p_m = manometric pressure in the flat jacks

b = flat jack width (see Figure 4)

r_1 = inner radius of lining

The equivalent pressure p_2 at a "measuring radius" r_2 just beneath the lining is calculated, this radius being outside the zone of irregular stresses beneath the flat jacks and the lining and loose rock.

$$p_2 = \frac{r_1}{r_2} \cdot p_1 = \frac{\Sigma b}{2 \cdot \pi \cdot r_2} \cdot p_m.$$

(c) Superposition of displacements (Figure 5) for two "fictitious" loaded lengths is used to approximate the equivalent displacements for an "infinitely long test chamber,"[7] based on the measured displacements of the relatively short test chamber with respect to its diameter.

$$\Delta = \Delta A_1 + \Delta A_2 + \Delta A_3 = \Delta A_1 + 2 \cdot \Delta B_1$$

(d) The result of the long duration test (Δ_d) under maximum pressure (max p_2) is plotted on the displacement graph (Figure 2). Test data for each cycle are proportionally corrected to give the complete long-term pressure-displacement curve. The elastic and plastic components of the total

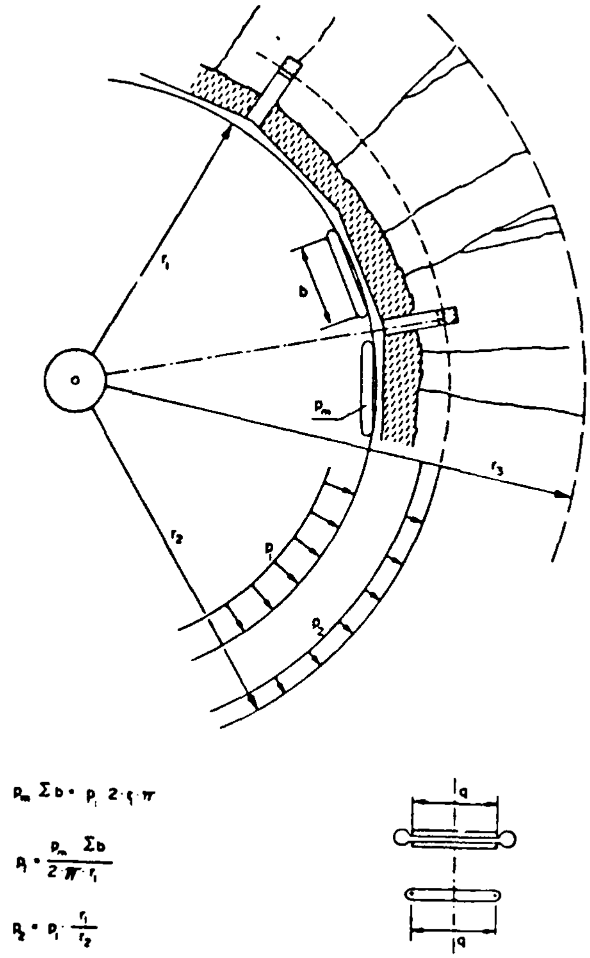


Figure 4. Schematic of loading with symbols used in calculations.

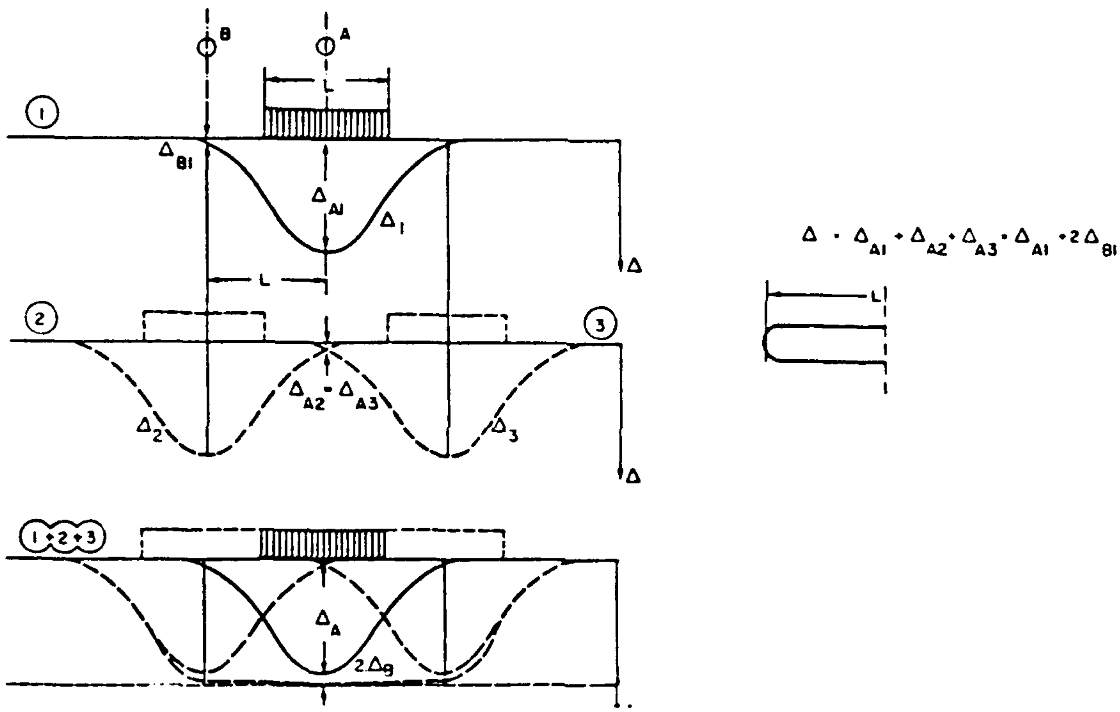


Figure 5. Method of superpositioning of displacements to eliminate end effects.

deformation are obtained graphically from the plotted deformation measurements at the final unloading:

$$\Delta_t = \Delta_p + \Delta_e$$

(e) The elastic modulus E and the deformation modulus V are obtained from the pressure-displacement graphs (Figure 2) using the following formulas based on the theory of elasticity:

$$E = \frac{p_2 \cdot r_2}{\Delta_e} \cdot \frac{(\nu + 1)}{\nu}$$

$$V = \frac{p_2 \cdot r_2}{\Delta_t} \cdot \frac{(\nu + 1)}{\nu}$$

where p_2 is the maximum test pressure and ν is an estimated value for Poisson's ratio.

(f) Alternatively to (e) above, the moduli of undisturbed rock may be obtained taking into account the effect of a fissured and loosened region by using the following formulas:

$$E = \frac{p_2 \cdot r_2}{\Delta_e} \quad \frac{\nu + 1}{\nu} + \ln \frac{r_3}{r_2}$$

$$V = \frac{p_2 \cdot r_2}{\Delta_t} \quad \frac{\nu + 1}{\nu} + \ln \frac{r_3}{r_2}$$

Where Δt is the total deformation, from all loading cycles, and Δe is the rebound deformation from the last loading cycle. Other variables are as previously defined. Where r_3 is the radius to the limit of the assumed fissured and loosened zone.

(g) As one example of the application of radial jack tests, the dimensions of pressure linings can be determined directly by graph. See Lauffer and Seeber 1961. However, such empirical applications should only be applied with caution and good judgement.

Reporting

10. The report should include the following:

(a) Drawings, photographs, and detailed description of the test equipment, chamber, chamber preparation, lining, and testing.

(b) Geological plan and section of the test chamber showing and describing features that may affect the test results. Logs of borings made for the extensometer installations, and indexed photographs of the rock cores.

(c) Tabulated test observations together with graphs of displacement versus applied pressure p_m or p_2, and displacement versus time at constant pressure for each of the displacement measuring locations including displacements at the rock to lining interface. Tabulated "corrected" values together with details of the corrections applied. See Figures 2 and 3 and Table 1 (graphs are usually drawn only for the maximum and minimum displacements).

(d) Transverse section of the test chamber showing the total plastic displacements resulting from the maximum pressure (Figure 6). The orientations of significant geological fabrics should be shown on this figure for comparison with any anisotropy of test results.

(e) Detailed test procedure actually used and any variations and reasons for these from the method described herein, as well as any pertinent or unusual observations.

(f) The graphs showing displacements as a function of applied pressure (Figure 2) should be annotated to show the corresponding elastic and deformation moduli and data from which these were derived.

(g) Equations and methods used to reduce and interpret results should be clearly presented, along with one worked out example.

(h) All simplifying assumptions should be listed, along with discussion of pertinent variations between assumptions and actual size conditions and their possible influence on the results measured. Any corrections should be fully documented.

(i) Results summary table, including test location, rock type, test pressure range, and modulus values for different depth increments around the chamber.

(j) Individual test summary tables for each measurement point.

(k) Results of complementary tests, such as laboratory modulus.

Table 1. Suggested Layout for Test Data Sheet

1	2	3	4	5	4 + 5	6	7	4 + 5 + 7	8	9
NR	time	p_2	Δ_A	Δ_B	$\Delta_A + \Delta_B$	Δ_d	Δ_d corr.	Δ_t	Δ_e	Δ_p
1						—	—			
2						—	—			
3a							—			
3b							—			
3c										
4										
5										
6a							—			
6b							—			
6c										
7										
8										
9a										
9*							—			

$$E = \frac{p_2 \cdot r_2}{\Delta_e} \cdot \frac{\mu + 1}{\mu} = \text{----------------}$$

$$V = \frac{p_2 \cdot r_2}{\Delta_t} \cdot \frac{\mu + 1}{\mu} = \text{----------------}$$

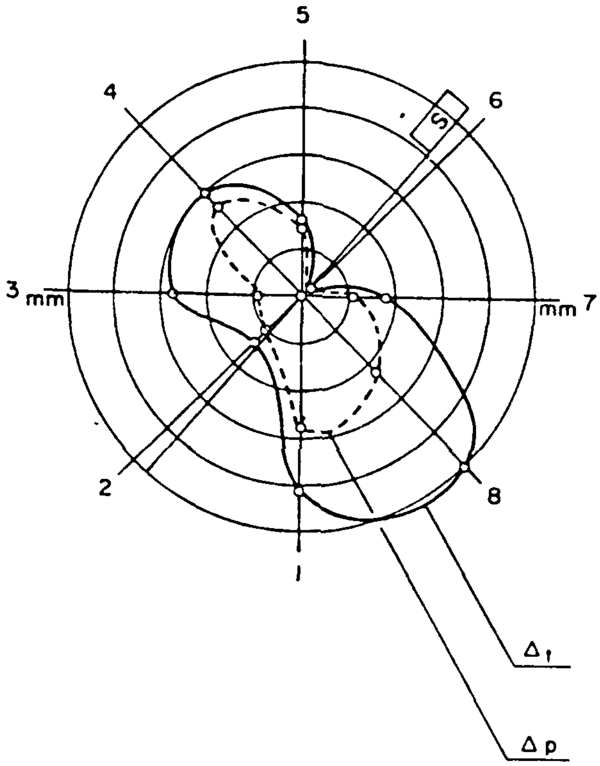


Figure 6. Typical illustration for showing displacement as a function of direction.

Notes

[1]The recommended diameters is 2.5 to 3 m, with a loaded length equal to 3 times this diameter. The chamber should be excavated with as little disturbance as possible, which implies the use of controlled blasting methods, such as line drilling or channel drilling. Alternatively, partial face mechanical excavation equipment may be used if available.

[2]When testing only the rock, the lining should be segmented so that it has negligible resistance to radial expansion; in this case the composition of the lining is relatively unimportant, and it may be of either shotcrete or concrete. Alternatively when it is required to test the lining together with the rock, the lining should not be segmented and its properties should be modeled according to those of the prototype.

[3]Measurements are usually made with mechanical gages. Particular care is required to guarantee the reliability of electric transducers and recording equipment.

[4]To assess the effectiveness of grouting, two test chambers may be prepared adjacent to each other. Grouting is carried out after completion of testing in the ungrouted chamber, and the equipment is then transferred to the grouted chamber. Alternatively the same chamber may be retested after grouting.

[5]Typically the maximum pressure applied in this test is 5 - 10 MPa.

[6]In the case of "creeping" rock it may be necessary to stop loading even though the displacements continue. Not less than 80% of the anticipated long-term displacement should have been reached.

[7]This superposition is made necessary by the comparatively short length of test chamber in relation to its diameter. Superposition is only strictly valid for elastic deformations but also give a good approximation if the rock is moderately plastic in its behavior.

References

International Society for Rock Mechanics, "Suggested Method for Measuring Rock Mass Deformability Using a Radial Jacking Test," International Journal of Rock Mechanics and Mining Sciences, v. 16, 1979, pp. 208-214.

American Society for Testing and Materials, 1986 Annual Book of ASTM Standards, Section 4, Construction; Volume 4.08 Soil and Rock; Building Stones, Standard D 4506-85, "Standard Test Method for Determining the In Situ Modulus of Deformation of Rock Mass Using a Radial Jacking Test."

Lauffer, H. and Seeber, G., "Design and Control of Linings in Pressure Tunnels and Shafts," 7th International Conference on Large Dams, 1961.

SUGGESTED METHOD FOR DETERMINING ROCK MASS DEFORMABILITY USING A DRILLHOLE-JACK DIALOMETER

Scope

1. (a) This test determines the deformability of a rock mass by subjecting a section of drill hole to mechanical jack pressure and measuring the resultant wall displacements. Elastic moduli and deformation moduli are calculated in turn.

(b) The results are employed in design of foundations and underground construction but are mostly used as semiquantitative index values revealing variability from point to point in a rock mass.

(c) The dilatometer is self-contained, and tests are relatively inexpensive compared to similar tests at a large scale.[1] Also, the wall is damaged only minimally by the drilling of the hole and usually remains representative of the undisturbed rock condition. These advantages, however, come at a sacrifice of representation of the effects of joints and fissures which are usually spaced too widely to be fully represented in the loaded volume around the drill hole.

(d) This method reflects practice described in the references at the end.

(e) Another type of dilatometer for drillholes transmits hydraulic pressure to the rock through a soft membrane. See RTH-363.

Apparatus

2. Drilling equipment to develop the access hole, in a given orientation without disturbing the wallrock.[2]

3. A drill hole-jack dilatometer similar to that in Figure 1, which consists of:

(a) Metal frame holding the other units and having parts and connections to external equipment.

(b) Two half-cylindrical, rigid steel plates of same curvature as wall of drillhole. Alternatively, the plates can be flexible as where they are faced with curved flat jacks as shown in Figure 2.

(c) Loading jacks or wedges functioning to drive the plates against the wall. A stroke of about 5 mm is needed beyond any requirements for seating the plates.

(d) Linear differential transformers oriented diametrically with potential resolution of 2 microns.

4. Hand-operated hydraulic pump and flexible hose and steel plumbing to withstand working pressure to 70 MPa.

5. Hydraulic pressure gages or transducer of suitable range and capable of measuring the applied pressure with accuracy better than 2 percent.

Procedure

6. Preparation

(a) The positions for testing are planned with due regard to the location of drilling station and the rock conditions to be investigated. The effects of geological structure and fabric are particularly important.

(b) The hole is drilled and logged. The log is studied for possible modifications in positions for testing. Multiple testing positions in one hole should not overlap but may join where two or more loading directions are to be distinguished.

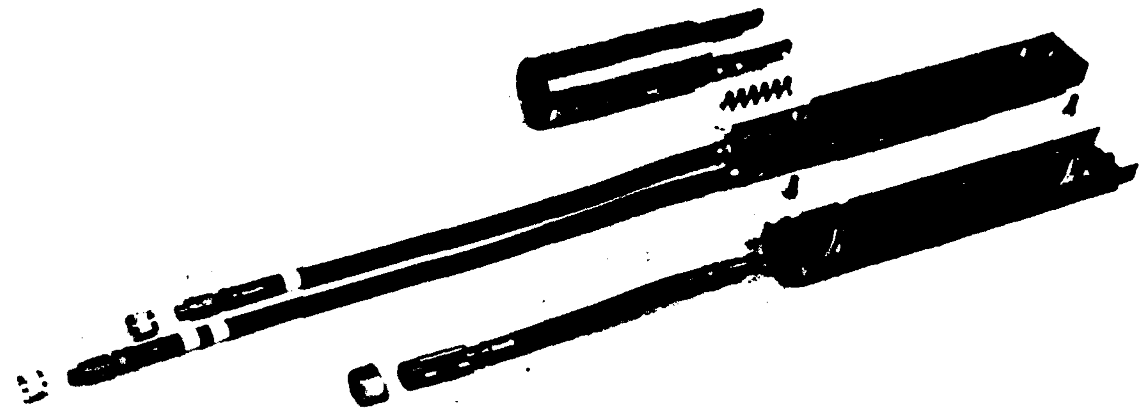
(c) Evaluate the texture and strength of the wall to confirm suitability of method. Otherwise, consider relocation or use of another dilatometer.

(d) The dilatometer is assembled and inserted into the hole, commonly using an attachable pole to position and rotate and taking special care with trailing lines.

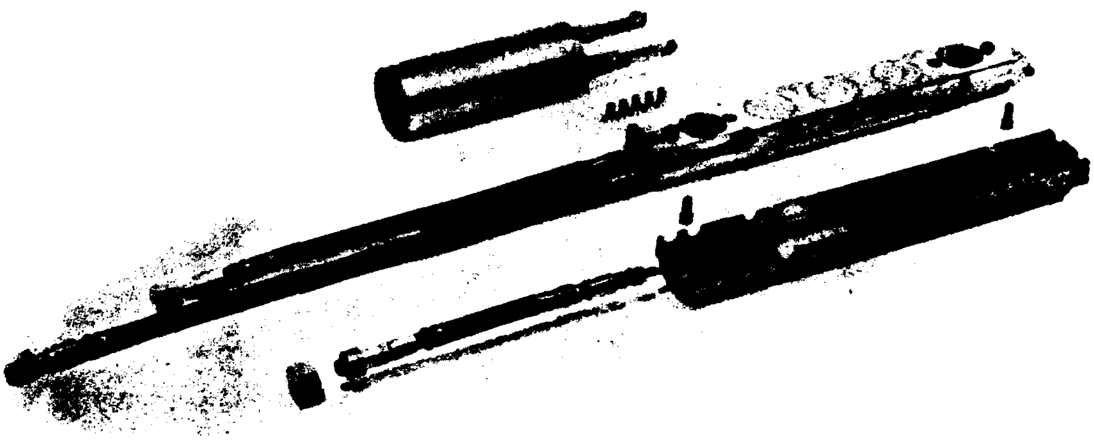
(e) The bearing plates are brought into initial contact with the wall with adjustments as necessary to minimize eccentricity and stress concentrations.

(f) The rods of the linear differential transformers are seated against the wall.

BOREHOLE TESTS



(a) Goodman jack model 52101 hard rock jack
(Courtesy Slope Indicator Co.).



(b) Goodman jack model 52102 soft rock jack
(Courtesy Slope Indicator Co.).

Figure 1. Example dilatometers using hard-faced jacks for loading. (Lama and Vukuturi, 1978)

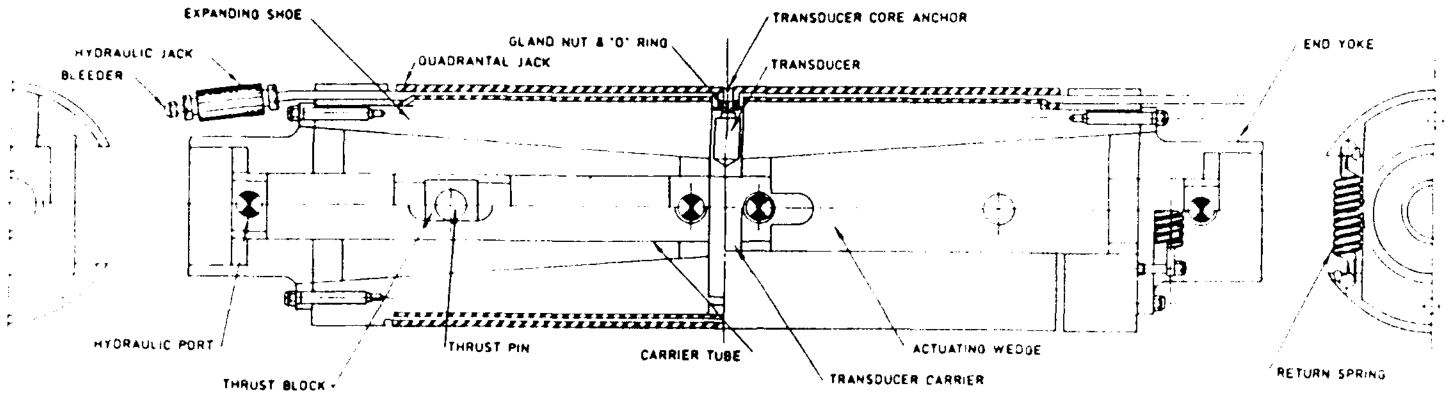


Figure 2. Example dilatometer using soft-faced flat jacks for loading. (Lama and Vukuturi, 1978)

7. Testing

(a) The dilatometer is pressurized in increased stages, with pressure released between stages. Typically the stage pressures are 25, 50, 75, and 100 percent of the planned maximum of the complete test.[3]

(b) Bearing pressure is increased at a rate of 0.5 MPa/min or less.

(c) On reaching the planned pressure for the stage, the pressure is held constant for at least 1 min to detect and define nonelastic deformation. Each stage is completed by releasing pressure at a prescribed rate up to 0.5 MPa/min.

(d) The test history is documented with four or more sets of measurements during pressure increase and two during pressure decrease. Supplementary notes are necessary to describe any complexities not otherwise revealed (such as nonelastic deformation).

(e) The hydraulic pressure is released from the loading jacks. The rods of the linear differential transformers are retracted. The loading plates are retracted from the wall and the dilatometer is removed from the hole.

Calculations

8. The axiosymmetrical relationship for elastic deformation does not apply directly for dilatometers with split loading. Modified expressions have been developed for the specific apparatus. The expression for the dilatometer in Figure 1 is

$$E = \frac{\Delta p d}{\Delta U_d} \cdot K(\nu, \beta)$$

where

Δp = pressure increment

ΔU_d = diametral displacement increment

d = diameter of hole

$K(\nu, \beta)$ = constant dependent on Poisson's ratio and angle of loaded arc β.

Such characteristics as $K(\nu, \beta)$ are supplied by the manufacturer since they are unique to each particular design.[4]

Where permanent deformation (nonelastic) occurs also, that portion of ΔU_d should be excluded from the equation for calculating modulus of elasticity. However, total deformation should be used to compute modulus of deformation, with the same equation.

Reporting

9. The report should include for each test or all tests together the following:

(a) Position and orientation of the test, presented numerically, graphically, or both ways.

(b) Logs and other geological descriptions of rock near the test. The structural details are particularly important.

(c) Tabulated test observations together with graphs of displacement versus applied pressure and displacement versus time at constant pressure for each of the displacement measuring devices (e.g., linear differential transformers).

(d) Transverse section of hole showing the displacements resulting from the pressure in all orientations tested. Calculated moduli are indicated also.

Notes

[1] See RTH-361, -366, -367 for similar test at tunnel scale.

[2] Diamond core drilling is recommended for obtaining the necessary close tolerance when using dilatometer only slightly smaller than the hole and displacement measuring devices with very limited stroke.

[3] Typically, the maximum pressure is about 15 MPa.

[4] Representation of the effects of a combination of complexities by this empirical factor $K(\nu,\beta)$ has been found unsatisfactory for most purposes except indexing (Heuze and Amadei 1985).

References

Lama, R. D., and Vutukuri, V. S., Handbook on Mechanical Properties of Rock Vol. III, TransTech Publications, 1978, 406 pp.

Stagg, K. G., "In Situ Tests on the Rock Mass," in Rock Mechanics in Engineering Practice, John Wiley & Sons, New York, 1968, pp 125-156.

Heuze, F. E., and Amadei, B., "The N-X-Borehole Jack: A Lesson in Trials and Errors," Int. Journal of Rock Mechanics and Mining Sciences, v. 22, No. 2, 1985, pp 105-112.

PART II. IN SITU TEST METHODS

E. Determination of Rock Mass Permeability

RTH 381-80

SUGGESTED METHOD FOR IN SITU DETERMINATION OF ROCK MASS PERMEABILITY USING WATER PRESSURE TESTS

1. Introduction

1.1 The water pressure test consists of the injection of water into a borehole at a constant flow rate and pressure. Water enters the rock mass along the entire length of borehole or along an interval of the borehole (test section) which has been sealed off by one or more packers (Fig. 1). Water pressure tests can be conducted in media above or below the groundwater table and anisotropic permeability can be estimated by orienting test boreholes in different directions. Permeability can be computed by assuming a continuous porous medium, or individual fissures and fissure sets within the rock mass can be considered.

2. Test Procedures and Interpretation

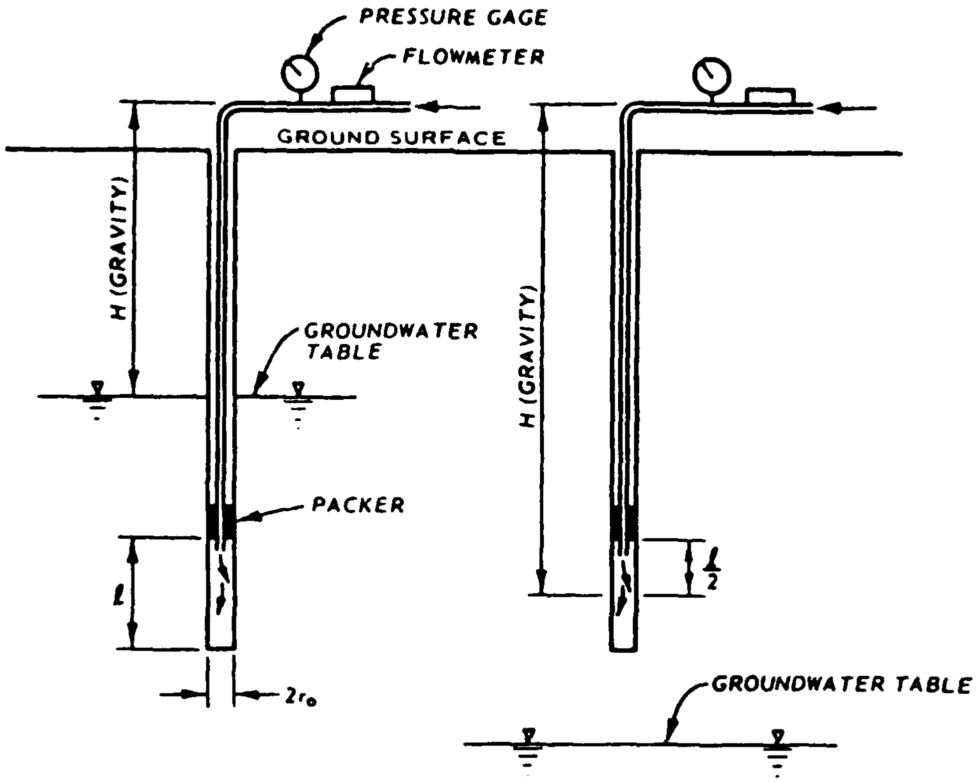
2.1 Test Layout and Setup - In many cases, initial exploration boreholes are routinely pressure tested prior to determining location and orientation of predominant fissure sets. Some boreholes should be specifically located for pressure testing as information concerning fissure networks is obtained. A pressure test affects a region, possibly within only a few feet of the borehole. Consequently, test boreholes should be as closely spaced as practicable. Extrapolation of test data between boreholes can be aided by determination of fissure continuity through examination of core logs or visual inspection of borehole walls with a borehole television or conventional type camera. Fault zones should be located and tested separately as they may be zones of exceptionally high or low permeability with respect to the surrounding region.

2.1.1 Where the scope of the exploration program will allow it, predominant fissure sets should be tested individually by orienting boreholes to intersect only the fissure set under investigation. Permeabilities of fissure sets can be combined to obtain overall directional permeabilities. Where fissures are numerous and randomly oriented, the borehole orientation should be perpendicular to the plane in which permeability is to be measured. The majority of pressure tests will be conducted in vertical boreholes; however, some test boreholes in other orientations are needed to estimate the anisotropy of the rock mass. In rock exploration programs, groups of inclined boreholes are generally needed to determine reliable estimates of joint set orientations. These boreholes could be pressure tested to aid in estimating anisotropic permeability.

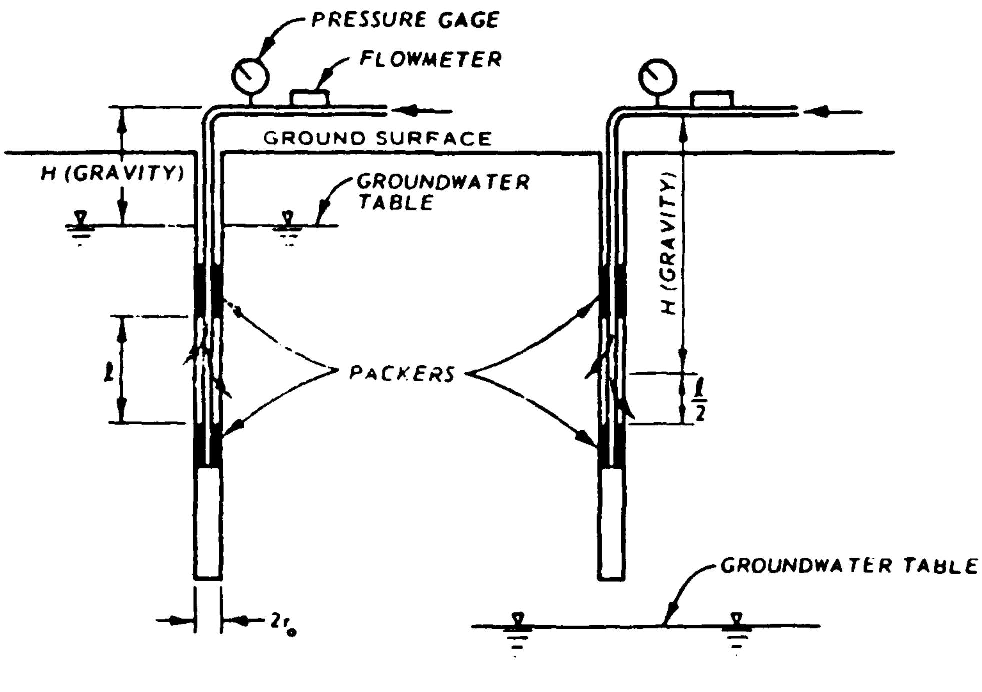
2.1.2 Generally, a borehole should be tested at intervals along its length to determine a permeability profile. A knowledge of the characteristics and location of intersected fissures is desirable in choosing test section lengths. Such information can be obtained from core examination (Note 1). When possible, test section lengths should be chosen to isolate fissure sets. Where fissures are numerous, test lengths can be limited to 5 or 10 ft (1.5 or 3 m). Where fissures are infrequent, a longer test length may be utilized. In many cases, fissure networks may be considered too complex to require special care in selecting test section lengths. However, it is good practice to test in 5- to 10-ft (1.5- to 3-m) intervals to allow detection of localized high- or low-permeability zones.

NOTE 1--Visual inspection with a borehole television camera or film camera would be beneficial and should be used where economically feasible.

2.1.3 Intervals along the borehole length should be tested using either the single or double packer method. The single packer setup shown in Fig. 1 is used when testing as drilling progresses. This technique is advantageous because it reduces the amount of drill cuttings available for clogging fissures since each section is tested before being exposed to the cuttings produced by further drilling. Also, errors due to packer leakage are minimized since only one packer is used. The double packer method (Fig. 1) can be used to test or retest sections of previously drilled boreholes.



a. SINGLE PACKER TECHNIQUE



b. DOUBLE PACKER TECHNIQUE

Fig. 1. Pressure test setups--pressure measured at the ground surface.

2.2 Drilling Operations - Prior to pressure testing, the borehole should be surged with water in an effort to remove some of the cuttings and dust. Reverse rotary drilling should be considered for use in boreholes drilled specifically for pressure testing. The removal of cuttings through the drill stem will minimize the clogging of rock fissures.

2.3 Test Equipment - The basic equipment consists of a water supply, pump, packers, flow pipe, and measuring devices. A pump with a minimum capacity of 50 gpm (3150 cu cm/sec) against a pressure of 100 psi (689.5 kPa) is recommended, and only clean water should be used. A progressing cavity-type positive displacement pump is recommended for pressure testing since it maintains a uniform pressure. The type and length of packer needed are dependent on the character of the rock mass to be tested. In most cases, the pneumatic packer will suffice; however, if problems arise, the cup leather or mechanical packers may be substituted. All packers should be at least 18 in. (450 mm) in length. The flow pipe should have a diameter as large as possible to reduce pressure losses between the ground surface and the test section.

2.3.1 Measuring devices are required for determination of volume flow rate and pressure. Flow rate is conveniently measured at the surface, and it is preferred that flow rate be measured continuously rather than averaged by measuring the volume of flow over a known period of time. Multiple gages may be required to measure flow rates ranging from less than 1 gpm (63 cu cm/sec) to as much as 50 gpm (3150 cu cm/sec).

2.3.2 It is recommended that pressure be measured directly within the test section by, for example, installation of an electric transducer. The transducer will also provide a measurement of the existing groundwater pressure at the level of testing. The transducer can be connected to a chart recorder and the initial groundwater pressure indicated as zero. Pressure changes recorded during testing would then be a direct measurement of the excess pressure which is needed in permeability calculations. In most cases, transducer systems will not be readily available. Consequently, pressure will be measured with surface gages. In these instances, pressure loss between the surface and test section must be considered.

2.3.3 When excess pressures are to be determined from surface gage readings, pressure loss between surface and test section must be estimated. Head loss during flow is generally caused by (a) friction, bends, constrictions, and enlargements along the flow pipe; and (b) exit from the flow pipe into the test section. The majority of pressure loss will be caused by friction. Friction losses are dependent on pipe roughness and diameter, and are directly proportional to the square of the flow velocity. Friction losses can be determined experimentally by laying the flow pipe on level ground and pumping water through it at several different velocities while measuring the gage pressure at two points along the pipe. The difference in the gage pressures is the friction loss over the distance between the gages. A plot of friction loss per unit length versus velocity can be obtained from the results. Friction losses can also be estimated from elementary fluid mechanics formulas, tables, and charts.

2.3.4 In most tests, pressure losses caused by pipe bends, constrictions, and enlargements will be insignificant; however, such losses can be checked from relationships given in elementary fluid mechanics textbooks or determined experimentally by pumping on the ground surface and measuring the pressure drop across critical portions of the flow pipe. The pressure loss at the exit from the flow pipe into the test section can be ignored since it is offset by the addition of a velocity head at the surface pressure gage (see paragraph 2.4.3).

2.4 Test Program - The general sequence of operations for using the single packer technique as the borehole progresses is listed below. Changes in the sequence applying to the double packer test are noted.

(a) Step 1 - Drill the desired length of test section, ℓ, (usually 5 or 10 ft (1.5 or 3 m)) and remove the drill equipment. Where the double packer method is to be used, the borehole is drilled to any desired depth.

(b) Step 2 - Study the core to determine the location, number, and characteristics of fissures intersecting the proposed test interval. If only equivalent permeability is to be computed based on test section length, ℓ, fissure information is not needed. However, such information, when correlated with measured permeability, is helpful in understanding the influence of various fissures or fissure sets on the overall permeability of the mass.

(c) Step 3 - Insert the flow pipe and packer, and seal the packer against the borehole wall. To ensure the best possible seal, additional inflation (or tightening) of packers should be accomplished under each test pressure. When tightening packers, a significant and lasting increase in test pressure accompanied by a decrease in flow rate is an indication that the seal has been improved.

(d) Step 4 - Conduct tests using a series of test pressure.

(e) Step 5 - Remove the packer and flow pipe, and begin drilling as in Step 1. In the double packer test, packers are moved to a new test zone; removal of test equipment will be required to alter the test section length as necessary.

2.4.1 The actual pressure testing is conducted in Step 4. The recommended test procedures are:

(a) Inject water into the borehole and establish a constant pressure.

(b) Take readings of pressure and flow rate over a 3- to 5-min period to ensure that steady-state conditions have been attained. If volume of flow rather than volume flow rate is measured, the average volume flow rate should be checked at 30-sec to 1-min intervals and compared with the overall average volume flow rate for the 3- to 5-min period. At the end of the test, conduct a pressure drop test. This is done simply by shutting off the pump and recording the drop in pressure with time.

(c) Increase or decrease the flow rate (and pressure) and conduct the next test.

2.4.2 The test program should be designed to check for turbulent flow and the effects of fissure widening. This requires that in a selected number of test sections, a series of tests be conducted at different pressures. A minimum of three tests, each at an increased pressure, are required to detect the nonlinear flow rate versus pressure relationship characteristic of turbulent flow. However, more tests should be conducted as necessary to completely describe any nonlinear behavior. Typical flow rate versus pressure curves are shown in Figs. 2-6. Consistency of results should also be checked by repeating the tests in the same sequence. A significant increase in permeability under the lower pressures would indicate the possibility of permanent fissure expansion.

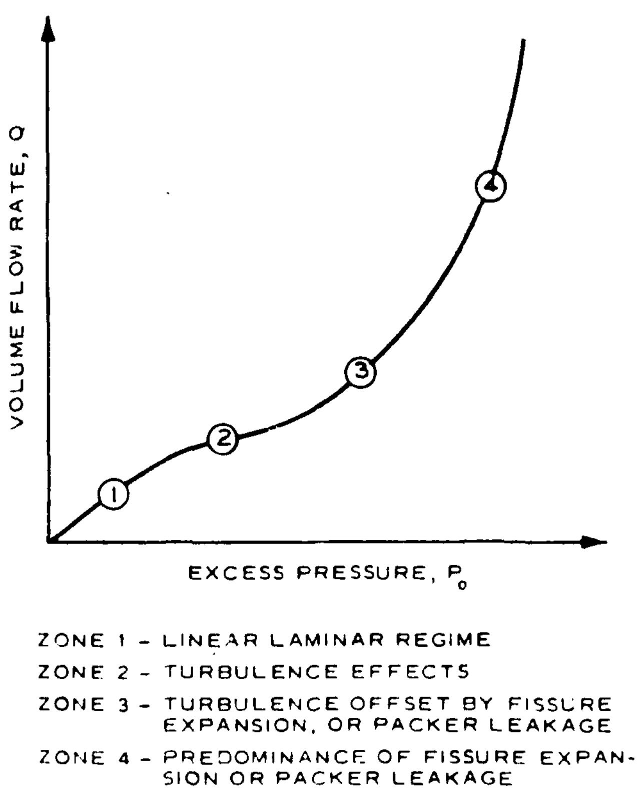
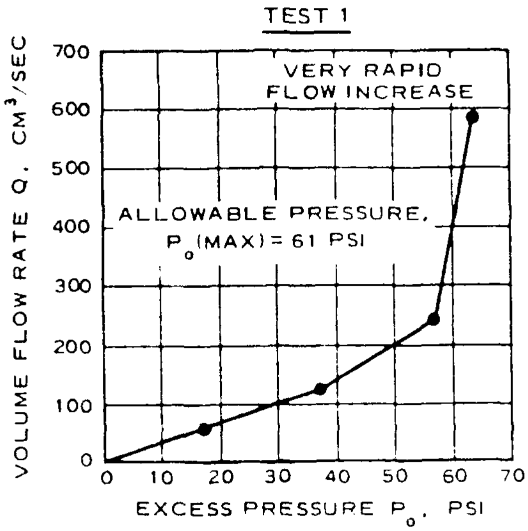
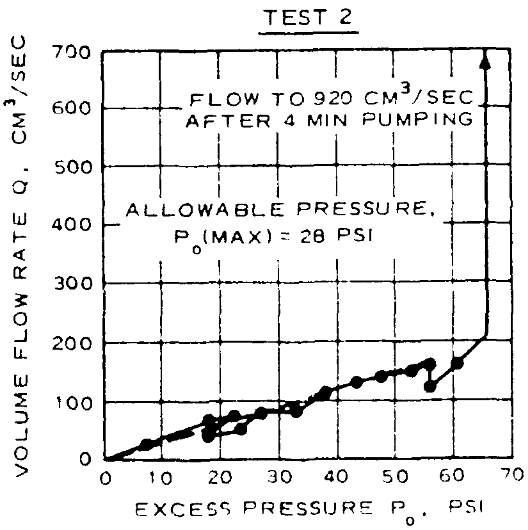


Fig. 2. Typical result of water pressure tests conducted at a series of increasing pressures (after Louis and Maini, 1970; Zeigler, 1975).



a. TEST CONDUCTED IN A VERTICAL BOREHOLE BETWEEN DEPTHS OF 97 AND 102 FT; GROUND-WATER TABLE AT A DEPTH OF 10 FT



b. TEST CONDUCTED IN A VERTICAL BOREHOLE BETWEEN DEPTHS OF 40 AND 45 FT; GROUND-WATER TABLE AT A DEPTH OF 10 FT

Fig. 3. Results of pressure tests in horizontally bedded sedimentary rock (after Morgenstern and Vaughan, 1963; Zeigler, 1975).

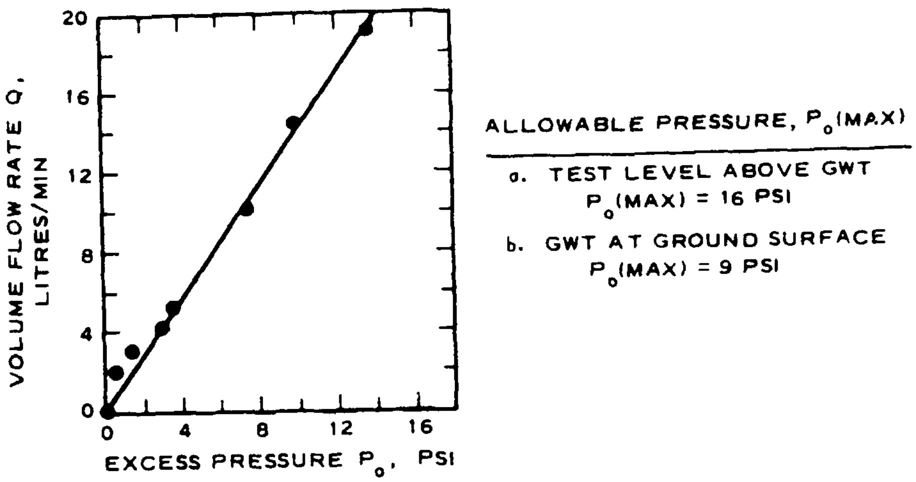


Fig. 4. Pressure test in a vertical borehole between depths of 13.3 and 18.8 ft (after Maini, 1971; Zeigler, 1975).

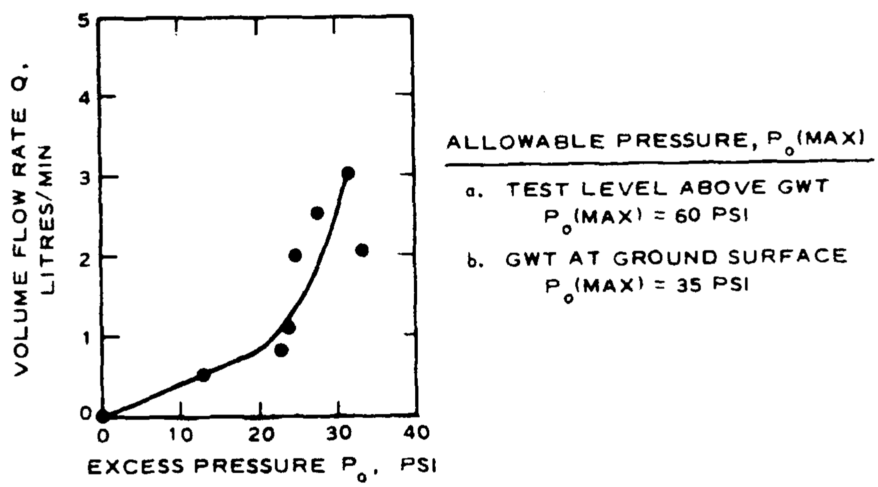
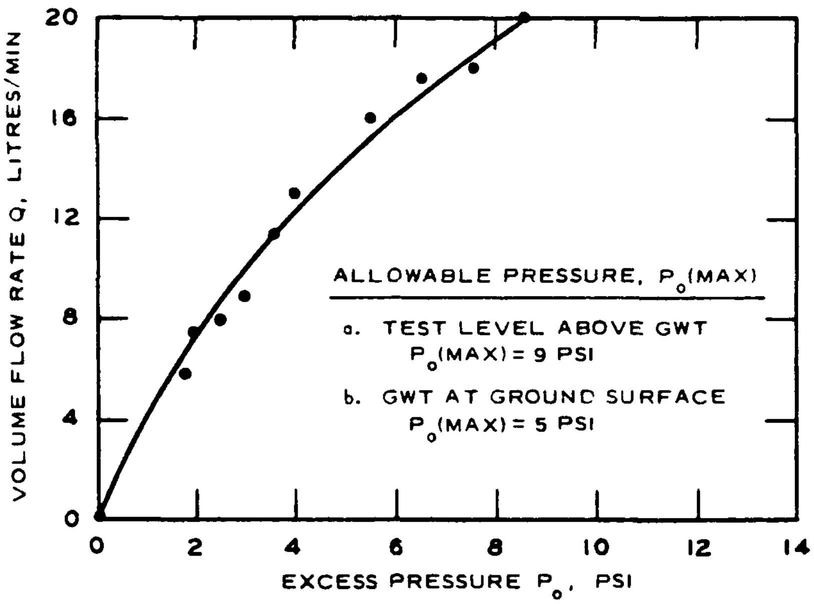
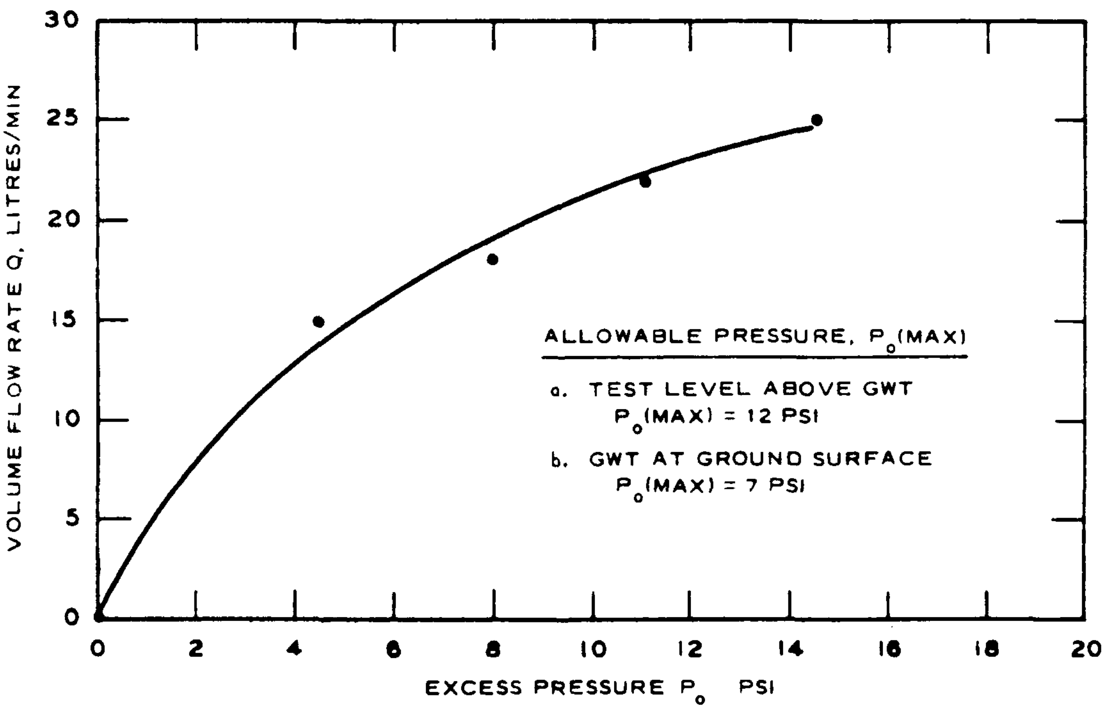


Fig. 5. Pressure test in a vertical borehole between depths of 58.3 and 63.8 ft (after Maini, 1971; Zeigler, 1975).



a. TEST CONDUCTED IN A VERTICAL BOREHOLE BETWEEN DEPTHS OF 6.0 AND 11.5 FT



b. TEST CONDUCTED IN A VERTICAL BOREHOLE BETWEEN DEPTHS OF 10.0 AND 15.5 FT

Fig. 6. Results of pressure tests (after Maini, 1971; Zeigler, 1975).

2.4.3 A typical test sequence using five separate pressures is as follows:

Test No.	Excess Pressure at Center of Test Section, P_o
1	1/5 P_o (MAX)
2	2/5 P_o (MAX)
3	3/5 P_o (MAX)
4	4/5 P_o (MAX)
5	P_o (MAX)
6	1/5 P_o (MAX)
7	3/5 P_o (MAX)
8	P_o (MAX)

The range of pressures over which tests should be conducted can be estimated by choosing P_o(MAX) to equal 1 psi/ft (22.62 kPa/m) of depth above the water table and 0.57 psi/ft (12.89 kPa/m) of depth below the water table. It is not intended that the computed P_o(MAX) be interpreted as a limit below which only laminar flow will occur; it should be used only as a guide in selecting a series of test pressures. Test results should be plotted as they are obtained to determine if further testing of the interval at other pressures is necessary to completely describe any nonlinear behavior.

2.5 Test Data Reduction - The quantities required for use in computing permeability parameters are:

(a) Length of the test section, ℓ (L).

(b) Radius of borehole, r_o (L).

(c) Number, n, and location of fissures intersecting the borehole test section.

(d) Elevation of groundwater table (L).

(e) Volume flow rate, Q (L^3/T).

(f) Excess pressure head at the center of the test section, H_o(L).

2.5.1 The test section length, ℓ, is simply the distance between the packer and borehole bottom (single packer test) or between the two packers (double packer test) as shown in Fig. 1. The radius of the borehole, r_o, is determined from the drill equipment. The number, n, and location of fissures intersecting the borehole test section are obtained from study of the core or a borehole camera survey. The fissure data are not needed if only equivalent permeability is computed based on the assumption that the tested medium is homogeneous and isotropic. The elevation of the groundwater table is determined before testing and assumed to remain constant during testing.

2.5.2 The volume flow rate, Q, will have been continuously recorded or determined by averaging the volume of flow over known time periods. The excess pressure head at the center of the test section, H_o, is a measure of pressure in height of water and is determined from

$$H_o = \frac{P_o}{\gamma_w} \tag{1}$$

where

P_o = excess pressure at center of test section (F/L^2)

γ_w = unit weight of water (F/L^3)

If total pressure in the test section is measured during the test with, for example, an electric transducer, the excess pressure head, H_o, is given by

$$H_o = \frac{P_t}{\gamma_w} - \frac{P_{t_i}}{\gamma_w} \tag{2}$$

where

P_t = total pressure at the center of the test section (F/L^2)

P_{t_i} = pretest (or natural) groundwater pressure at the center of the test section (F/L^2)

The pressure head (P_{t_i}/γ_w) will generally be equivalent to the height of the groundwater table above the center of the test section. In tests above the groundwater table, P_{t_i}/γ_w will be zero. If a natural groundwater pressure exists and is set equal to zero on the recording device, the excess pressure, P_o, and not total pressure will be recorded during testing and H_o would be determined from Equation 1.

2.5.3 The excess pressure head, H_o, can also be determined from gage pressure measured at the ground surface. The following relationship is derived by application of Bernoulli's equation

$$H_o = \frac{P_g}{\gamma_w} + \frac{v_g^2}{2g} + H(\text{gravity}) - h_t \tag{3}$$

where

P_g = pressure measured at the surface gage (F/L^2)
V_g = flow velocity at the surface gage (L/T)
g = acceleration due to gravity (L/T^2)
H(gravity) = excess pressure head due to the height of the water in the flow pipe (Fig. 1) (L)
h_t = sum of all the head losses between the surface gage and the test section (L)

By assuming the test section and surrounding medium to behave as a large reservoir, the head loss at the exit from the flow pipe to the test section can be approximated as $v_e^2/2g$, where v_e is the flow velocity at the exit point as noted by Vennard.[3.6] Equation 3 can be revised to

$$H_o = \frac{P_g}{\gamma_w} + \frac{v_g^2}{2g} + H(\text{gravity}) - h_L - \frac{v_e^2}{2g} \tag{4a}$$

where h_L = friction head loss plus minor losses due to pipe bends, constrictions, and enlargements (L). Pipe diameters at the surface and test section are usually equal, such that $v_g = v_e$. Also, the minor pressure losses due to pipe bends, constrictions, and enlargements can normally be ignored. Consequently, the pressure head, H_o, can be expressed as

$$H_o = \frac{P_g}{\gamma_w} + H(\text{gravity}) - h_f \tag{4b}$$

where h_f = friction head loss (L).

2.6 Equivalent Permeability - An equivalent permeability should be computed for each test section. Equivalent permeability is computed based on the assumption that the tested medium is homogeneous and isotropic. An equivalent permeability can be computed for laminar or turbulent flow, whichever is indicated by the test data. Radial flow will be assumed since the geometry of the test section (in particular, the high borehole length to diameter ratio) tends to dictate radial flow in a zone near the borehole which is most affected by the pressure test:

(a) Laminar flow governed by Darcy's law ($v = k_e i$, where v = discharge velocity (L/T), k_e = laminar equivalent permeability (L/T), and i = hydraulic gradient (L/C):

$$k_e = \frac{Q}{\ell H_o} \frac{1}{2\pi} \ln (R/r_o) \tag{5}$$

The radius of influence, R, can be estimated from $\ell/2$ to ℓ. To compute k_e, a value of volume flow rate, Q, and corresponding excess pressure head, H_o, are chosen from a straight-line approximation of a plot of Q versus H_o. The straight line must pass through the origin as shown in Fig. 4.

(b) Turbulent flow governed by the Missbach law ($v^m = k'_e i$, where k'_e = turbulent equivalent permeability $(L/T)^m$, and m = degree of nonlinearity):

$$k'_e = \frac{Q^m(R^{1-m} - r_o^{\,1-m})}{(2\pi\ell)^m\, H_o(1 - m)} \qquad (6)$$

The radius of influence, R, can be estimated from $\ell/2$ to ℓ. The degree of nonlinearity, m, is determined as the arithmetic slope of a straight-line approximation to a plot of log H_o versus log Q. The log-log plot may involve all or only a portion of the test data. The value of m should be between 1 and 2. To compute k'_e, values of flow rate, Q, and corresponding excess pressure head, H_o, are chosen from the approximated straight-line log-log plot.

2.6.1 In computing equivalent permeability of particular fissure systems, test section length, ℓ, should be replaced by the term nb_{avg} where n is the number of fissures intersecting the test section, and b_{avg} is the average spacing between fissures intersecting the test section. Substitution of nb_{avg} for ℓ is important where fissures are clustered over a small portion of the test interval. A fairly even distribution of fissures along the test length will normally yield ℓnb_{avg}.

2.7 <u>Permeability of Individual Fissures</u> - Laminar or turbulent permeabilities are estimated for individual fissures by assuming the test section to be intersected by a group of parallel and identical fissures. Each fissure is assumed to be an equivalent parallel plate. Flow is assumed to be radial and to occur only within the fissures. The material between fissures is assumed impermeable. The following equations are applicable:

(a) Laminar flow governed by Darcy's law ($v = k_j$ (where k_j = laminar fissure permeability (L/T))). The equivalent parallel plate aperture, e, is computed from Equation 7 below and used to compute the permeability of each fissure, k_j, from Equation 8:

$$e = \left[\frac{Q \ln (R/r_o)}{2\pi n H_o} \frac{12\mu_w}{\gamma_w}\right]^{1/3} \tag{7}$$

where μ_w = dynamic viscosity of water $(F\text{-}T/L^2)$

and

$$k_j = \frac{e^2 \gamma_w}{12\mu_w} \tag{8}$$

To compute e, corresponding values of Q and H_o are chosen from a straight-line approximation of Q versus H_o, which must pass through the origin as shown in Fig. 4. The radius of influence, R, can be estimated between $\ell/2$ and ℓ.

(b) Turbulent flow governed by Missbach's law ($v^m = k'_j i$, where k'_j = turbulent fissure permeability $(L/T)^m$):

$$k'_j = \frac{Q^m (R^{1-m} - r_o^{\,1-m})}{(2\pi n e)^m H_o (1 - m)} \tag{9}$$

To apply Equation 9, an equivalent parallel plate aperture, e, must first be estimated from the linear portion of the flow rate, Q, versus pressure, H_o, curve (i.e., zone 1, Fig. 2) as given by Equation 7. The degree of nonlinearity, m, is the slope of a straight-line approximation to a log-log plot of H_o versus Q. The log-log plot may involve all or only a portion of the test data. Corresponding values of Q and H_o can be chosen from the straight-line log-log plot for substitution in Equation 9. The radius of influence, R, can be chosen between $\ell/2$ and ℓ.

2.8 <u>Directional Permeability</u> - Equivalent permeabilities computed for fissure sets must be interrelated to obtain overall directional permeabilities which are needed in continuum seepage analyses. Directional permeabilities can be obtained by adding the equivalent permeabilities of fissure sets (computed via Equation 5 or 6) oriented in the same direction. This procedure is illustrated for an assumed laminar flow in the three cases shown in Fig. 7. In case (a), the zone tested contains one set of horizontal fissures (fissure set 1). The vertical borehole in case (a) will give a measure of the laminar equivalent permeability, k_{e_1}, of fissure set 1. Permeability in the vertical direction, $k_e(V)$, would be that of the intact rock since there are no vertical fissures. Permeabilities in a direction contained within the horizontal plane (such as $k_e(H1)$ and $k_e(H2)$ in Fig. 7) would be interpreted as k_{e_1}, since k_{e_1} is based on a radial flow.

2.8.1 In case (b) there are two intersecting fissure sets: the horizontal fissures (fissure set 1) and a series of vertical fissures (fissure set 2). The pressure test boreholes are oriented so that each intersects only one of the fissure sets. It is assumed that each test measures only the permeability of the intersected fissure set. In computing directional permeabilities both fissure sets 1 and 2 can transmit flow in the horizontal direction, H2; consequently, their equivalent permeabilities are summed ($k_e(H2) = k_{e_1} + k_{e_2}$). In the vertical direction, V, and horizontal direction, H1, the equivalent permeability of each fissure set is considered separately ($k_e(V) = k_{e_2}$, $k_e(H1) = k_{e_1}$).

2.8.2 In case (c) there are three intersecting fissure sets. Three boreholes are each oriented to intersect only one of the fissure sets. The directional permeabilities are each the sum of equivalent permeabilities corresponding to two fissure sets ($k_e(V) = k_{e_2} + k_{e_3}$; $k_e(H1) = k_{e_1}$; $k_e(H2) = k_e + k_{e_2}$).

2.8.3 When only vertical boreholes are tested, but structures such as in case (b) and case (c) of Fig. 7 are known to exist, the additional permeability added by the other fissure sets must be estimated. This

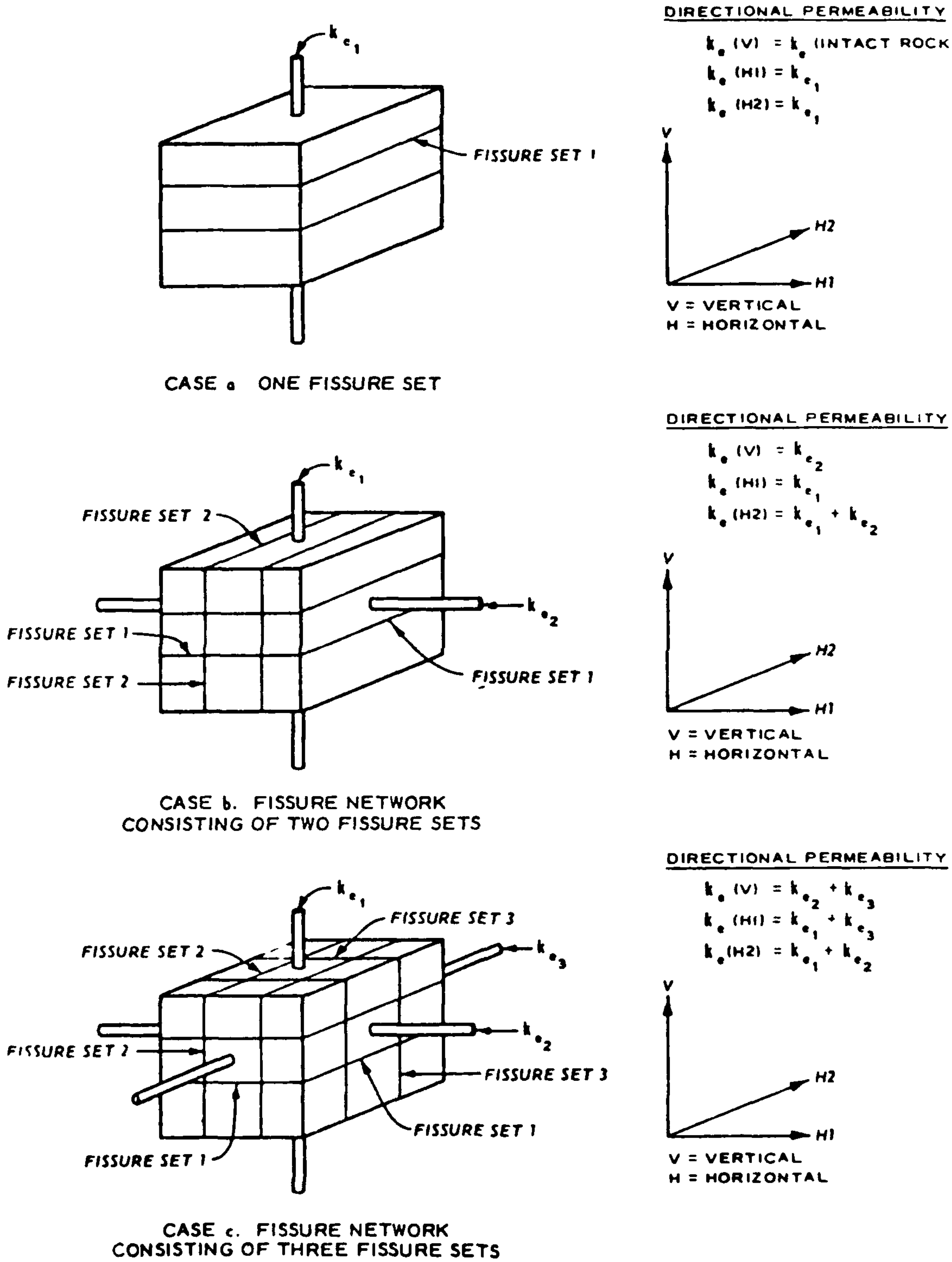


Fig. 7. Directional permeability from superposition of laminar equivalent permeabilities of fissure sets.

can be done based on the assumption that fissures not tested have the same equivalent parallel plate aperture as the tested fissures. Any difference in equivalent permeability between the fissure sets would be a function of the difference in fissure frequency. For example, in case (b) in Fig. 7 under conditions of laminar flow,

$$k_{e_2} = k_{e_1} \frac{b_{avg_1}}{b_{avg_2}} \tag{10}$$

where

b_{avg_1} = average fissure spacing of tested fissure set 1

b_{avg_2} = average fissure spacing of untested fissure set 2

2.8.4 The procedure of adding permeabilities of separate fissure sets relies heavily on the assumption that pressure tests reflect only the permeability of the fissure sets intersecting the test section. This assumption is based on the theoretical rapid loss in pressure away from the borehole. The assumption is likely to become less accurate as average fissure spacing within secondary fissure sets (i.e., fissures tending to parallel the borehole) is decreased. In complex fissure networks with fissure spacings less than 1 ft, it is recommended that the method of Snow,[3.4, 3.5] based on the assumption of a homogeneous anisotropic continuum, be used in computing directional permeabilities.

2.8.5 The problem of combining fissure set permeabilities is avoided by using discontinuum rather than continuum seepage analyses. In the discontinuum analyses, fissures can be oriented to correspond to the field geologic structure and assigned individual permeabilities and/or equivalent parallel plate openings as determined from pressure tests. However, a three-dimensional analysis such as that presented by Wittke et al.[3.7] would be required in many situations. In structures similar to case (c) in Fig. 7, a two-dimensional seepage analysis in any of the indicated directions would consider only two of the three fissure sets. For example, in direction H1. fissure set 3 would be ignored, although it may be a major contributor to seepage in direction H1.

3. References

3.1 Louis, C. and Maini, Y. N. T. (1970), "Determination of In Situ Hydraulic Parameters in Jointed Rock," Proceedings, Second Congress on Rock Mechanics, Belgrade, Vol 1.

3.2 Maini, Y. N. T. (1971), In Situ Hydraulic Parameters in Jointed Rock; Their Measurement and Interpretation, Ph.D. Dissertation, Imperial College, London.

3.3 Morgenstern, N. R. and Vaughn, P. R. (1963), "Some Observations on Allowable Grouting Pressures," Grouting and Drilling Mud in Engineering Practice, Butterworth and Company, London.

3.4 Snow, D. T. (1966), "Three-Hole Pressure Test for Anisotropic Foundation Permeability," Rock Mechanics and Engineering Geology, Vol IV, No. 4, pp 298-316.

3.5 Snow, D. T. (1969), "Anisotropic Permeability of Fractured Media," Water Resources Research, Vol 5, No. 6, Dec, pp 1273-1289.

3.6 Vennard, J. K. (1965), Elementary Fluid Mechanics, 4th ed., John Wiley and Sons, Inc., New York.

3.7 Wittke, W., Rissler, P., and Semprich, S. (1972), "Three-Dimensional Laminar and Turbulent Flow Through Fissured Rock According to Discontinuous and Continuous Models," Proceedings of the Symposium on Percolation Through Fissured Rock, Stuttgart, Germany.

3.8 Zeigler, T. W. (1976), "Determination of Rock Mass Permeability," Technical Report S-76-2, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi.

BIBLIOGRAPHY

Davis, S. N. and DeWiest, R. J. M. (1966), Hydrogeology, John Wiley and Sons, Inc., New York.

Hvorslev, M. J. (1951), "Time Lag and Soil Permeability in Groundwater Observations," Bulletin No. 36, Apr, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi.

Lynch, E. J. (1962), Formation Evaluation, Harper and Row, New York.

Muskat, M. (1946), "The Flow of Homogeneous Fluids Through Porous Media," J. W. Edwards, Inc., Ann Arbor, Michigan.