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	Engineering and Design CONSTRUCTION CONTROL FOR EARTH AND ROCK-FILL DAMS	
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US Army Corps
of Engineers

ENGINEERING AND DESIGN

Construction Control for Earth and Rock-Fill Dams

ENGINEER MANUAL

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DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

CECW-ED

Manual
No. 1110-2-1911

30 September 1995

Engineering and Design
CONSTRUCTION CONTROL FOR EARTH AND ROCK-FILL DAMS

1. Purpose. The purpose of this manual is to present principles and methods for construction control of earth and rock-fill dams.

2. Applicability. This manual applies to all Corps of Engineers divisions and districts having responsibility for construction of earth and rock-fill dams.

3. General. This manual is a guide to construction and inspection of earth and rock-fill dams in those aspects that pertain to safe and satisfactory performance.

FOR THE COMMANDER:

ROBERT H. GRIFFIN
Colonel, Corps of Engineers
Chief of Staff

This manual supersedes EM 1110-2-1911, dated 17 January 1977.

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Table of Contents

Chapter 1
Introduction

1-1. References

References pertaining to this manual are listed in Appendix A. Additional reference materials pertaining to the subject matter addressed in this manual are also included in Appendix A.

1-2. General Considerations

The safety and satisfactory performance of earth and rock-fill dams require competent and adequate supervision, careful inspection, and control testing. It is the responsibility of the Resident Engineer to bring to the attention of the Engineering Division any design or construction detail not adequately covered by the plans and specifications. The Resident Engineer must provide the field supervision and control required to accomplish the intent of the plans and specifications. The resident engineer must also coordinate the work with design engineers and provide guidance to the contractor if unexpected conditions are discovered during construction.

a. Importance of construction control. Many earth and rock-fill dams have shown signs of distress or experienced partial failure (necessitating expensive remedial measures) from causes traceable to poor construction practices or to unexpected adverse conditions. Close observations by soils engineers and geologists of foundation and abutment preparation, excavation, fill operations, movements and deformations of fill and foundation, and seepage can often enable early detection and correction of undesirable conditions. Construction control should ensure that:

(1) Necessary actions are taken to remedy or allow for unexpected conditions. Frequent and careful observations by inspectors, geologists, and field engineers, who are familiar with conditions assumed for design, are essential during stripping of the foundation, opening of borrow areas, and excavating operations. Immediate reporting of unexpected conditions will enable the Resident Engineer to coordinate and plan, with design engineers, any additional investigations needed to establish design modifications.

(2) Equipment and procedures are adequate to satisfactorily accomplish the work. Review of the contractor's plans for quality control, dewatering and draining work areas, and haul roads, together with inspections of the actual operations, is an important aspect of construction control.

(3) The completed structure meets the requirements and intent of the plans and specifications. This involves continuous inspection of foundation and abutment preparation, material processing, and embankment construction, and a comprehensive control testing program to ensure proper material placement and compaction.

(4) Adequate construction records are maintained. Preparation of completion reports of construction operations and maintenance of records of test results are essential aspects of field control. Such reports and records are often required to evaluate claims by the contractor and to determine possible causes of distress that might later occur in portions of the completed work. These documents should include as many detailed photographs as necessary. Video photography may also be included in the methods to document and record construction procedure and methods.

b. Relation of construction and design. The design of an earth or rock-fill dam is not finished until construction is complete, the reservoir has been filled, and the dam is functioning satisfactorily. During construction, design engineers should frequently reassess design concepts and assumed conditions in light of actual conditions observed in the field. This involves frequent visits to the project to observe actual conditions and construction procedures. Consultation with specialists may be required to evaluate unusual problems or foundation conditions. Design evaluation must include analyses of compaction control results. It may also require reanalysis of stability conditions based on results of laboratory tests on record samples and additional foundation samples and field observations of pore water pressures, settlements, and lateral displacements. A high degree of coordination between design and construction engineers is mandatory.

Chapter 2
Field Organization and Responsibility

2-1. Resident Inspection Force

Each levee, earth or rock-fill dam, or other embankment must be adequately inspected to ensure that plans and specifications are observed and followed by the contractor. This requirement applies to all divisions and districts having responsibility for design and construction of civil works projects. The size and composition of the resident inspection force for foundation and earthwork control operations should be adequate to provide for continuous inspection of construction activities, field testing and sampling, observation of field instrumentation, laboratory testing, data compilation, maintenance of records, and preparation of reports. On large projects, the contractor may operate as many as three shifts, and materials-handling for fill operations may be highly mechanized to obtain high production. The inspection force must be so staffed and organized that inspectors and technicians are available for continuous inspection of the contractor's operations. In discussing contractor quality control, ER 1180-1-6 states, "The Government is responsible for all phases of the construction project, including the activities necessary to assure that the contractor has complied with the requirements of the contract plans and specifications..." and "In contrast to the Contractor's quality control the government is responsible for quality assurance. This includes checks, inspections, and tests of the products which comprise the construction, the processes used in the work and the finished work for the purpose of determining whether the Contractor's quality control is effective and he is meeting the requirements of the contract. These activities are to assure that defective work or materials are not incorporated in the construction."

a. Technical responsibilities. The Resident Engineer is responsible for constructing the embankment and related appurtenances in compliance with plans and specifications. On large projects, resident geologists and soils engineers provide technical assistance. Assisting the Resident Engineer are office and field engineering staffs. The office engineering staff is responsible for preparing field modifications to the plans and specifications in accordance with applicable district regulations, reviewing plans submitted by the contractor such as those for quality control and dewatering, evaluating results of construction control tests, and compiling instrumentation data to send to the Engineering Division for evaluation. The field engineer in charge of field supervision and inspection is responsible for planning, executing, and coordinating all field inspection and testing to ensure compliance with established standards and detail drawings and specifications. The field engineer is assisted by one or more chief construction inspectors on each shift who coordinate the activities of subordinate inspectors, and by a materials testing or soils engineer who supervises a number of technicians in obtaining samples and performing required field and laboratory tests. Detailed technical and organizational responsibilities may differ in the various districts and divisions; however, construction projects are to be staffed with the number of experienced Government laboratory technicians needed to perform Government acceptance testing on compacted fill, including filter and drainage fills. Acceptance testing should be performed immediately after placement and compaction of the lift material to be sampled and tested. Attention should be given to selection of samples for acceptance testing so that all materials of generally different descriptions being placed in the same compacted zone of the embankment will be tested.

b. Preconstruction training. Every earth or rock-fill dam is designed for specific foundation conditions and to utilize locally available materials. Unique features are inherent in each project, and a wide variety of construction methods may be utilized. Therefore, good communication between design and construction personnel is essential. The construction staff should be familiar with the design memoranda pertinent to the work. Preconstruction instructions and training should be given to field inspection personnel to acquaint them with the design concepts and to provide them with a clear understanding of expected conditions, methods of construction, and the scope of plans and specifications. This may be done by training sessions, preferably with design personnel present, using a manual of written instructions prepared especially for field personnel, to discuss engineering considerations involved and to explain control procedures and required results. Inspection personnel should be familiar with the plans and specifications; excavation boundaries; types of materials to be excavated; temporary and permanent drainage and seepage control measures; approved sources of borrow materials; procedures and equipment most suitable for excavating, processing, and hauling borrow materials; characteristics of fill materials and compaction requirements; capabilities of various types of compaction equipment; and procedures required to obtain desired or specified compaction. Closely supervised on-the-job training should be given to inspectors and materials testing personnel during initial stages of construction to increase their proficiency in recognizing signs of inadequate compaction, in using expedient methods of checking water content and density of fill materials, in using selected methods for field density measurements and laboratory compaction, and in detecting inadequate construction procedures and unsafe conditions.

c. Number of personnel and skills required. Experienced construction engineers, inspectors, and technicians are required for construction control operations on earth and rock-fill dams. The Resident Engineer, the field engineer in charge of supervision and inspection, chief inspector, materials engineer or chief soil technician, and geologists should have been associated with the project from the time of any preliminary construction operations such as test fills, quarry blasting and rock production tests, and excavations of tunneling made to inspect subsurface conditions. Augmentation of this cadre with less experienced inspectors and technicians will provide a sufficiently capable inspection force. An example of a resident inspection force organization for a large earth and rock-fill dam is shown in Figure 2-1. Additional inspectors and technicians may be required during certain phases when construction operations are at their peak or when several major portions of the earth or rock-fill dam are being constructed concurrently. An example of a typical similar organization for a smaller earth dam project is shown in Figure 2-2. It should be noted that the organization for a small earth dam is very flexible, being dependent on the magnitude and extent of the construction. Small projects often require temporary assignment of specialized personnel, such as soils engineers and geologists, during certain construction phases.

d. Quality control.

(1) The contractor is responsible for quality control, and the contract specifications give requirements for the contractor quality-control organization, personnel qualifications, facilities and types of tests, and reporting of test data and inspections. The Government field inspection force has the responsibility of accepting completed work and must have a staff large enough to accomplish the following:

(a) Check the effectiveness and adequacy of the contractor's quality-control system and take action to have deficiencies corrected.

(b) Inspect construction operations to prevent defective work and placement of unsatisfactory materials.

(c) Monitor progress.

(d) Perform check tests and acceptance tests.

(e) Resolve or report field problems and conflicts with contract documents to higher authority.

(f) Make acceptance inspections.

(2) Contractor quality-control operations will, if properly implemented, assist in achieving adequate construction,

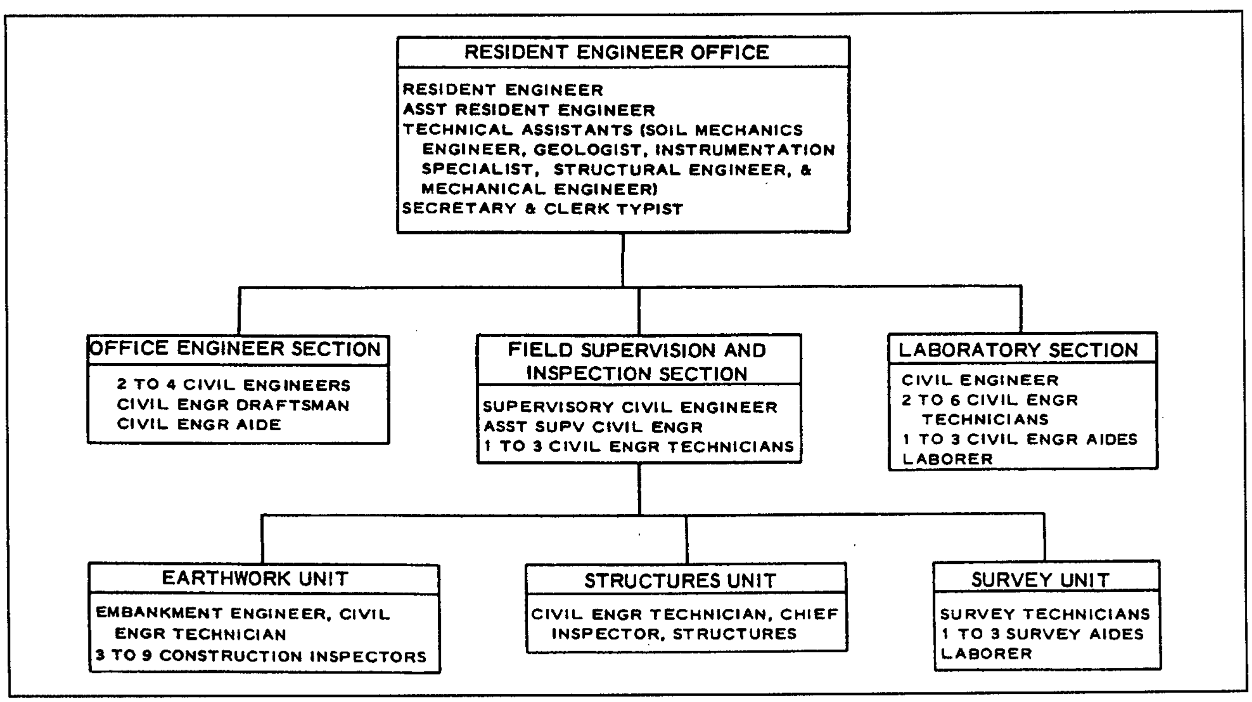


Figure 2-1. Example of resident engineer's staff organization for large earth dam project

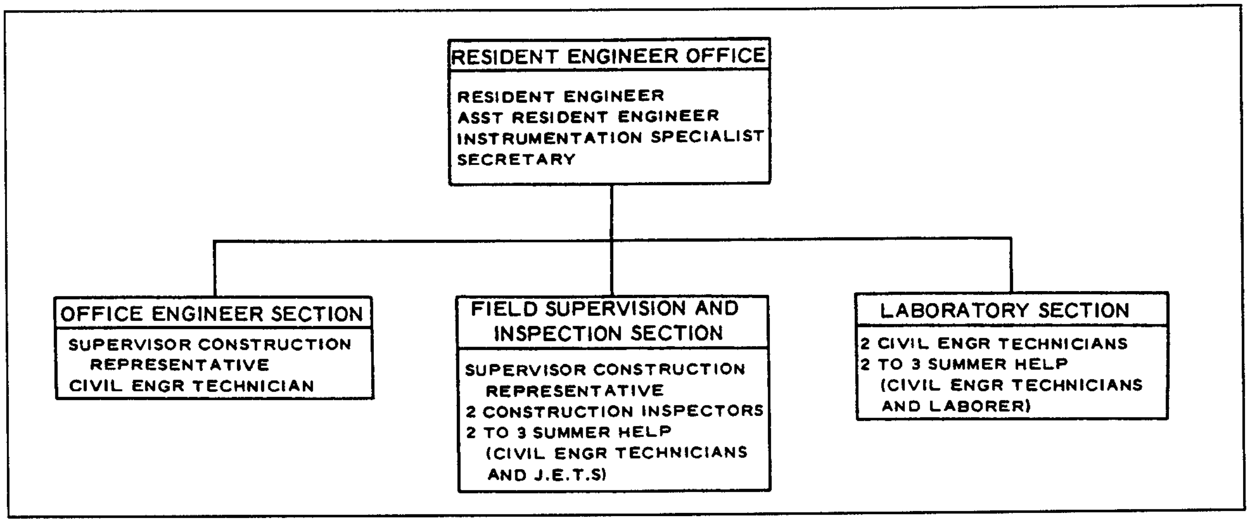


Figure 2-2. Example of resident engineer's staff for small earth dam project

particularly on those operations where specifications contain product requirements such as water content limits and gradation of filter material. However, Government field inspectors must be present to witness operations for which construction procedures are specified and to conduct tests to ensure that results obtained are those required by design. Several important considerations are listed below:

(a) The contractor is not required to conduct tests on the quality or durability of materials used for pervious fill, filter zones, bedding, spalls, or rip-rap. The Government is responsible for determining such properties in approving sources of natural or processed (blasted, ripped, or crushed) pervious materials for use in embankments.

(b) The contractor is not required to provide tests on compaction characteristics when fill placement procedure is specified. Government inspection forces are responsible for inspecting the contractor's specified construction activities and for testing embankment or slope materials to establish that excavation and fill placement procedures result in an excavated slope or fill that conforms to properties assumed in the design.

e. Relationship with contractor. Mutual understanding between the Resident Engineer and his staff and the contractor of the requirements of the contract specifications is necessary to obtain desired construction quality. The quality of the work must never be compromised. Unnecessary requirements and restrictions should not be imposed over and above the specification requirements. Firmness by Resident Engineers and inspectors who are efficient and know their job will gain the respect and cooperation of the contractor.

2-2. Field Laboratory

Foundation and earth-fill materials are tested in the field laboratory to determine gradation, water content, in situ density, compaction, relative density, and Atterberg limits. The data obtained serve as a basis for ensuring compliance with specifications and design requirements, for guidance toward maximum utilization of available materials, and for providing a record of the properties of materials placed in all parts of the project.

a. Size. The size of the field laboratory and satellite testing units depends on the magnitude and areal extent of construction operations. Remote borrow areas, dikes, or relocation fills may require a central laboratory, supplemented by one or more skid- or wheel-mounted portable units. An existing structure, a prefabricated structure, or trailer may be specified. For relatively large projects, about 500 to 1,500 sq ft may be required for a laboratory space. The laboratory should have a sound floor, necessary worktables, benches, storage cabinets, equipment pedestals (for compaction mold base and sieve shakers), service sinks, and utilities. Additionally, it may be very advantageous to have an awning-covered work slab with an area of from 500 to 1,000 sq ft to serve as a work space on which to dry and work large samples, to perform coarse aggregate gradations, etc.

b. Types of tests and facilities required. The major portion of testing in the field laboratory is for acceptance testing of compacted fill. These tests must be conducted in a manner that will yield results of a quality comparable to the initial laboratory tests upon which the design was based.

Discrepancies resulting from variations in testing equipment and techniques should be carefully avoided. Water content, compaction, relative density gradation, and Atterberg limits tests are the most common tests conducted. Water content and compaction tests are used for control of cohesive soils in impervious and random fills. Atterberg limits tests may be used for control of fills of fine-grained cohesive soils where good correlations of optimum water content and maximum dry density with the Atterberg limits have been established. Gradation and relative density tests are used for control of pervious fills. Gradation tests are also used for control of rock fill. Field density tests are performed on the fill, but compaction tests on the material and water content determinations may be made either in the central laboratory or in the field. The central laboratory must have equipment for these tests, but supplemental portable units may be advisable for gradation, compaction, and water content testing at remote locations. Panel pickup trucks are often used to transport equipment for field density testing, undisturbed (record) sampling in the fill, and sampling in borrow areas. Specially equipped pickup trucks with a small hoist may be required where large-scale field density tests are to be performed on material such as rock fill or soils with a high percentage of large gravel sizes.

2-3. Assistance by Higher Echelon

Unusual conditions encountered during construction generally require special attention. The advice of specialists in soil and rock mechanics, geology, and instrumentation of earth and rock-fill dams, and additional evaluation by design engineers are often required to obtain effective solution to unusual problems and conditions.

a. Geologists and soils engineers. Specialists with experience in soil and rock mechanics, geology, and instrumentation are found in division and district offices, at Headquarters, U.S. Army Corps of Engineers (HQUSACE), Waterways Experiment Station (WES), and on Corps of Engineers boards of consultants. The services of soil and rock mechanics engineers and geologists are particularly valuable during early stages of construction when the foundation, abutment, and any diversion tunnels or excavations (such as for spillway foundations) expose existing conditions. At this time, it is vital that actual conditions be evaluated to determine if they are consistent with conditions assumed for design. In addition, it is necessary to recognize any unusual conditions that may affect construction. The advice of specialists in soil mechanics and rock mechanics is also valuable in establishing, from observed field conditions, modifications that may significantly improve the design without increasing the cost of the project.

b. Design engineers. The engineer who designs the earth or rock-fill dam should visit the project during construction and assist field personnel in interpreting plans and specifications and observe problems that may not have been fully evaluated in the design. Visits should also be made whenever unexpected conditions are encountered that may require changes in the plans or specifications. A cooperative attitude must be maintained between design engineers and construction personnel so that mutual understanding is reached on existing problems and feasible solutions are developed. In some cases, conferences at the construction site may be necessary between construction personnel, designers, and specialists to review conditions and determine if design modifications are required. A regular schedule of visits should be set up so that design personnel and representatives from the division office and HQUSACE can inspect field conditions at critical construction stages.

c. Instrumentation. Instrumentation of earth and rock-fill dams is becoming increasingly important. The main reasons are that many higher earth and rock-fill dams are being constructed, sites having unfavorable foundation conditions must be used more frequently, interest is increasing in obtaining meaningful data for evaluating dam behavior, and continually increasing downstream land development is increasing the consequences of failure on property damage and loss of life. Monitoring of pore water pressure, settlements, and deformations of the foundation and embankment is necessary to check the safety of the dam during construction and to control the rate of construction. The instrumentation must be of the proper type, placed in critical locations, and installed properly. Valid readings depend on techniques and procedures used in installing and observing the instrumentation. For this reason, specialists experienced in field instrumentation should plan and supervise the field installations. These specialists can be from the district and/or division office, from WES, or from firms specializing in installations of instrumentation of earth and rock-fill dams. This applies whether the instrumentation is furnished and installed by the Government or furnished and installed by the contractor. Because proper interpretation of instrumentation data is vital to the safety of the dam, the responsibility for collecting and reporting data to the Engineering Division should be carefully delegated. Installation and observations of instrumentation are discussed in paragraph 6-5; the general use of instrumentation is described in Appendix E and by Dunnicliff (1988).

2-4. Records and Reports

Construction records and reports are needed to document results of tests made to verify specification requirements and action taken to correct deficiencies and to provide a record describing the field conditions, modifications to plans and specifications, construction procedures, sequence of operations, and the location and as-built dimensions of important features. These are necessary to evaluate claims made by the contractor based on changed conditions, or claims by the Contracting Officer that work performed does not meet contract requirements. Progress reports are also needed for the district office and to provide a basis for payments to the contractor for work accomplished. Inspectors must maintain a daily inspection report (or log), and a master diary must be kept by the Resident Engineer. The required content of these documents is outlined in EM 415-1-302, "Inspection and Work Records." Details of specific construction control records and reports are described in Chapter 7.

a. Construction records. These records provide useful data for designing future alterations and additions to the structure, determining causes of later undesirable movement or seepage, or interpreting piezometric data. As-built drawings, construction photographs, descriptions of foundation conditions encountered and various treatments, compaction data, and test data on record samples should be included in the records.

b. Construction reports. The construction foundation report should include details such as dip and strike of rock, faults, artesian and other groundwater conditions, and other characteristics or conditions of foundation materials. A complete history of the project in narrative form should be written, giving the schedule of starting and completing various phases of the work, describing construction methods and equipment used, summarizing quantities of materials involved, and including other pertinent data. Foundation reports should be supplemented by photographs that clearly depict foundation conditions. Routine photographs should be taken at regular intervals, and additional pictures should be taken of items of specific interest, such as the preparation of foundations and dam abutments. For these items, colored photographs should be taken to provide a better depiction of construction conditions. The captions of all photographs should contain the name of the project, the date on which the photograph was taken, the identity of the feature being reported, and the location of the camera. In reports containing a number of photographs, an alternative would be an index map with a circle indicating the location of the camera with an arrow pointing in the direction the camera was pointing, with each location keyed to the numbers on the accompanying photographs. Details concerning the use and preparation of construction foundation reports are presented in ER 1110-1-1801.

Chapter 3
Foundation and Abutment Treatment

3-1. General

The preparation of the foundation and abutments for an earth or rock-fill dam is a most difficult and important phase of construction; the thoroughness with which it is done is reflected in the performance of the completed structure. It is often difficult or sometimes impossible to correct foundation and abutment deficiencies that show up after construction is well underway or completed. The primary purpose of foundation and abutment treatment is to obtain positive control of under seepage, prepare surfaces to achieve satisfactory contact with overlying compacted fill, and minimize differential settlements and thereby prevent cracking in the fill. Inspection of the work must ensure that the foundation and abutments are stripped to depths sufficient to remove soft, organic, fractured, weathered, or otherwise undesirable materials; depressions and joints in rock surfaces are cleaned and adequately filled; rock surfaces are made relatively smooth and uniform by shaping and filling; subsurface cavities are detected and grouted; and cutoffs extend to suitable impervious materials. During this phase of construction, close liaison must be maintained between construction and design personnel since most discrepancies between design and field construction occur in this portion of the work. Few dams are constructed without encountering some undesirable foundation conditions that were not discovered in exploration for design, such as zones of weathered or fractured rock, cavities, soft soil areas, abandoned pipes or drains, or abandoned stream channels filled with sand and gravel. This is the reason that inspection trenches are generally required beneath the impervious zone of a dam when cutoff trenches are not specified. These inspection trenches provide the means for careful examination of the foundation along the entire length of the dam to ensure that undesirable foundation conditions are detected.

3-2. Clearing, Grubbing, Stripping, and Cleaning

Clearing, grubbing, stripping, and cleaning of areas upon which a compacted earth or rock-fill dam will lie are required to remove those materials having undesirable engineering qualities such as low shear strength, high compressibility, undesirable permeability, or other characteristics which would interfere with compaction operations; and provide a surface favorable for a good bond with the overlying fill. Specifications should provide adequate time for inspection by the Contracting Officer's representative of exposed foundation and abutments. In some cases where abutments will require special treatment, a separate contract for such work is awarded.

a. Soil foundation and abutments.

(1) Clearing consists of removal of all aboveground obstructions, including trees, vegetation, felled timber, brush, abandoned structures, local dams, bridges, and debris. Grubbing includes removal of all objectionable belowground obstructions or material including stumps, roots, logs, drain tiles, and buried structures or debris. Foundation or abutment soils disturbed during clearing and grubbing operations must be removed. Blasting should be avoided if possible; if unavoidable, explosive charges should be kept as small as possible.

(2) Foundation and abutment stripping generally follow clearing and grubbing operations. Stripping consists of the removal of sod, topsoil, boulders, and organic or foreign materials. Stripping beneath closure sections should be performed in the dry after diversion of the river. Necessary or inadvertent deviation from stripping limits identified in the plans and specifications should be reported to the design office so that effects of the changes can be evaluated. Personnel inspecting stripping operations should be able to identify the materials to be removed. Inspectors should look for soft pockets as well as old sloughs or river meanders that may not have been found during design investigations; the resident geologist can assist in locating such features. Several passes of a heavy roller should be made over the stripped surface to "proof test" the area to reveal any unsuitable materials overlooked during stripping.

(3) The sides of holes and depressions left by grubbing and stripping should be flattened and scarified and the depressions filled with material of the same type and compacted to a density at least equal to that of the surrounding foundation material by the specified method and equipment. Where areal dimensions of depressions are small, power hand tampers are required to compact fill. Final preparation of the foundation surface, immediately prior to placing embankment fill, should include adjusting soil water contents as near optimum as possible, compacting as prescribed for the overlying fill, and scarifying the compacted surface to receive the initial embankment lift.

(4) Compaction of some types of saturated soils in wet foundation areas may do more harm than good. When it is not feasible to dry such areas out, it may be necessary to place a thick initial lift to permit compaction equipment to operate without remolding and disturbing the foundation soil. Also, the weight of compacting equipment operating on the initial lift might be reduced and progressively increased as more lifts are placed. However, this should not

be done for foundation areas under the embankment unless specifically permitted by the plans and specifications or approved by the design office, as the effects of a lightly compacted layer at the base of the dam could adversely affect stability.

(5) Preparation of soil abutments prior to fill placement should be the same as that for soil foundations. To ensure bonding of the embankment to the natural soil of the abutments, it is necessary to remove some of the abutment surface soil. Inspection should confirm that all loose, wet, or soft soils are removed. In addition, abutment slopes should be smooth and as flat as economically feasible at contact with the embankment to improve compaction of fill against the abutment and to minimize the probability of differential settlement causing cracking (paragraph 3-4*a*(5)). Depressions should be filled with concrete or soil compacted at proper water contents to densities equal to or greater than those of the materials to be placed above them in the embankment fill. See paragraph 3-4*a*(2) for discussion of treatment of abutment slopes of clay shales.

b. Rock foundations and abutments.

(1) After all rough excavations of overburden and/or weathered rock have been completed, all grouting is completed, and the surface of the rock foundation is exposed, shaping and cleaning operations should begin. Shaping and cleaning a rough rock foundation are necessary to provide a smooth, uniform, and clean surface against which fill can be compacted. The procedure generally consists of removing large loose rocks, overhangs, and projecting knobs by scaling, handpicking and wedging, and light blasting pressure washing followed by some form of "dental treatment" to fill all holes, cracks, joints, crevices, and depressions. Dental treatment involves cleaning the cavities and backfilling them with concrete, and is discussed in more detail in paragraph 3-4*b*(3). The resident geologist or embankment engineer should inspect and approve this phase of the work.

(2) The final preparation of almost all rock foundations requires hand labor. The use of heavy or tracked vehicles on the final foundation should be avoided, especially if the rock is thinly bedded or badly jointed. Blasting to remove knobs or overhangs may prove more harmful than helpful, and extreme caution must be exercised to prevent the opening of cracks or actual displacement of blocks or rocks that would otherwise provide adequate bearing. It is generally desirable to place concrete fill beneath or around projections if, by so doing, blasting can be avoided. Where concrete fill is used, materials and procedures should be directed towards ensuring good concrete/rock bond; subsequent fill operations should avoid dislocating the concrete. Hand methods involve removal of all loose or "drummy" rock (rock that sounds hollow when struck with a steel hammer or bar), and the scaling down of sloped surfaces to provide an even, uniform slope.

(3) Washing the hard rock foundation surface with water under high pressure and dry brooming to remove loose residue are generally the last step in foundation preparation. This is done to clean the surface to the maximum extent possible and to remove fines that may have worked into seams. All seams or cracks should be cleaned to a depth of at least twice their width. Removal of these fines will facilitate complete filling of seams in subsequent operations (such as dental treatment) taken to prevent seepage. Pressure washing also serves to detect rock projections overlooked during hand excavation which might otherwise work loose during compaction of the first lift or lifts of fill. Washing should be performed to clean from higher elevations to lower elevations.

(4) Particular attention should be given to cleaning openings that cross the axis of the dam. Accumulated water from the washing process must be removed. Small air pumps, hand bailers, or aspirators may be used to empty narrow, water-filled fissures. If the foundation consists of blocky rock with frequent joints, caution must be used to avoid removal of satisfactory foundation material (such as stiff clay in joints) by overzealous pressure washing. When the nature of the rock is such that it could be softened by washing with water, compressed air should be used instead of water. Air pressure is also often used as a final step in cleaning sound rock surfaces. Figure 3-1 shows the rock foundation at DeGray Dam being cleaned with compressed air.

Figure 3-1. Final foundation cleaning using compressed air, DeGray Dam, Arkansas

(5) Where rough and irregular surfaces remain after hand excavation, troughs, pits, and other depressions are filled with concrete to provide a more even surface on which the first layer of the embankment may be compacted. As previously noted, this procedure is termed dental treatment and is discussed further in paragraph 3-4*b*(3). If foundation grouting has been performed, cleanup operations should include removal of any spilled or washed grout that might otherwise conceal surface imperfections and pockets of undesirable material.

(6) Before placing the first layer of embankment material, the cleaned and prepared rock surface should be moistened, but no standing water should be permitted. Moistening the rock surface is recommended instead of using overly wet soil in the first lift to obtain good contact. Use of heavy pneumatic equipment (preferably a rubber-tired roller) is recommended for compacting the first lift on rock surfaces. This will enable the rock surface to be kept intact, especially where the rock surface is irregular or composed of thin beds of alternating hard and soft rock.

(7) Foundations consisting of compaction-type shales and slaking tuffs should be protected from disintegration caused by drying due to exposure to air. The handling of clay shales is discussed in paragraph 3-4*a*(2).

(8) The same degree of care should be exercised in abutment treatment as in foundation treatment. A good bond between the embankment and the abutment is critically important. Areas to be cleaned at rock abutments should include not only those beneath the embankment core but also those beneath transition or filter zones. Within these areas, all irregularities should be removed or trimmed back to form a reasonably uniform slope on the entire abutment with vertical surfaces no higher than 5 ft. Benches between near-vertical surfaces should be of such width as to provide a stepped slope comparable to the slope on adjacent areas but not steeper than 1V on 1H. Overhangs should not be permitted at any locations. Methods of overhang removal are discussed in paragraph 3-4*b*(4).

(9) The treatment of cracks, fissures, and other undesirable conditions in rock foundations and abutments is discussed in paragraph 3-4*b*(2).

3-3. Seepage Control

a. Cutoffs. Foundation cutoffs or core trenches serve as barriers to underseepage. The design of foundation cutoffs is based largely on borings made during field investigations for design. Therefore, the open excavation of a cutoff trench provides the first real look at actual foundation conditions; frequent inspections, particularly by the field geologist and embankment engineer, should be made. Some common types of cutoffs are discussed in the following paragraphs.

(1) Compacted backfill trenches. Backfill compacted into a seepage cutoff trench is one of the most effective construction devices for blocking foundation seepage. Material and compaction requirements are the same as for the impervious section of the embankment. When required by contract specifications, the trench must fully penetrate the pervious foundation and extend a specified distance into unweathered and relatively impervious foundation soil or rock. Treatment (as described in paragraph 3-2*a*) of the exposed surface in the bottom and sides of the trench is essential to ensure firm contact between foundation and backfill. The trench excavation must be kept dry to prevent sloughing of the side slopes and to permit proper backfill placement and compaction. When the water table is near the ground surface, dewatering the excavation is required and is frequently a major expense in cutoff construction. Dewatering and drainage methods are discussed in paragraph 3-5. In any trenching operation, a qualified geotechnical engineer should inspect the construction at regular intervals to monitor stability of the side slopes.

(2) Slurry trenches.

(a) The slurry trench method of constructing a seepage cutoff involves excavating a relatively narrow trench with near-vertical walls, keeping the trench filled with a bentonite slurry to support the walls and prevent inflow of water, and then backfilling with a plastic impervious mixture of well-graded clayey gravel to protect against piping, to reduce seepage, and to minimize consolidation of the backfill material.

(b) The backfill should be a mixture of impervious borrow, sand, gravel, and bentonite slurry (U.S. Army Engineer District, Savannah 1968). The backfill may be a mixture of material excavated from the cutoff trench and other material to provide an acceptable blend.

(c) Depending on the required depth, the excavation may be accomplished with a dragline, backhoe, clamshell, or trenching machine. A trenching machine is limited to depths less than about 40 ft, provided no cobbles exist. Unmodified backhoes are limited to depths less than about 45 ft but with special modification can reach depths of 55 to 60 ft; their main advantage is that they can be used in areas where cobbles exist. Maximum depths of about 100 ft have been achieved with a dragline. Required equipment modifications for excavation to a great depth (with a dragline) include weighting the bucket to overcome the buoyant effect of the slurry and providing heavy-duty bearings and hydraulic systems. A dragline excavating a slurry trench is shown in Figure 3-2.

(d) The specific gravity of the slurry must be high enough to ensure that hydrostatic pressure exerted by the slurry will prevent caving of the sides of the trench and yet not be so high as to limit the depth to which the excavating bucket will operate. Typical values of specific gravity of slurries used in past jobs range from 1.05 to 1.2, with some values as high as 1.5. The slurry level is generally maintained 2 to 3 ft above the groundwater level.

(e) Procedures for cleaning the bottom of the trench, removing sand which settles out of the slurry, continuous control of viscosity and specific gravity of the slurry, and mixing and placing the backfill are critical in achieving successful results. An example of successful slurry trench construction is that at West Point Dam, Chattahoochee River, Alabama and Georgia, in which the bottom of the trench was cleaned with a modified dragline bucket (Jones 1967; U.S. Army Engineer District, Savannah 1968). A scraper blade was attached to the bucket which, when dragged along the bottom of the trench, removed coarser soil particles and some of the finer loose material at the top of the rock. An air jet was used to remove sand, gravel, and other undesirable material from potholes, cracks and crevices; these materials subsequently became entrained in the slurry. Suction and discharge pipes were used to remove contaminated slurry (from the trench), which was cleaned by sending it to shallow sediment ponds along the sides of the trench where the contaminants settled out of suspension. The clean slurry was then placed back into the trench. Mechanical desanders are available and may be desirable or even required for removing sand (from the bottom of the trench) in some situations. The bottom of a trench should be sampled after it has been cleaned to ensure that it is properly free of undesirable material.

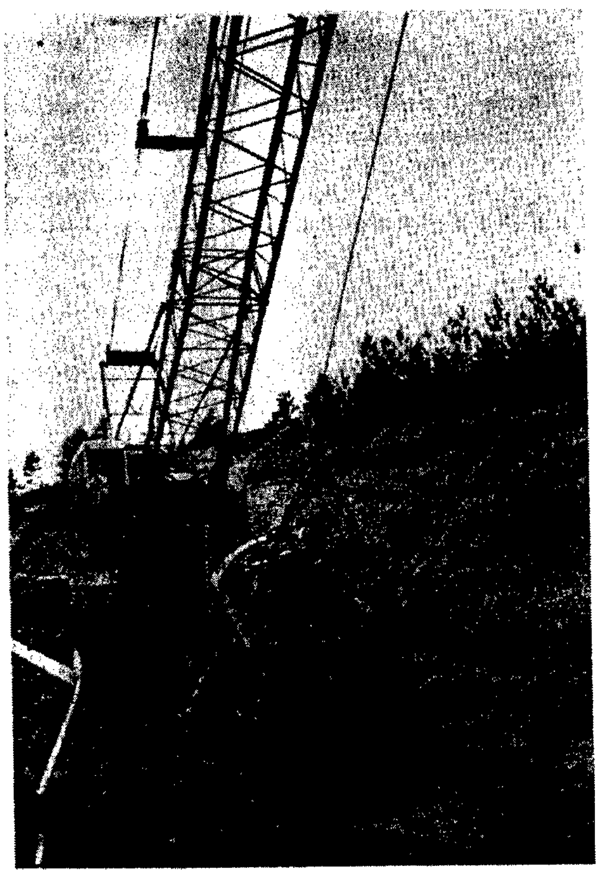


Figure 3-2. Dragline excavation of slurry trench, West Point Dam, Georgia

(f) After the bottom of the trench has been cleaned, backfill is placed in the trench with a clamshell to form a gentle slope parallel to the axis of the trench; backfill is then successively pushed into the trench with a bulldozer and allowed to slide down the slope, intermixing with and displacing the slurry. The slope of the backfill should be flat enough to prevent sliding and sloughing. The trench surface should be observed/inspected as long as possible to detect unusual settlements which might indicate slurry pockets entrapped during the backfilling process. A sketch/schematic of the progressive excavation and backfilling scheme used at West Point Dam is shown in Figure 3-3. The ultimate objective is to achieve a positive cutoff by the combined effects of the backfill and a "filter cake" formed on the sides of the trench by the slurry. Since the integrity of the filter cake after backfill placement cannot be assured, it is recommended that the added benefit of the cake be considered as an additional safety factor with the backfill as the primary element of seepage cutoff.

(g) The slurry trench as a construction technique for forming a cutoff barrier was first used in the United States in the 1940s, and its widespread use began in the late 1960s and early 1970s. Presently, the technique is used routinely although design studies for cutoff walls must be comprehensive and include parameters such as wall depth, thickness, layout, grade, and preparation of the working surface. Design studies must also consider properties of the slurry. Water of adequate quality for slurry mixing must be provided. Typical water quality requirements for bentonite slurry are hardness less than 50 parts per million (ppm), total dissolved solids content less than 500 ppm, organic content less than 50 ppm, and pH of about 7. A satisfactory procedure for the design of slurry trenches and slurry trench construction is given by Winterkorn and Fang (1975).

(3) Grout curtains.

(a) Grouting is the injection by pressure of grout (a mixture of water, cement, and other chemical compounds) into openings (voids, cracks, or joints) in a rock mass. The grout is designed to be injected as a fluid and to stiffen or solidify after injection.

(b) The rock foundation and abutments of most large dams require grouting to reduce seepage and to reduce hydrostatic uplift pressures in dam foundations. Grout curtains are frequently tied into the bottom of cutoff trenches which extend through soil overburden to the rock foundation. Grouting procedures must be tailored to the formation characteristics of the foundation being grouted, and close supervision and inspection are required to obtain satisfactory and economical results. The resident geologist should direct and supervise the inspection of grouting work; he should be experienced in this type of work since many decisions must be made as the work progresses based on judgment and evaluation of results. Successful and economical grouting requires a complete and reliable subsurface investigation to allow determination of the volume which must be grouted. Items which must be determined by the grouting inspector are grout hole location, geometry, length and inclination; injection pressure and rate; grout properties (liquid, transition, set); and necessary degree of improvement in soil properties.

b. Blankets, relief wells, galleries, and toe drains.

(1) Upstream impervious blankets. A horizontal upstream impervious blanket controls underseepage by lengthening the path of underseepage. The effectiveness of the blanket depends on its length, thickness, continuity, and the permeability of the material/soil from which it is constructed. At sites where a natural blanket of impervious soil already exists, the blanket should be closely examined for breaches such as outcrops of pervious strata, root holes, boreholes, and similar (seepage) paths in the foundation which, if present, should be filled or covered with impervious material to provide a continuous impervious blanket. This is especially important in areas where old stream beds may exist. It may be necessary to make additional shallow auger borings during construction to define the extent of breaches, if any, in the natural blanket.

(2) Pressure relief wells. Relief wells are installed along the downstream toe of an embankment to intercept underseepage water and relieve excess uplift pressures that would otherwise develop at the toe of an embankment. Relief of hydrostatic pressure and removal of the associated water volume prevents the transport of soil which might occur in the formation of sand boils and also prevents heaving at the toe. The installation of relief wells is discussed briefly in EM 1110-2-1901 and TM 5-818-5 and in greater detail in EM 1110-2-1914.

(3) Drainage galleries and tunnels. To facilitate foundation and abutment grouting and interception of seepage water, drainage galleries and tunnels are sometimes used in high dams. Drainage tunnels into rock abutments should be examined by a geologist or rock mechanics expert to obtain information on in situ jointing and rock types. Information on the subterranean makeup of the site should be acquired, collated, and interpreted by experienced personnel with geological training for the express purpose of evaluating the site for grouting. The plan for grouting should include design of the location, spacing, depth, and size of grout holes as well as a method to install drain pipes required to block or intercept seepage. The inspector must also be experienced in rock tunneling to ensure satisfactory

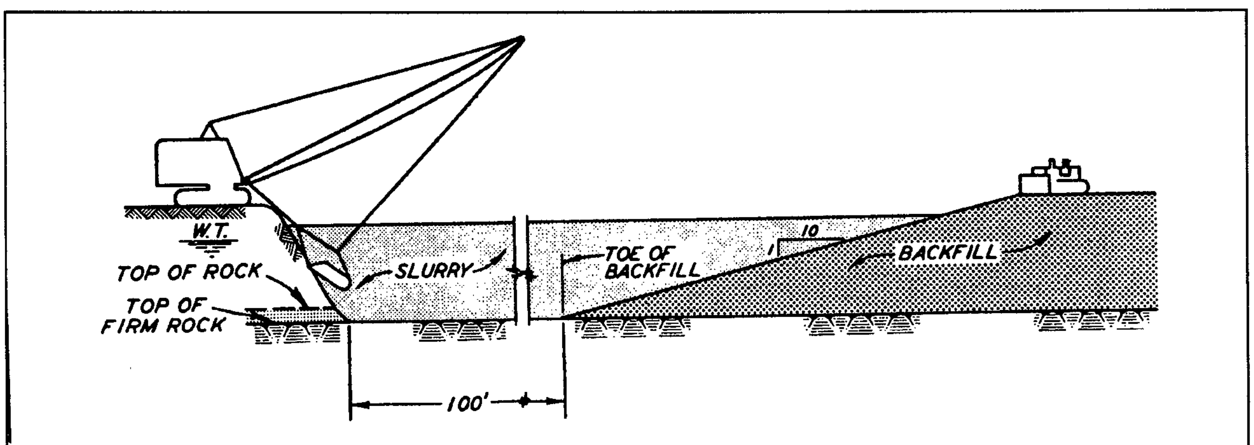


Figure 3-3. Progressive excavation and backfilling scheme for slurry trench construction

installation of rock bolts and other structural support features by plans and specifications. Drainage galleries at the base of a dam or in an abutment of soil or weathered rock are usually concrete-lined tunnels. Inspection of concrete-lined tunnels requires knowledge of concrete placement and backfill practices around concrete structures in addition to knowledge of grouting and seepage control in pervious soils. Inspection of concrete, including proper placement techniques, is thoroughly discussed in the *ACI Manual of Concrete Inspection* (1967). EM 1110-2-2000 also contains information related to the inspection of concrete placement.

(4) Toe drains.

(a) Toe drains collect and facilitate removal of seepage water at the downstream toe of the dam to prevent formation of soft boggy areas and/or boils. Toe drains are generally connected to the horizontal drainage blanket and sometimes to the relief well system to collect and remove seepage water in thin pervious strata in the upper foundation that the deeper relief wells cannot drain.

(b) Toe drains generally consist of a trench containing a perforated collector pipe surrounded by filter gravel with the remainder of the trench backfilled with sand. Particular care must be exercised in placement of the backfill. Unless the sides of the trench are approximately sloped at the angle of repose of the filter material, a wood or steel form must be used to keep the filter layers separated as the backfill is brought up. Additionally, filter materials must be protected from contamination which could result from inwash during a rainstorm. Construction (backfilling) of toe drains in short sections could minimize contamination.

(c) The same control procedures are used for toe drains as those that are used in construction of impervious fill in the main embankment; these are described in Section IV of Chapter 5. Gradation tests on filter materials should be run at least twice each day during placement operations. Stockpiled as well as in-place filter material should be tested. Handling and compaction of the filter material must be closely controlled to avoid segregation and particle breakage.

3-4. Treatment of Unfavorable Conditions

Unexpected unfavorable conditions are frequently discovered during early construction, and may range from undesirable deposits of material not detected in exploratory drilling to adverse seepage conditions that were impossible to predict. Very often, when undesirable materials are found, additional exploration by test pits, borings, or calyx holes is necessary to define the extent of the unexpected deposits and their characteristics. In this way, the impact of a problem deposit can be properly evaluated in relation to the original design. Some common undesirable conditions are discussed in the following paragraphs.

a. Unfavorable soil conditions.

(1) Highly compressible and low strength soils. Organic soils exhibit high compressibility and low shear strength and are generally recognized by their dark color, the presence of organic particles, and often a distinctive "organic" odor. Inorganic clays with high water content also exhibit high compressibility and low shear strength. If an embankment is constructed on a deposit of either highly organic soil or highly compressible inorganic soil, excessive differential settlement could cause cracking of the embankment, or shear failure might occur; if significant deposits of either of these materials are discovered during early construction, their extent should be established and, if it is feasible, they should be removed and replaced with acceptable compacted backfill. If extensive and/or deep deposits of such materials are found, engineering personnel should be consulted to determine if design modifications (such as flattening embankment slopes or adding berms) are required.

(2) Clay shales.

(a) Clay shales are among the most troublesome and unpredictable soils. They are often termed "compaction" or "soil-like" shales if they have been highly overconsolidated by great thicknesses of overlying sediment and have no appreciable cementation. Clay shales tend to slake rapidly when subjected to cycles of wetting and drying; some exhibit very high dry strength, but upon wetting swell and slake profusely, losing strength rapidly. They vary in color from brown to green to black and are often slickensided (a slickenside is a smooth, shiny, striated, discontinuous surface that shows evidence of movement). Problem clay shales can be identified on the basis of slickensides found by breaking undisturbed blocks or chunks apart, and from the speed of slaking during cycles of wetting and drying. Clay shales that are slickensided may be unstable even in relatively flat slopes. Rapidly slaking shales will deteriorate into soft clays with low strength upon exposure to air and water and require protection of exposed surfaces prior to fill placement. Stability in deposits of problem clay shales is further compounded if they are highly fractured or jointed or show evidence of faulting.

(b) Some clay shales also tend to swell or expand considerably when unloaded by excavating overlying material. Expansion may progress deeper into the clay shale deposit with time and cause nonuniform rebound across excavation surfaces. This is caused by stored strain energy that is released with time after overlying materials are removed. Therefore, excavating in clay shales should be completed and backfilled without delay. The last foot or so of excavation into a slaking clay shale should be deferred until just prior to backfill operations in order to minimize the time of exposure of the final clay shale surface. During winter, the depth of cover should be no less than the frost penetration depth; operating in this manner will provide a fresh surface to compact the fill against and eliminate the chance of a soft stratum between the unweathered shale and the fill. This is generally a costly procedure for steep slopes, but becomes more economical for slopes flat enough for equipment to work on.

(c) Only rubber-tired equipment should be used in final excavation, cleanup, and initial fill placement on clay shales to minimize disturbance. Final clay shale surfaces should *not* be scarified prior to covering with fill. If pressure cleaning is required, only air pressure (i.e, no water) should be used.

(d) Various types of coatings have been applied to protect exposed clay-shale surfaces; they include gunite, sprayed asphalt, and other bituminous materials and resin emulsions. Gunite is reliable when reinforced and anchored to the shale, but particular care must be exercised to avoid a drummy condition. This type of protection was successfully used at Waco and Proctor Dams in Texas. Although bituminous coatings and resins have been used successfully, they do not always provide adequate protection for the clay shale. At Waco Dam, an asphalt emulsion membrane used on near-vertical cuts was not always adequate, even with multiple application. Evidence of its inadequacy was that the shale surfaces spalled and slaked. Concrete slabs, whether placed specifically for protective purposes or as slabs for an overlying structure, provide good protection. Exposed surfaces may also be protected by wet mats. Burlap has proven to be an unsatisfactory mat because it is too porous to retain water for any length of time. Maximum allowable exposure time can vary from a few minutes to several hours depending on the characteristics of the shale and the prevailing weather conditions.

(3) Collapsible soils.

(a) "Collapsible" soils are generally soils of low density and plasticity which are susceptible to large decreases in bulk volume when they are exposed to water. Collapsible soils are characterized by bulky grains (in the silt-to-fine-sand grain size) along with some clay. Collapse results from softening of clay binder between larger particles or the loss of particle-to-particle cementation due to wetting. Volume change from collapse occurs rapidly (relative to consolidation) and can be very significant especially if the soil is under high stress. If an embankment is founded on a collapsible soil which is subjected to wetting for the first time, substantial settlement and possibly cracking in the overlying embankment could result. Therefore, unaltered collapsible soils should not be allowed in a dam foundation. If it is not practicable to remove such deposits, they should be treated to break down their structure prior to construction.

(b) Prewetting has been used as a treatment for collapsible soils; the deposits are flooded with water in flat areas where ponding is possible, or by continuous sprinkling on slopes where ponding is not possible. Later, as the embankment is constructed, its weight compresses the foundation soil, causing primary consolidation to take place during construction rather than a sudden and possibly catastrophic foundation collapse when the reservoir is filled for the first time.

(4) Loose granular soils. Loose, water-saturated sands and silts of low plasticity may have adequate shear strength under static loading conditions; however, if such materials are subjected to vibratory loading, they may lose strength to the point where they flow like a fluid. The process in which susceptible soils become unstable and flow when shocked by vibratory loading is called liquefaction, and it can be produced by vibration from blasting operations, earthquakes, or reciprocating machinery. In very loose and unstable deposits, liquefaction can occur as the result of disturbances so small that they are unidentifiable. Loose silt and sand deposits have been compacted by blasting (Layman 1942), vibroflotation, and driving compaction piles; however, the effectiveness of these procedures for deposit densification is not predictable. Vibroflotation has been successfully used in treating limited areas, but it is very expensive. Blasting is generally not effective in densifying loose granular deposits because the vibratory energy produced is of such high frequency.

(5) Steep abutment slopes. Steep abutment slopes of earth tend to increase the possibility of transverse cracks developing in the embankment after construction. During construction, they may become unstable and endanger construction personnel. Slides can occur in clays, sands, and gravel, particularly in slopes subjected to seepage. Slides may damage completed works and require costly repairs. In many cases, it may be necessary to bench the slopes to provide safety against sloughing material and sliding. Frequent inspection should be made by the resident geologist or other experienced personnel to determine whether flattening of specified slopes is required.

(6) Old river channels. Old abandoned river channels filled with pervious or impervious materials are often

encountered unexpectedly during construction. As mentioned earlier, the extent of these deposits is often impossible to establish accurately during the exploratory stages, and in some cases an entire deposit may be missed. Old river channels beneath a dam foundation, filled with course-grained pervious material, would constitute a dangerous open path of seepage. Channel fillings of soft fine-grained materials can cause differential settlements and cracking of the embankment if not removed and replaced with properly compacted material. Where the existence of such deposits has been revealed, additional exploration by borings, test pits, etc., to establish their extent may be necessary. The design engineer can then decide what measures will have to be taken to modify the design or to remove the deposit. An old river channel found during foundation excavation for the core trench at Fall Creek Dam, Fall Creek, Oregon is shown in Figure 3-4.

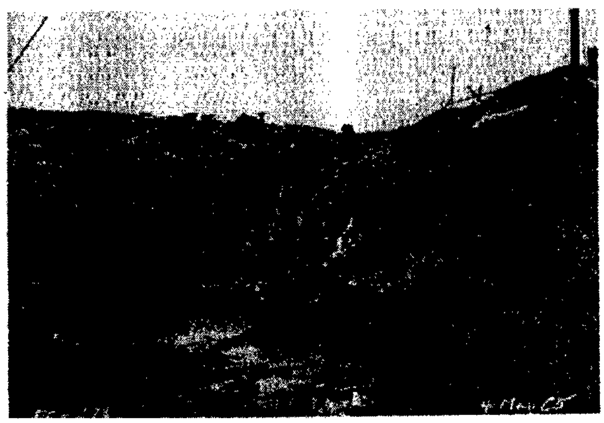


Figure 3-4. Old River Channel in foundation of Fall Creek Dam, Oregon

b. Unfavorable rock conditions.

(1) Weathered rock. Weathered rock may have undesirable characteristics such as high compressibility, low strength, and high permeability. Removal of weathered rock is generally required for embankments founded on rock to obtain impervious contact beneath the core and to eliminate the possibility of differential settlements and low shear strengths beneath the core and other zones. The weathering of rock is a transitional process; a sharp line of demarcation does not exist between weathered and unweathered zones, and the degree of weathering usually decreases with depth. It may be necessary to excavate deeper in some areas than in others to remove weathered rock. The elevation to solid rock sometimes shown on plans can be used only as a guide. Excavation in abutment and foundation areas being stripped should be inspected by a geologist as work progresses to determine when solid rock is reached.

(2) Open joints and fractures. All open joints, cracks, fissures, and fractures in the foundation rock surface must be filled to prevent erosion or scour of embankment material at the rock contact. A sand-cement mortar is generally used to fill these openings. The mortar is worked into the fractures using a stiff broom, taking care to prevent the accumulation of mortar on unfractured surfaces, where it would not be needed in any event and might be harmful if it cracked or broke off during rolling of embankment fill. The water-to-solids ratio of the mortar should be varied as required to accommodate the conditions encountered. If the rock is closely fractured with fine cracks, the water content may be increased and a fine sand used to permit easy entry of the mortar into the minute seams. If wide, deep cracks are present, a stiffer mortar with coarser sand should be employed to reduce the extent of shrinkage cracking (in the cured mortar). A rock surface after mortar treatment is shown in Figure 3-5.

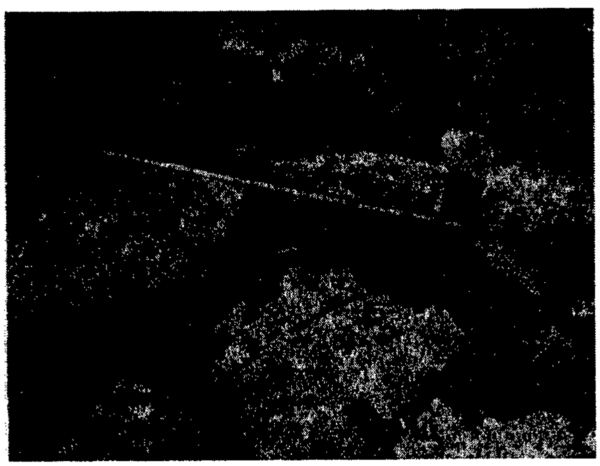


Figure 3-5. Mortar-treated rock surface

(3) Cavities and solution features.

(a) Cavities, potholes, and other voids caused by solution of the rock are dangerous, and field personnel should always be on the lookout for such conditions during foundation preparation. Personnel should be especially alert where a dam is being built on rock susceptible to solution, such as limestone or gypsum. Potholes and cavities exposed or "daylighted" on the foundation surface are usually remedied by dental treatment. Concrete should be thoroughly vibrated or rodded into the voids and its upper surface brought up to the general level of the surrounding rock. Dental treatment serves to smooth up the foundation to reduce compaction difficulties as well as provide a nonerodable impervious seal as a measure of protection against scour of the embankment fill along the rock contact. Figure 3-6 shows a solution channel located directly under the center line of

Figure 3-6. Solution channel, Mississinewa Dam, Indiana

Figure 3-7. Abutment overhang, DeGray Dam, Arkansas

Mississinewa Dam, Indiana, discovered during excavation for the cutoff trench. After extensive exploration to trace its limits, the channel was backfilled with concrete.

(b) The presence of "daylighted" cavities on the foundation surface indicates the possibility of buried cavities or even caverns below grade. Cavities left untreated are highly dangerous, as emphasized by past experience; there have been cases where leakage through underground cavities was so great that it was impossible to fill the reservoir and the dams were eventually abandoned. Hence, any indication of underground cavities (such as sink holes, disappearance of surface water, etc.) should be reported so that further exploration may be undertaken if required. If an extensive network of solution cavities is found, extensive grouting may be required to ensure the imperviousness of the foundation.

(4) Overhangs and surface depressions. Overhangs and other irregularities in the rock surface of an abutment or foundation must be corrected. An abutment overhang at DeGray Dam, Arkansas, is shown in Figure 3-7. Overhangs should be removed by drilling and blasting with care so as not to disturb the adjacent sound rock. Line drilling and blasting (or blasting with presplitting) have been used successfully to form a relatively smooth surface (EM 1110-2-3800). Presplitting has generally given better results than line drilling for a variety of rock types. Figure 3-8 shows the left abutment at J. Percy Priest Dam, Tennessee, formed by the presplitting method. Concrete dental treatment can be used to fill depressions created by blasting and to remedy some types of overhangs. An example of the use of dental concrete to eliminate an abutment overhang is shown in Figure 3-9. Tamping of soil under overhangs instead of removal or dental treatment must not be permitted. Surface depressions are generally filled with select impervious borrow using heavy mechanical hand tampers to even up the foundation surface in preparation for the first lift of material to be compacted by heavy roller equipment. If the rock is very irregular, it may be more economical to cover the entire area with a concrete slab. It should be noted that a gently undulating rock surface is desirable, and only when surface depressions interfere with compaction of the first lift should concrete backfilling be required.

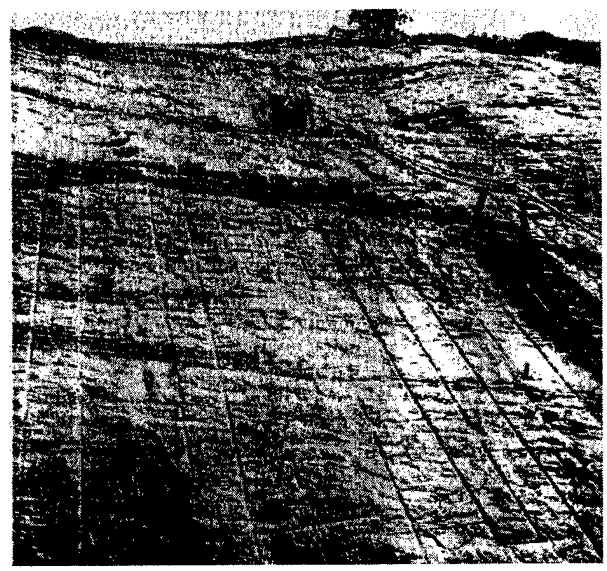


Figure 3-8. Presplit abutment, J. Percy Priest Dam, Tennessee

(5) Springs. Springs, often encountered in rock foundations and abutments, are simply groundwater sources

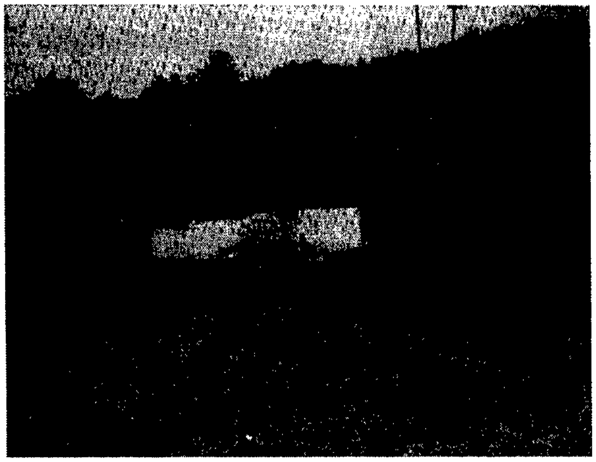


Figure 3-9. Abutment overhang after concreting

Figure 3-10. Seepage from spring in abutment at Green River Dam, Kentucky

seeping to the ground surface driven by artesian pressure. Attempts to place impervious fill over springs issuing from joints or rock fractures will result in extremely wet soil in the vicinity of the spring which is impossible to properly compact. Depending on the flow rate and pressure driving the spring, seepage will continue through the wet soil, creating an uncompacted weak zone of increasing size if fill placement is continued without properly removing this source of water. The zone created around an improperly controlled spring is a very dangerous situation which will cause problems both during construction and over the life of the embankment. Where the water is under a low head and has a single point of issue, a standpipe can usually be installed. A corrugated metal pipe of a diameter depending upon the size of the spring is placed over the spring area, and a damp mixture of quick-setting cement, sand, and gravel is packed around the standpipe base. Earth is then compacted around the outside of the pipe at the base. The water is kept pumped down within the standpipe until an impervious seal is obtained and enough pipe sections have been added to retain the head of water in the pipe. The pipe is then filled with vibrated concrete or grout, and construction of the fill continued upwards and across the top of the plugged pipe by conventional methods. The area is then examined for evidence of new springs, which often appear after an old spring is plugged. This procedure can also be used for springs on the abutment when the fill reaches the same elevation as the spring. While filling operations are progressing below the spring, a small pipe can be grouted into the source of seepage and discharged away from the fill as a temporary measure. This procedure was used at Green River Dam, Kentucky, to eliminate seepage from the abutment spring shown in Figure 3-10. If the springs are not fully localized in area, more extensive methods as described in paragraph 3-5 may be required.

3-5. Dewatering and Drainage of Excavated Areas

Inadequate control of groundwater seepage and surface drainage during construction can cause major problems in maintaining excavated slopes and foundation surfaces and in compacting fill on the foundation and adjacent to abutment slopes. Plans and specifications seldom contain detailed procedures for dewatering and other drainage control measures during construction, and the contractor is responsible for dewatering systems adequate to control seepage and hydrostatic uplift in excavations, and for collection and disposal of surface drainage and seepage into excavations. Inspections and observations must ensure that dewatering and drainage control systems are installed correctly and are functioning properly.

a. Dewatering.

(1) Potential troubles can often be detected in early stages by visual observation of increased seepage flow, piping of material from the foundation of slopes, development of soft wet areas, uplift of excavated surfaces, lateral movement of slopes, or failure of piezometer water levels to drop sufficiently as pumping is continued. Water pumped from dewatering systems must be observed daily at the discharge outlet; if the discharge water is muddy or contains fine sand, fines are being pumped from the foundation. This can be observed by obtaining a jar of the water and observing sediment settling near the bottom. The pumping of fines from the foundation can cause internal erosion channels or piping to develop in the embankment structure; if this happens it is crucial that corrective measures be

taken. Wells or wellpoints from which fines are being discharged must be sealed and replaced with wells having adequate filters. Piezometers should be installed with dewatering systems to monitor drawdown levels in the excavated area. Piezometers should be read daily and the readings plotted to enable continuous evaluation. Daily pumping records should also be kept and evaluated to determine the quantity of water removed by dewatering systems and sump systems. These records are valuable for detecting inadequate seepage control and for evaluating claims by the contractor of changed conditions with respect to the plans and specifications. A detailed description of various types of dewatering systems, installation procedures, and performance evaluation is given in TM 5-818-5. A sketch of a single-stage wellpoint dewatering system is shown in Figure 3-11a, and a sketch of a multistage dewatering system with provisions for drainage of surface water is shown in Figure 3-11b. The two-stage wellpoint system used to dewater the core trench at Carlyle Dam, Illinois, in the St. Louis district is shown in Figure 3-12.

(2) Failure of the dewatering system can result in extremely serious problems, often requiring extensive and expensive remedial work. In excavations bottoming in impervious material, unchecked artesian pressure in underlying pervious strata can cause heaving of the excavation bottom. If the impervious stratum ruptures under these pressures, boils (violent emission of soil and water) will develop, causing the loss of the underlying foundation material and thereby endangering the entire structure. Figure 3-13 shows sand boils that developed at Friars Point, Mississippi, landward of a levee during a high river stage. Similar boils could develop on the bottom of an excavation from excessive artesian pressures in the underlying strata. Failure of excavation slopes may also occur because of excessive artesian pressures. In order to prevent failure of the dewatering system, all power sources should have standby gas or diesel-powered pumping or generating equipment, and standby pumps should be available.

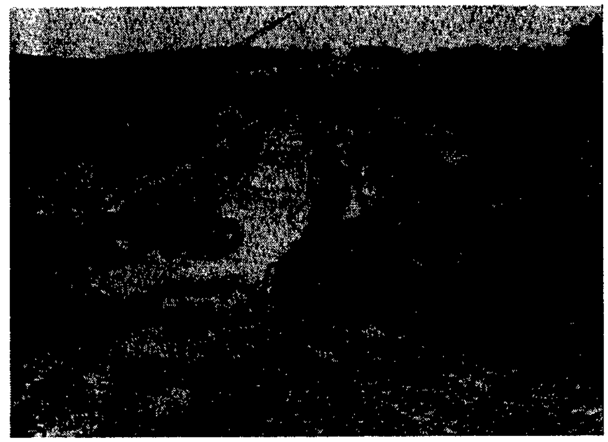


Figure 3-12. Core trench at Carlyle Dam, Illinois, with two-stage wellpoint dewatering systems on each slope

b. Sumps and ditches.

(1) When an excavation such as a cutoff trench is extended to rock or to an impervious stratum, there will usually be some water seeping into the excavation and/or

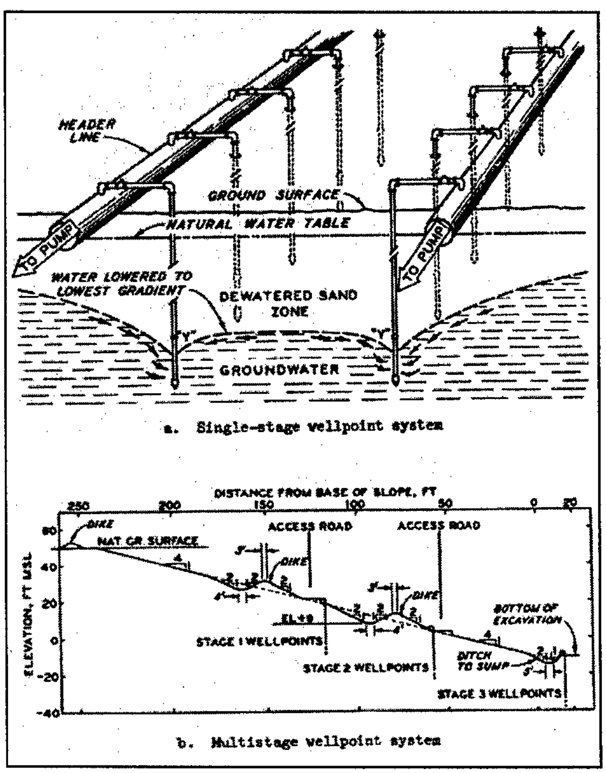


Figure 3-11. Dewatering systems

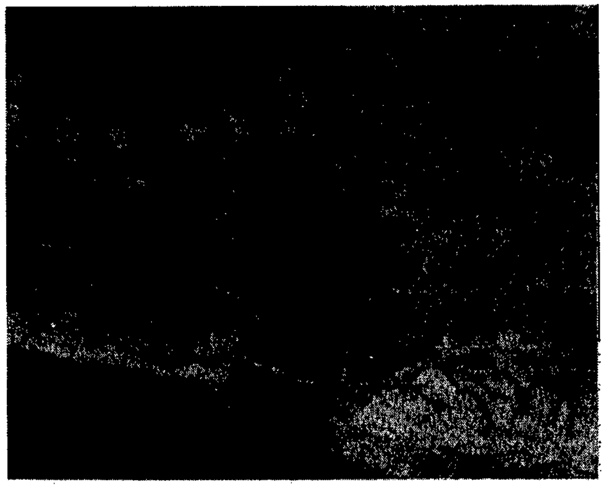
Figure 3-13. Sand boils at Friars Point, Mississippi

"wet spots" in the bottom of the excavation, even with deep wells or wellpoint systems in operation. Water seeping into the excavation from the upstream and downstream slopes of a long cutoff trench can usually be captured by excavating narrow longitudinal ditches or drainage trenches at the intersection of the slopes and the bottom of the excavation (see Figure 3-11b), or by forming such trenches with sandbags, with sumps located as necessary for pumping the water out. If the bottom of the excavation will still not dry out, smaller ditches can be cut through the problem areas and sloped to drain to the side trenches.

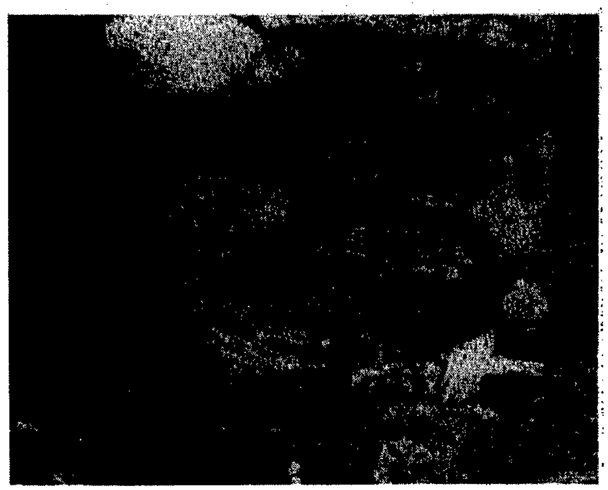
(2) To keep the bottom of the cutoff trench dry while placing backfill, the drainage ditch system can be filled with gravel so that the system can continue to function even after being covered with soil. These gravel-filled ditches should constitute only a very small portion of the overall area being drained to preclude the necessity of later grouting large portions of the bottom of the cutoff trench containing gravel. The surface of the gravel can be covered with a layer of heavy felt paper, burlap, or plastic sheeting, or by a layer of stiff concrete to prevent migration of fines from the fill material. The ditches are blocked at the ends of the excavation by means of concrete plugs. Riser pipes are brought up from each sump (low point in the ditch) with the result that water can be pumped from one or more of the riser pipes as necessary, with the remaining riser pipes serving as vent pipes.

(3) After the backfill is brought to a height that will counteract the hydraulic head, the gravel-filled ditches are grouted. Cement grout is introduced under gravity through one riser pipe with the vent pipes serving as an escape for air and water in the gravel. When grout issues from the vent pipes, the vent pipes are shut off and a slight pressure is maintained in the system until the grout has set. After grouting, normal fill placement operations are continued. If only one sump is used in the drainage system, a vent pipe will have to be installed at an appropriate location before backfilling starts.

(4) Figures 3-14a and 3-14b show the drainage system successfully used at Laurel Dam, Kentucky, to remove localized flow cracks in the shale. The system provided a dry foundation for impervious material. Figure 3-14a shows a trench dug along a water-producing crack in the shale. A perforated collector pipe was placed in the trench with a vent hose and grout pipe attached. The collector pipe was surrounded with stone and the remaining portion of the trench filled with concrete. Water volume from various collector pipes discharged into a collection box from which a single pipe carried the water to a pump located in a 60-in. corrugated metal pipe (CMP) sump (see Figure 3-14b). The vent hoses, grout, pipes, and CMP were extended as the fill was brought up. After 20 to 30 ft of embankment fill had been placed, the collector pipes were grouted and the CMP was filled with -3-in. stone and grouted.



a. Drainage trench dug along water-producing crack in shale



b. Collection point for drainage trenches

Figure 3-14. Foundation drainage system at Laurel Dam, Kentucky

(5) In many cases where a dewatering system is being used, a 4- to 5-ft-high impervious blanket placed at the toe of the slope will prevent the minor seepage flow that otherwise might occur and will therefore provide a dry bottom.

c. Surface erosion. Surface erosion may present problems on slopes cut in silts, fine sands, and lean clays.

Eroded material will wash down and fill in the excavation below the slope. The slope itself will be left deeply scoured and rutted, making it necessary for costly smoothing operations to be performed before the fill can be placed against it. Figure 3-15 shows surface erosion on an unprotected excavation at Kaskaskia Dam, Illinois. The best way to combat surface erosion of temporary excavation slopes is to backfill as soon as possible, thus cutting down on exposure time. This often cannot be done, however, and it becomes necessary to take other measures. Grass cover on the slopes is a good means of preventing surface erosion if it can be readily established and if the slopes are to remain open for a season or two. Other slope protection measures such as rip-rap or asphaltic treatment are rarely justified for construction slopes. Thus, it is necessary to keep as much water off the slope as possible. Most slopes can withstand rain falling directly on them with only minor sloughing. Perimeter ditches and/or dikes (see Figure 3-11b) at the top of the slope are needed to carry other surface waters away from the excavation if surface waters outside the excavation would otherwise run into it. Ditches may be needed at several elevations along the excavation slopes to catch surface waters, as shown in Figure 3-11b.

Figure 3-15. Surface erosion of unprotected construction slope of Kaskaskia Dam, Illinois

d. Other seepage control measures. Other means of stabilizing excavation slopes and preventing seepage from entering an excavation (such as electro-osmosis, freezing, sheet-piling, and grouting) have been used for structure excavations. These methods are not economically feasible for extensive foundation excavations for dams but might be used in structures where conventional dewatering methods are not suitable for various reasons.

Chapter 4
Borrow Areas and Quarries

Section I
Earth Fill

4-1. Excavation, Handling, and Hauling Equipment

Over the past several decades, significant improvements in earth-moving equipment have been made at an increasing rate, and there is no indication that this trend will slow. While the basic types of equipment have remained virtually the same, speed, power, and capacity have continuously increased. Some of the basic principles of the more common units are discussed in the following paragraphs.

a. Excavation equipment.

(1) Power shovels, draglines, elevating graders, wheel excavators, and scrapers. Excavation is usually accomplished with power shovels, draglines, scrapers, wheel excavators, or side-delivery loaders. Each offers certain advantages and has certain disadvantages; therefore, several types are often used on the same job. Discussion of the four major types of excavation units is given in Figures 4-1 through 4-3.

(2) Dredges. Dredging is sometimes employed to move material from borrow areas to the damsite. Dredges are particularly suitable for use when large quantities of material are to be obtained from borrow areas submerged in rivers, lakes, etc. The two basic types of dredges are the bucket dredge and the hydraulic dredge.

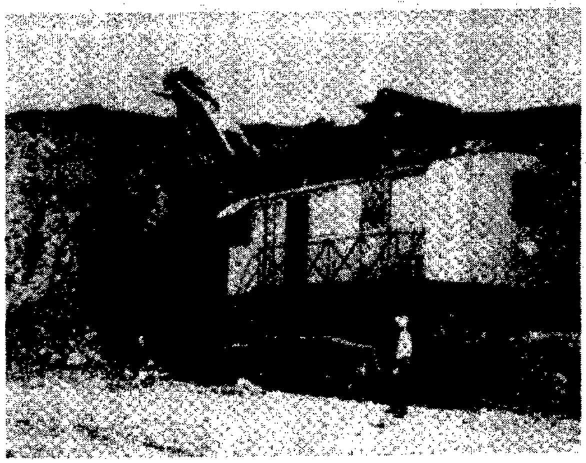
(a) Bucket dredge. A limiting disadvantage of bucket dredges is that the discharge is alongside the place of excavation. They can best be used for localized dredging or where the borrow area is located so that the material can be economically transported by trucks or barges to the site. There are three types of bucket dredges: grab dredges, dipper dredges, and ladder dredges. The grab dredge is essentially a grab bucket operated from a derrick mounted on a flat-topped barge. The dipper dredge is simply a power shovel operating from a barge. The ladder dredge excavates with a continuous chain of buckets supported on an inclined ladder. Bucket dredges have the advantage that they can excavate in most any material. Dredging depths greater than 100 ft are not uncommon for grab dredges. Digging depths of the dipper dredge are limited by the length of the boom (65 ft is about the maximum, although greater lengths are available for special projects) while the digging depth of the ladder dredge is limited by the length of the ladder (usually around 40 ft, but lengths greater than 75 ft are not uncommon).

(b) Hydraulic dredges. All hydraulic dredges basically consist of a centrifugal pump and a suction line through which the pump is supplied with material, but different means are used to loosen and pick up material. Hydraulic dredges can transport material without rehandling for practically unlimited distance if booster pumps are used; this feature is particularly useful in the construction of temporary dikes and permanent structures by the hydraulic fill method. However, it must be mentioned that material placed by the hydraulic fill method will be loose, essentially water-saturated, and probably susceptible to liquefaction if sufficiently disturbed by the mechanisms specified in paragraph 3-4*a*(4). The most common dredge in current use is the cutterhead dredge, which excavates material with a rotating cutterhead then removes it by suction. The cutter is attached to the forward end of a ladder or to the suction pipe itself; rotation of the cutter agitates soft material and/or cuts hard material. Various types of cutters are available, and the type to be used depends on the hardness of the material to be excavated. Some cutters can cut into sound rock. The maximum suction available at the pump is about 28 ft of water; therefore, head losses must be kept to a minimum in order to provide adequate suction.

b. Hauling, handling, and separating equipment. Borrow material excavated by scrapers or by dredging is usually taken directly to its point of deposition on the embankment under construction. However, when most other excavation procedures and equipment are used, additional hauling and handling are required. Regardless of the equipment used, borrow material may require sorting and/or separation according to size or material type. Some of the equipment used for hauling, handling, and separating borrow materials is discussed in the following paragraphs.

(1) Trucks. Trucks of many types are used to transport embankment material from the borrow pit to the dam, capacities of up to 50 tons being common. They can be categorized according to method of unloading as bottom dump, end dump, or side dump.

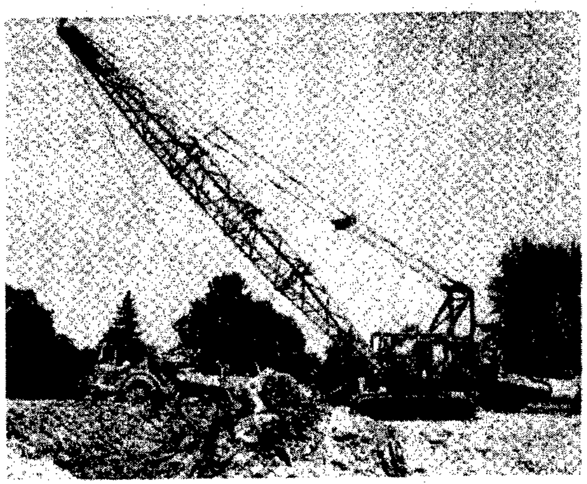
(2) Belt conveyors. Belt conveyors (Figure 4-4) are used to transport material from the borrow area to the damsite when very large quantities of material must be moved under difficult conditions. Belt conveyors are most suitable for moving large quantities of material over rough terrain where there are large differences in elevation between the borrow pit and the dam, and where the cost of building and maintaining haul roads would be high. Conveyor systems are adaptable to different types of automated processing procedures: screening plants, crushing plants,



Courtesy of Koehring, Milwaukee, Wisconsin

POWER SHOVEL

Primary Use:	Excavation above working level
Advantages:	Very versatile for mixing layered materials during excavation; can excavate much harder materials than can a dragline
Disadvantages:	Fairly accurate truck spotting required when loading; excavates hard materials in large chunks that must be broken prior to compaction
Capacities:	Common bucket capacities are 4 to 14 cu yd although larger capacities are not uncommon; typical rates of excavation for average construction conditions range between 70 and 90 cu yd/hr per cu yd of bucket capacity

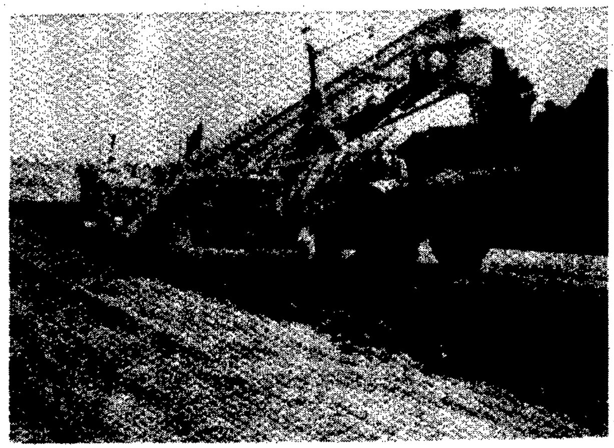


Courtesy of Koehring, Milwaukee, Wisconsin

DRAGLINE

Primary Use:	Excavation below working level
Advantages:	Especially effective in excavating pervious materials from below water table; effective where borrow pit bottom is too soft to support trucks or scrapers
Disadvantages:	Generally slower than shovels and scrapers; cannot excavate hard materials; not as efficient as power shovel in mixing layered material in borrow pit
Capacities:	Bucket capacity frequently about same as that of power shovel, although some draglines are used with buckets much larger than those usually used on shovels

Figure 4-1. Power shovel and dragline



Courtesy of Barber-Greene, Aurora, Illinois

ELEVATING GRADER

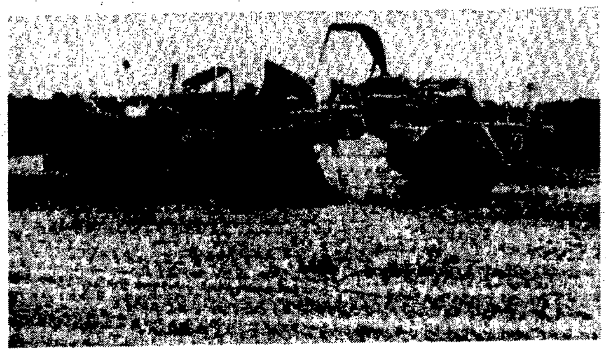
Primary Use:	Excavation in large, flat areas of uniform material with direct discharge into hauling equipment
Advantages:	When the borrow areas are large with flat topography and uniform soil conditions, side delivery loaders generally provide the most economical and rapid means of excavation
Disadvantages:	Not particularly efficient in borrow pits containing cobbles and sands
Capacities:	Normal cut 3 to 4 ft deep and common production rates between 800 and 1,200 cu yd/hr

Courtesy of Barber-Greene, Aurora, Illinois

WHEEL EXCAVATOR

Primary Use:	Excavation in large areas of reasonably uniform topography and material, with direct discharge into hauling equipment
Advantages:	High volume of excavation; exceptionally good blending of material; good conditioning of material; can work in granular materials, materials containing cobbles, and materials with some cementation
Disadvantages:	Borrow areas must have reasonably flat and uniform topography; slow travel speed to other areas; if unit breaks down, all borrow operations cease
Capacities:	Face cut up to 10 ft wide and 13 ft deep; 1,750 nominal bank yd/hr capacity

Figure 4-2. Elevating grader and wheel excavator



SCRAPER

Primary Use:	Excavation in uniform material or where it is desired to excavate horizontally stratified deposits selectively
Advantages:	Serve as both excavating and hauling units; larger and faster units can compete with trucks as hauling units; deposit borrow material in even layers, sometimes eliminating use of additional spreading equipment; borrow material is broken up when excavated, making compaction easier.
Disadvantages:	Unsatisfactory for use in very soft materials; pusher tractor sometimes required when loading
Capacities:	Capacities of 30 cu yd are common

Note: Self-propelled rubber-tired scrapers are rapid-moving type; scrapers towed by slower moving crawler tractors may be preferred where better traction is needed in the pit and the haul is relatively short.

Figure 4-3. Scraper

blending operations, water additions, etc. Transfer points (where the material is transferred from one belt to another) are usually required. Automatic facilities for loading trucks at the terminal points can be easily provided, or sometimes the material can be dumped directly from the belt onto the embankment, spread with bulldozers or grader, and compacted.

(3) Separation plants. Separation or screening plants (Figure 4-5) are employed where it is desired to separate different particle sizes of a granular material. Generally, the purpose is to remove oversize rocks or cobbles to facilitate compaction or to remove fines from filter material. There are four principal types of screening plants:

(a) Horizontal or sloping stationary screens. With this equipment, material is directed through a stack of stationary screens/sieves with screen opening sizes decreasing toward the bottom of the stack. Larger soil particles are retained on the upper screens while smaller particles fall through to be retained on a lower screen.

(b) Vibrating screens. This equipment is basically like the stationary system described above except that soil separation through the screens is facilitated and expedited by vibrating the screens.

(c) Rotating trommels. This equipment consists of an inclined rotating cylinder with screens or holes of different sizes around the periphery. Separation is accomplished when soil particles of different sizes fall through the appropriate hole size as the mixed material rolls around inside the rotating cylinder.

(d) Wobblers (or rotating cams). In wobblers, rotating cams produce vibrations which cause fines to fall through sloping screens, whereas the larger (rock) particles tumble down the sloped screens and fall over the edge onto a pile or a conveyer belt. Each type of separator is discussed in more detail in paragraph 4-4*c*. Various plants have capacities ranging from 100 to 2,000 cu yd/hr, but most plants process 300 to 500 cu yd/hr. Wet materials having appreciable clay content are the most difficult to process, since the clay tends to clog the screen openings.

4-2. Borrow Area Operation

a. Plans for development. The contractor should be required to submit detailed plans prior to construction for the development of borrow areas and quarries. These plans should be carefully reviewed prior to approval and rigidly followed during construction.

b. Inspection. Inspection of borrow pits includes observing and recording all earthwork operations performed in the borrow pit prior to dumping the material on the embankment. Working under the supervision of the construction engineer and chief inspector, the borrow pit inspector observes areas excavated, depth of cut, and adequacy of the contractor's equipment for the tasks at hand. Inspectors should inform the chief inspector of conditions that deviate from the plans and specifications, so that corrective action can be taken if necessary; these deviations include:

(1) Borrow materials that are different from those expected to be obtained in the borrow area.

(2) Borrow excavation operations that are not producing the desired blend or type of material required.

(3) Borrow materials that are too wet for proper

Figure 4-4. Belt conveyor system

Figure 4-5. Vibrating screen separating plant

compaction. During excavation and processing, the inspector observes all adjustments made to the water content of the material. If separation or blending is required, the inspector performs tests to ensure that the soil type and/or gradation of the processed material meets the specifications.

c. *Water content control.* Water content changes occur in borrow pits because of rain, evaporation, or the addition of water for the direct purpose of raising the borrow material water content. Earthwork contractors should be encouraged to take steps to hold the water content of the excavated borrow material as close to the desired placement water content as possible prior to delivery to the embankment.

(1) Dry soils.

(a) For most clays it is not desirable to add more than 3 to 4 percent water on the fill, and in arid regions the average natural water content of soils in borrow areas may be 10 to 15 percent below the desired value for compaction. Under such conditions, irrigation of borrow areas generally results in more uniform water content distribution, while also being the most economical method of adding water. Borrow areas are frequently wetted to depths of 5 to 15 ft or more by surface irrigation.

(b) The water content of soils in a borrow area may be increased by constructing low dikes and ponding/flooding the area or with a pressure sprinkling system. Controlled ponding/flooding is most suitable in low-lying flat areas and tight soils for which long wetting periods are needed. Sprinkling is advantageous on sloping ground and in large borrow areas where only relatively shallow wetting is needed. It is desirable in some borrow areas not to strip topsoil before ponding/flooding, since stripping tends to seal natural holes and cracks in the ground surface which facilitate the entry of water. Ripping tight surface layers has been found effective in speeding up the wetting process. When sprinkling is used on hillside/sloped borrow, it may be desirable to use contour plowing to prevent surface runoff. Good judgment should be used in prewetting steep hillside borrow so that slides are not induced. The length of time required for wetting/hydration may vary from a few days to several months, depending on soil permeability and the depth to which moistening is desired. A curing period is desirable after wetting to allow added water to be absorbed uniformly by the soil. The time needed and the best technique to use can be determined by experimentation.

(c) If general borrow pit irrigation is not satisfactory, supplemental water can be added by sprinkling the face of a shovel excavation; the water is mixed into the matrix as the material is shoveled, hauled, dumped, and spread. If soil is being processed through a screening plant to remove oversize cobbles, a considerable amount of water can be blended into the soil by sprinkling within the plant.

(2) Wet soils.

(a) It is generally easier to add water to dry soil than to reduce the water content of wet soil. The difficulty of lowering the water content of a soil deposit will depend on the plasticity of the deposit and on the amount and type of rainfall during construction. The rainfall pattern is important; for example, a few scattered cloudbursts are less harmful than the same amount of precipitation falling as rain over a longer period of time. It is practically impossible to dry out borrow material to any extent without excessive work and cost unless a dry season of sufficient length permits evaporative water loss.

(b) The first step in either drying out or maintaining the in situ water content of borrow material is to provide surface drainage in the borrow area; this is done by cutting ditches and sloping surfaces to drain to these ditches. Since water is drained away from the borrow material, absorption of subsequent rainfall is minimized. Wet soil can sometimes be dried by ripping, plowing, disking, or otherwise aerating the soil to a depth of several inches. The time required for drying (and hence the production of usable material) will depend on soil plasticity, the depth to which the material can be aerated, and climatic conditions. After the soil has dried to a usable water content, it may be removed with elevating scrapers or graders and the process of aeration, drying, and removal repeated. The procedure described is relatively effective for silty and sandy soils, but is not effective for plastic clays; if the water content of plastic clays is lowered by aeration in dry climates, the result may be hard dry chunks which are difficult to process. Open ditches in borrow areas of sandy and silty soils with a high water table will drain off excess water and lower the water table.

(c) In the construction of Dorena Dam, Oregon, by the Portland district, disking the borrow areas did not break up and mix the clay material enough to obtain uniform water content distribution within the soil. This problem was solved by using a heavy rotary pulverizer pulled by a crawler tractor after the disking. Shortly after pulverizing, the material was at or near placement water content, was easy to load, and was in excellent condition for compaction.

(d) In very wet climates and adverse weather conditions, it may only be possible to prevent borrow material from becoming wetter during construction. This may be accomplished by such techniques as providing surface drainage and/or using equipment that minimizes the chance for material to absorb additional water. Excavating with power shovels on a vertical face is an example of this strategy.

d. Blending soil layers with excavating equipment.

(1) Blending two or more soil types may be required where different soil strata are present in borrow areas or required excavation. Reasons for blending soils are to obtain borrow materials having acceptable characteristics for a particular embankment zone and to utilize borrow materials so stratified in situ that it would not be feasible to load and place material from individual soil strata.

(2) Where materials to be blended occur as horizontal strata, shovels, draglines, wheel excavators, or in some cases, scrapers have been used to blend them during excavation. Excavation with a power shovel on a vertical cut will blend the materials. Where more extensive mixing is required, it can be achieved by running the open bucket through the mixture several times before loading. Construction control in this case will require maintaining the height of cut necessary to obtain the desired proportions of each type of material and to ensure that the materials are blended thoroughly.

(3) Blending different materials from different sources can be accomplished by stockpiling one layer on the other so that excavation can be made through the two materials as in a stratified natural deposit. However, this procedure is expensive and is seldom used.

(4) Scrapers have been used to mix stratified deposits by developing the excavation in such a way that the scraper is loaded on an incline, cutting across several horizontal strata of different materials; however, this procedure is generally not as effective in mixing as the use of a shovel, dragline, or wheel excavator.

e. Selection of materials intended for different embankment zones.

(1) The borrow pit inspector must assure that materials intended for a certain embankment zone are within specification limits. Selection of materials will, to a large extent, have been accomplished on the basis of design studies; that is, borrow areas for the various zones will have been designated. Design studies should have disclosed the nature of the materials and the expected ranges of variation. Therefore, field personnel should review the results of all investigations and know what materials are acceptable; the inspector must be able to identify these materials visually as far as possible, and with a minimum of index tests.

(2) The use of proper equipment by the contractor will also aid in preventing undesirable mixing of soil types. For instance, stratified deposits of distinct soil types to be kept separate should be excavated with a scraper since scrapers excavate by cutting relatively thin strips of soil, thereby avoiding mixing of strata. Screening plants are sometimes employed to obtain required gradations. Although screening of natural materials for major embankment volumes is an expensive process, it may not be excessively so when compared with the benefits. The use of screening plants may avoid major placement problems, allow steeper embankment slopes, and employ a lesser volume of material. Screening is used most often in connection with filter materials.

(3) If any materials are encountered in borrow areas having characteristics that differ appreciably from those anticipated, such materials should not be used unless approved by the design office.

(4) The borrow pit inspector must ensure that materials of possible short supply are conserved.

f. Oversized materials. The maximum diameter of stone or cobble allowed in compacted fill is generally limited to about three-fourths the thickness of the compacted layer. Where a high percentage of oversized cobbles or stones is present, oversize material removed may be used in the outer portions of the dam. Oversize materials can be removed on the fill surface by hand labor or by special rakes mounted on tractors, or they can be screened out in the borrow areas. Generally, removal of oversized rock is more efficiently accomplished in the borrow areas. Rock separation plants are usually employed for this purpose. Processing methods are discussed further in paragraph 4-4*c*.

g. Stockpiling. When excavation of fill material from borrow sources progresses at a faster rate than its placement in the embankment, the material can be stockpiled near locations where it is to be used. Stockpiling involves expensive rehandling and is generally only used on large projects where borrow is to be used from an excavation made before embankment construction, where borrow areas will be flooded during construction, or when material must be stockpiled close to its point of intended placement for rapid construction of a closure section. Unless stockpiling is a specified item, its use is at the expense of the contractor. Stockpiling is advantageous in cases where borrow must be transported long distances and moved by conveyor belt or other means at the site. Filter materials are often stockpiled when it is necessary to obtain them from commercial sources or to manufacture them on the site. Care should be taken in stockpiling filter materials to avoid segregation, contamination, and particle breakage. In dumping filter material onto a stockpile, drop heights should be kept at a minimum; the filter material stockpile should be located well away from other types of material, the area should be sloped so that water drains away from filter stockpiles, and heavy equipment should not be operated over filter materials. Gradation tests should be performed on samples of filter material from a number of locations around the stockpile before and after it is placed to ensure that specifications have been met. The advantages of maintaining stockpile quality should be well understood and appreciated by site personnel, especially if many contractors will use the stockpile.

h. Cold weather operations. Borrow area operations can often continue into freezing weather without loss of embankment fill quality. Frost penetration progresses slowly in undisturbed (in situ) fine-grained soils except in extremely cold weather, and soils will generally remain unfrozen if borrow operations are conducted continuously. Material satisfactory for fill placement can be obtained if the in situ water contents do not require adjustment on the fill. Sands and gravel can generally be excavated and handled effectively under very low temperatures, but the addition of water on the fill for compaction may present problems. Borrow excavation in cold weather is usually limited by fill placement requirements; it should be limited to use only in special situations and should be practiced with considerable caution.

Section II
Quarries and Rock Excavation

4-3. General

Even experienced geologists and engineers often cannot predict how rock obtained from a quarry or excavation will break down after blasting. Consequently, field personnel must be particularly observant of the contractor's methods and the results obtained. The most frequent trouble occurs when the quarried material either contains more quarry fines and dust or more oversized material than had been anticipated in the design. It has sometimes been necessary to make major design changes because rock behavior or breakdown was contrary to that anticipated by the designers. See EM 1110-2-3800 for further guidance on the subject.

4-4. Equipment

a. Loading.

(1) Power shovels and front-end loaders are used almost exclusively today for loading trucks or other vehicles in rock excavations or quarries. Power shovels have been used for many years; large front-end loaders have recently come into prominence with the advent of more powerful units with large capacity buckets. In a deep quarry, the walls and face must be carefully scaled as rubbelized material from a blast is cleaned up to prevent rockfall accidents.

(2) The power shovel, either electric or diesel, is still the most common piece of equipment for loading directly from the muck pile, although front-end loaders have been used in this capacity also. The power shovel is generally desired because of its large capacity, its powerful bite, and its efficiency in getting the load from the bucket to the carrier.

(3) Front-end loaders are used most often to load processed material from stockpiles. A front-end loader may be either tracked (crawler) or rubber-tired. The crawler type has been used often in the past, but the four-wheel-drive rubber-tired type has recently become popular. Although the rubber-tired loader lacks the traction of the crawler, it is faster and usually has sufficient traction on most surfaces to load a full bucket efficiently.

b. Hauling. Trucks are generally used as prime movers of rock fill. The three basic truck types are the end dump, the bottom dump, and the side dump. Side dumps are rarely used for the construction of compacted rock-fill dams; they are more useful for building out the edges of fills. Bottom dumps are more frequently used, but they have definite limitations; they are somewhat unwieldy, and oversize rock has a tendency to become trapped in their discharge gates, requiring bulldozers to push them off the rock and thus costing time and disrupting the hauling schedule. At the Lewis Smith Dam, Alabama, all bottom dumps had to be taken off the job and replaced with end dumps for this reason. End dump trucks are probably the most frequently used vehicles for rock hauling because of their speed, mobility, and generally lower first costs to the contractor. End dumps vary in size from the light "dump truck" to semitrailer types with capacities in excess of 100 cu yd. Since in most cases hauling from rock excavations and quarries is "off-the-road" hauling, these trucks are not subject to size and weight limitations imposed upon carriers that travel on public highways, thus allowing the large capacities.

c. Processing. It is usually necessary to process fragmented rock produced by blasting since it is unlikely that the required sizes and/or gradation would occur directly from blasting. Processing may involve removal of oversize or undersize (fines) material, or obtaining a specific size range for use in a particular zone of the embankment such as a graded filter. Preparations should be made to stockpile filter material when it is convenient to prepare the proper gradation; this ensures an adequate supply of filter material during rainy periods when the screens of a processing plant tend to clog. There are several types of separation or processing plants, each with its own advantages and disadvantages; the use and end product desired will dictate the choice of plant. Some of the more common plants are briefly discussed below.

(1) Grizzlies. The grizzly is perhaps the most common separating device; it is used only for removing oversize rock from the material which, in some cases, is all that is needed to obtain the gradation specified. A grizzly consists of a sloped grate made of heavy bars which are wider at the top than at the bottom to ensure that particles do not bind partway through the gratings and clog the openings between the bars. Grizzlies are often constructed with sloped vibrating screens or with rotating cams (grizzly wobblers) so that oversize material passes over the grate and falls off the end while the desired material falls through the grate. Grizzlies may be constructed in many ways, but they always involve the use of a grate or lattice of heavy bars. Figure 4-6 shows a sloping grizzly in operation at Gathright Dam, Virginia. A grizzly wobbler used at Stockton Lake Dam, Missouri, is shown in Figure 4-7.

(2) Trommel. A trommel is a separating device which consists of a rotating cylinder of perforated sheet metal or wire screen. Like the grizzly, it is used for eliminating oversize particles, but it can also separate the remaining material into various size fractions. A trommel can be open at either one end or both ends with the axis of the cylinder horizontal or slightly inclined so that the material is advanced by rotation of the cylinder. Size of the perforations in the sheet metal or of the openings in the screen can be varied to obtain more than one size fraction. As material is fed into the rotating cylinder, the oversize material passes through and is discharged at the other end, while each of the fractional sizes falls through a properly sized opening into hoppers or onto conveyor belts which transport the various sizes to stockpiles.

(3) Shaking or vibrating screen. This device is generally used for sizing. It consists of a metal screen or multiple screens with desired opening sizes, mounted either horizontally or inclined on a rigid frame and given either a reciprocating motion (in the case of a shaking screen) or a vibrating motion (in the case of a vibrating screen). A vibrating screen separating plant is shown in Figure 4-5. Material retained on a given screen passes on to one end of the screen, where it is discharged into a hopper or onto a belt. More than one size can be separated by having screens of successively smaller openings below the initial screen.

4-5. Test Quarries

a. Test quarries aid the designer in determining the sizes, shapes, and gradations of rock produced by excavation and handling. Test quarries are usually operated in conjunction with a test fill so that all aspects of rock behavior from blasting to compaction can be evaluated. Variables in quarrying operations include drilling, blasting techniques, loading, processing, and hauling procedures and equipment. A properly executed test quarry program will provide the designers and field personnel valuable information pertinent to cut slope design, evaluation and control of geologic structures, best blasting technique, and rock fragmentation control to be used; provide representative materials for test fills; give prospective bidders a better understanding of the drilling and blasting behavior of the rock; and determine what processing, if any, of the rock will be required. A test

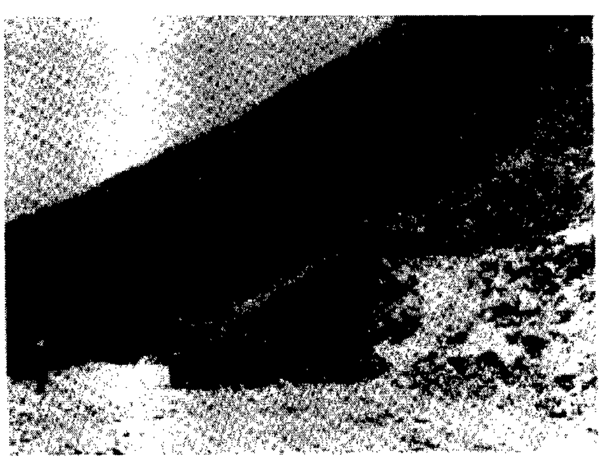


Figure 4-6. Rock separation by means of a sloping bar (grizzly) arrangement at Gathright Dam, Virginia

Figure 4-7. Grizzly wobbler discharging +2-in. rock into end-dump truck and -2-in. material into bottom-dump truck (spillway-powerhouse excavation at Stockton Lake Dam, Missouri)

quarry in operation at Foster Dam, Oregon, is shown in Figure 4-8.

Figure 4-8. Test quarry in operation at Foster Dam, Oregon

b. Even though test quarries are usually established during design, contracts are frequently administered by the construction division, working in cooperation with the design engineers. It is desirable that construction personnel assigned to the test quarry be later assigned to the dam construction, if possible. These personnel will have gained valuable insight into construction procedures and material behavior and would form an important nucleus when staffing for dam construction.

c. The purpose of construction control is primarily to ensure that different test methods and procedures called for in the plans and specifications for a test quarry are followed so that comparison of different methods and analysis of data acquired will be meaningful. It is important that complete and accurate records are kept throughout the job. Records should include types of drilling equipment used; rates of drilling in each type of rock; hole diameter, depth and spacing; explosive type, charge, and spacing; stemming material and procedure; time sequence of firing; and details of results obtained by blasting.

4-6. Obtaining Specified Rock Fill

The specifications will prescribe the gradations of the rock fill to be placed in the various zones. It may be possible to obtain acceptable material directly from blasting, but the rock may have to be processed in one form or another. In any case, the inspector must ensure that the placed rock meets the criteria set forth in the plans and specifications. The contractor's method of operation has a significant effect on the gradation of rock obtained. Therefore, the inspector should be familiar with methods of operation that affect the rock fill.

a. Drilling and blasting.

(1) It would be very desirable and economical to obtain the required gradation directly from the piles of blasted rock, but this is generally difficult and some experimentation by the contractor will be necessary. Preliminary experimental shots should provide criteria for establishing satisfactory drilling and blasting patterns. The spacing of boreholes and intensity of charge required will depend on in situ rock conditions. See EM 1110-2-3800 for information required from contractors and for guidance on approving blasting procedures and inspecting blasted areas.

(2) Rock is excavated or quarried either by the "bench method" using vertical holes, or by "coyote holing," which involves firing large explosive charges placed in tunnels driven into a rock face at floor level. Coyote holing is initially cheaper than the bench method, but frequently results in excess fines and oversize stones that require secondary blasting. As a general rule, the coyote method should not be used when quarrying for rock-fill structures.

(3) In quarry experimentation, the material broken by the first blast should be cleared away not only to determine if the correct gradation has been achieved, but to examine the excavation rock face and the condition of the excavation floor. The power shovel has been mentioned above as the best means of loading/moving blasted rock in quantity, but it must be operated from a relatively level floor.

(4) In a normal shot, the coarser material will be located away from the face and on the top; the finer materials will be concentrated toward the working face and at the bottom. The operator of the loading equipment should be cautioned and instructed on loading procedures that will help ensure a uniform distribution of sizes in each load if the rock is used without processing. In many quarries, a layer of fines will become concentrated on the quarry floor as loading progresses and must be either used elsewhere or wasted.

b. Processing. If the contractor cannot obtain the correct rock sizes by blasting or if special treatment is called for in the specifications, processing will be required. The degree of processing will vary from using a simple grizzly to remove oversize rocks, or washing to remove excess fines, to running material through a crusher plant. Rock crusher plants are not generally used for processing rock fill due to high operational cost. However, crusher plants are sometimes used to produce select filter materials such as transition or bedding material. The amount of processing required will depend on the results of the blasting unless special processing is expressly called for in the specifications. Different types of separation equipment and their utilization have been described above.

Section III
Final Condition of Borrow Areas, Quarries, and Spoil Areas

4-7. Borrow Areas

Generally, no treatment is required for reservoir-side borrow areas located below minimum operating pool elevation. However, for reservoir-side borrow areas located above this elevation and land-side borrow areas, some treatment is generally required to eliminate what would otherwise be unsightly scars. Treatment of these borrow areas usually begins with the contractor stockpiling topsoil when first opening the borrow pit. After all usable borrow has been removed (the contractor should not be allowed to excavate to bedrock, but should be required to leave a foot or two of soil over rock), the previously stockpiled topsoil should be spread back over the borrow area and graded to a smooth, uniform surface, sloped to drain. On steep slopes, benches or terraces may have to be specified to help control erosion. As a final step, the entire area should be fertilized and seeded. These procedures are intended to prevent erosion and bring the borrow area to a pleasing aesthetic appearance and possibly make it available for future use.

4-8. Quarries

Abandoned quarries are unsightly and may be a safety hazard. As far as practicable, old quarries should be drained and provisions made to prevent any further ponding of water in them. All slopes should be scales and trimmed to eliminate the probability of falling rocks and debris. All areas that can support vegetation should be seeded. In some cases, fencing may be needed to restrict free access.

4-9. Spoil or Waste Areas

Specific areas must be provided for the disposal of waste or spoil materials. These areas should be clearly shown on specification drawings. Material which is unsuitable for other purposes is usually used to fill and shape depressions such as ditches, sloughs, etc., located outside the limits of the embankment. Waste material in excess of that required to fill these depressions may be used upstream to reinforce seepage blankets or used as berms to add to the stability of the structure. Waste areas located downstream of the embankment must be carefully selected to avoid interference with any component of the structure or creation of undesirable areas requiring excessive maintenance or remedial work. Waste materials should never be permitted to block seepage from drains, pervious zones, or rock toes. Additionally, disposal areas should not be located such that they could possibly cause contamination of natural groundwater. Generally, no compaction of material in waste areas is needed other than that produced by hauling equipment except on the finished outside slopes. Surfaces should be left reasonably smooth and sloped to provide drainage away from the embankment and construction activities. Outside slopes of waste areas composed of easily erodible materials should be compacted or track-walked to prevent erosion. If possible, the areas should be fertilized and seeded to improve their appearance and prevent erosion, thus possibly making them suitable for recreational purposes.

Chapter 5
Earth-Fill and Rock-Fill Construction

Section I
Fill Processing and Compaction Equipment

5-1. Heavy Compaction Equipment

The three principal types of heavy equipment used to compact embankment fill are the tamping roller (sheepsfoot), the rubber-tired roller, and the vibratory steel-wheel roller. Corps of Engineers specifications, uses, advantages, and disadvantages of each roller are summarized in Figures 5-1, 5-2, and 5-3. Other general-purpose equipment sometimes used to compact fill are the fill hauling and spreading equipment itself, which is routed over fill to be semicompacted (paragraph 5-25), and crawler tractors, which sometimes can produce adequate compaction of pervious cohesionless materials.

5-2. Hand-Operated Compaction Equipment

For compaction in restricted areas such as those immediately adjacent to concrete walls, around conduits, or in depressions in rock surfaces, hand-operated power tampers or vibrated plate compactors are used. Power tampers should weigh at least 100 lb, and vibrated plate compactors are effective only in clean cohesionless backfill.

5-3. Spreading and Processing Equipment

Spreading and processing equipment commonly used on embankment fills is as follows.

a. Crawler and rubber-tired tractors and bulldozers. This equipment is used to tow compactors, plows, harrows, etc., with bulldozer blades to move and spread material and to remove oversize stones from embankment fill.

b. Motor graders (road patrols). Graders are used to spread and mix material, dress up boundaries between different zones (such as core, transition and filter), work out oversize stones, and scarify surfaces of previously compacted lifts.

c. Disks. Disks are typically towed by rubber-tired or crawler tractors (Figure 5-4) and used to scarify surfaces of previously compacted lifts or to aerate and blend water into uncompacted lifts before compaction.

Section II
Test Fills

5-4. Rock Test Fills

a. In the design of rock-fill dams, the construction of test embankments can often be of considerable value, and in some cases it is absolutely necessary. Design engineers should manage the test fill program, although construction personnel may administer the contract under which testing is performed. Test fills aid the designer by defining the effects of variables which would otherwise remain unknown. A properly executed test fill program should determine the most effective type of compaction equipment, the lift thickness, and the number of passes; maximum rock sizes; amount of degradation or segregation occurring during rolling; and physical properties of the in-place fill, such as density and grain-size distribution. The knowledge developed and the consequent improvement of design can significantly influence the cost of the structure. Hammer and Torrey (1973) provide guidance on the design and construction of test fills.

b. Test fills are often operated in conjunction with test quarries. This practice not only provides information about rock behavior during quarrying procedures, but also ensures that material used in the test fill is representative of material that will be produced by the proposed excavation. As in test quarries, test fill programs are often administered by both construction and design personnel, and it is advisable, when possible, to assign construction personnel involved in the test fill program to actual dam construction.

c. Construction of a test fill should be very strict, otherwise data obtained may be of questionable value. Plans and specifications for the test fill are prepared by the design engineer to evaluate construction procedures and material behavior so that results of the test fill can be used in design and construction of the prototype; therefore, changes or additions should not be made without approval of the design engineer.

d. Records and data required should be established by the design office; records should be kept up to date, and data plotted daily, as changes in the test program may be necessary to obtain desired information. It is important to record any and all observations made by field personnel, no matter how insignificant they may seem at the time. Photographs and notes or visual observations are extremely important, as they often provide answers to perplexing questions that would otherwise go unanswered.

TAMPING ROLLER (SHEEPSFOOT)

Specifications:	
Towed:	
Double-drum unit:	Water or sand and water ballasted. Towed by crawler or rubber-tired tractor at not more than 5 mph.
Drum:	Diameter, 60 in. (minimum); length, 60 in. (minimum).
Weight:	Weighted: at least 4,000 lb/ft of drum length. Empty: not more than 2,500 lb/ft of drum length.
Feet:	Uniformly spaced. Approximately three feet per each 2 sq ft of drum surface. Foot length: 9 to 11 in. Face area: 7 to 10 sq in.
Cleaning fingers:	Provided to prevent accumulation of material between feet.
Self-propelled:	May be used in lieu of towed roller if it causes no shearing of or laminations in fill. Specifications same as above except that (a) empty weight greater than 2,000 lb/ft of drum length may be used with face areas of feet not greater than 14 sq in. to approximate nominal foot pressure of towed roller, (b) inflation pressures of rubber-tired front wheels not greater than 40 psi. Speed not greater than 5 mph.
Use:	To compact fine-grained soils or coarse-grained soils with appreciable plastic fines.
Advantages:	Kneading, churning, and tamping action mixes soil and water better than other compaction equipment (this does not preclude proper processing of material prior to compaction, however); produces good bond between lifts; and breaks down weak rock or cemented soils.
Disadvantages:	Leaves surface rough and loose, and therefore susceptible to wetting by rains or surface waters. Compacts to shallower depth than other equipment. Effectiveness diminished in compacting soils containing cobbles or large rock fragments. Self-propelled rollers sometimes cause shearing of or laminations in fill.

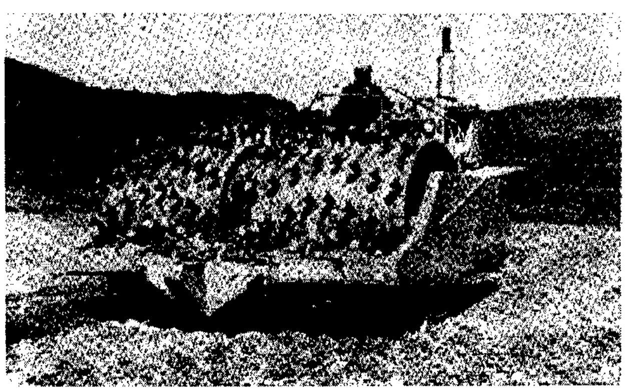


Figure 5-1. Tamping roller (sheepsfoot)

RUBBER-TIRED ROLLER

Specifications:

Unit with pneumatic-tired wheels towed at speeds not exceeding 5 mph.

Wheels:	Minimum of 4 wheels abreast, each carrying equal load in traversing uneven surfaces.
Wheel spacing:	Distance between nearest edges of adjacent tires not to exceed one-half of tire width under 25,000-lb wheel load. Tire pressure 80 to 100 psi.
Weight:	Ballast loading to provide wheel loads from 18,000 to 25,000 lb.
Use:	To compact cohesive (and sometimes cohesionless) soils.
Advantages:	Compacts to greater depths than sheepsfoot roller. Produces relatively smooth compacted surface which is rain-resistant. Effective in compacting in closer quarters than sheepsfoot (i.e., against rock abutments and concrete structures). More effective than sheepsfoot in compacting cohesive soils containing large particle sizes. Wet areas of fill can be determined by observation of roller rutting.
Disadvantages:	Compacted surfaces must be scarified before placing next lift. Not as effective as sheepsfoot roller in breaking down soft rock or in mixing fill material.

Figure 5-2. Rubber-tired roller

VIBRATORY STEEL-WHEEL ROLLER

Specifications:

Single drum unit. Towed by crawler tractor with minimum drawbar horsepower of 50 at speed not to exceed 1.5 mph (when compacting rock fill or sands and gravels), or self-propelled at speed not to exceed 1.5 mph (when compacting sands and gravels only).

Weight:	Minimum total weight, 20,000 lb; 90 percent transmitted to ground by smooth drum with roller in level position attached to towing vehicle. Unsprung weight of drum shaft and internal mechanism not less than 12,000 lb. (Note: while guide specification CW 02212 specifies a roller with a minimum static weight of 20,000 lb, lighter rollers (7,000 to 10,000 lb) have been effectively used to compact pervious sand and/or gravel, and to compact soft rock that would be broken down too much by heavier rollers.)
Vibration:	Frequency: between 1,100 and 1,500 vpm. Dynamic force: not less than 40,000 lb at 1,400 vpm.
Use:	To compact cohesionless materials.
Advantages:	Greater densities can be obtained in cohesionless soils than with tamping or rubber-tired equipment. Fill may be flooded with water to improve compaction.
Disadvantages:	May cause degradation of soil or rock-fill particles and create layers of fines.

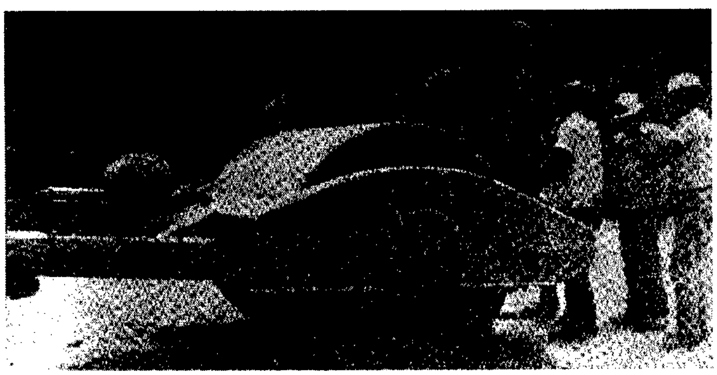


Figure 5-3. Vibratory steel-wheel roller

Figure 5-4. 36-in.-diam disk towed by D-8 tractor at DeGray Dam, Arkansas

e. Gradation tests should be performed on rock before and after compaction. A comparison of the before and after gradation curves will indicate the probable amount of particle breakage to be expected during handling and compaction. Lift thicknesses must be measured. In-place density of the compacted material after rolling must be determined directly by large-scale conventional methods or indirectly by observing settlement of the fill. The latter method is generally used because conventional density tests in rock fills are difficult and settlement measurements provide a better relative measurement of density. If densification of a layer is determined from settlement readings, caution must be exercised to ensure that settlement is in fact that of the layer in question and does not include settlement of the foundation or underlying layers. Settlement in the foundation and within lifts can be determined from settlement plates. A number of conventional density tests should be made to supplement settlement data in any case.

f. Test trenches should be excavated through the completed test fill to allow visual observation of compacted lift thicknesses, distribution of fines, and distribution of density. Test fill operations, including inspection of test trenches, should be thoroughly documented with measurements, photographs, and written results of visual observations. A test trench cut through a portion of the test fill for New Melones Dam, California, is shown in Figure 5-5.

5-5. Earth Test Fills

Test fills for earth embankments are often necessary to establish proper loose lift thicknesses and the number of

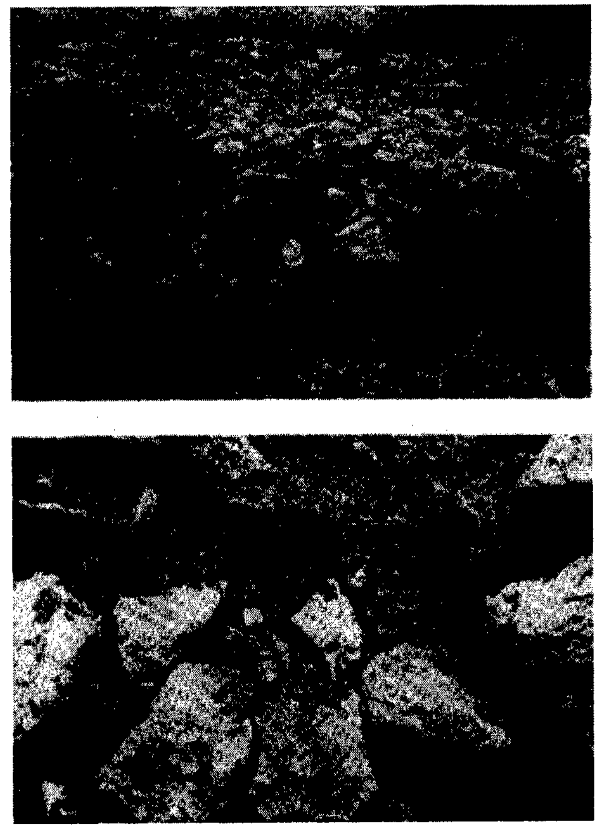


Figure 5-5. Test trench through portion of rock test fill at New Melones Dam, California. General view of trench (top); closeup view of trench wall (bottom)

passes required to compact soils for which there is no previous compaction experience. Test fills may be needed to determine the best procedure and equipment for adding and mixing in water to obtain the desired water content uniformity throughout the lift. Test fills are also constructed (at the contractor's expense) when the contractor wants to use equipment other than that permitted by the specifications, as the contractor must prove that desired results can be obtained with the proposed equipment. Test fills are also used to determine if desired densities can be economically obtained in the field, (for example, at Canyon Dam, Texas, Figure 5-6). A test fill is often a part of the embankment, and if satisfactory results are achieved, the compacted fill can be utilized as part of the embankment fill.

Section III
Impervious and Semipervious Fill

5-6. Definitions

Impervious materials include clays of high and low plasticity (CH and CL), clayey sand or gravel (SC and GC), and

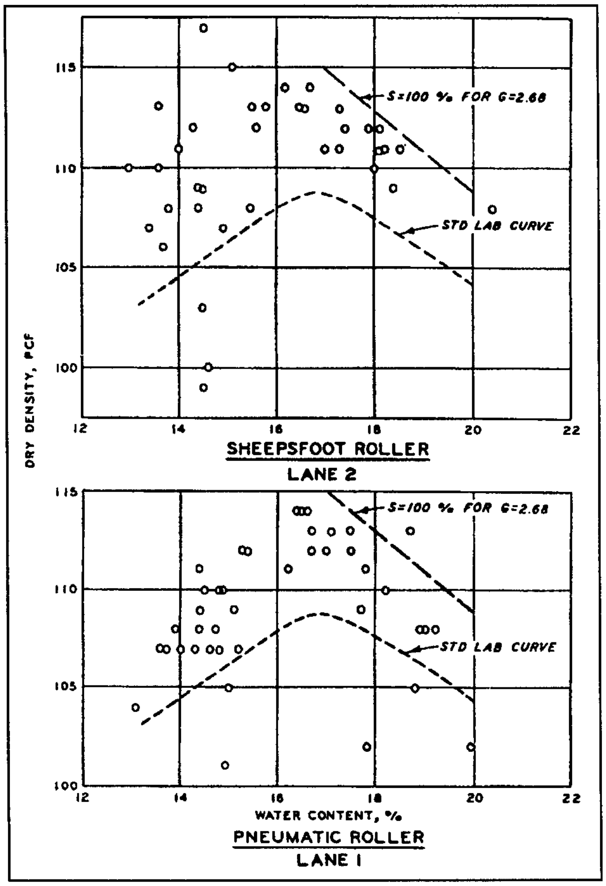


Figure 5-6. Test fill data from Canyon Dam, Texas

clayey silt (CL-ML). Semipervious materials include silts (ML) and silty sands and gravel (SM and GM). Note that sands with borderline gradations classifying as SM-SP, that is, sands having as high as 12 percent passing the No. 200 sieve (5 percent is the usual upper limit for a material to be classified as pervious), may have the characteristics of semipervious material even though such materials may be allowed by the specifications in embankment zones designated "pervious fill." Generally, compaction curves indicating adequately defined optimum water contents and maximum dry densities can be developed using the standard compaction tests on impervious and most semipervious materials.

5-7. Compaction Fundamentals

a. Soils containing fines can be compacted to a specific maximum dry density with a given amount of energy; however, maximum density can be achieved only at a unique water content called the optimum water content. Maximum dry density and optimum water content are determined in the laboratory by compacting five or more specimens of a soil at different water contents using a test procedure which utilizes a standard amount of energy called "standard compactive effort." A detailed description of the laboratory compaction procedure to establish maximum dry density and optimum water content is contained in EM 1110-2-1906.

b. In the field as in the laboratory, the two variables that control fill density are placement water content and field compactive effort applied by the passage of a piece of equipment a certain number of times over a lift of a specific thickness. When the water content of a soil being compacted with a certain compactive effort deviates from the optimum water content for that effort, a dry density less than maximum will result; the greater the water content deviation from optimum, the lower the resulting density will be. For soil with a water content on the dry side of optimum or at optimum, an increase in compactive effort will generally increase the density. For soil with water content considerably greater than optimum, an increase in compactive effort will tend to shear the soil but not further compact it.

c. Compactive effort can be increased by increasing contact pressure of the roller on the soil, increasing the number of passes, or decreasing the lift thickness. Combinations of these procedures to increase and control compaction on a job will depend on difficulty of compaction, degree of compaction required, and economic factors.

5-8. Compaction Specifications

a. Requirements of the more important compaction features (such as water content limits, layer thickness, compaction equipment, and number of passes) will be contained in the specifications and must be checked closely by the inspection force to ensure compliance.

b. Specifications will generally state the type and size of compaction equipment to be used and require that the contractor furnish the Government manufacturer's data and specifications on the equipment. Those data should be checked against job specification requirements along with a visual inspection to ensure that the equipment is in condition to produce the required compaction. If a sheepsfoot or tamping roller is to be used, some of the items to be checked include: drum diameter and length; empty weight and ballasted weight; arrangement of feet and length and face area of feet; and yoking arrangement. For a rubber-tired roller, tire inflation pressure, spacing of tires, and empty and ballasted wheel loads should be checked. For a vibratory roller, static weight, imparted dynamic force, operating frequency of vibration, and drum diameter and length should be checked.

c. Uncompacted or loose lift thickness will be specified. Lift thickness specified will be based on type of material and compacting equipment used. Impervious or semi-pervious materials are commonly placed in 6-to-8-in. loose lift thicknesses and compacted with six to eight passes of a sheepsfoot roller or in 9-to-12-in. loose lift thicknesses and compacted with four coverages or a 50-ton rubber-tired roller. When using a rubber-tired roller or any roller that leaves a smooth surface after compaction, scarification of the compacted lift prior to placing the next lift is specified to ensure a good bond between the lifts. In confined areas where hand-operated power tampers must be used, fill is commonly place in 4-in. loose lifts and compacted to densities achieved with sheepsfoot or pneumatic-tired equipment under the above-mentioned conditions.

d. In-place water content and density must be related to optimum water content and to maximum dry density to judge whether a compacted soil is suitable or unsuitable. Minimum acceptable field density is normally established in design as a percent (usually 95 or above) of maximum dry density[1], and an allowable range of placement water contents is given in the specifications relative to optimum water content of the soil being compacted. Each soil type has a different maximum dry density and optimum water content for a given compactive effort, and it is necessary that in-place field densities and water contents be compared with laboratory-determined optimum water contents and maximum densities of the same soil. Because mixing different soil strata in borrow areas can result in materials with unexpected compaction characteristics, if a material being compacted in the field cannot be related to available laboratory compaction data, a laboratory compaction test should be performed on that material. Check companion tests should be performed by field personnel before fill placement to ensure consistency with target values for a given soil.

e. Assumptions are made in design regarding shear strength, permeability, and deformation characteristics of the embankment fill. These properties vary with density and water content of the compacted soil. Therefore, soil must be placed as specified; otherwise, design assumptions are not met and problems may occur in the completed structure. Thus, desired density and placement water content range are not arbitrarily established but are specified for very definite reasons, and both requirements must be satisfied. If the water content is outside its specified range, even though the desired density is obtained, the soil must be reworked and recompacted. If the minimum density is not obtained even though the water content is within the specified limits, additional roller passes at Government expense will be required. Procedures for performing tests to determine field densities and water contents are contained in paragraph 5-10, and application of these tests to compaction control is included in Appendix B.

5-9. Simple Control Procedures

a. Simple controls using both visual observations and rough measurements are the primary means by which construction control is carried out. However, they must not be used as the only means of control, but must be supplemented by an extensive program of control testing. For any estimate to be meaningful and accurate, the observer must have his eye and hand calibrated to all conditions expected. It is desirable to construct a small test section prior to the beginning of major fill placement so inspectors and the contractor can become familiar with the behavior and compaction characteristics of the fill material and with the performance of the compacting equipment. Noncritical locations are often used for such experimentation, such as in reaches where embankment heights are low.

b. An inspector should be familiar enough with the materials at a job site to recognize when the soils are too dry, too wet, or at optimum water content. To gain needed familiarity with site materials, an inspector should spend time in the field laboratory performing compaction tests and index tests such as Atterberg limits so as to become familiar with differences in appearance and behavior of site fill materials.

c. A trained inspector should be able to pick up a handful of soil and make a reasonable estimate of its water content relative to optimum by feel and appearance. Experienced inspectors can often estimate deviation from optimum water content to within 1 percent. Material may be examined by rolling a small amount on a clipboard or between the hands to get an indication of how close to the plastic limit the soil is. Comparison with the plastic limit is a good rule-of-thumb because there is often good correspondence between optimum water content and the plastic limit of a soil. However, after the inspector has made visual and contact examination, a water content test should be performed on the material in question for confirmation of water content.

d. In addition to having a feel for how a soil looks and feels when it is at the proper density, a penetration resistance index test is often devised by inspectors. The resistance index test itself can range from the use of a Proctor needle (Proctor penetrometer) to that of a common

[1] When the compaction procedures are set forth in the specifications, the percentage of maximum dry density is not specified, but the desired value is given to field inspection forces by the design office.

spade. Many inspectors, in fact, have had success in judging density by noticing the resistance of the compacted soil to penetration by a spade.

e. Proper lift thickness is fairly easy to estimate when the inspector's judgement has been calibrated by actual thickness measurements. However, many contractors are interested in placing lifts as thick as they can get by with, and conflict often arises on this point. Therefore, control of lift thickness by visual observation alone is not sufficient and must be supplemented with measurements. Contractor behavior dictates the level of force that must be exercised to maintain proper lift thicknesses. As a minimum practice by the inspector, it is necessary to make measurements on the same point on the construction surface after every few layers.

f. Much useful information can be gained by observing the action of compacting and heavy hauling equipment on the construction surface. If the water content of the fill material is uniform and the lift thickness is not too great, the action of the roller will indicate whether water content of the material is satisfactory and good compaction is being obtained. For example, it is likely that soil-water content is too high if on the first pass of a rubber-tired roller the tires sink to a depth greater than or equal to half the tire width, after several passes, excessive rutting of the soil surface is observed, the surface ahead of the roller shows signs of weaving or undulating (as opposed to "springing"). It should be noted that the characteristics just described may sometimes be caused by tire pressure which is too high, but in most instances they are caused by water content which is too great. On the other hand, if the roller tracks only vary slightly or not at all and leave the surface hard and stiff after several passes, the soil is probably too dry. For most soils with the proper water content, the roller will track nicely on the first pass and wheels will embed 3 to 4 in.; there should always be some penetration into soil at its proper water content, although penetration will decrease as the number of passes increases. After several passes of a sheepsfoot roller, the roller should start walking out of the fill if adequate and efficient compaction is being obtained. Walking out means that the roller begins bearing on the soil through its feet with the drum riding a few inches above the soil surface. If the roller walks out after only a few passes, the soil is likely too dry. If the roller does not walk out but continues churning up the material after the desired number of passes, either the soil is too wet or foot contact pressure is too high. Another significant observation during compaction by sheepsfoot roller is whether or not the feet are coming out clean. Soil is generally too wet when large amounts of material are being picked up by the feet and knocked off by the cleaning teeth. If soil is at the proper water content, only a small amount of sticking should occur.

g. At a proper water content there will always be a noticeable "springing" of the embankment surface as it reacts to the passage of any heavy construction equipment; the amount will depend largely on soil type. However, a sudden sinking or rising of the surface under the weight of the passing equipment is a good indication that a soft layer or pocket exists below the surface; if there is no spring at all, it is probable that several lifts of fill have been placed too dry. If such a condition is noticed, it should be checked by the laboratory and the condition corrected if the underlying layers do not meet specifications.

5-10. Field Control Testing and Sampling

a. General. Field control testing (field density tests) and record sampling of compacted fill are conducted for two basic reasons: to ensure compliance with design requirements, and to furnish a permanent record of as-built conditions of the embankment. Field control testing consists largely of determinations of the water content, density, and classification of the field-compacted material. Record sampling consists of obtaining undisturbed samples (often with companion disturbed bag samples) at selected locations in the embankment during construction.

b. Field density testing and record sample programs.

(1) Frequent control tests should be performed at the start of fill placement; after rolling requirements have been firmly established and inspection personnel have become familiar with material behavior and acceptable compaction procedures, the amount of testing can be reduced. Many factors influence the frequency and location of control tests and record samples. Frequency of testing will depend on the type of material and how critical the fill being compacted is relative to the overall job (for example, an impervious core will naturally require more control than will a random berm). Sampling should be carried out at locations representative of the area being checked. It is vitally important in control tests that soil specimens be properly sized; specimens that are too small yield inaccurate and misleading results. Guidance regarding proper specimen sizing is given in EM 1110-2-1906 and by Gilbert (1990).

(2) A systematic testing and sampling plan should be established at the beginning of the job. Control tests are usually designated as routine and are performed at designated locations, no matter how smoothly the compaction operations are being accomplished. A routine control test should be performed for every 1,000 to 3,000 cu yd of compacted material and even more frequently in narrow embankment sections where only a small volume of material raises the section height considerably. In the first lift above the foundation, tests should be made more frequently to

ensure that proper construction is attained in this important area. The locations of record samples should be at the discretion of the design engineer and should also be stated on a predetermined plan of testing. A rough guide for taking record samples is one for every 30,000 cu yd of core fill and every 30,000 to 50,000 cu yd of compacted material outside the core. Since the record samples are taken primarily to determine the shear strength of the fill, it may be more important in many dams to concentrate more tests in the material outside the core because this is where a major portion of the resistance to sliding is developed. For dams with narrow central plastic clay cores placed wet of optimum water content for impermeability and flexibility, flanked by large lean clay zones, record samples should be taken mainly in the clay shells.

(3) In addition to routine control tests, tests should be made in the following areas: where the inspector has reason to doubt the adequacy of the compaction, where the contractor is concentrating fill operations over relatively small areas, where special compaction procedures are being used (power tampers in confined areas, etc.), where instruments are located, and adjacent to abutments.

c. Record samples. Undisturbed record samples may be obtained by carefully carving out about a cubic foot block of the compacted fill. The sample is then sealed in wax and encased in a wooden box or protected by other methods of packaging against disturbance or water loss. Undisturbed record samples are also taken by trimming around a large steel cylinder as it is pushed into the fill (e.g., the Fort Worth district has used a sampler 7-1/2 in. in diam by 10 in. high). Details for obtaining and preserving record samples are described in EM 1110-2-1907. Undisturbed record samples are subjected to shear and perhaps consolidation testing by the division laboratory, and the material from trimmings and unused portions of the record samples or of the companion bag samples are used for laboratory compaction, gradation, specific gravity, Atterberg limits, and other laboratory tests. Undisturbed record samples and bag samples must be tested promptly if the results are to be useful in construction control.

d. Field density tests. Field density determination consists of volume and weight measurements to determine wet density of in-place fill and water content measurement to determine fill water content and dry density. Volume and weight measurement can be determined by direct or indirect methods. In direct measurements, weight of the material removed from a hole in the fill and hole volume are used to determine wet density. Direct water content determination involves drying the soil in an oven at 110 ± 5 °C, then weighing the dry soil to determine water loss. Determining density and water content by indirect methods involves measuring a characteristic of the material that has been previously correlated with density and/or water content. As a rule, field density tests should be taken one lift thickness deep.

(1) Direct methods.

(a) Direct methods of measuring volume include sand displacement, water balloon, drive cylinder, piston sampler, and water displacement. Apparatus, procedures, and guidance in obtaining satisfactory results for the sand displacement, water balloon, drive cylinder, and piston sampler are given in EM 1110-2-1907. The sand displacement and water balloon methods are the most widely used for measuring in-place density because of their applicability to a wide range of material types and good past performance records. Apparatus for these two methods is shown in Figure 5-7. Sand displacement is the most reliable and most frequently used method; it should be the referee test for all other control methods. The drive cylinder and piston sampler are good for obtaining samples from which the density can be ascertained, but are limited to moist fine-grained cohesive soils containing little or no gravel and moist fine sands that exhibit apparent cohesion. The water displacement method is generally used for testing gravelly soils where holes as large as one cubic yard are needed to obtain accurate results. A water-displacement density test is shown in Figure 5-8. A thin plastic sheet is necessary to line the hole to prevent leakage, and special equipment is often required for handling and weighing the large volume of excavated material and measuring the large volume of water. A 3-to-5-ft-diam steel ring with a height about equal to the compacted lift thickness is often used where the fill surface is rough and uneven. The volume of water required to fill the ring with the plastic liner in place is determined, the water and liner removed, and then the hole is dug without moving the ring. The liner is then placed in the hole, and the volume of water required to fill the hole to the top of the ring is determined. The difference between the two volume measurements is the in-place volume of excavated fill material. Apparatus and procedures for large volume water displacement tests are described by Hammer and Torrey (1973) and by Gordon and Miller (1966).

(b) Water content measurement is required to control placement water content and to determine dry density for field tests. Methods for direct water content determination include conventional oven drying, hot plate or open flame drying, drying by forced air, and drying in a microwave oven. In conventional oven drying, a soil specimen is dried to a constant weight in an oven maintained at a temperature of 110 ± 5 °C and the weight loss determined. Conventional oven drying is the standard for accuracy in water content measurement; details of the test are described in EM 1110-

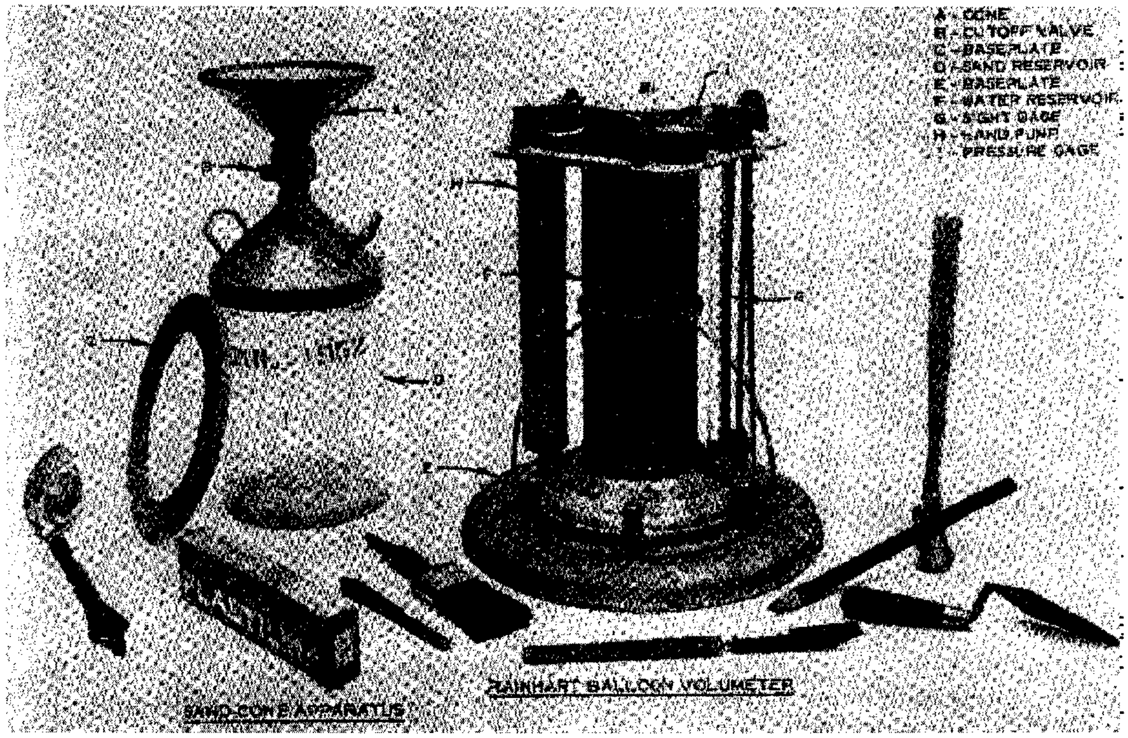


Figure 5-7. Displacement density apparatus

Figure 5-8. Large-scale water displacement density test in progress at New Hope Dam, North Carolina

2-1906. The hot plate method utilizes a small tin pan and a hot plate, oil burner, gas burner, or some other device to apply high temperature quickly; the test is performed by weighing a sample of wet soil, drying the soil on a tin pan over high heat, and weighing again to determine how much water was lost. Hot plate drying is fast but can result in inaccuracy in water content because high uncontrolled temperature applied to the soil can drive off adsorbed water and burn or drive off volatile organic matter, neither of which should be removed in a normal water content test. In the forced hot air or "moisture teller" procedure, a specimen is placed in a commercially available apparatus containing an electric heater and blower. Hot air at 150 to 300 °F is blown over and around the specimen for a preset time. A 110- or 230-V source is required for the apparatus, which can accommodate specimen masses from 25 to 500 g. Drying times will depend on specimen size and material type but are estimated to vary from 5 min for a sand to as long as 30 min or more for a plastic clay. Care must be exercised when using this method also, since the soil may be overdried or underdried with a resulting inaccuracy in water content.

(c) Microwave oven. A Computer Controlled Microwave Oven System (CCMOS) has been developed at WES and demonstrated to be an acceptable and useful piece of equipment for rapid determination of water content for compaction control. The principal of operation of the system is that water content specimens are weighed continuously while being heated by microwave energy; a small computer monitors change in water content with time and terminates drying when all "free" water has been removed. CCMOS is essentially automatic; after the operator has placed a specimen in the oven system, the controlling computer performs all required tasks (including calculations) through software, and returns the final water content with no additional input required from the operator. A water content test in the CCMOS typically requires 10 to

15 min; the system has been field-tested at the Yatesville Lake and Gallipolis Lock projects in the Ohio River division. At the projects, companion tests were performed in a conventional oven at 110 ± 5 °C and in CCMOS. Data returned from the projects are shown in Figure 5-9; statistical analysis of the data shows that CCMOS produces water contents that are within 0.5 percent of the conventional oven water content. Special procedures must be used when drying materials which burst from internal steam pressure during microwave drying (which includes some gravel particles and shales) and highly organic material, which requires a special drying cycle. CCMOS will not produce correct water contents in soils with high gypsum content; therefore, no attempt should be made to use the system to dry such materials. (However, it must be noted that a special drying procedure is required to dry gypsum rich soils in the conventional constant temperature oven). CCMOS and its operation and use are described by Gilbert (1990). Components of the system are shown in Figure 5-10.

(d) Pressure tester methods. The pressure tester method for water content determination involves combining moist soil in a sealed chamber with calcium carbide (these react with water in the soil to release gas) and relating the resulting gas pressure to soil water content. Accuracy can be a problem when using this technique since soils and especially fine grained clays bind and hold water at different energy levels. Consequently, there is no assurance that calcium carbide will react correctly with bound or adsorbed water; calibration tests must be performed to correlate pressure tester water content with conventional oven water

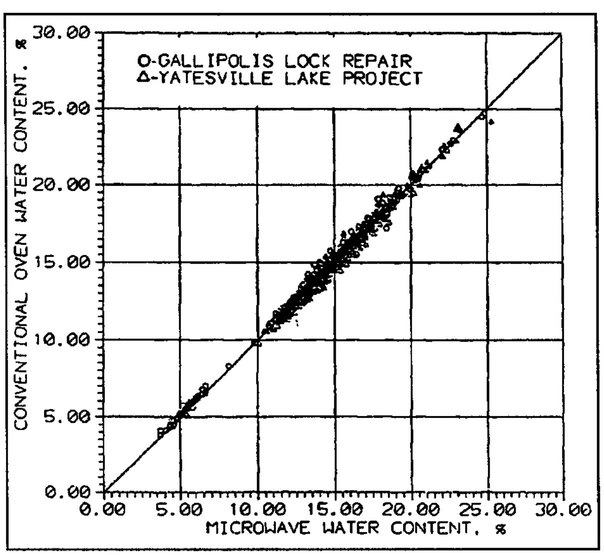


Figure 5-9. Conventional oven versus CCMOS water content data

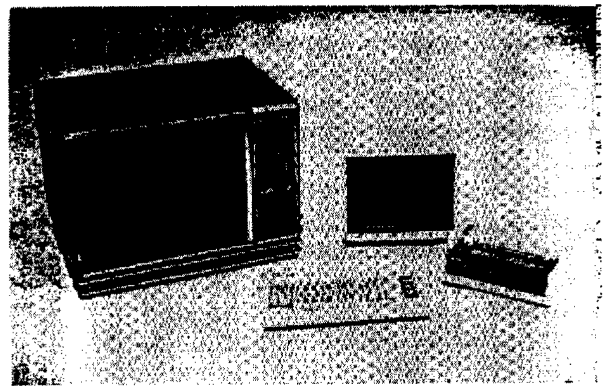


Figure 5-10. Components of the microwave drying system

content. Special care should also be taken in using the pressure tester technique with organic soils, since accuracy is affected by the presence of organic matter in soils. The pressure tester technique is most effective and accurate on relatively dry soils (less than 20 percent water content) which readily disaggregate; the technique becomes cumbersome and possibly dangerous when testing excessively wet soils, as very high gas pressure may develop in the test chamber. The American Society for Testing and Materials (ASTM) has recently prepared a standard for the calcium carbide pressure tester method of water content determination (ASTM D 4944). The procedure states that uncertainty and sources of error in using the procedure arise from the fact that water entrapped in soil clods does not react correctly or completely with calcium carbide; additionally, some soils contain chemicals which react unpredictably with the reagent to give erroneous results. It is important to realize that when calcium carbide reacts with water, acetylene gas is released which is highly flammable to the point of being explosive; additionally, the gas is irritating to the skin and eyes. Therefore, appropriate safety measures must be exercised when using this procedure.

(2) Indirect methods. Indirect methods can often be successfully used to measure density and water content; however, indirect methods should never be used instead of direct methods or without careful calibration and correlation with results obtained from direct methods. Additionally, indirect measurement results should be periodically checked against direct measurement results during construction. Indirect methods include the use of the nuclear moisture-density meter, the Proctor penetrometer (often called the "Proctor needle") and the pressure tester method of water content measurement.

(a) Proctor penetrometer. The Proctor penetrometer is generally accurate only under ideal conditions; it requires careful calibration using soils of known density and water content and considerable operating experience. Even then the results may be questionable due to the significant influence of nonuniform water content or variation due to the influence a small piece of gravel can have on the penetration resistance. The Proctor penetrometer is, therefore, not recommended for general use in compaction control; it can be a very useful tool in supplementing the inspector's visual observations and providing a general guide for detecting areas of doubtful compaction. The procedure for determining the relation between wet unit weight, penetration resistance, and water content is described in the ASTM Standards, Designation D 1558-63.

(b) Nuclear method.

- The nuclear method is an expedient means by which both water content and density determinations can be made more rapidly than by conventional direct methods. Improvements in the design of nuclear equipment and a better understanding of the nuclear principles have led to increasingly widespread use of nuclear gauges. The nuclear method is not permitted as a primary control, but is used to supplement direct methods. A 1969 survey of Corps of Engineers use of nuclear gauges showed 13 Corps offices were using such instruments in various applications. A 1990 survey of nuclear density gauge use in earthwork construction within the Corps showed that seven districts were using nuclear instruments to supplement other methods of density and water content determination. Guidance given by Webster (1974) and by Rosser and Webster (1969) requires that before a nuclear density gauge is used on a Corps of Engineers job, results obtained using factory curves must be compared with density and water contents determined by conventional methods. Based on this comparison, corrections may be required to the factory curve or a new calibration curve may have to be developed. It should be noted here that recent research has shown that the calibration of nuclear gauges is highly nonlinear in determination of water content or soil density at water contents greater than about 40 percent, and steps should be taken to account for this nonlinearity.

- Most nuclear gauges are built to measure density by one or more methods, classified as the direct transmission, backscatter, and air-gap density methods as shown in the schematics of Figure 5-11; however, all nuclear gauge methods are based on the principle of using gamma radiation to establish a density relationship. The direct transmission method is reported to yield the best accuracy, in that material composition and surface roughness influences are minimized. The backscatter method avoids the need to create an access hole in the compacted soil because the unit rests on the surface. The air-gap method (shown in Figure 5-11c) raises the device above the surface to lessen composition error, but accuracy will still not match that of the direct transmission method. Moisture measurements utilize a method based on the principle of measuring the slowing of neutrons emitted into the soil from a fast neutron source, usually using the backscatter method. Generally, the density and water content measuring devices are incorporated into a single self-contained unit. Both surface-type nuclear gauges, which test materials at depths greater than 1 ft, are now available. Descriptions of gauges available from a number of manufacturers are given by Smith (1968). Modern nuclear gauges contain a microcomputer which processes gauge readings to directly calculate and display wet density, dry density, degree of compaction, and water content.

- No license is required by the Atomic Energy Commission (AEC) for using nuclear gauges when the radiation-emitting source is a naturally occurring radioactive nuclide. A license is needed when the radiation-emitting source is a by-product radioactive nuclide. All Corps applications for AEC licenses, renewals, amendments, and correspondence thereto must be forwarded through normal Corps channels to HQDA (CESO-ZA), Washington, DC 20314, for processing. AEC standards are contained in Title 10, Part 20, Code of Federal Regulations (Atomic Energy Commission 1966). Full time radiation inspectors with special training must be present on Corps projects where nuclear gauges are used. This requirement can be a barrier on small jobs or jobs with marginal funding.

- The advantage of the nuclear method is the speed with which density and water content determinations can be obtained as compared with conventional methods. An in situ density and water content determination can be made in approximately 15 min as compared with a period as long as 24 hr for conventional methods when oven drying is used. In addition, the possibility of human error is minimized. However, the field density and water content must still be related to a compaction curve or to maximum and minimum densities, as is the case with data obtained by conventional methods. Consequently, it is necessary to obtain a sample of the material at the location of the nuclear test in order to relate the field and laboratory date.

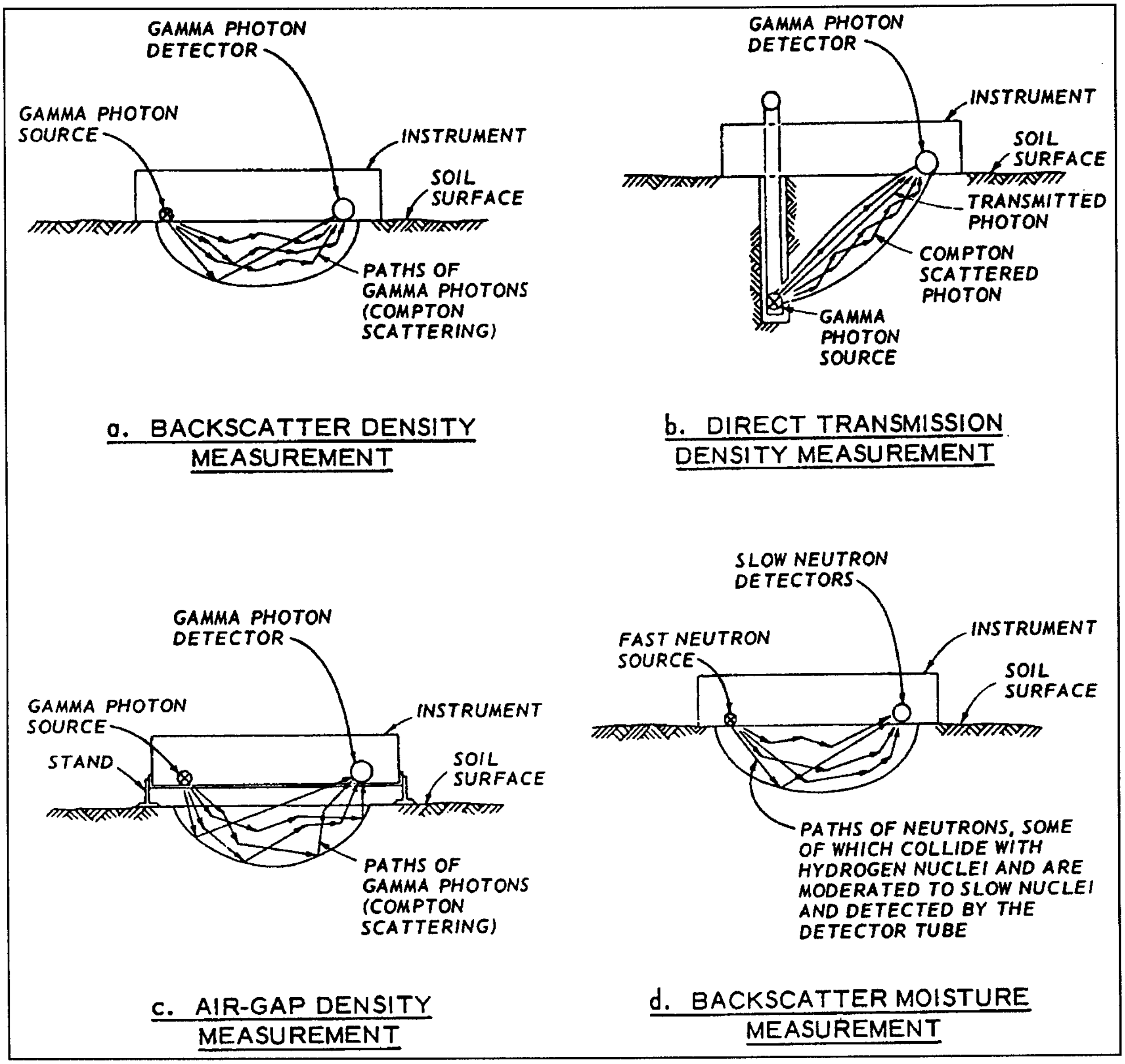


Figure 5-11. Schematic diagrams of density and moisture measurements

- The operating principle of the nuclear moisture gauge is, very basically, that neutrons from a radioactive source are released into a soil/water mixture then detected some distance away after they have traveled through the soil. The (initial) statistical energy spectrum of the released neutrons is known; after reacting with the soil/water medium for a period of time, the neutrons reach a state of energy equilibrium which is detected and measured by a probe. Detected energy level is then related to soil water content through calibration. Neutrons lose energy primarily by colliding with chemically bound hydrogen present in the (soil-water) medium, and neutrons are absorbed by certain elements which may be present in the soil. Therefore, some of the factors that adversely affect water content measurement using this procedure may be more clearly visualized: (1) *All* chemically bound hydrogen causes neutron energy loss, including that in organic matter, adsorbed water, and structurally bound water as well as "free" water. Only free water should be included in a normal water content determination; the gauge cannot discriminate between hydrogen in free water and hydrogen in other sources. (2) Certain elements (such as iron, potassium, manganese, boron, and chlorine) are highly absorptive of neutrons. The presence of these elements in soils will cause erroneous water content determination using a nuclear gauge. Because of the possible presence of generally unknown quantities of organic matter, adsorbed water, structurally bound water, and highly absorptive elements, water content measured by the nuclear gauge must be frequently checked against that determined in the conventional oven to account for the factors which are known to influence nuclear gauge results. In addition, nuclear gauges react with and are affected by other nuclear gauges in close proximity; therefore, a nuclear gauge should not be used within 30 to 40 ft of another nuclear gauge in use in the field. A major disadvantage of nuclear gauges is that specimen size is unknown and can never be established with certainty; the volume "probed" by a nuclear gauge is determined by water content, soil mineralogy, grain-size distribution, and geometry of the test configuration (for example, results determined in a narrow utility trench may be in considerable error relative to results obtained on a flat, obstruction-free soil surface).

- Additional disadvantages of nuclear methods for determining field densities and water contents are general lack of understanding of the method as well as factors affecting the results and, consequently, lack of confidence in the results; calibration curves must be developed and/or verified by field tests for each instrument; and although the proper use of nuclear gauges presents no health hazards, rigid safety regulations and documentation requirements must be met. For this last reason, field parties are sometimes reluctant to use nuclear equipment.

e. Test pits. It is sometimes desirable to excavate deep test pits to determine the overall condition of the compacted embankment. Field density tests can be made and undisturbed record samples can be obtained at various elevations as the pit is being dug, and the degree of uniformity or water content with depth can be obtained by testing samples at frequent depths. An important advantage of the test pit is that it allows a visual inspection of the compacted fill; soft spots can be detected, and it can be determined whether or not successful bonding of the fill lifts has been accomplished. Large-diameter bucket auger holes (30 to 36 in.) can also be utilized effectively for this purpose. All tests and visual observations should be thoroughly documented, including numerous photographs. Test pits must be backfilled with properly compacted soil.

f. Methods of relating fill density and water content to maximum density and optimum water content.

(1) The fill density and water content must be related to laboratory values of maximum density and optimum water content of the same material in terms of percent compaction and variation of fill water content from optimum. For this comparison to be meaningful, valid laboratory values must be selected.

(2) Performance of the standard five-point compaction test on the field density test material is ideal, as it gives the correct values of maximum dry density and optimum water content directly. However, the five-point test is time-consuming and generally not possible on material from each field density test.

(3) There are other, less time-consuming methods based on identification of the field density material with one of the soils on which standard compaction tests have been performed in connection with design studies and during construction. The means of identification are as follows:

(a) Two-point compaction test.

(b) One-point compaction test.

(c) U.S. Bureau of Reclamation (USBR) rapid compaction control.

(d) Atterberg limits correlations.

(e) Grain-size distribution correlation (sometimes used for coarse-grained soils).

(f) Visual comparison.

These methods are discussed in detail in Appendix B. The two-point and one-point methods follow essentially the same procedure as the five-point method, but are quicker since fewer points need to be run. It should be noted that the five-point method requires wetted soil cured overnight prior to compaction to allow uniform distribution of added water. In the one- and two-point methods, whether adding water to or drying back the fill material, thorough mixing is required to obtain valid results. Water contents and dry densities from the one- and two-point methods are plotted on the same plot as the five-point laboratory compaction curves used for control. The curve best fitting the plotted points is selected, and the field values are compared with the maximum density and optimum water content of that curve. Atterberg limits correlations are based on correlations of liquid limit, plastic limit, or plasticity index with optimum water content and maximum dry density. In the USBR rapid method, a wet density compaction curve is developed from three wet density compaction points, and the percent of maximum dry density and deviation from optimum water content are computed without having to perform water content tests. The visual method consists of establishing by visual examination that the field density material is the same as one of the materials on which laboratory compaction curves were developed. It is a frequently used method, but is the least desirable because materials that look very much alike and have the same soil classification can have widely varying compaction characteristics.

g. Procedures for gravelly soils. Results of the five-point, two-point, one-point, and visual methods are usually correlated directly with field density test results if appropriately sized compaction molds are used (see EM 1110-2-1906). However, the Atterberg limits correlations and USBR rapid methods are based on the minus No. 4 sieve fraction; consequently, the field density test results must be corrected to obtain the water content and density of the minus No. 4 fraction. Corrections must also be applied to the field results if the laboratory compaction curve is based on a scalped material (corrections would not be made if particles larger than 2 in. are replaced with an equal weight of particles from 2 in. to the No. 4 sieve and tested in the laboratory in a 12-in. compaction mold). The equations necessary to make these corrections and procedures for applying them are given in Appendix B.

h. Evaluation of test results and subsequent actions to be taken. As soon as field test results are obtained, they must be compared to appropriate values of maximum dry density and optimum water content to determine if specification requirements have been met. If measured values match or exceed specification requirements, the next lift can be added. If test results show that specifications have not been met, corrective measures must be taken immediately. A lift must be rejected if the material is too wet or too dry. If density is too low but water content is acceptable, additional rolling is all that is required. If, however, water content is outside specifications, the entire lift should be reworked and rerolled. A lift that is too wet should be worked by disking until the water content is lowered to an acceptable value and then recompacted. A lift that is too dry should be disked, sprinkled, and redisked until the additional water is uniformly distributed, then recompacted. It is important when reworking a rejected lift that the full lift depth be reworked, not just the upper portion. All reworked lifts should be retested for density and water content. It is desirable to determine the reason(s) for an unsatisfactory lift in either borrow or fill operations, so that conditions causing the problem may be corrected on future lifts.

5-11. Operations in Adverse Weather

a. Cold weather.

(1) Research (Sherards et al. 1963) has shown that good compaction is not obtained on frozen soil or on soil at temperatures near freezing. Contractors will often want to keep working as long as possible in cold weather, and the Resident Engineer may be faced with a difficult problem in deciding exactly when it becomes too cold for further fill placement. There are no definite criteria for establishing the temperature below which satisfactory work is impossible. The rate at which unfrozen soil loses heat and freezes depends on the size of the construction surface and the rate of fill placement. In cold weather it is important to keep the construction surface "active," i.e., fill placement must continue without lengthy interruptions. Work has been continued at some dam sites in 20 to 30 °F weather 24 hr a day, 7 days a week to keep the construction surface active. Work must be terminated whenever water in the soil freezes quickly and equipment operation becomes awkward. Underwater dumping in water with floating ice should not be allowed because of the possibility of entrapping ice in the fill. Construction in cold weather must be limited to special situations and always performed under close observation with extreme care.

(2) Protecting the construction surface during the winter when operations are shut down is another problem. The degree of protection required depends on the severity of the winter. In most parts of the United States, it is not necessary to use any protection if the embankment surface has been properly seal-rolled; the worst damage is a heaving and loosening of the upper few inches of the embankment fill by frost action. Before construction starts again in the spring, the surface material should be excavated to a depth below the line of frost action. The depth at which to excavate is best determined by visual examination of shallow test holes. In colder climates where the embankment freezes to a depth of several feet, it may be desirable to protect the construction surface during winter with several feet of loose material. Other methods of protection have been used in extremely cold areas, such as ponding water over the construction surface and even using some type of heating coils on foundations for structures (spillway, outlet works tower, etc.).

b. Wet weather.

(1) Maintaining proper water content during periods of high precipitation is always a problem. Impervious materials should never be placed on embankments during rain, although construction operations can often be continued

successfully between rains. Water content of material spread on embankments can be reduced somewhat during periods between rains by plowing or disking before rolling.

(2) It is desirable to compact fill material as soon as possible after spreading to minimize the time loose fill is exposed to precipitation. Rubber-tired rollers are superior to sheepsfoot rollers when rains are frequent because they leave a relatively smooth compacted surface, whereas the sheepsfoot roller leaves a loose rough surface that readily soaks up rain water. If a sheepsfoot roller is used for general compaction, smooth-wheel rollers (steel or rubber) can be employed to seal the surface when rain is imminent. In any event, the construction surface should be kept sloped to allow the water to run off instead of standing in puddles and soaking in. After a rain, if some ponding does occur, it is usually easy for the contractor to install a few small ditches to drain these areas.

(3) It is often necessary after a rain to scarify and work the construction surface to a depth below that of excessive moisture penetration until it is dried to a satisfactory water content or, to remove and waste all affected material. If procedures to facilitate runoff are followed (sloping the surface, sealing the surface with smooth rollers, etc.), the depth of moisture penetration will be kept to a minimum.

c. Dry weather.

(1) If material being dumped on the fill is too dry for proper compaction, water must be added by sprinkling after it is spread and before it is rolled. The amount of water added and the blending required will depend on grain size and plasticity of the soil, fine-grained soils of high plasticity requiring the greatest amount of blending. Soil must be worked with disks to thoroughly blend and homogenize added water into the soil. The importance of uniform moisture distribution cannot be overemphasized; if pockets of wet and dry soil are allowed in uncompacted material, very poor compaction will result.

(2) Sprinkling the soil can be accomplished by hosing from a pipeline, located along either the embankment toe or the crest, or by the use of water trucks. The latter method is the most effective and the most commonly used today. Pressure sprinkling systems on trucks are superior to gravity systems and should be employed if at all possible. Water sprays must not be directed on the soil with such force as to cause fines to be washed out. Until the inspectors and contractor personnel have gained a "feel" for the amount of water needed, rough computations of the number of gallons to add for a given area should be made, and water applied accordingly. After a few trials, a feel for the proper amount will develop. The coarser and less plastic the soil, the more easily water can be added and worked uniformly into it. It is very difficult to obtain uniform water content distribution in plastic clays containing lumps without a "curing" period of a few days; this is, of course, not practical on the embankment surface. Consequently, disking followed by addition of water and then thorough mixing with a heavy rotary pulverizer may be required to obtain uniform distribution of water in such soils.

5-12. Compaction in Confined Areas

a. Confined areas are those where normal rolling operations with heavy equipment are restricted or where heavy equipment cannot be used at all and hand compactors must be used. Compaction with hand compactors should be avoided and heavy equipment used in these areas if at all possible. Confined areas, where heavy equipment can often be used on a careful basis if maneuvering room is available, include fairly smooth abutments (rock or earth), conduit barrels, towers, etc. Confined areas where hand compactors often must be employed are adjacent to thin concrete structures, such as wing walls, guide walls, etc., where heavy equipment might damage the structure; adjacent to rough, irregular rock abutment slopes where heavy equipment cannot get in close enough to the surface to squeeze the fill into all the irregularities and openings in the rock; and around seep rings or plugs where maneuverability is a problem.

(1) Heavy equipment. When conditions are such that heavy compaction equipment can be used to compact the soil against rock abutments or walls of concrete structures, the construction surface of the embankment should be sloped at about 1V on 6H for a distance of 8 to 12 ft away from the rock or concrete. This will allow the roller to act more directly in compacting the soil against the abutment or structure. The area can then be rolled perpendicular to the face of the abutment or structure by heavy pneumatic equipment or a sheepsfoot roller or by heavy pneumatic equipment in a direction parallel to the face.

(2) Hand compactors. If heavy rollers cannot be used in this manner, the roller should be allowed to work as close as possible, and the portion of embankment directly against the rock or concrete should be compacted with smaller equipment in thinner lifts. Smaller equipment refers to hand-operated power tampers, as shown in Figure 5-12, or power tampers mounted on small tractors. These tampers are usually gasoline-operated or operate on compressed air. Hand-operated power tampers (sometimes called rammer compactors) are probably the most widely used equipment for compacting fine-grained soils in confined areas. A loose lift thickness of not more than 4 in. should be employed in conjunction with these power tampers. Hand compactors should have a minimum static weight of 100 lb, and the

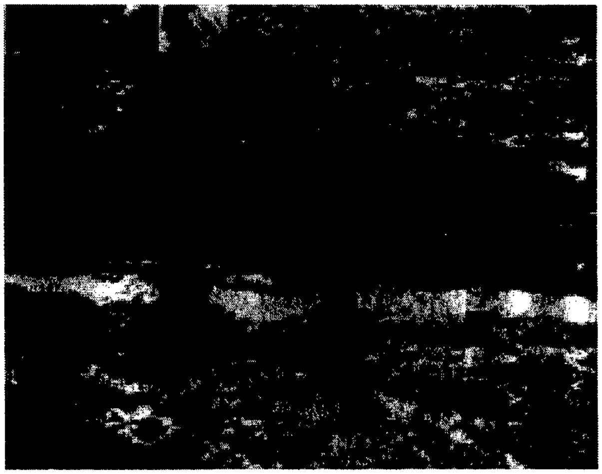


Figure 5-12. Compacting against rock abutment with a hand-operated power tamper, Green River Dam, Kentucky

inspector should carefully check to ensure that the manufacturer's recommended air pressure is being developed. Experience has shown "two-by-four" wood rammers, or single-foot compressed air tampers (commonly referred to as "powder puffs" or "pogo sticks") do not produce adequate compaction. It is important that zones of hand compaction and compaction by heavy equipment overlap so that no uncompacted material exists between them.

b. Where impervious material is to be placed adjacent to abutments or concrete structures, it should be as fine-grained as practicable. Soil must be plastic enough to penetrate all irregularities in the abutments and to form a well bonded seal.

c. Close compaction control must be exercised in these areas since they are generally more critical with respect to seepage and damaging settlements causing cracking and piping than the main embankment. A special sampling program should be established, and an inspector must watch operations involving the use of power tampers at all times.

Section IV
Pervious Fill

5-13. Definition

Pervious fill material as used in this manual is defined as free-draining cohesionless sand and/or gravel, containing less than approximately 5 percent passing the No. 200 sieve.[1] Standard impact compaction tests on such materials do not yield well-defined values of optimum water content and maximum dry density, and field densities are related to maximum and minimum density determinations made according to the relative density procedures in EM 1110-2-1906 rather than to maximum dry density as determined in the standard compaction test.

5-14. Compaction Equipment

a. Vibratory steel-wheel rollers in the weight range of 5 to 10 tons are the best equipment for compacting pervious sands and/or gravel. It should be noted that compaction equipment has been steadily increasing in weight and it is not uncommon to find steel-wheel roller compactors with a weight of 15 or more tons. Drum-drive self-propelled vibratory rollers have been found to be effective on fine uniform sands when other vibratory rollers were not. Rubber-tired rollers are sometimes specified, and crawler tractors are sometimes used if they can produce required densities. Crawler tractors may be effective for compaction in rough or hilly areas where a vibratory roller cannot operate well—for example, in compacting a horizontal drainage layer on an undulating foundation. The contact pressure of the tractor should be at least 9 PSI, and the tractor should operate at that speed which imparts the greatest vibration to the fill.

b. Where free-draining pervious material is placed as backfill against concrete walls and around concrete structures such as outlet conduits, small compaction equipment is needed because of restricted area or because heavy equipment is not allowed close to the walls. Two common types of vibratory compactors are a small steel-wheel vibratory roller and a "vibrating plate" compactor. Air-operated concrete vibrators have also been used successfully in densifying narrow, relatively deep zones of pervious backfill. Figure 5-13 shows pervious backfill being placed against a concrete wall with a hand-operated compactor in the background. Vibratory-type units should be checked frequently to ensure that they are operating at a frequency level giving the highest possible density. For cohesionless materials, the frequency of vibration should generally fall between 1,100 and 1,500 vibrations/cycles per minute.

[1] Outer embankment zones of some dams composed of coarse-grained soils containing appreciable amounts of fines (i.e., greater than 5 percent) are sometimes designated as "pervious zones." The compaction equipment, procedures, and control for materials comprising the pervious zones referred to here are those presented in *Section III* on impervious and semipervious fill.

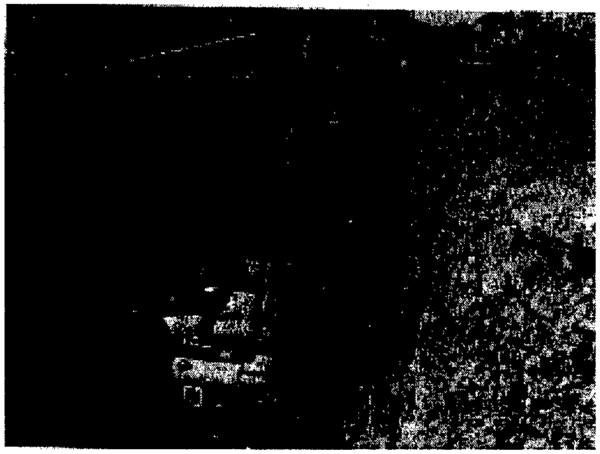


Figure 5-13. Placement of pervious material adjacent to stilling basin wall, Brookville Dam

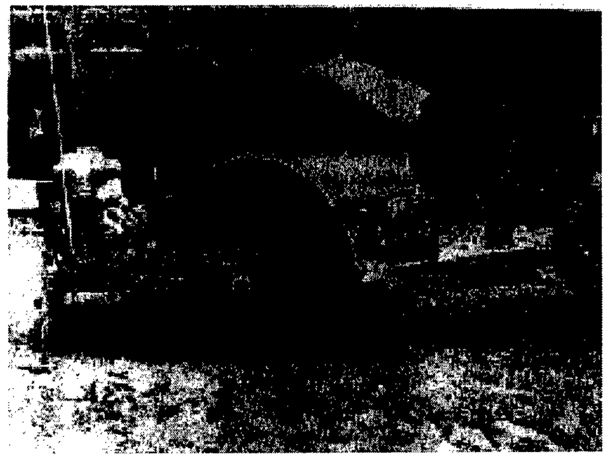


Figure 5-14. Sprinkling system built on frame of vibratory roller (feeder pipes are shown above frame, spray bars are attached below) used at DeGray Dam, Arkansas

5-15. Material Gradations

Gradations of materials to be placed in large pervious embankment zones are generally not specified except to restrict the maximum size and/or the percentage by weight of particles smaller than the No. 200 sieve. On the other hand, a particle-size distribution within limits defined by roughly parallel gradation curves on the standard grain-size (semi-log) plot is specified for select pervious materials to be used in horizontal and inclined drainage layers and in pervious transition layers.

5-16. Water Content Control

Water content control is unnecessary in gravel, and the material may simply be compacted in its as-received condition. If the material is sand or contains a significant proportion of sand sizes, the material must be maintained in a high degree of saturation during compaction using water trucks with pressure spray bars, hoses connected to header pipes laid along the embankment, or other approved methods of water application. If pervious sand is compacted at a low degree of saturation (with insufficient water), surface tension between the water present and the sand grains will cause the moist soil to "bulk," and in this state it will not densify efficiently under an applied compactive effort; the result will be a poorly compacted weak layer which may cause problems. It is therefore imperative that sand be in a high degree of saturation as the roller passes over it, but it is often difficult to achieve a high degree of saturation because surface tension will also prevent water from flowing freely through sandy soils. Figure 5-14 shows the unique wetting system used to saturate the sand for the vertical sand drain at DeGray Dam. The entire spray system was attached to the frame of the vibratory roller itself. The water was fed to the system by means of a long hose connected to a water truck which moved as necessary to accommodate the roller. Very good results were achieved with this arrangement. If a so-called pervious material contains appreciable fines (say, more than 5 to 10 percent passing the No. 200 sieve), it is probable that water content control must be exercised to ensure that the water content is within a range that will permit desired compacted densities to be obtained.

5-17. Lift Thicknesses and Number of Passes or Coverages

These will be established in the specifications. Pervious fill is commonly placed in 6- to 15-in. lifts when it is to be compacted by three or four passes of a vibratory steel-wheel roller or a 50-ton rubber-tired roller, or in 6- to 8-in. loose lifts when it is to be compacted by three to six coverages by a crawler tractor. A note of clarification is necessary here. Guide Specification CE-1306 calls for a specified number of "complete passes" of a rubber-tired roller or a vibratory steel-wheel roller, where a "complete pass" is defined to be complete areal coverage of the lift. This concept does not retain the same meaning in a crawler tractor where the tracks (because of their wide separation) do not make complete areal coverage of the lift. Therefore, when compaction by a crawler tractor is specified, the specification should require coverages by the *tracks* of the tractor. In confined areas where small vibratory rollers or hand-operated vibrating compactors are required, material is normally placed in 2- to 3-in.-thick layers with vibratory compaction applied until densities comparable to those required for areas compacted with heavy equipment are achieved.

5-18. Density Requirements

Compaction equipment and procedures are normally specified without field densities being specified. The expectation is that the prescribed field compaction methods will produce the desired densities. If these are not achieved in the field, then the contractor is paid for additional rolling. Requirements established by the Office of Chief of Engineers are that the average in-place relative density of pervious fill zones should be at least 85 percent and that no portion of the fill should have a relative density less than 80 percent.

5-19. Construction Control

a. Simple control procedures.

(1) Checking lift thicknesses of pervious fill can be accomplished by the simple procedures described for impervious and semipervious materials, except that in using a shovel or rod it is often difficult to determine when the top of the underlying compacted layer is reached. This procedure is also not practical in fills containing large gravel particles. In some cases, it is possible to excavate a small hole in the loose material to the top of the underlying layer, which is identified by a relatively higher resistance to digging.

(2) The inspector must make certain that the embankment is always graded so that surface waters will not wash fines from impervious or semipervious fill materials into the pervious fill. During construction of earth dams, placement of filter materials for drainage layers should be kept higher than adjacent fill containing fines in order to prevent spillage of fine-grained soil onto the pervious material or to reduce the washing of fine-grained soils into the materials by surface runoff. The inspector should be trained to recognize the appearance of pervious material meeting specifications so that he can more easily detect, without the delay of testing, the presence of excess fines. A good indication of excessive fines is when the hauling and compacting equipment sinks in and causes ruts in the fill surface. This usually indicates that water applied during compaction is not draining through the material as it should because of clogging by excess fines.

(3) In general, a vibratory roller should push only a small amount of material ahead of it and leave a smooth surface behind on the first pass. If the roller sinks in and pushes a large amount of material in front of it, either the frequency of vibration is not correct for the particular soil being compacted or the material contains too many fines.

(4) It is more difficult to judge the compacted density of pervious material than that of fine-grained material. Resistance to penetration of a shovel or a reinforcing steel rod often is not a suitable way of checking density, and it is necessary for the inspector to rely more heavily on field control tests. Some inspectors can judge the compaction obtained in pervious fill by walking over it and feeling the reaction of the material. The uniformity of particle-size gradations in the lift should be observed. Too much vibration may cause segregation of the fill material, causing the fines to settle to the bottom of the lift. Conversely, if particles are being crushed during compaction, a layer of fines will develop in the upper few inches of the lift.

(5) The inspector should observe loading, dumping, and spreading operations, particularly if the pervious fill is well-graded material, to ensure that undesirable segregation of particles is not occurring as a result of such operations.

(6) The particular importance of horizontal and inclined drainage layers to the function of a dam, and the fact that these features are so limited in thickness, justify the special attention of inspectors to see that gradations and densities of the in-place filter materials meet the specifications.

b. Gradation. Gradation tests should be performed to ensure that the material being placed is within specification limits. The number of gradation tests needed will, of course, depend on the variability of natural pervious material as obtained from the borrow areas. Complete gradation tests should be performed on material for which the entire range of particle sizes is specified. For those materials for which only the percent finer than the No. 200 sieve or some other sieve is specified, the material should be soaked and then washed over the No. 4 and the designated lower-limit sieve in accordance with procedures given in EM 1110-2-1906. Gradation tests should also be performed on compacted material especially when it is suspected that there has been contamination with fines from surface waters or when the fill material may have been degraded by breakage of particles during compaction.

c. Field density testing and relative density determinations.

(1) The water balloon and sand volume density test methods described in paragraph 5-10*d*(1) can be used to determine the in-place density of pervious fill. It is, of course, more difficult to dig holes in pervious materials. When the fill material contains high percentages of large particles, it may be necessary to increase the volume of holes substantially and to line the holes with plastic film so that the volume may be determined by the quantity of water or oil needed to fill it. The nuclear density meter can be used for supplementary density determinations under the conditions stated in paragraph 5-10*d*(2)(b). The density of

free-draining pervious fills and filter materials cannot be related to standard impact compaction test results since water content/density relations are not valid for such materials, as they are for materials having varying degrees of plasticity. Field densities must be expressed in terms of maximum-minimum densities as determined by laboratory tests described in EM 1110-2-1906. Field densities are expressed in terms of their relation to these laboratory values, i.e., in terms of relative density. The percent relative density, D_d, of the in-place material can be computed by the equation:

$$D_d = \frac{\gamma_d - \gamma_{min}}{\gamma_{max} - \gamma_{min}} \times \frac{\gamma_{max}}{\gamma_d} \times 100$$

where

γ_d = dry unit weight of the pervious fill in place (the in-place density), pcf

γ_{min} = minimum density, pcf, from laboratory tests

γ_{max} = maximum density, pcf, from laboratory tests

(2) Field density determinations using the water balloon or sand volume procedures should be made for every 1,000 cu yd of pervious fill placed at the beginning of the job and for every 3,000 cu yd thereafter, with more frequent determinations desirable for testing in drainage layers. These tests generally should be taken one lift thickness deep, especially in sands. Although the performance of maximum and minimum density determinations on material from each field density test would give the most accurate determination of the relative density of the in-place material, this is frequently not feasible because of time and manpower restrictions. Therefore, it is often advisable to attempt to develop correlations between the gradation data and the maximum-minimum density values on materials representing the range of gradations to be expected from the sources of supply. Figure 5-15 is an example of a correlation between the percent finer than the No. 16 sieve and the maximum-minimum density values. Where a good correlation like this is developed, a simple determination of the percent finer than the No. 16 sieve is all that is needed to obtain the appropriate maximum-minimum density values in the in-place material. In other instances, good correlations may be developed between maximum-minimum density values and the percent of material passing other sieve sizes or the coefficient of uniformity. Correlations developed between minimum and maximum density values can be used to obtain minimum density after the performance of a maximum density test alone. Caution should be exercised in using such correlations for uniform sands, since the spread between the maximum and minimum density is very small and large errors may result.

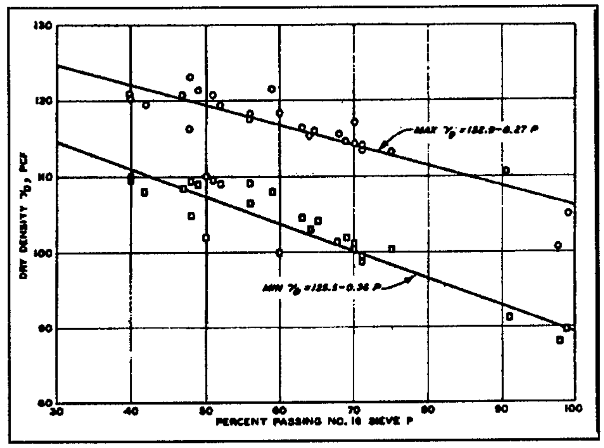


Figure 5-15. Correlation between density and percent passing No. 16 sieve

(3) Correlations such as those described above should be used only on materials from the particular sources for which the correlations were developed. Application to materials from other, geologically different sources may lead to considerable error because of differences in particle shapes and degradation characteristics. The selection of maximum-minimum density values by visual comparisons of field density test material with samples of materials on which maximum-minimum density determination have previously been made is generally not a good procedure; small differences in particle sizes that cannot be adequately detected by visual comparisons have significant effects on these density values.

5-20. Test Results and Actions to be Taken

Assume, for example, that it has been established that a lift has in-place relative densities between 80 and 85 percent. A review must be made of the relative densities of all previous lifts to ensure that a minimum average of 85 percent relative density will be maintained if the questionable lift is to be accepted. The intent of the relative density criteria is that the relative density of the material measured immediately after compaction must conform to the requirements stated in paragraph 5-18 without any consideration of increase in density caused by the placement and compaction of subsequent lifts.

Section V
Rock Fill

5-21. General

a. Embankments with large rock-fill zones are becoming more common. This is primarily due to the necessity of using sites where rock foundation conditions are unsuitable for concrete dams, the suitability of modern construction equipment to handle rock, the increasingly higher dams being constructed, and the economic benefit obtained by the maximum use of rock from required excavation.

b. Rock for construction purposes falls into two categories, soft rock and hard rock. Soft rocks such as shales, mudstones, siltstones, claystones, chalk, earthy limestones, weakly cemented sandstones, and badly weathered igneous and metamorphic rocks break down in varying degrees during compaction, some into tight compact masses similar to impervious soils. Soft rocks are generally susceptible to further softening by exposure to air and water. Conversely, hard rocks do not readily break down during handling and compaction, and their use results in a pervious to very pervious fill depending on the amount of fines present.

c. There are no well-established criteria for construction methods best suited to either type of rock. The breakdown of most types of rock is very unpredictable, and this fact has led to the widespread use of test quarries and test fills for large rock-fill dams.

5-22. Hard Rock

a. Specifications. The specifications for pervious rock-fill sections generally require that the rock be sound, well graded, and free draining. Gradation is not usually specified, but the maximum permissible size is normally specified together with lift thickness. Rock fill is usually required to be placed by dumping from trucks, with bulldozers spreading the material to the desired lift thickness. The required placement and spreading operations should be that segregation of rock sizes is avoided. This is discussed in more detail in the following paragraphs. The type of roller, lift thickness, and number of passes will be specified, preferably based on results of test fills.

b. Placement operations.

(1) When rock is dumped on the fill surface and pushed into place by a bulldozer, the fines are moved into the upper part of the lift, thereby creating a smoother working surface for the compacting equipment. If, however, a layer of fines of such a thickness as to choke the upper part of the lift and prevent distribution of the fines throughout the lift is produced, it is necessary to specify that the rock be dumped directly in place.

(2) All oversized rock must be removed prior to compaction. This is usually done with bulldozers, crawler tractors fitted with special "rock rakes," or cranes. Oversized rocks are often pushed into a specified zone in the outer slopes. Excessively large rocks are sometimes hauled off to be dumped elsewhere, or they are broken up with a drop weight or explosives and used in the rock fill or riprap zone. Oversized rocks should never be pushed to accumulate along the contact slope of a closure section.

(3) Close inspection is also required to ensure that the material does not contain excessive fines, which can cause excessive post-construction settlements when the reservoir is filled. It is difficult to define a limiting amount of fines in the specifications and, consequently, this is rarely done. However, the design office should provide some guidance to field personnel, based on results of rock tests and fill studies, which will aid in determining when excessive fines are present. The inspector must be alert for material variations that could result in undesirable changes in gradation of the material being brought to the embankment. Such changes should be called to the attention of the contractor so that quarrying techniques can be changed.

c. Compaction.

(1) With the introduction of smooth steel-wheel vibratory rollers to this country in the last 20 years, current practice is to compact all sound rock fill, using these rollers, in comparatively thin lifts of unsluiced rock. Vibratory rollers having static weights of 5, 10, and 15 tons have been used to construct several Corps rock-fill dams.

(2) The lift thickness specified is dependent on the size and type of rock and the type of compaction equipment to be used, and is usually determined from results obtained during construction of a test fill. Generally, specified lift thickness will be no thicker than 24 in. unless test fills show that adequate compaction can be obtained using thicker lifts. Maximum size rock should not exceed 0.9 of the lift thickness.

(3) Scarification of compacted lift surfaces is not necessary and should not be allowed because it disturbs the compacted mass.

5-23. Soft Rock

a. The use of soft rocks in the past has been dictated by their availability in large quantities from required excavations. The main concern about these materials is their tendency to weather and soften with time when exposed to air and water. However, cases in which large portions of embankments were composed of soft rocks have shown that they can be used satisfactorily in random and semipervious zones, attaining adequate shear strength and experiencing no appreciable softening after placement. Where soft rocks will constitute a significant structural portion of a fill, their properties and the best methods of compaction should be determined by means of a test embankment constructed during the design studies.

b. Some types of soft rock have been compacted by first rolling over the loose lift with a heavy tamping roller equipped with long spike or chisel-type teeth ("shell breaker"), and then compacting the lift with conventional tamping or rubber-tired rollers. A summary of this technique is given by Bennett (1958).

Section VI
Semicompacted Earth Fills

5-24. Uses

Spoil berms, channel fillings, and low levees to protect farmlands are often constructed of semicompacted fills.

5-25. Specifications

Semicompacted fills are those specified to be compacted by the routing of hauling and spreading equipment over the spread layer. Lift thickness is specified, but the range of placement water contents is either not specified or permitted to vary widely.

5-26. Construction Control

Inspection of semicompacted fill is usually entirely visual, although a few density tests may be made for record purposes. The primary concern of the inspector is to ensure that the specified lift thickness is not exceeded, suitable materials are being used, and hauling and spreading equipment covers the fill uniformly.

Section VII
Sequence of Placement and Measurement of Quantities

5-27. Schedule of Construction

The schedule for construction of an earth or rock-fill dam may require stage (or phase) construction. In a wide flat valley, the embankment on one side of the river may be constructed to the full height under one contract, with subsequent portions constructed during following years. Where foundations are soft, the embankment may be constructed to a specified elevation and further fill placement deferred for a year or more to permit dissipation of foundation pore pressures or to achieve an adequate degree of consolidation. In a narrow steep valley with rock foundations, the entire embankment may have to be completed to a stipulated elevation by a certain date to prevent overtopping during the flood season. The construction schedule is developed to make maximum use of available borrow and excavation materials, considering river diversion requirements, foundation conditions, and seasonal weather conditions. The contractor is responsible for constructing the particular stage or section of embankment within the time limit specified. The inspection force is responsible for determining that each stage or section of embankment is being built in proper sequence and also that each stage or section is constructed using proper placement sequence. Changes in sequence or timing of stages should not be made without approval of the design engineer.

5-28. Placement Sequence

a. It is usually required that the embankment be brought up fairly uniformly over the entire width and length of the section under construction. Interim embankment crests should be crowned slightly to provide surface drainage during wet weather. Specified transverse slopes of interim crests may range from 1 to 5 percent. During periods of dry weather, the fill heights of central impervious zones are sometimes allowed to exceed the heights of adjacent pervious zones by as much as 5 ft to permit continuous placement of impervious material. However, special precautions such as sloping the impervious fill material away from the pervious zone are required to keep impervious material out of inclined filter zones.

b. Placing material in a cutoff trench should be accomplished by dumping and spreading the first lift of the downstream filter zone material (if such a filter is required at the downstream trench slope) and then dumping and spreading the first lift of impervious material. This should be followed by compaction of both zones concurrently, with separate equipment being used on each zone. Dumping and spreading filter layers first will help to maintain the specified width of the filter zone. A downstream horizontal drainage zone should be completely placed and covered by two lifts of downstream shell materials as soon as possible to prevent contamination of the blanket by exposure to surface waters carrying fines.

5-29. Measurement of Quantities

Measurement of excavated materials is usually based on cross-section surveys of the area before and after excavation, using the average end area method for computing quantities. For embankment fill, a cross-section survey of the outer boundaries and average end area method are used in computing quantities. For separate zones within an embankment, the theoretical quantities are computed from the lines and grades shown in the construction drawings. Inspection personnel should be completely familiar with provisions of the specifications and lines and grades shown on construction drawings so that instances of overexcavation or fill placement outside contract lines and grades are recorded. This will assist in preventing possible errors in measurement and certification of payment for quantities in excess of contract provisions.

Section VIII
Slope Protection

5-30. Areas to be Protected

Slope protection is required to protect upstream slopes against damage from wave erosion, weathering, ice damage, and damage from floating debris. Upstream slope protection of earth dams usually consists of riprap, although soil cement, concrete paving, and asphalt paving have been occasionally used when riprap was not economically justified. Dams with outer shells of sound, durable, large rock may not require further protection. Downstream slope protection is required to protect against damage from surface erosion by wind and rain. Downstream slope protection includes gravel for dry climates, turf in humid climates, riprap where tailwater may create wave action, and waste rock. Proper field construction procedures and enforcement of specifications are particularly important in obtaining slope protection that will remain in place and in minimizing maintenance during the life of the dam.

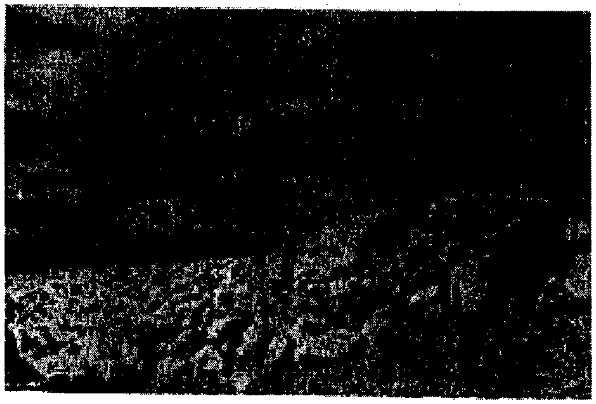
5-31. Upstream Slope Protection

Placement of upstream slope protection may be accomplished either as the embankment is being built or after the embankment is completed. This depends on the elevation limits of slope protection, the schedule for impounding reservoir water, and the type of slope protection. The best procedure is to require that the slope protection construction not lag behind earth-fill construction more than 10 ft in elevation.

a. Riprap. Riprap is the most commonly specified type of upstream slope protection. Properly graded riprap, placed to provide a well-integrated mass with minimum void spaces so that underlying bedding cannot be washed out, provides excellent slope protection. Two primary factors govern successful construction and are discussed below.

(1) Loading from the quarry to provide a good mixture of different sizes within the required gradation in each load. Proper loading from the quarry requires that blasting operations produce proper sizes and that the inspector be experienced in inspecting the loading operations.

(2) Placing loads on the slope to provide uniform distribution of different sizes without segregation and rearrangement of individual rocks to provide a rock mass without large voids. A gradation test (performed by weighing a sufficient quantity) should be made for each 10,000 cu yd of placed riprap. Two ENG forms for plotting gradations curves are available. These forms are ENG Form 4055 dated April 1967, "Riprap Gradation Curves" and ENG Form 4056 dated April 1967, "Gradation Curves for Riprap, Filter, and Bedding". Placement should be accomplished by placing loads along the slope against previously placed riprap; this will reduce segregation of sizes that would otherwise occur if loads were dumped in separated piles. The best method of placement to avoid segregation is to use a skip as shown in Figure 5-16a. Dumping rock at the top of the slope into a chute should never be allowed since this will result in segregation. If dumping is done from trucks, it is usually necessary to winch load haulers down the slope to the placement location. Dumping should proceed along horizontal rows and progress up the slope; loads should not be dumped to form rows up the slope. If very large (4 to 5 ft diam) rock is specified, a crane with an orange peel attachment (Figure 5-16b) operating on a platform built up on the slope can be used. Other equipment such as Gradalls, cranes with clamshell buckets, and rubber-tired front end loaders, can be used to place riprap. These are preferable to dumping from haulers. Close visual inspection after dumping and spreading is required to determine the degree of uniform distribution of different sizes and close-knit arrangement of individual pieces. Reworking, generally by hand, will almost always be required; however, reworking can be kept to a minimum if care is taken when loading to ensure that each individual load has the proper amount of each size rock (i.e., the proper gradation). Supplemental gradation checks might be made by the photogrid method, described by Curry (1964), in which a 10-ft-by-10-ft aluminum pipe form containing a 1-ft-by-1-ft grid or rope is placed on the riprap and photographed. From the photograph, the number and size of stones visible at the surface are determined. However, for materials that have been selectively arranged, as by hand, this may not provide an accurate determination of size distribution. Firm enforcement of specifications is required, especially during early stages of riprap placement



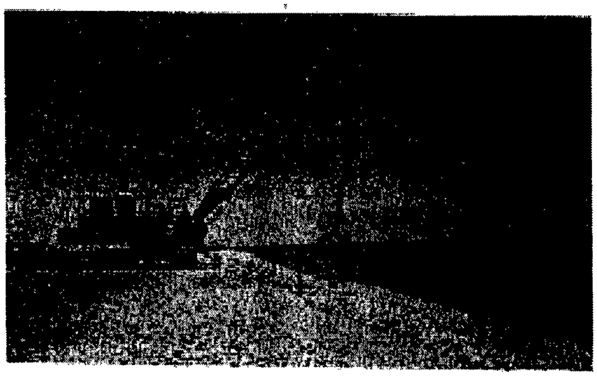


Figure 5-16. Placement of riprap and bedding material. (a) Dragline with skip placing riprap at Texarkana Dam, Texas; (b) Dragline with orange peel attachment placing riprap at Lavon Dam, Texas; (c) Gradall used to place bedding material, main stem levee, Chariton River Project, Missouri

to ensure a well-graded mass having no large voids.

(3) A bedding layer or layers must be provided between the riprap and embankment material (except in the rare case that the latter is highly cohesive) to protect the embankment material from eroding by wave action and to provide a stable base for riprap. Placement is shown in Figure 5-16c. Many riprap failures have occurred because the bedding material was not large enough to preclude being washed out by wave action through interstices of the riprap. Removal of the bedding causes settling and dislodging of overlying riprap, further exposing bedding and embankment material to direct wave action. It is, therefore, necessary that bedding material meet specifications relating to gradation and layer thickness. It is good practice to place rock spalls or crushed stones of like size between the bedding and riprap if they are available from quarry or required excavation. The term spalls refers to the finer materials resulting from rock excavation for materials such as riprap. Spalls must be durable fragments of rock, free of clay, silt, sand, or other debris. The gradation of spalls will vary and must be specified for each particular job. The use and minimum and maximum size of spalls or equivalent crushed stone should be established in the design memo and should be specified if it is appropriate. In some cases, spalls may replace graded bedding material.

b. Soil cement. Soil-cement slope protection has not been used for Corps of Engineers dams in the past, but may be used in the future if riprap would be excessively expensive. The recommended method is to use plant-mixed rather than mixed-in-place soil cement. Inorganic sandy or gravelly soils with not more than 35 percent finer than the No. 200 sieve are suitable for soil cement use. The plasticity of the fines should be low. Soils classified as GW-GM, GP-GM, GM-GC, SW-SM, SP-SM, and SM-SC would be suitable. The amount of cement and water added to the soil is based on laboratory tests to determine compaction properties and resistance to freezing and thawing cycles. Plant-mixed soil cement is usually spread in 6-in. horizontal lifts along the slope in a strip, 7 to 10 ft wide (depending on the slope angle and specified thickness perpendicular to the slope), and compacted by sheepsfoot or rubber-tired rollers. Succeeding lifts are stairstepped up the slope. Control of cement and water content, uniformity of mixture, and density tests are required, as well as measurement of both lift thickness and lift width. Bonding of the compacted layers is important, and scarifying is usually specified on surfaces of compacted layers prior to placing the next lift. Curing is also important, and in dry climates special measures may be needed to prevent drying of compacted layer surfaces. Reference material on construction operations can be obtained from the Portland Cement Association. Another source of information is the U. S. Bureau of Reclamation, which has used soil-cement slope protection extensively.

c. Other upstream protection. Monolithic concrete, hand-placed riprip paving (grouted and ungrouted), and asphalt paving are other methods of protection for upstream slopes. These methods have been used infrequently, if at all, for Corps of Engineers dams in the past.

5-32. Downstream Slope Protection

a. Grass turf. Grass turf for protection of downstream slopes is usually specified in humid climates for earth embankments. Where the downstream embankment zone is composed of pervious material, sufficient fine-grained soil or topsoil must be placed to support vegetation growth. The method usually specified consists of clearing the slope of any roots and stones, tilling to a depth of at least 4 in., fertilizing, seeding or sprigging, compacting, watering, and maintaining as required to establish the turf. Temporary or permanent protection should be established on completed portions of the embankment as soon as possible. The usual practice of waiting until near the end of construction and trimming slopes by filling erosion channels with loose material and then fertilizing and seeding has resulted in continuing maintenance problems at several projects. Specifications usually provide detailed instructions with which inspection personnel should be familiar. Frequent inspection should be made to ensure the following:

(1) That the soil is properly tilled and not allowed to migrate down the slope during tilling, which might create depressions or undulations.

(2) That the specified type, quality, and quantity of fertilizer and seed are used.

(3) That turfing operations are conducted during good weather conditions, and necessary interim precautions are taken to prevent erosion, such as mulching with hay or burlap for a protective covering.

b. Riprap. Riprap placed on the lower downstream slope to protect against wave action from tailwater should be controlled in the same manner as upstream riprap. Above the elevation needing such protection, dumped rock (usually waste rock) is used only when readily available from required excavation or stripping operations. (Control should conform with specification requirements).

c. Gravel. Gravel or rock spalls (depending on available material) are sometimes used for downstream slope protection. Where the outer downstream shell contains random granular materials, it is often specified that cobbles and rocks be pushed out to the edge and used in the slope protection. In this case, it is desirable that placement of downstream slope protection be kept 5 to 10 ft behind embankment placement. The gravel or spalls are usually dumped and spread horizontally along the outer slope to depths of at least 1 ft measured perpendicular to the surface.

Chapter 6
Miscellaneous Construction Features

6-1. River Diversion

Control of the river during construction is accomplished by the construction of cofferdams and rerouting of the river to permit foundation preparation and construction of the embankment in the dry. In narrow steep valleys, a diversion tunnel is usually constructed in one of the abutments, and the river is diverted through the tunnel by an upstream cofferdam. A downstream cofferdam may also be required to control the tailwater. In wide valleys, after construction of the outlet works and most of the embankment except for a gap at the river, river flow is diverted through the outlet works. To do this, it may be necessary to excavate a diversion channel from the river and to block off the river channel by dumping fill. An earth cofferdam can then be constructed upstream across the gap, tying into embankment sections already constructed, or as the case may be, to an abutment on one side. Following construction of a downstream cofferdam, if needed, the foundation of the closure section of the dam is prepared in the dry and the closure section constructed. Cellular-type cofferdams constructed in connection with lock-and-dam projects are beyond the scope of this manual, and only embankment-type cofferdams are discussed below.

a. Cofferdams. The schedule and method of cofferdam construction and the degree of construction and control required are influenced by the diversion scheme. Sometimes the cofferdams can be built in the dry using normal embankment construction procedures, but in other cases cofferdam construction may involve placement of fill under water. Current Corps of Engineers policy requires that the location, sequence of construction, source of materials, zonation, section geometry, top elevations, construction methods, and time schedule be specified.

(1) River closure. Cofferdam construction for river closure can be accomplished by placing loose material such as earth, sand, gravel rock, or precast concrete elements from one bank to the other. The type of material required depends on the severity of river currents. The closure should be scheduled during low river stages when conditions are favorable for embankment placement and flood possibility is at a minimum. Two methods of placement are generally used. One is the end fill method, as shown in Figure 6-1, where fill is dumped or pushed off the end of the embankment to advance it across the river. The other is the level method, in which the fill is placed across the river at the same height. The first method is simpler, but may require large rock or concrete elements to withstand erosion by high current velocities caused by the channel width reduction. The second method keeps velocities low, since water flows over a long section of the cofferdam crest as it is being built. However, special facilities such as a bridge or trestle, a cableway, or barges may be required over deep water to provide a means of dumping fill into the river. In addition, continuous monitoring of underwater fill elevations is required to detect detrimental erosion. A summary of construction methods and equipment used in cofferdam construction is found in EM 1110-2-1602.

Figure 6-1. Examples of end fill closure method. (a) River closure, cofferdam, Mica Dam, British Columbia; (b) two days before final closure, Dalles Closure Dam, Columbia River

(2) Design and construction requirements.

(a) Normally, cofferdams will be designed by the Government and the contractor may be permitted to recommend changes leading to a superior or more economical and adequate plan (ER 1110-2-8152). Design of a cofferdam by the contractor is allowed only when the construction schedule provides ample review and design time to ensure a competent and safe design or where no major damage or significant delays would occur from a failure. Compaction procedures as well as other pertinent construction aspects, including construction and surveillance requirements, would be covered in the contract plans and specifications. Inspection and control testing, as outlined in Chapter 5, are required especially for cofferdams which will become part of the dam to ensure compliance with specifications or to determine when specified procedures need to be modified. The safety of cofferdams depends to a large extent on competent construction. Intensive inspection is required to ensure compliance with design requirements and safe construction practices. A contract requirement for quality control by the contractor does not relieve resident inspection forces from performing necessary and adequate inspection and surveillance.

(b) Slopes of temporary cofferdams depend on the type of material used, degree of compaction, and their height. Cofferdams are often designed on the basis of past experience. If the contractor is permitted to design a cofferdam, it should be required that an analysis of slope stability with plans for construction be furnished. Slopes of cofferdams designed by the Government are specified in the contract drawings.

(c) Seepage under cofferdams and through uncompacted rock-fill cofferdams can be a serious problem unless seepage control provisions such as a cutoff and/or an upstream impervious zone are included. For cofferdams designed by the Government the plans provide necessary measures for controlling underseepage and through-seepage. However, particular attention should be given to any contractor's plans for temporary cofferdams to ensure that adequate seepage control measures are included. It is also important that where thin upstream blankets are placed against rock-fill or coarse gravel, careful attention be given to providing properly graded transitions between the impervious and coarser materials.

(3) Slope protection. Slope protection against wave action is usually not required for cofferdams. However, where a river is restricted to a channel by temporary cofferdams prior to diversion, slope protection against current action may be required. EM 1110-2-1601 should be referred to for guidance on rock weights required to resist various current velocities.

b. Closure sections. After portions of the dam have been constructed with the river confined to a natural or constructed channel, cofferdams are constructed to divert the river through the outlet works so that the closure section can be constructed in the dry. This may cause flooding of upstream borrow areas, in which case alternate borrow areas or stockpiling of borrow materials for the closure section should be considered prior to river diversion.

(1) Foundation preparation and fill compaction. Clearing, stripping, and cleaning of the foundation area in the closure section must be carefully inspected to ensure an adequate foundation. The closure section is generally the highest section of the dam and is constructed to final grade in a much shorter time than the rest of the embankment; consequently, special emphasis on inspection and control testing is justified to ensure that proper materials are placed at the specified water contents and compacted to the desired or specified densities.

(2) Cleaning of adjacent embankment slopes. Riprap placed on the end slopes of completed embankment sections to prevent stream erosion must be removed prior to placing fill for the closure section. In addition, the end slopes must be cut back as necessary to provide unweathered and adequately compacted material adjacent to the closure section fill.

(3) Observations during construction. Because of the rapid construction, the following problems may occur in earth-fill closure sections:

(a) High excess pore water pressures in impervious zones of the embankment and in impervious foundations.

(b) Transverse cracking between the closure section and previously completed embankment sections caused by differential settlement.

(c) Bulging of outer slopes of largely impervious sections from the use of material which is too wet.

The procedures for construction and criteria for compaction stipulated in the plans and specifications for minimizing these problems may include drainage layers in the embankment and at the contact between embankment and foundation (*but never under or through an impervious core*) to help dissipate pore pressures, flattening of slopes, and fill water contents wet of optimum to provide a more plastic fill to minimize cracking. The use of higher fill water contents may, however, accentuate problems of bulging and high pore water pressures. If potential problems are indicated, surface movements data and piezometer readings must be obtained daily and continuous plots maintained so that the development of excessive pore water pressures and/or bulging can be detected early and corrective action taken. Corrective measures taken after consultation with the design office may include slowing down fill placement, removal of fill, flattening slopes, addition of stability berms, or installation of drains.

6-2. Stage Construction

a. Stage construction refers here to construction of an embankment in stages with substantial intervals of time during which little or no fill is placed. This may be necessitated by environmental conditions which make the construction season very short or because fill placement must be restricted or even stopped to allow excess pore water pressure in the foundation and/or fill to dissipate.

b. Important features of construction control for stage construction include the following:

(1) Review of contractor's plans of operation as required by specifications to establish the adequacy of methods and procedures, and field inspection to ensure that approved time schedules and construction methods and procedures are followed.

(2) Inspection to ensure that the surfaces of completed sections of embankment on which additional fill will not be placed for several months are sealed and/or shaped to drain readily to prevent saturation and adequately protected against erosion.

(3) Inspection to ensure that outside slopes of stage-constructed embankments that will eventually form the final slopes of the dam are within specified tolerances.

(4) Observation and evaluation of piezometer and settlement data for soft foundations and consultation with the design office to determine when fill placement can be safely resumed. The design office has the responsibility to determine when fill placement can be resumed.

(5) Inspection to ensure that protective materials such as riprap, grass sodding, and trash or other debris are removed from previously completed embankment surfaces prior to resuming fill placement.

(6) Inspection of completed embankment surfaces prior to resuming fill placement to determine the need for removal of pervious surfaces materials which have become contaminated with fines or recompaction of surface materials where water content and density are unsatisfactory. Field density and water content tests should be performed to check visual observations.

6-3. Surface Drainage Facilities

Temporary surface drainage facilities are required to keep surface runoff and slope seepage out of excavations, to prevent contamination of filter zones by muddy runoff, to prevent saturation of loose fill before and during compaction, and to prevent ponding of water on compacted fill. Surface drainage is the responsibility of the contractor, as the specifications require certain phases of foundation preparation and embankment construction to be performed in the dry. Past field experience has shown that the provision and maintenance of adequate drainage have frequently been neglected, and more attention needs to be given to them by the inspection force. Grading to direct surface water away from excavations, ditching to intercept water before it reaches an excavation or work area, provision of sumps to collect seepage water, and pumping from the sumps are common means of handling surface drainage. The extent of needed facilities is dependent on the frequency, intensity, and seasonal distribution of precipitation. For permanent drainage facilities for roads and other features, reference should be made to TM 5-820-4.

6-4. Service Bridge Pier Foundations

Piers for service bridges to intake structures are sometimes constructed during early stages of embankment construction. At several dams, lateral deformations of the embankment under the stresses imposed by fill placed after pier construction have caused significant upstream movements of the piers. Therefore, construction of piers should be delayed until the embankment has been carried to near its final height. After construction of the piers, surveys should be made at regular intervals to monitor any movement of the piers no matter what steps are taken to prevent movement.

6-5. Instrumentation

Instrumentation of earth and rock-fill dams includes piezometers, slope indicators, settlement devices, surface movement monuments, internal vertical and horizontal movement indicators, and seismic movement devices (see paragraph 7-3 for required records). Seismic movement devices are described in detail in ER 1110-2-103. Various types of instrumentation devices, procedures for installation, observation and maintenance, frequency of observations, collection, recording, analysis, and reporting of data, and possible causes of malfunctions are discussed in EM 1110-2-1908. The basic description and operating mechanism of instruments generally installed in embankment dams are given in Appendix E. A comprehensive reference on geotechnical instrumentation for monitoring field performance including calibration, use, installation, and maintenance is given in EM 1110-2-1908.

a. Installation. Installation of instrumentation devices, particularly electronic types, should be supervised, if not actually done, by experienced personnel from within the Corps of Engineers or by firms specializing in instrumentation installation. The Resident Engineer staff must be familiar with the planned locations of all instruments and appurtenant apparatus or structures (such as trenches for piezometer lines and terminal house, etc.) so that necessary arrangements and a schedule for installation can be made with the contractor and/or with the office or firm that will install the devices. Inspectors should inspect settlement gauges furnished and installed by the contractor. Records must be made of exact locations and procedures used for installation and initial observations. Riser pipes, tubes, or leads extending above the embankment surface must be protected from damage by earth mounds, guard posts, or other means. Inspectors should ensure that necessary extensions are added as the fill is constructed to higher elevations. This feature of construction generally occurs at a time when the contractor is anxious to begin or accelerate embankment construction. Therefore, patience, cooperation, and understanding must be exercised by the contractor and the Resident Staff.

b. Observations. Schedules for observations during construction are generally established by the design office. Pore pressure piezometers are observed frequently during construction to provide data for use in slope stability checks and to control, if necessary, the rate of fill construction on soft foundations. Initial observations should be checked to ensure their validity and accuracy, since these readings usually form the basis to which subsequent observations are related. Observations should be plotted immediately after each set of readings is taken and evaluated for reasonableness against the previous set of readings. In this way, it is often possible to detect errors in readings and to obtain check readings before significant changes in field conditions occur. On large projects all records should immediately be processed by computers. This will generally result in the quickest results with a minimum amount of effort. Possible sources of errors other than erroneous readings are discussed in EM 1110-2-1908. ER 1110-2-1925 prescribes the forms for recording instrumentation data and instructions for reporting them.

6-6. Haul Roads, Maintenance Roads, and Public Roads

Haul roads are temporary roads built by the contractor for access to work areas. Maintenance roads (or service roads) are temporary or permanent roads for access by Government forces to facilities requiring maintenance. Public roads include relocated permanent roads and roads for access to public-use facilities at the site.

a. Haul roads. Haul roads, although usually not shown on the plans, should be discussed in the specifications in general terms. The contractor should be required to submit detailed plans for all haul road layouts including grades, widths, locations, and post-construction obliteration and cleanup. Construction control consists generally of seeing that the contractor maintains and operates on the haul roads in accordance with sound safety practices, and enforces adequate traffic control where public-use roads are crossed. Haul roads should not be located near the edge of excavated slopes where the weight of road fill and/or heavy equipment or ponding of water could endanger the stability of slopes. Haul roads up embankment slopes should be scabbed on the outside or final embankment slope and relocated periodically where highly compacted zones could develop in the embankment under the road which could lead to cracking. The contractor should be required to remove haul roads that would endanger slope stability. The practice of placing pervious material across impervious zones to support heavy hauling equipment should be discouraged. If allowed, all pervious material must be carefully removed. Lastly, it is important that environmental considerations be made in connection with haul roads so that no permanent scars in the project aesthetics remain.

b. Maintenance and public roads. Construction control of maintenance or surface roads and public roads is required to ensure compliance with specifications relating to fill, filter, base coarse, and pavement materials; compaction of fills, subgrades, base course, and wearing course; and installation of drainage structures. Field compaction control is similar to that required for earth fill as discussed in Chapter 5.

Chapter 7
Records and Reports

7-1. Daily Reports

Daily reports or logs are prepared by inspection personnel covering their assigned areas of work. The reports are prepared on ENG Forms 2538-1-R (MILITARY) and 2538-2-R (CIVIL), "Daily Log of Construction" (Figures 7-1, 7-2, 7-3, 7-4). Similar locally prepared forms or books may also be used. The importance of daily reports should be stressed at all levels of the Resident Engineer's force, since daily reports form a part of official Government records and may by used as evidence in court or in other legal action. The reports are also valuable in determining possible causes of distress, unusual seepage, or other potentially critical conditions during and after construction is completed. Inspectors' reports are not personal records and should be carefully controlled and accounted for as official records. The daily reports must be thorough, accurate, neat, and legible. Detailed information on the following items should be included as outlined in ER 1180-1-6:

a. Contract number and contractor's name.

b. Description and location of the work.

c. Date.

d. Weather.

e. Items of equipment and procedures used.

f. Type and amount of work performed.

g. Type and number of field control tests performed by the contractor and by Government forces, and brief comments on results obtained.

h. Progress of work, delays, causes of delays, and extent of delays.

i. Instructions given to contractor, including name of contractor representative talked to, and resulting actions taken by contractor.

j. Details of any controversial matters.

k. Visitors to the inspector's area of responsibility.

l. Safety infractions/violations observed and corrective actions taken.

Daily reports are often included in a weekly progress report to the District Construction Division. The weekly report also contains information on overall progress on unusual conditions, and on instructions and directions given to the contractor by the Resident Engineer to attain desired results in accordance with plans and specifications.

7-2. Compaction Control Reports

Records of compaction control are required to document the procedure used and the adequacy of results obtained. The records also provide information for evaluation of compaction control and for use in determining causes of distress or unusual conditions that may develop during or after construction. Forms for use in tabulating daily field control data are contained in Appendix C, together with instructions for their preparation and additional information required when submitting the forms to higher echelons. These forms can also be used in submitting monthly reports of data, as required by ER 1110-2-1925. Evaluation of compaction control is necessary to ensure that the method and procedures are producing the quality of in-place fill required by the specifications. Summaries of compaction control data described in Appendix D provide a convenient means of evaluating the adequacy of compaction. These summaries are required to be submitted by the district office for review by higher echelons (ER 1110-2-1925).

7-3. Instrumentation Observations

a. Records of instrumentation observations are important in determining changes in pore water pressure, deformations, and settlements. The data, when summarized and evaluated, are useful in substantiating design assumptions and thus in verifying stability during construction, designing modifications, and additions to structures, or determining causes of operating difficulties. Standard forms and instructions for recording the following types of instrumentation readings are contained in Appendix E.

(1) Closed-system piezometer data.

(2) Open-system piezometer data.

(3) Subsurface settlement plate data.

(4) Surface reference point data.

b. For guidance on recording of data not covered by Appendix E, frequency of observations, and evaluating instrumentation data, see EM 1110-2-1908. Instructions for submission of data to a higher echelon are contained in ER 1110-2-1925.

QUALITY ASSURANCE REPORT (QAR) DAILY LOG OF CONSTRUCTION - MILITARY *(ER 1180-1-6)*	THE QCR WILL BE ATTACHED TO OR FILED WITH THE QAR. REPORT NUMBER
PROJECT	DATE CONTRACT NUMBER
CONTRACTOR *(Or hired labor)*	WEATHER

CQC Control phases attended and instruction given:

Results of QA activities and tests, deficiencies observed, actions taken and corrective action of contractor. Include comment pertaining to contractors CQC activities.

VERBAL INSTRUCTION GIVEN TO CONTRACTOR: *(Include names, reactions and remarks)*

HAS ANYTHING DEVELOPED ON THE WORK WHICH MIGHT LEAD TO A CHANGE ORDER OR FINDING OF FACT? ☐ NO ☐ YES

ENG Form 2538-1-R, May 94 (MILITARY) EDITION OF AUG 89 MAY BE USED UNTIL EXHAUSTED (Proponent: CEMP-CE)

Figure 7-1. Front of ENG FORM 2538-1

Information on progress of work, causes for delays and extent of delays, weather, plant, material, etc.
Information, instructions or actions taken not covered on QCR report or disagreements:
SAFETY: *(Include any infractions of approved safety plan, safety manual or instructions from Government personnel. Specify corrective action taken.)*
REMARKS: *(Include visitors to project and miscellaneous remarks pertinent to work.)*

QA REPRESENTATIVES SIGNATURE	DATE	SUPERVISOR'S INITIALS	DATE

(REVERSE OF ENG FORM 2538-1-R) Page 2 of 2 Pages ★ U.S.G.P.O.: 1990-718-543/10302

Figure 7-2. Back of ENG FORM 2538-1

QUALITY ASSURANCE REPORT (QAR)
DAILY LOG OF CONSTRUCTION - CIVIL
(ER 1180-1-6)

THE QCR WILL BE ATTACHED TO OR FILED WITH THE QAR.

TO	REPORT NUMBER
	DATE
PROJECT	CONTRACT NUMBER
CONTRACTOR *(Or hired labor)*	WEATHER

PORTION OF SCHEDULED DAY SUITABLE FOR OPERATIONS					TEMPERATURE	
STRUCTURAL EXCAVATION %	BORROW EXCAVATION %	EMBANKMENT %	CONCRETE %	STRUCTURE %	MINIMUM	MAXIMUM

HAS ANYTHING DEVELOPED ON THE WORK WHICH MIGHT LEAD TO A CHANGE ORDER OR FINDING OF FACT? ☐ NO ☐ YES *(Explain)*

24 HOUR PRECIPITATION	
INCHES	ENDING M

NUMBER OF GOVERNMENT EMPLOYEES						RIVER STAGE	
SUPERVISORY	OFFICE	LAYOUT	INSPECTION	TOTAL	LABOR	FEET	TIME M

NUMBER OF CONTRACTOR'S EMPLOYEES				NUMBER OF SHIFTS ☐ 1 ☐ 2 ☐ 3					
SUPERVISORY	SKILLED	LABORERS	TOTAL	FROM M	TO M	FROM M	TO M	FROM M	TO M

Attach list of the following: (a) Major items of equipment either idle or working, and (b) Number and classification of contractor personnel onsite.
Note: If the contractor's Quality Control Report (QCR) contains the information it need not be repeated.

CONTRACTOR/SUBCONTRACTORS AND AREA OF RESPONSIBILITY FOR WORK PERFORMED TODAY:

a. __________

b. __________

c. __________

d. __________

e. __________

f. __________

g. __________

WORK PERFORMED TODAY: *(Indicate location and description of work performed. Refer to work performed by prime and/or subcontractors by letter in Table above.)*

Days of no work and reasons for same:

Information on progress of work, causes for delays and extent of delays. Plant, material, etc.

ENG Form 2538-2-R, May 94 (CIVIL) Edition of Aug 89 may be used until exhausted. (Proponent: CEMP-CE)

Figure 7-3. Front of ENG FORM 2538-2

CQC CONTROL PHASES ATTENDED AND INSTRUCTIONS GIVEN:

RESULTS OF QA INSPECTIONS AND TESTS, DEFICIENCIES OBSERVED, ACTIONS TAKEN AND CORRECTIVE ACTION OF CONTRACTOR. INCLUDE COMMENT PERTAINING TO CONTRACTORS CQC ACTIVITIES

VERBAL INSTRUCTIONS GIVEN TO CONTRACTOR: *(include names, reactions and remarks)*

CONTROVERSIAL MATTERS IN DETAIL:

INFORMATION, INSTRUCTIONS OR ACTIONS TAKEN NOT COVERED IN QCR REPORT OR DISAGREEMENTS:

REMARKS: *(include visitors to project and miscellaneous remarks pertinent to work)*

SAFETY: *(include any infractions of approved safety plan, safety manual or instructions from Government personnel. Specify corrective action taken.)*

QA REPRESENTATIVES SIGNATURE	DATE	SUPERVISOR'S INITIALS	DATE

(Reverse of ENG Form 2538-2-R) Page 2 of 2 pages.

Figure 7-4. Back of ENG FORM 2538-2

c. Observations of seepage quantities from relief wells, toe drains, seepage galleries, and other seepage control installations should be recorded on appropriate locally developed forms for evaluation at the project and district levels. Unusual seepage conditions should be reported immediately to the district office along with available observational data for evaluation of the effect of existing conditions on the safety of the dam.

7-4. Construction Foundation Report

a. Construction foundation reports are to be prepared for major projects as required in ER 1110-1-1801. Voluminous records are maintained during construction and are often filed on completion of the project without regard for possible future usefulness. Such information is readily available if it is assembled in a concise foundation report. Because of the complexity of this report, responsibility for its preparation should be assigned at a very early stage of construction so that its preparation can progress as the work progresses.

b. Instructions for preparation of the foundation report, including a suggested outline, are contained in ER 1110-1-1801. Drawings in the reports should accurately pinpoint major features and not simply be rough sketches. Photographs are especially useful if foundation problems arise in the future. Therefore, good photographs of major features, labeled accurately as to location, should be used liberally throughout the report.

7-5. Final Construction Report

A final construction report should include the foundation report outlined in ER 1110-1-1801 and the following:

a. A narrative history of the project, including schedules of starting and completing various phases, treatment of unusual conditions, construction methods and equipment used, quantities of materials involved, and other pertinent information.

b. As-built drawings.

c. Construction photographs.

d. Summary of field compaction control data and laboratory test results.

e. Summary of other tests for acceptance control.

f. Test results on record samples.

g. Results of stability and other analyses during construction.

h. Summary of instrumentation data.

i. Summaries or references to conferences and inspection visits and resulting actions implemented.

Appendix A
References

A-1. Required Publications

TM 5-818-5
Dewatering and Ground Water Control

TM 5-820-4
Drainage for Areas Other Than Airfields

ER 1110-1-1801
Construction Foundation Report

ER 1110-2-103
Strong Motion Instrument for Recording Earthquake Motions on Dams

ER 1110-2-1925
Field Control Data for Earth and Rockfill Dams

ER 1110-2-8152
Planning and Design of Temporary Cofferdams and Braced Excavations

ER 1180-1-6
Construction Quality Management

EM 1110-2-1601
Hydraulic Design of Flood Control Channels

EM 1110-2-1602
Hydraulic Design of Reservoir Outlet Works

EM 1110-2-1901
Seepage Analysis and Control for Dams

EM 1110-2-1906
Laboratory Soils Testing

EM 1110-2-1907
Soil Sampling

EM 1110-2-1908
Instrumentation of Earth and Rock-Fill Dams

EM 1110-2-1914
Design, Construction and Maintenance of Relief Wells

EM 1110-2-3800
Systematic Drilling and Blasting for Surface Excavations

U. S. Army Engineer District, Savannah 1968
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A-2. Related Publications

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A-3. Selected Bibliography

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TM 5-332
Pits and Quarries

TM 5-818-4
Backfill for Subsurface Structures

TM 5-818-6
Grouting Methods and Equipment

TM 5-852-2
Arctic and Subarctic Construction

ER 1110-2-100
Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures

ER 1110-2-101
Reporting of Evidence of Distress of Civil Works Structures

ER 1110-2-1200
Plans and Specifications for Civil Works Projects

ER 1110-2-1901
Embankment Criteria and Performance Report

EM 1110-2-1902
Stability of Earth and Rock Fill Dams

EM 1110-2-1909
Calibration of Laboratory Soils Testing Equipment

EM 1110-2-2000
Standard Practice for Concrete for Civil Works Structures

EM 1110-2-2300
Earth and Rock-Fill Dams, General Design and Construction Considerations

CE-1201
Subsurface Drilling, Sampling and Testing

CE-1305.01
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CE-1306
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CE-1307
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CE-1308
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Appendix B
Methods of Relating Field Density Data to Desired or Specified Values

B-1. General

Compaction control of soils requires the comparison of fill water content and/or dry density values obtained in field density tests with optimum water content and/or maximum dry density or determination of relative density if the fill materials are cohesionless. For fine-grained soils or coarse-grained with appreciable fines, field results are compared with results of laboratory standard effort (or in special cases 15-blow or modified effort) compaction tests performed in accordance with procedures given in EM 1110-2-1906 (Appendices VI and VIA). For free-draining cohesionless soils, relative density of the fill material is determined using test procedures described in EM 1110-2-1906 (Appendices XII and XIIA). However, see Control Using Relative Density under paragraph B-4*a* below.

B-2. Fine-Grained Soils

a. Standard compaction test. The performance of a standard laboratory compaction test on material from each field density test would give the most accurate relation of the in-place material to optimum water content and maximum density, but it is not generally feasible to do this because testing could not keep pace with the rate of fill placement. However, standard compaction tests should be performed during construction when an insufficient number of the compaction curves were developed during the design phase, when borrow material is obtained from a new source, and when material similar to that being placed has not been tested previously. In any event, laboratory compaction tests should be performed periodically on each type of fill material (preferably one for every ten field density tests) to check the optimum water content and maximum dry density values being used for correlation with field density test results.

b. One-point compaction method.

(1) In the one-point compaction method, material from the field density test is allowed to dry to a water content on the dry side of estimated optimum and is then compacted using the same equipment and procedures used in the five-point standard compaction test. (It must be mentioned that during drying, the material must be thoroughly and continuously mixed to obtain uniform drying; otherwise, erroneous results may be obtained). The water content and dry density of the compacted sample are then used to estimate its optimum water content and maximum dry density as illustrated in Figure B-1.

(2) In Figure B-1, the line of optimums is well-defined and the compaction curves are approximately parallel to each other; consequently, the one-point compaction method could be used with a relatively high degree of confidence. In Figure B-2, however, the optimums define not a line but a broad band. Also, the compaction curves are not parallel to each other and in several instances cross on the dry side. To illustrate an error that could result from using the

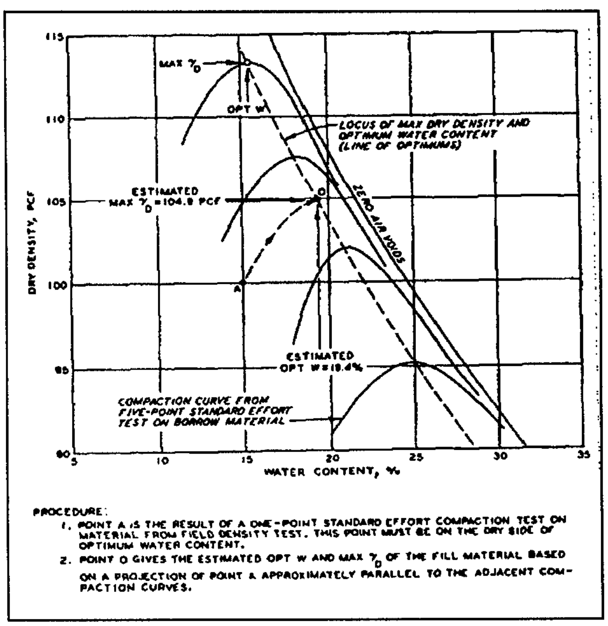


Figure B-1. Illustration of one-point compaction method

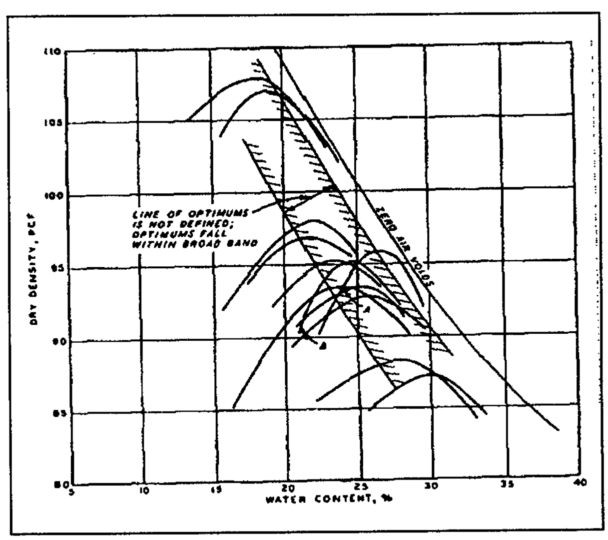


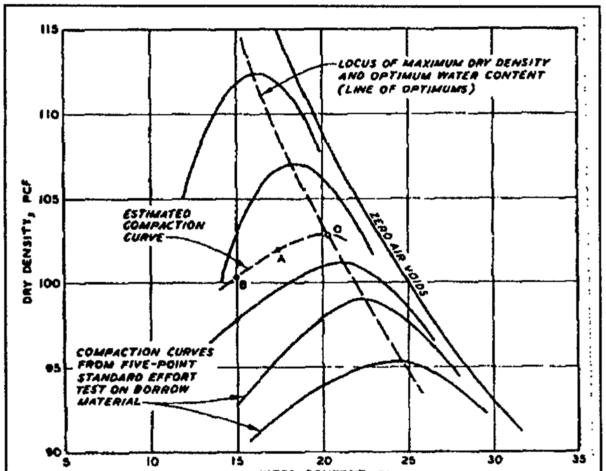
Figure B-2. Illustration of possible error using one- and two-point compaction methods

one-point method, consider the field density and water content shown by point B in Figure B-2. Point B is close to three compaction curves. Consequently, the correct curve cannot be determined from the one point. The estimated maximum dry density and optimum water content could vary from about 92.8 pcf and 26 percent, respectively, to 95.0 pcf and 24 percent, respectively, depending on which curve was used. Therefore, the one-point method should be used only when the data define a relatively good line of optimums.

c. Two-point compaction test results.

(1) In the two-point test, using the same equipment and procedures used in the five-point compaction test, one sample of material from the location of the field density test is compacted at the fill water content, if thought to be at or on the dry side of optimum water content (otherwise, reduce the water content by drying to this condition). A second sample of material is allowed to dry back about 2 to 3 percentage points dry of the water content of the first sample and then compacted in the same manner. After compaction, the water contents of the two samples are determined by oven drying or other more rapid means, and the dry densities are computed. The results are used to identify the appropriate compaction curve for the material tested as shown in Figure B-3.

(2) The data shown in Figure B-3 warrant the use of the two-point compaction test since the five-point compaction



PROCEDURE:

1. POINTS A AND B ARE RESULTS OF A TWO-POINT STANDARD EFFORT COMPACTION TEST ON MATERIAL FROM FIELD DENSITY TEST. POINTS A AND B MUST BE ON THE DRY SIDE OF OPTIMUM WATER CONTENT.
2. THE ESTIMATED COMPACTION CURVE BASED ON POINTS A AND B ESTABLISHES POINT O ON THE LOCUS, WHICH IS THE ESTIMATED MAXIMUM DRY DENSITY AND OPTIMUM WATER CONTENT OF THE FILL MATERIAL.

Figure B-3. Illustration of two-point compaction method

curves are not parallel. Using point A only, as in the one-point test method, would result in appreciable error as the shape of the curve would not be defined. The established compaction curve can be more accurately defined by two compaction points as shown. Although the two-point method is more accurate than the one-point method, neither method would have acceptable accuracy when applied to the set of compaction curves shown in Figure B-2. There are materials and instances when the two-point compaction test fails to identify the proper compaction curve. Experienced embankment construction engineers suggest that when this occurs, a third compaction point is necessary, and is performed for proper definition of the soil compaction curve.

d. Visual comparison. In the visual comparison method, selection of an appropriate compaction curve is based on visual identification of the type of material from the field density test with material (usually jar samples) on which five-point compaction tests have been run. Unfortunately, materials that appear similar can have widely varying compaction characteristics, and this method is not considered reliable.

e. Atterberg limits correlations. To develop Atterberg limits correlations, liquid limit, and plastic limit determinations and five-point compaction tests are made and plots are prepared of optimum water content versus liquid limit, versus plastic limit, and versus plasticity index. Similar plots are made of the limits values versus maximum densities. The plots are then analyzed to determine if adequate correlations exist (exhibited by plotted points falling in a narrow band across the plot). Figures B-4 and B-5 are examples of such plots. If a good correlation exists, appropriate limits tests are performed on the field density test material and the plots used to estimate optimum water contents and maximum densities of the in-place material. This method is applicable to fine-grained cohesive soils classified as CL and CH. Statistical analyses of the data shown in Figures B-4 and B-5 indicate relatively good correlations. Least square linear regressions were performed on the data shown in Figures B-4 and B-5 to determine the "best fit" linear equations to correlate optimum water content and maximum dry density with liquid limit and plasticity index. Using properties of statistical parameters, it can be shown that about 68 percent of the data points (of true optimum water content) on Figure B-4a will lie within plus or minus 1.4 percentage points of the indicated line of best fit; similarly, about 65 percent of the maximum dry density data points will lie within plus or minus 2.7 pcf of the indicated line. (Conversely, 32 percent of the data points will fall outside of these limits around the respective lines). Optimum water content and maximum dry density did not correlate as well with plasticity index. Approximately 68 percent of the actual optimum water contents and

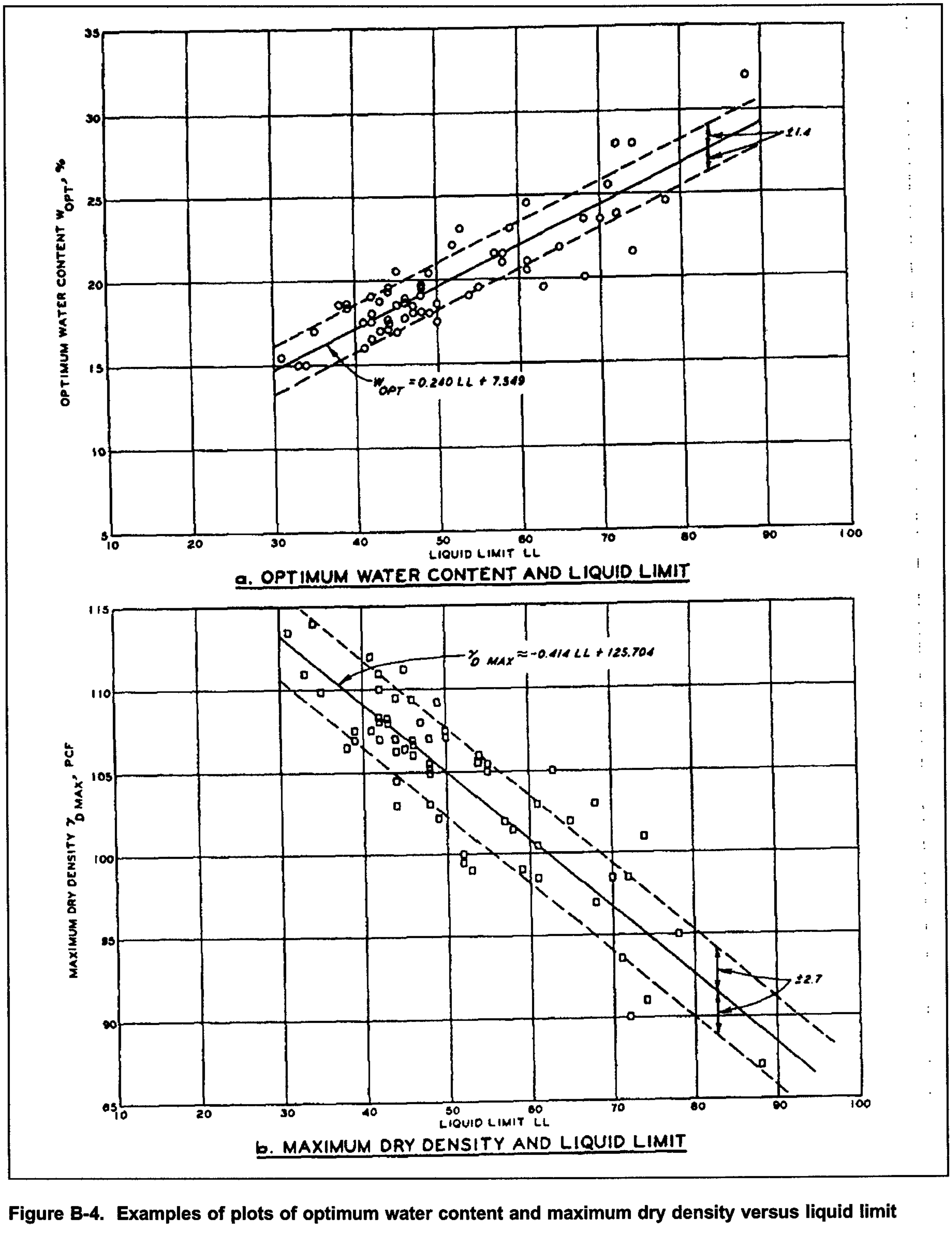


Figure B-4. Examples of plots of optimum water content and maximum dry density versus liquid limit

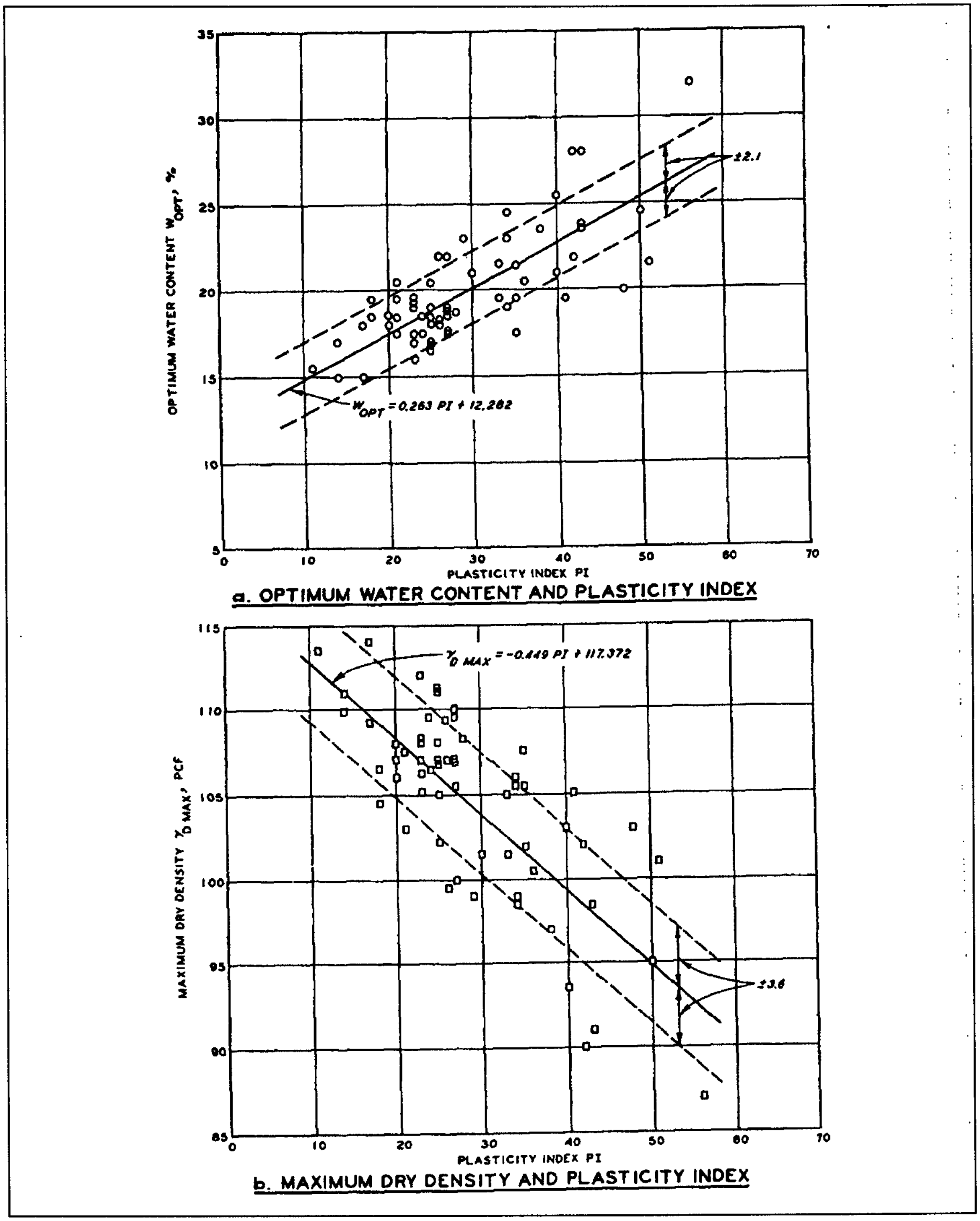


Figure B-5. Examples of plots of optimum water content and maximum dry density versus plasticity index dry densities will be within ± 2.1 percent and 3.6 pcf, respectively, of those indicated by the lines of best fit. Therefore, when this method is used, it is very important that additional five-point compaction tests and Atterberg limits tests be performed to check the correlation and to extend the correlation for new borrow material for mixtures not previously tested. The Atterberg limits correlation method includes more variables than the two-point method and thus can be less accurate, depending on how carefully the particular method is used. However, the limits correlation method has the advantage of providing the exact classification of the soil, and of providing data that can be correlated with design strength studies.

f. Analysis of Atterberg limits correlations. A discussion of Atterberg limits correlations and comparison of results with the one-point method are given in by Torrey (1970). However, additional discussion of the method is deemed appropriate here to point out mathematical weaknesses in the procedure. In order to determine a mathematical relationship between the variables of interest (that is liquid limit, plastic limit, optimum water content, maximum dry density) using the methods of statistics, it is necessary to assume a frequency distribution between the variables. It was assumed that there is a normal or Gaussian distribution between the variables. A normal distribution has a very specific mathematical definition and, although the assumption of normal distribution is reasonable, it must be pointed out that there is no insurance that the assumption is valid. Additionally, it was assumed that the relationship between the variables of interest is linear; again, there is no evidence to support such an assumption; in fact, it is very likely that there is a curvilinear relationship between the variables of interest. Analysis of the data presented in Figures B-4 and B-5. showed that the linear correlations between optimum water content and liquid limit (shown in Figure B-4a) and maximum density and liquid limit (shown in Figure B-4b) explain only 77.6 percent and 76.3 percent, respectively, of variation between the regression line and the data points. This means that unidentified mechanisms explain about one quarter of the variation between the regression line and the points. Similarly, the linear correlations between optimum water content and plasticity index (shown in Figure B-5a) and maximum dry density and plasticity index (shown in Figure B-5b) explain only 57.8 percent and 55.7 percent of the variation, respectively; about 43 percent of the observed variation is unexplained by the mathematical model chosen. In this light, the correlation between the variables appears less sound, especially considering that there is no mathematical assurance that a relationship exists between these variables; the mathematical curve-fit procedure used in the analysis ensures only that the mathematical expressions given are the best possible linear fits. The numbers defining the error bands of the regression lines of Figures B-4 and B-5 are called the standard error of the estimate. Again, if the data are normally distributed about this line, theory predicts that about 68 percent of the points lie between the (error band) lines. However, this also indicates that 32 percent (about one-third) of the points will statistically lie outside the band. For example, since the standard error between maximum dry density and liquid limit is 2.7 pcf, if maximum dry density were estimated based on a determination of liquid limit of a soil sample taken from the area, chances are about one in three that the error in maximum density would be greater than 2.7 pcf. In this light, the use of this procedure to estimate either maximum dry density or optimum water content appears to be unsound and inappropriate. The use of one- and two-point compaction test results appears to be much more sound, especially considering that the results of a one-point compaction test may be obtained in about 40 min using microwave drying techniques outlined in paragraph 5-10*d*(1)(c). Conversely, the time required to obtain the results of a liquid and/or plastic limit test may be prohibitive in a construction environment where large volume rates of earth are being placed.

g. USBR rapid compaction control method. Details of this method are described in the U.S. Bureau of Reclamation Earth Manual (1963). The test is applicable to fine-grained (100 percent minus No. 4 sieve) cohesive soils with liquid limits less than 50. The method, however, is applicable to soils containing oversize particles providing the proper corrections, as stated in Torrey (1970) or in the Earth Manual (1963), are applied. It is a faster method than the standard compaction test, and is often more accurate than other methods. The method usually requires adding water to or drying back sampled fill material, and thorough mixing is required to obtain uniform drying or distribution of added water. Otherwise, the results may be erroneous, especially for highly plastic clays. In highly plastic (and probably difficult) clays, it is likely to be inaccurate because of the lack of sufficient curing time of the specimens.

B-3. Cohesive Soils

a. Oversize particles. The term "oversize particles" as used in this work refers to those particles larger than the maximum size allowed when using a given mold (i.e., No. 4 for a 4-in. mold, 3/4-in. for a 6-in. mold, 2-in. for a 12-in. mold). The term "fine fraction" refers to that part of the soil composed of particles equal to and smaller than the maximum size allowed for a given mold. Results of field density tests made in fill material containing oversize particles must sometimes be related to results of compaction tests made on materials from which oversize particles have been scalped, if the USBR rapid compaction control method is used, or if it has not been possible to perform compaction tests using molds of sufficient size to accommodate the large particles.

b. Correction of field density test results. When the proportion of oversize material is not greater than about 35 percent, the dry density of the fine fraction can be calculated with reasonable accuracy from the following equation which associates all voids with the fine fraction:

$$\gamma_f = \frac{f\gamma_t\gamma_w G_m}{\gamma_w G_m - c\gamma_t} \tag{B-1}$$

where

γ_f = dry density of fine fraction, pcf
f = proportion of fine fraction by weight expressed as a decimal fraction
γ_t = dry density of total field sample, pcf
γ_w = unit weight of water, 62.4 pcf
G_m = bulk specific gravity of oversize particles (dry method), dimensionless
c = proportion of oversize particles by weight expressed as a decimal fraction

The water content of the fine fraction can be calculated from the following equation:

$$w_f = \frac{w_t - cw_c}{f} \times 100 \tag{B-2}$$

where

w_f = water content of fine fraction, percent
w_t = water content of the total field density sample, expressed as a decimal fraction
w_c = water content of oversize fraction, expressed as a decimal fraction

At the beginning of construction, charts can be prepared for materials having oversize particles relating dry densities and water contents of total samples to dry densities and water contents of fine fractions (Figures B-6a and B-6b) if it is desired to use the original Ziegler equation. Different charts are required for materials having oversize particles with significantly different bulk specific gravity and/or absorption values. In field density testing, the appropriate chart is entered using the percent oversize particles and the water content or dry density determined on the total sample to obtain the water content, dry density, and wet density of the fine fraction. Corrections for oversize particles will be subject to large errors if the percent of oversize particles is greater than about 35; in such cases, compaction control should be based on laboratory compaction tests performed in molds of appropriate sizes.

c. Modified Ziegler equation to estimate maximum density. A procedure to compute dry density of earth-rock mixtures has been determined as an extension of the Ziegler procedure and is discussed by Torrey and Donaghe (1991); the procedure is a modification of the development which resulted in Equation B-1. The modified equation accounts for the actual percent compaction of the gravel fraction when the total material (gravel and fines) is at its maximum density. This is done by incorporating the effect of a factor called the density interference coefficient, which is defined as

$$I_c = \frac{R_c}{P_g G_m} \tag{B-3}$$

where

R_c = decimal fraction of the percent compaction of the minus No. 4 or -3/4-in. fraction
P_g = decimal fraction of percent gravel in the total material
G_m = bulk specific gravity of the gravel

To determine the maximum dry density corresponding to the gradation of the total fill sample, use the equation

$$\gamma_{tmax} = \frac{P_g I_c \gamma_{fmax} \gamma_w G_m}{f\gamma_w + P_g c I_c \gamma_{fmax}} \tag{B-4}$$

where

γ_{tmax} = maximum dry density of the gradation of the total fill
γ_{fmax} = maximum dry density of the finer fraction determined at its optimum water content, W_{fopt}, by the one- or two-point compaction method

It should be noted that γ_{fmax} may be determined based on gravel content defined as either the -3/4-in. or the minus No. 4 sieve fraction of the total material to be placed in the fill. However, it is more efficient to use the minus No. 4 fraction because percent oversize particles (c) and percent gravel in the total material (P_g) are the same number. This will eliminate an extra sieving operation which would be required if γ_{fmax} and W_{fopt} are used for the -3/4-in. fraction, since both the percent oversize (+3/4-in. material) and the percent gravel in the total material would have to be determined. To facilitate its numerical evaluation, Equation B-4 may be rewritten in the form

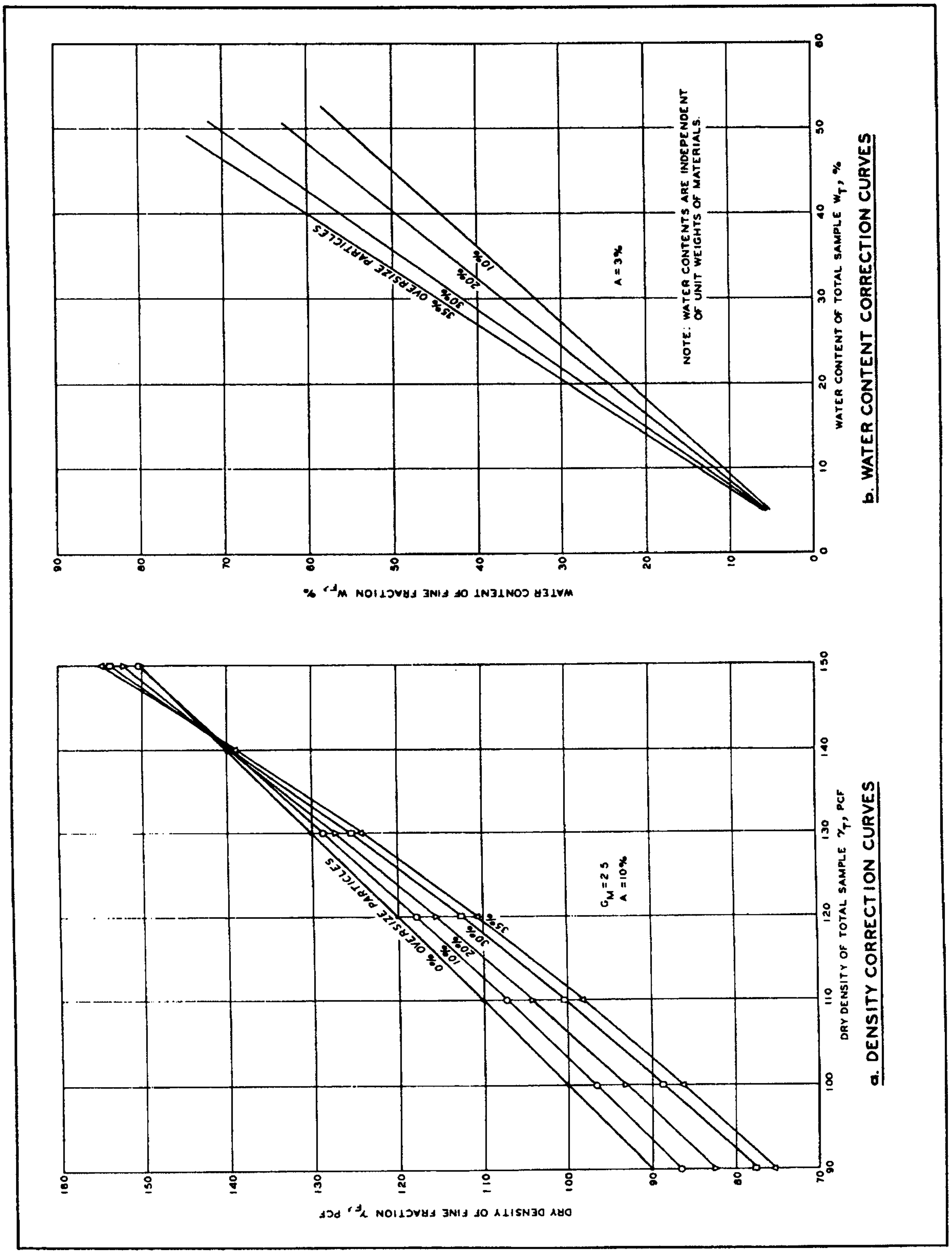


Figure B-6. Density and water content correction curves

$$\gamma_{tmax} = \frac{P_g I_c \gamma_w G_m}{f\frac{\gamma_w}{\gamma_{fmax}} + P_g^2 I_c} \qquad \text{(B-5)}$$

which consists of three groups of terms. Figures B-7 and B-8 were prepared from data presented by Torrey and Donaghe (1991) and allow numerical evaluation of the groups of terms $P_g I_c \gamma_w G_m$ and $P_g^2 I_c$, respectively, in terms of the percent gravel in the total material. It should be noted that the term c in Equation B-4 is the decimal value of percent oversize particles by weight, and is equal to P_g if I_c is based on the minus No. 4 fraction. The value of bulk specific gravity used to determine the relationship presented in Figure B-7 is 2.5. The remaining group of terms in Equation B-5 is $f\gamma_w/\gamma_{fmax}$, and is easily evaluated. Accordingly, the maximum dry density of the fill containing up to 70 percent gravel may be determined from Equation B-5.

As an extension of Equation B-2, the optimum water content of the total material is given as

$$w_{topt} \approx f w_{fopt} + c w_c \qquad \text{(B-6)}$$

where

w_{fopt} = optimum water content of the fine fraction

The optimum water content of the fine fraction, w_{fopt}, can be directly related to that of the total material, w_{topt}, and the gravel content of the total material, P_g, by an optimum water content factor, F_{opt}, defined as

$$F_{opt} = \frac{\frac{w_{fopt}}{w_{topt}}}{P_g} \qquad \text{(B-7)}$$

When the optimum water content factor, F_{opt}, versus gravel content in the total material is plotted in log-log coordinates, the relationship is linear over a significant range of gravel content, up to more than 60 percent gravel content for some gravels. However, it appears necessary to demonstrate linearity above gravel contents of 50 percent, since some data examined deviated from linearity above about 50 percent gravel content. Linearity of the water content factor, F_{opt}, versus gravel content in the total material in log-log coordinates may be used to establish the total material curve without testing the total material, which would require large-scale compaction equipment. This may be achieved if the -3/4-in. fraction of the total material contains a sufficient range in gravel content to base the water content factor, F_{opt}, on the minus No. 4 fraction, while treating the -3/4-in. fraction as a total material.

B-4. Cohesionless Soils

Gradation tests are performed on sands and gravels used in pervious zones to determine compliance with specifications, and field density tests are performed and compared with laboratory relative density tests to ensure that in-place densities are adequate. Gradation tests are required on compacted filter layer samples to ensure specification compliance after compaction.

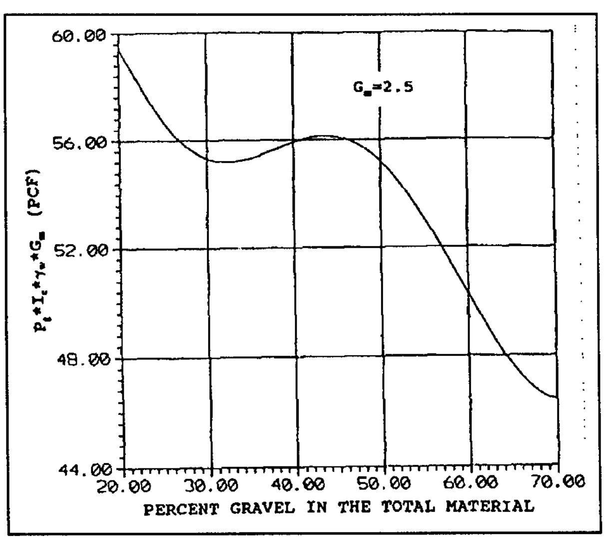


Figure B-7. Relationship between gravel content and parameters in the numerator of Equation B-5

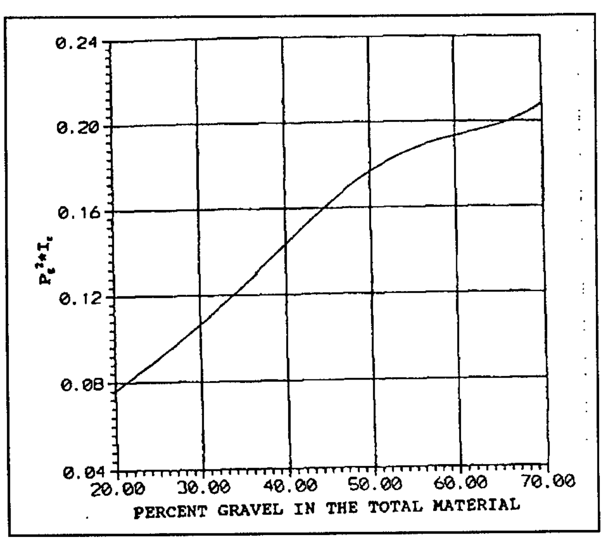


Figure B-8. Relationship between gravel content and parameters in the denominator of Equation B-5

a. Control using relative density. Where materials available for cohesionless fill vary significantly in gradation, maximum-minimum density tests should be performed on material from each field density test at least in the initial construction stages. Where cohesionless materials can be grouped into categories with relatively constant gradations, relative density tests and gradation tests can be performed on each different material. Gradation tests on material from field density tests can then make it possible to match field densities with appropriate relative density test results. However, it is necessary to point out that relative density is computed from maximum and minimum densities determined on the material in question, using the procedure outlined in EM 1110-2-1906. It was concluded by ASTM (1973) that the maximum density of cohesionless materials as determined on the "vibratory table" (as described in EM 1110-2-1906) is subject to considerable uncertainty. Further, the conclusions are that vibratory tables cannot, in general, be successfully calibrated for repeatable energy application to the soil specimen, large local density variations exist throughout the vibrated soil specimen, and density results obtained with the vibratory table are generally not repeatable from laboratory to laboratory. Therefore, control of the gradation and density of cohesionless fill using the method of relative density may be unacceptable, especially if the procedure involves coordinated effort and testing between two laboratories. An example is given by ASTM (1973) in which the standard deviation in maximum density of one sand tested by 14 laboratories is greater than 6 pcf. It is specified in EM 1110-2-1906 that minimum density tests be repeated until densities from two successive runs agree within ±1 percent. Maximum density is then obtained by placing a minimum density specimen on the vibratory table; only one maximum density test is required. Variation and uncertainty in laboratory-measured values of maximum density can cause serious problems in the construction of cohesionless fill and graded filters. Basic laboratory research is needed to resolve difficulties with the shaking table test for maximum density. Until research is performed and the source of uncertainty identified and resolved, particular care and caution should be used in determining maximum density. One method of minimizing uncertainty is to perform several maximum density tests to determine and ensure that large variations in maximum density are not being observed. A control criterion for maximum density specimens similar to that for minimum density specimens may be used—that is, agreement between two successive specimens within ±1 percent.

b. Alternative maximum density procedure. In light of the difficulty of obtaining duplicate results of maximum density on the vibratory table, consideration must be given to eliminating the test. A possible alternative procedure for maximum density determination is the Modified Providence Vibrated Density Test as described in EM 1110-2-1906. In this test, a sample of oven-dried soil is placed in a heavy steel mold, compressed under a surcharge, and vibrated to a maximum density by repeatedly striking the side of the mold with a hammer. Research presented by Tokue (1976) suggests that the level of shear strain, not acceleration, is directly related to densification of cohesionless soil. Many of the unknown uncertainties associated with the vibratory table may be avoided by use of this relatively simple procedure.

c. Materials with +3-in. particles. Relative density tests described in EM 1110-2-1906 are performed on cohesionless soils with particle sizes not greater than 3 in. If cohesionless soils contain a large amount of +3-in. material, large-scale field density tests would be needed for comparison with results of field density tests performed during construction of test fills to develop adequate compaction procedures. When no field density test results are available, control is achieved by careful inspection to ensure that the specified gradation is being met and that the specified compaction procedures are followed. Visual inspection of the sides of a test pit dug in the compacted fill can provide qualitative indications of the denseness of the material and of the existence of any significant voids.

Appendix C
Field Compaction Control Forms and Supplemental Instructions

C-1. General

ENG Form 4080 (Figure C-1) is for use where water content control is required to obtain adequate compaction; the title of this form is "Summary of Field Compaction Control of Impervious or Semipervious Soils for Civil Works Projects." ENG Form 4081 (Figure C-2) is for use when water content control (other than complete saturation) is not required; the title of this form is "Summary of Field Compaction Control of Pervious Soils and Rockfill for Civil Works Projects." Use of these forms is described in ER 1110-2-1925.

a. A database package utilizing the commercially available software routine dBase III Plus (Trademark of Ashton-Tate) was developed around the information required for ENG Forms 4080 and 4081; the system uses the microcomputer to analyze data for use in the quality assurance (QA) program. Specially prepared Computer Applications in Geotechnical Engineering (CAGE) software interacts with the database to reduce data, perform statistical analysis, and generate ENG Forms 4080 and 4081 with the information required for reporting. Data from field notes is entered into a microcomputer in a dBase III-driven format; for data manipulation, the system is menu driven and user interactive. CAGE is designed to store, retrieve, and display earthwork construction control data as well as provide summaries of required construction parameters. Hard copies of any of the summaries, reports, and graphs generated may be printed by the system along with computer-generated copies of ENG Forms 4080 and 4081 if they are required in hard copy form. The use and operation of the CAGE system is described by Edris, Strohm, and Woo (1991).

b. If construction control data are recorded manually on ENG Forms 4080 and 4081, information at the top of each form could be placed on a master sheet from which reproducible copies could be made for recording data and for making subsequent copies for submission.

c. Explanation of any abbreviations used which are not explained in the forms, should be furnished with the first report submitted on a project.

d. Lower lines of the forms may be used for necessary remarks.

C-2. Additional Information

Information on items below, as appropriate, should be submitted with the initial reports and also whenever changes are made.

a. Borrow sources and operations.

(1) Description of borrow materials (each borrow or excavation area).

(2) Natural water content.

(3) Method of adding water in pit.

(4) Method of reducing water content in pit.

(5) Method of excavating and mixing (describe equipment used).

(6) Equipment used for loading and transporting material.

b. Compaction equipment. Describe in detail.

(1) Sheepsfoot roller.

(a) Make, model, and whether self-propelled or towed.

(b) State size (diameter and length) and number of drums.

(c) Describe tamping feet: number of rows, feet per row, and total number of feet per drum; length, shape and base area of foot.

(d) Give weight of roller empty, weight as used, type of ballast, and unit pressure (weight of roller divided by total contact areas of tamping feet).

(e) Specify type of frame (rigid or oscillating), speed of travel during compaction, and if cleaners are used.

(2) Rubber-tired roller.

(a) Make and model.

(b) Number of boxes or sections, rolling width, overall width and length.

(c) Number of tires, tire size, ply rating and spacing.

MAIN FORM

RCS: ENGCW-E-11(R1)

(1) Daily SUMMARY OF FIELD COMPACTION CONTROL OF IMPERVIOUS OR SEMIPERVIOUS SOILS FOR CIVIL WORKS PROJECTS

PROJECT	RIVER / STATE / TOWN	CONTRACT NO.	CONTRACTOR:	DATE OF REPORT:	REPORT NO.: 73
Main Embankment Closure Section Dam	(nearest)	(Fill in if desired for district or division use)		(date transmitted from field)	SHEET 1 OF 1

SPACE RESERVED FOR PUNCH CARD OPERATOR

1-2 ____ 3-4 ____ 5- ____ 6-7 ____ 8-9 ____ 10-11 ____

EMBANKMENT ZONE	(S) SPECIFIED OR (D) Desired % COMP	w % LIMITS	COMPACTION EQUIPMENT (3)	LOOSE LIFT THICKNESS (IN.)	NUMBER OF PASSES	CONTACT PRESSURE (PSI)
Upstream	95 D	+2 to -3 S	SFR	8	8	517
Central	95 D	+2 to -2 S	SFR	8	8	517
Downstream	95 D	+2 to -3 S	SFR	8	8	517

TEST NO (COL 1)	DATE MADE (COL 2)	TYPE TEST (COL 3)	STATION (COL 4)	OFFSET (FT) (COL 5)	ELEVATION (FT) (COL 6)	DEPTH (IN.) (COL 7)	EMBANKMENT ZONE (COL 8)	BORROW SOURCE (COL 9)	CLASS WORD OR LETTER SYMBOL (COL 10)	Gradation: MAX PART SIZE (IN.) (COL 11)	Gradation: % PASSING #4 SIEVE (COL 12)	Gradation: % PASSING #200 SIEVE (COL 13)	Atterberg Limits: LL (COL 14)	Atterberg Limits: PI (NOT PL) (COL 15)	In-Place Total Sample: DRY DENSITY (PCF) (COL 16)	In-Place Total Sample: WATER CONTENT (%) (COL 17)	In-Place -#4 Fraction: DRY DENSITY (PCF) (COL 18)	In-Place -#4 Fraction: WATER CONTENT (%) (COL 19)	Lab Standard Method: TEST (COL 20)	Lab Standard Method: MAX DRY DENSITY (PCF) (COL 21)	Lab Standard Method: OPT WATER CONTENT (%) (COL 22)	Liquid Limit Correlation: MAX DRY DENSITY (PCF) (COL 23)	Liquid Limit Correlation: OPT WATER CONTENT (%) (COL 24)	Field and Lab Correlation: w' % + OR - (COL 25)	Field and Lab Correlation: PERCENT COMPACTION (COL 26)	COMMENTS ON TESTS (COL 27)
488	1965 4 Mar	SV	147+50	480 DS	93	*	DS	*	SP	*	*	7	NP	NP	-	-	114.8	9.8	-	-	-	112.7	12.9	-3.1	102	
489	4 Mar	CYL	149+30	20 DS	104	*	Central	*	CH	*	*	62	52	32	-	-	110.0	16.5	-	-	-	107.2	17.1	-0.6	103	
490	4 Mar	CYL	149+00	175 US	105	*	US	*	CL	*	*	44	37	25	-	-	111.9	14.1	-	-	-	116.0	12.9	+1.2	96.5	
								Following data from					Dam,				District					Visual Class				
545	1963 9 Jul	NM	10+05	110 US	1375	*	A	Spillway Excav	CL(E)	*	*	*	*	*	113.0	*	-	-	-	-	-	113.5	*	*	98.8	Check test on NM
3	9 Jul	WV	10+05	110 US	1375	*	A	Spillway Excav	CL(E)	*	*	*	*	*	110.7	*	-	-	-	-	-	113.5	*	*	97.7	
								Following data from					Dam,				District					1 Pt. Std.				
CRF-7 DM D-30	1963 6 Sep	SV	7+25	85 DS	797	*	Random	Excav	SM	*	*	*	*	*	136.7	7.2	121.5	11.5	-	-	-	124.8	10.2	-3.0	97.3B	
CRF-8 DM D-48	9 Sep	SV	8+00	90 US	826	*	Impervious	Borrow	SM	*	*	*	*	*	126.2	9.6	118.9	11.9	-	-	-	126.3	10.2	-0.6	94.1B	
								Following data from					Dam				District									
404	1964 19 Ag	CYL	30+00	8 DS	705	*	Impervious Core	*	MH	*	68	27	75	34	82.3	34.6	68.4	44.4	(A)	84.0	37.0			+7.4	81	
707	21 Sep	CYL	20+00	℄	801	*	Impervious Core	*	MH	*	76	37	74	32	91.1	27.4	81.0	34.9	(A)	81.0	*			*	100	

* Information not given in field report but should have been.

ATTN: Compaction procedure using 12-in. mold was added to EM 1110-2-1906 after note on COL 20 (see below) was prepared.

NOTE (1) FILL IN APPROPRIATE WORD DAILY, WEEKLY, MONTHLY, OR PERIODIC
(3) ENTER TYPE OF EQUIPMENT USED, E.G. SHEEPSFOOT ROLLER (SFR) PNEUMATIC ROLLER (PR) VIBRATORY ROLLER (VR) PNEUMATIC HAND TAMPER (PHT): OR VIBRATORY TAMPER (VT)

COL 3 CYLINDER (CYL) CHUNK (CK) SAND VOLUME (SV) WATER VOLUME (WV) NUCLEAR METHOD (NM) OTHER ______ (__)

COL 5 RECORD OFFSET BY DISTANCE UPSTREAM OR DOWNSTREAM OF CENTERLINE OF DAM OR RIGHT OR LEFT OF AXIS

COL 7 DEPTH FROM FILL SURFACE TO TOP OF DENSITY TEST

COL 8 E.G. AREA C, SPILLWAY EXCAVATION, ETC

COL 10 ENTER SOIL COLOR BEFORE CLASS. WHEN CLASS ESTIMATED NOTE BY LETTER (E)

COL 11-13 WHEN BOTH GRADATION AND WATER CONTENT CONTROL OF THE FILL MATERIAL IS A SPECIFICATION REQUIREMENT, USE THE SUPPLEMENTAL FORM ALONG WITH THIS FORM, USING THE SAME SHEET NUMBER FOR THE SUPPLEMENTAL SHEET FOLLOWED BY AN S

COL 20 STANDARD METHOD (A) 4-IN.-DIAM MOLD, (B) 6-IN.-DIAM MOLD AS OUTLINED IN MANUAL 1110-2-1906. IF MATERIAL WITH MAX PARTICLE SIZE GREATER THAN 3/4 IN. OR OTHER EFFORT IS USED IN THE LAB COMPACTION TEST, NOTE AS NS UNDER COL 20 AND REPORT DETAILS OF PROCEDURE

COL 21-22 1- OR 2-POINT COMPACTION DATA ON FIELD SAMPLES CAN BE RECORDED IN COL 21 AND 22, MAKING APPROPRIATE CHANGES IN COLUMN HEADINGS

COL 23-24 FOR USE WHEN MAX DENSITY AND OPTIMUM WATER CONTENT ESTIMATED. STATE HOW ESTIMATED BY FILLING IN BLANK, E.G. 1-POINT STANDARD METHOD, RAPID COMPACTION CONTROL; LIQUID LIMIT CORRELATION, VISUAL COMPARISON, ETC

COL 25 COL 17 OR COL 19 MINUS COL 22 OR COL 24.

COL 26 INDICATE DERIVATION, E.G. FOR COL 16 ÷ COL 21 OR COL 23, INDICATE BY A OR B, RESPECTIVELY; FOR COL 18 ÷ COL 21 OR COL 23, INDICATE BY C OR D, RESPECTIVELY

COL 27 IF TEST RESULTS DO NOT MEET SPECIFICATIONS, DENOTE BY LETTER (U) AND NOTE REMEDIAL TREATMENT IN FOOTNOTE. WHEN UNSATISFACTORY AREA IS REWORKED AND RETESTED, DENOTE BY (R)

U.S. ARMY ENGINEER District, ______
CORPS OF ENGINEERS

PREPARED BY (Inspector)

SUBMITTED BY
RESIDENT ENGINEER

ENG FORM 4080
JUN 67

Figure C-1. Specimen sheet showing data actually reported from several projects

(a) Daily SUMMARY OF FIELD COMPACTION CONTROL OF PERVIOUS SOILS AND ROCKFILL FOR CIVIL WORKS PROJECTS — RCS: ENGCW-E-11(R1)

PROJECT	RIVER / STATE / TOWN	CONTRACT NO.	CONTRACTOR	DATE OF REPORT	REPORT NO
Main Embankment Dam	(nearest)	(Optional entry if desired	for district or division use)	(date transmitted from field)	96 — SHEET 1 OF 1

SPACE RESERVED FOR PUNCH CARD OPERATOR — 1 — CARD NO. 1 AND CARD NO. 2

EMBANKMENT ZONE	SPECIFIED (%) COMP REL DEN	COMPACTION EQUIP (b)	LOOSE LIFT THICKNESS (IN)	NO. OF PASSES	WEIGHT (LB) OR CONT PRESS (PSI)
Pervious	(Data on	PR	8 Dam)	3	50 T

EMBANKMENT ZONE	ROCK DESCRIPTION TYPE	COLOR	HARDNESS	% WEATHERED
	(Data on		Dam)	
Class II 18"	Slabby Basalt	Gray		
Class I 18"	Basalt	Black		

TEST NO (COL 1)	DATE MADE (COL 2)	TYPE TEST (COL 3)	STATION (COL 4)	OFFSET (FT) (COL 5)	ELEVATION (FT) (COL 6)	DEPTH (IN) OR VOLUME (CF) (COL 7)	EMBANKMENT ZONE (COL 8)	SOURCE (COL 9)	CLASS LETTER SYMB (COL 10)	MAX PART SIZE (IN) (COL 11)	Total sample 3" (COL 12)	1-1/2 (COL 13)	3/4 (COL 14)	4 (COL 15)	10 (COL 16)	40 (COL 17)	100 (COL 18)	200 (COL 19)	-3" fraction 3/4" (COL 20)	#4 (COL 21)	-#4 fraction 40 (COL 22)	#200 (COL 23)	UNIF COEF C_u D_{60}/D_{10} (COL 24)	APPARENT SPECIFIC GRAVITY +#4 (COL 25)	-#4 (COL 26)	DRY DENSITY FIELD (PCF) (COL 27)	LABORATORY TEST METHOD (COL 28)	MAXIMUM (PCF) (COL 29)	MINIMUM (PCF) (COL 30)	% COMP OR REL DEN (COL 31)	COMMENTS ON TESTS (COL 32)
CF-10 D-D361	1 Oct	WV	10+70	10 US	338	*	Pervious	Borrow C	GP	3	100	92.9	77.4	44.2	33.6	25.1	-	0.7	77.5	44.2	56.9	1.6	82.5	*	*	135.8	PVD	138.8	-	D_M 97.8	
CF-15 D-D364	1 Oct	WV	15+00	25 DS	343	*	Pervious	Borrow C	GP	3	100	96.6	67.1	40.8	31.5	14.0	-	0.6	67.1	40.8	34.4	1.4	46.7	*	*	134.3	PVD	139.8	-	D_M 96.1	
								Rock-fill data from					Dam,						District												
											Total Sample 12"	6	3	1	4	10	40	200	-6" Fraction 1"	4											
2026	1962 18 Apr	DT	21+00	240 US	1400	*	Class II 18"	Spillway	-	18	100	89	64	34	16	10	6.4	3.7	44	23.4	-	-	3.4	*	-	112.1	V	102.7	94.4	D_M 109	
2176	25 Jun	DT	21+20	100 DS	1461	*	Class I 18"	Helga Burgeple	-	18	100	89	68	22	2.9	1.7	0.7	0.2	25	3.3	-	-	4.7	*	-	118.6	V	114.1	100.8	D_M 104	

* Information not given in field report but should have been.

Remarks: Col 3 - Density Test (DT) performed using plastic membrane or tarpaulin and water.
Col 28 - V - laboratory test using 3/4 cu yd vibrating apparatus (job-built).

NOTE (a) FILL IN APPROPRIATE WORD: DAILY, WEEKLY, MONTHLY, OR PERIODIC
(b) ENTER TYPE OF EQUIPMENT USED, E.G. PNEUMATIC ROLLER (PR), VIBRATORY ROLLER (VR), CRAWLER TRACTOR (CT), RUBBER TIRED TRACTOR (RTT), STEEL TAMPING PLATE (STP), OTHER ________
COL 3 FOR GRAVEL OR SAND FILL: WATER VOLUME (WV), SAND VOLUME (SV), NUCLEAR METHOD (NM), GRADATION SAMPLE (GS), OTHER ________. FOR ROCKFILL: GRADATION SAMPLE (GS), DENSITY TEST (DT) ________
COL 5 RECORD OFFSET BY DISTANCE UPSTREAM OR DOWNSTREAM OF CENTERLINE OF DAM OR RIGHT OR LEFT OF AXIS
COL 7 GRAVEL OR SAND FILL: SHOW DEPTH BELOW COMPACTED SURFACE TO TOP OF DENSITY TEST (E.G. "60"); ROCKFILL: SHOW VOLUME OF DENSITY TEST HOLE, E.G. "215 CU FT"
COL 9 E.G. AREA C, SPILLWAY EXCAVATION, ETC.
COL 10 WHEN CLASSIFICATION ESTIMATED NOTE BY LETTER (E)
COL 27 IF ESTIMATED SO NOTE AS 110-E AND STATE HOW ESTIMATED IN FOOTNOTE
COL 28 VIBRATORY TABLE (VT), PROVIDENCE VIBRATED DENSITY (PVD), OTHER ________. REPORT SEPARATELY DETAILS OF SPECIAL LABORATORY PROCEDURES USED
COL 31 INDICATE HOW DERIVED, E.G. PERCENT COMPACTION (D_M) OR RELATIVE DENSITY (D_R) BY PLACING APPROPRIATE SYMBOL (D_M OR D_R) OVER VALUE OBTAINED
COL 32 IF TEST RESULTS DO NOT MEET SPECIFICATIONS DENOTE BY LETTER (U) AND NOTE REMEDIAL TREATMENT IN FOOTNOTE. WHEN UNSATISFACTORY AREA REWORKED AND RETESTED DENOTE BY LETTER (R)

U.S. ARMY ENGINEER District, ________ CORPS OF ENGINEERS
PREPARED BY (Inspector)
SUBMITTED BY ________ RESIDENT ENGINEER

ENG FORM 4081, JUN 67

Figure C-2. Specimen sheet showing both pervious fill and rock-fill data actually reported from two districts

(d) Weight empty and as used, type of ballast, tire pressure, and load per tire as used.

(e) Speed of travel during compaction.

(3) Vibratory roller.

(a) Make and model.

(b) State size (diameter and length) and number of drums.

(c) Give weight of roller empty, static weight per roller used, dynamic ground pressure exerted, type of ballast, and vibrating frequency.

(d) Speed of travel during compaction.

c. Embankment operations.

(1) Type of equipment used in spreading and mixing the material.

(2) Method of removing oversize rock fragments.

(3) Method of adding water on the fill.

(4) Method of reducing water content of the fill.

d. Compaction control methods.

(1) For impervious or semipervious fill: Describe the methods used to determine in-place density and water content. Also report method of correcting for oversize particles and for correlating field density and water content for material with oversize particles with laboratory density and water content. Submit a copy of any reference curves used for correlating the field data with the laboratory data.

(2) For pervious fill or rock fill: Describe in detail methods used in determining laboratory maximum and minimum densities (if different from those specified in EM 1110-2-1906) and in determining field densities of pervious soils and rock fill. Also include details of methods used for correlating field and laboratory densities in determining percent compaction or relative density and details of methods used in correcting for oversize particles.

Appendix D
Instructions for Preparing Periodic Summaries of Field Compaction Control Data on Earth and Rock-Fill Dams

D-1. Compaction Control Data Summary Forms

Summaries of compaction control data are prepared at least monthly, using tabular summary form, ENG Form 4287, and one or both of two summary plots: ENG Form 4287A for soils requiring control of both water content and density and ENG Form 4287B for soils requiring only density control.

D-2. Separate Summary Forms and Plots

Separate summary forms and plots should be prepared for (a) significantly different materials (impervious, random, pervious, etc.) used in different zones of the embankment and (b) materials compacted by different equipment (e.g., impervious fill compacted by towed rollers and impervious backfill compacted by hand-operated power tampers).

D-3. Example Summary Forms and Plots

Examples of prepared summary forms and plots are shown in Figures D-1 through D-4. Examples of appropriate entries for tabular summaries are given in Table D-1.

D-4. Summary Plot for Materials Requiring Water Content and Density Control

A summary plot for materials requiring water content and density control is illustrated in Figure D-2. Two vertical lines are first drawn on the plot to show the limiting values of water content in percentage points wet or dry of standard optimum. A horizontal line is drawn to show the desired or specified minimum percent of maximum standard and dry density. The top margin and right side margin of the plot are marked to show the limiting values illustrated in Figure D-2. The data are then plotted using symbols shown on the legend. Should an area be reworked more than once or reworked and tested more than once, only the last test result or last set of test results should be plotted. The test results are summarized in the tabulation form on the right side of the plot in Figure D-2. Total number of tests is the total number of plotted data points excluding retests and check tests. Check tests should not be included in the number retested.

D-5. Summary Plot for Materials Requiring Only Density Control

Use of the summary plot for material requiring only density control is illustrated in Figure D-4. Inappropriate labels at the top and bottom of the plot are lined out. If control is based on maximum density determined using a vibratory procedure, "STD" should also be lined out. Suitable scales are added to the plot, and a vertical line is drawn to indicate the minimum value of relative density, minimum percent of maximum standard dry density, or minimum percent of maximum dry density by a vibratory procedure, whichever applies.

PERIODIC SUMMARY OF FIELD COMPACTION CONTROL DATA

Project Dam — Resident Engr S. J. Smith
District — Insp. or Tech J. S. Jones
Location of Project (River) (Nearest town, state)
Report No. 12 Period 5 Nov 68 to 5 Dec 68

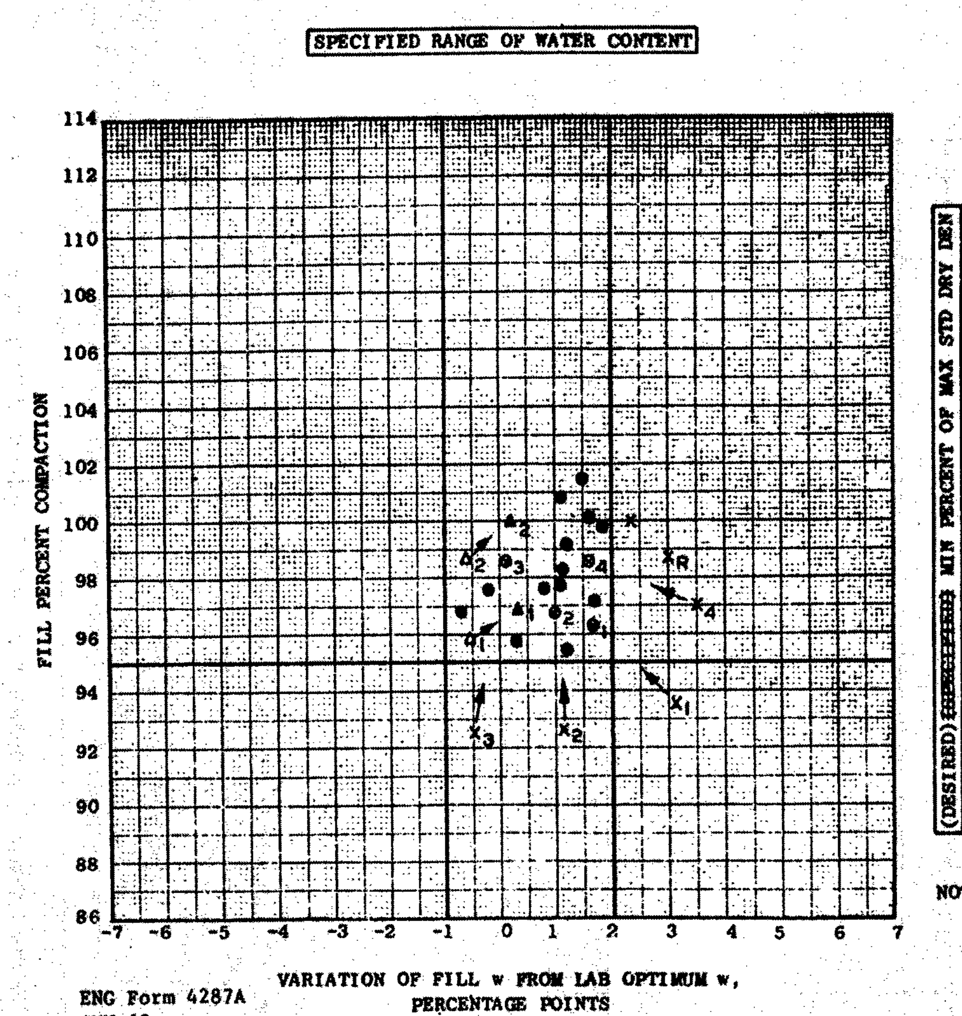
TYPE OF FILL	IMPERVIOUS (CORE)
Soil Classification (USCS Symbols)	CH, CL
Stationing of Areas Tested	14+75 to 46+50
Elevation of Areas Tested	832 to 840
Compaction Equipment	Sheepsfoot roller, Ferguson self-propelled model SP-120B (615 psi)
Number of Passes	8
Uncomp. Lift Thick.	8 in.
Roller Speed, MPH	3 to 5
In-Place Density Method (Give % of tests made with each method)	Sand Volume (90%) Nuclear (10%)*
Method of Determining Field w	Oven Drying
Method of Relating Field w to Std Opt w, and Field Density to Max. Dry Density, or Relative Density	Field results compared to laboratory compaction curve for similar soil. Appropriate laboratory curve selected by 1- or 2-point Std compaction test at field w or drier, supplemented with liquid limit test correlation
Specified Range of w (Percentage Points Above & Below Std Opt w)	Opt -1 % to Opt +2 %
(Desired) (Specified) Min. (% Comp. or Rel. Density)**	95%
No. Areas Tested	21
No. with w Outside Acceptable Limits	3
No. with Density Below Min.	2
No. with w and Density Outside Acceptable Limits	1
No. Areas Reworked	5
No. Areas Retested	4

Remarks

* Only the two "initial" tests shown on the summary plot were by nuclear method; check tests and remainder of tests were by sand volume method.

**Strike out inapplicable words. Summary Prepared by ARG Date 6 Dec 68
ENG Form 4287 Summary Checked by JSJ Date 7 Dec 68
JUN 69 (ER 1110-2-1925)

Figure D-1. Example of a prepared summary form, field compaction control data, impervious (core)



ENG Form 4287A
JUN 69

PROJECT: ______________ Dam
DISTRICT: ______________
REPORT PERIOD: 5 Nov - 5 Dec 68
REPORT NO.: 12
TYPE OF MATERIAL: IMPERVIOUS (CORE)

	CUMULATIVE TO THIS REPORT	THIS REPORT
TOTAL NO. OF TESTS	270	21
NO. OUTSIDE LIMITS:		
TOTAL	57	6
w	42	3
DENSITY	12	2
w & DENSITY	3	1
NO. REWORKED	50	5
NO. RETESTED AFTER REWORKING	44	4

LEGEND:

- ● WITHIN ACCEPTABLE LIMITS
- x OUTSIDE ACCEPTABLE LIMITS
- x_R LATER REWORKED ONLY (NO RETEST)
- x_1, x_2 LATER REWORKED AND RETESTED
- $●_1$, $●_2$ RESULT OF RETEST AFTER REWORKING
- Δ_1, Δ_2 INITIAL TEST (USE ONLY w/CHECK TEST)
- $▲_1$, $▲_2$ CHECK TEST

NOTE: USE SAME NUMBER FOR LATER REWORKED AND RETESTED AND RESULT OF RETEST AND FOR INITIAL AND CHECK TEST. INDICATE PAIRS BY SHORT ARROWS, e.g., $x_1 \rightarrow ●_1$, $\Delta_1 \rightarrow ▲_1$

(ER 1110-2-1925)

Figure D-2. Example of a prepared summary plot for materials requiring water content and density control

PERIODIC SUMMARY OF FIELD COMPACTION CONTROL DATA

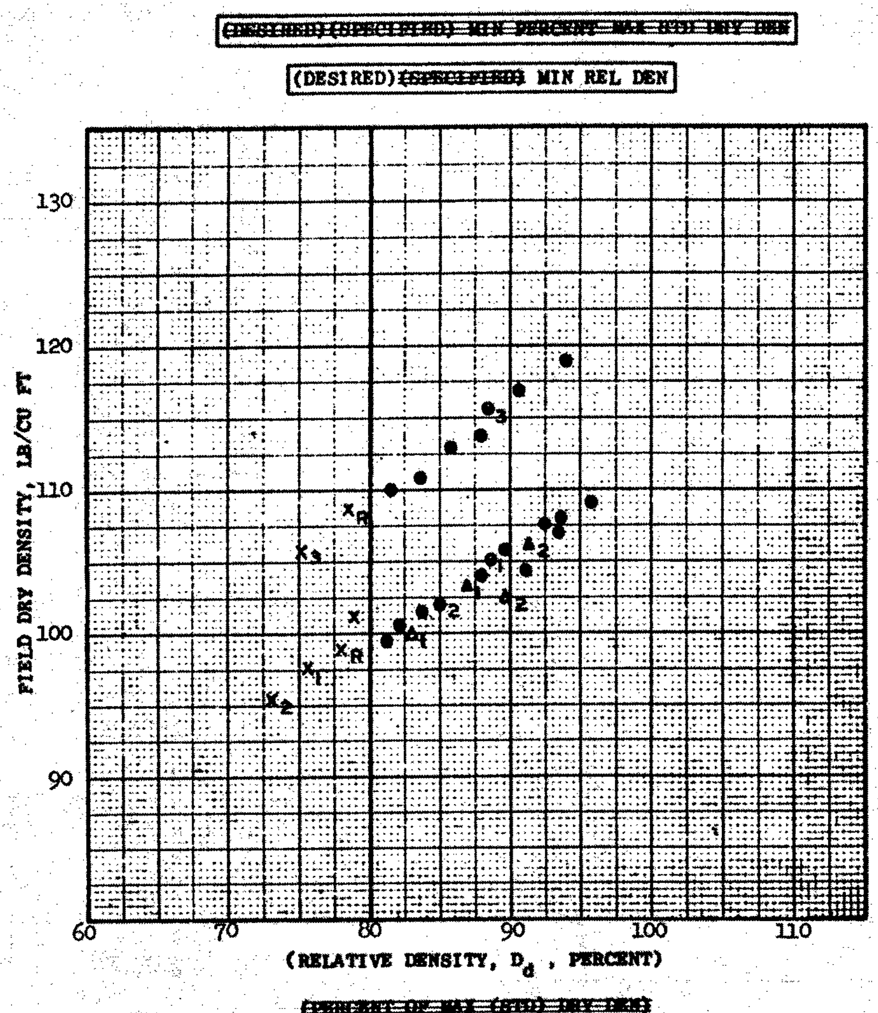
Project Dam Resident Engr J. S. Smith
District Insp. or Tech J. S. Jones
Location of Project (River) (nearest town, state)
Report No. 12 Period 5 Nov 68 to 5 Dec 68

TYPE OF FILL	PERVIOUS (SAND DRAIN)
Soil Classification (USCS Symbols)	SW
Stationing of Areas Tested	15+50 to 37+50
Elevation of Areas Tested	830 to 839
Compaction Equipment *	Vibratory Roller, Tampo Model VC80 (static wt. = 3.5 tons, centrifugal force of 7.5 tons at 1600 rpm)
Number of Passes *	4
Uncomp. Lift Thick.	6 in.
Roller Speed, MPH	2
In-Place Density Method (Give % of tests made with each method)	Sand Volume (90%) Nuclear (10%)**
Method of Determining Field w	Visual Observation
Method of Relating Field w to Std Opt w, and Field Density to Max. Dry Density, or Relative Density	Field results compared to results of laboratory maximum (modified Providence vibrated) and minimum density tests on similar material. Appropriate laboratory results selected by gradation correlation.
Specified Range of w (Percentage Points Above & Below Std Opt w)	Saturated during compaction
(Desired) (Specified) Min. (% Comp. or Rel. Density)+	80%
No. Areas Tested	25
No. with w Outside Acceptable Limits	Not Applicable
No. with Density Below Min.	6
No. with w and Density Outside Acceptable Limits	Not applicable
No. Areas Reworked	5
No. Areas Retested	3

Remarks
* If compacted by crawler tractor, change "passes" to "coverages"
** Only the two "initial" tests shown on summary plot were by nuclear method. Check tests and all other tests were by the Sand Volume method.

+Strike out inapplicable words. Summary Prepared by ARG Date 6 Dec 68
ENG Form 4287 JUN 69 (ER 1110-2-1925) Summary Checked by JSJ Date 7 Dec 68

Figure D-3. Example of a prepared summary form, field compaction control data, pervious (sand drain)



PROJECT: ____________ Dam
DISTRICT: ____________
REPORT PERIOD: 5 Nov - 5 Dec 68
REPORT NO. 12
TYPE OF MATERIAL: PERVIOUS (SAND DRAIN)

	CUMULATIVE TO THIS REPORT	THIS REPORT
TOTAL NO. OF TESTS	175	25
NO. BELOW MINIMUM	26	6
NO. REWORKED	23	5
NO. RETESTED AFTER REWORKING	18	3

LEGEND:

- ● ABOVE ACCEPTABLE MIN
- X BELOW ACCEPTABLE MIN
- X_R LATER REWORKED ONLY (NO RETEST)
- X_1, X_2 LATER REWORKED & RETESTED
- $●_1$, $●_2$ RESULT OF RETEST AFTER REWORKING
- Δ_1, Δ_2 INITIAL TEST (USE ONLY w/CHECK TEST)
- $\blacktriangle_1$, $\blacktriangle_2$ CHECK TEST

NOTE: USE SAME NUMBER FOR LATER REWORKED AND RETESTED AND RESULT OF RETEST AND FOR INITIAL AND CHECK TEST

ENG Form 4287B
JUN 69

(ER 1110-2-1925)

Figure D-4. Example of a summary plot for material requiring only density control

Table D-1
Samples of Appropriate Entries on Tabular Summary

Compaction Equipment	Method of Relating Field *w* to Standard Optimum *w* and Field Density to Maximum Dry Density or Relative Density
Sheepsfoot roller, Bros, self-propelled, SP244DA (636 psi)	Field results compared with results of complete standard compaction test on material from field test.
Pneumatic roller, 50-ton Ferguson Model RT-100 S, 4-wheel (80 psi)	Field results compared with laboratory curves selected by two-point standard compaction test on material from field test.
Sheepsfoot roller, Southwest Model 2DM-120S, 25,335 lb (towed)(527 psi)	Field results compared with results of rapid compaction (USBR) tests on fill material.
Sheepsfoot roller, Ferguson Model SP-120B, self-propelled (615 psi)	Field results compared with laboratory standard compaction results for minus 1-in. material, corrected for percent plus 1-in. material. Appropriate laboratory results selected by Atterberg limits correlations.
Sheepsfoot roller, (towed), American Steel Works, similar to Model ABD 120 (547 psi)	Compared visually with materials on which laboratory standard compaction tests were performed.
D-8 crawler tractor (12.2 psi)	Maximum (vibratory table)[1] and minimum density determined for each field density test.
Pneumatic roller, 50-ton Bros Model 450, 4-wheel (80 psi)	Compared with results of laboratory maximum (modified Providence vibrated) and minimum density test on minus 2-1/2-in. fraction.
Vibratory roller, Bros Model VP-20D (static weight = 10 tons; centrifugal force = 20 tons at 1,300 rpm)	Appropriate laboratory results selected by gradation correlation.

Note: if more than one method is used, show percentage use of each method.

[1] Use care to confirm reliability of maximum density as determined on the vibratory table. See the caution in Appendix B, paragraph B-4*a*.

Appendix E
Description and Use of Instruments During Earth and Rock-Fill Dam Construction

E-1. General

Stability of an earth and rock-fill dam is often more uncertain during construction than upon completion. Unfavorable conditions produced, for example, by inclement weather during construction may result in transient conditions of marginal stability, which may be the most dangerous over the life of an embankment. For that reason, it is important to carefully and continuously monitor the state of a compacted earth structure during construction. Instruments to monitor displacement, slope (change), pore water pressure, soil stress, and water flow are necessary to monitor changes in an embankment that may signify the onset of instability and indicate the need of a change in construction technique or procedure. The basic instruments, their function and basic operating principal will be described here. However, EM-1110-2-1908 is under preparation at this writing and will clarify the philosophy, policy, use, and installation of instrumentation with respect to earth and rock-fill dams.

E-2. Displacement Measuring Techniques and Instruments

a. Slope indicators (inclinometers). Slope indicators are used primarily to monitor earth movement in undisturbed soil masses as well as compacted embankments by detecting changes in slope within the soil structures. A specially designed plastic or aluminum casing with an alignment groove along one edge may be installed in a bore hole up to 900 ft deep. Slope indicator instruments are lowered into the casings on spring-loaded rollers which ride into the grooves to maintain alignment. Deviation from the vertical is detected by monitoring an electronic signal from either a Wheatstone Bridge circuit or piezoelectric crystals within the sensor, which is generated by a change in stress in a mechanical system such as a pendulum or cantilever arm. Slope versus electronic signal from a slope indicator instrument is established by calibration; therefore the slope of the bore hole with depth is determined from the output signal of the sensor as it is lowered into the bore hole. Change in slope with time is an indication of embankment movement. Plots of slope versus depth at various locations over the site of the embankment will indicate patterns of movement and will therefore index the stability of the embankment. Manufacturers of slope indicator instruments claim that some models have the sensitivity to detect a deviation in slope of one part in 10,000. A careful calibration is recommended before installation of this or any instrument to confirm that devices are operational, and that they deliver the measurement accuracy required for proper monitoring.

b. Settlement/heave measurement devices. Settlement devices are sometimes used in the same casings as slope indicators. Settlement meter casings are designed to "telescope" in order to follow settlement or heave within the soil structure. Special couplings in the casing may allow from 6 to 12 in. of movement per casing section. A probe which hooks to the bottom lip of a casing section is lowered into the bore hole. After hooking onto the bottom lip of a casing section, distance from the top of the casing is measured with a surveyor's chain. The elevation of the casing top may be determined by standard level survey techniques based on bench marks outside the settlement zone. Calibration required for this procedure is accomplished with a surveyor's chain. A slight variation to this procedure is to measure settlement in emplaced structures by placing ferrous metal washers around the casings. The vertical position of the washers may be monitored using magnetic or inductive pickup sensors.

E-3. Stress Measurement

a. Carlson soil stress meters. These instruments are designed for the direct measurement of soil pressure against a solid structure. The meter consists of two steel discs connected along their circumferences/edges by a flexible rim. A thin film of mercury fills the space between the discs. When subjected to stress, the mercury deflects an internal diaphragm. The device is calibrated in a pressure chamber under hydrostatic pressure such that a relationship between pressure and diaphragm deflection is established. When implanted in a soil structure, the stress pattern over the area/face of the meter is averaged. The Carlson soil stress meter is usually placed next to a solid structure with its top side exposed to soil. In such an installation, the flexible rim is covered with neoprene rubber to prevent binding and damage to the instrument.

b. Flat jacks. These stress measurement devices are variations of the Carlson soil stress meter and consist, essentially, of a fluid-filled space between two flat parallel plates with a pressure-tight hinged seal around the periphery. These instruments are permanently installed in structures of interest at the desired location and orientation. The average pressure exerted by the soil on the face of the jack is transmitted to the fluid inside, which is measured electronically or mechanically. The main advantage of these devices is that they require little deformation for activation; the main disadvantage is that their stiffness may not match that of the structure in which they are installed and, as a result, the stress measured may be in error. Calibration of these devices can be difficult, and installation must be performed by an experienced technician.

E-4. Pore Pressure Measurement

Piezometers are instruments permanently installed in a soil or rock structure to measure fluid pressure. Several types of piezometers based on different pressure sensing mechanisms are available for general use. Each type will be identified and the operating mechanism briefly described.

a. Open standpipe piezometer. This type of device consists of an open tube in which the level of fluid is measured by sounding, or by lowering a tape into the tube to measure the water level.

b. Casagrande piezometer. The tip of the device consists of a 2-ft-long porous tube connected to a riser pipe of 3/8-in. tubing. Water level (pressure) is measured with an electronic sounding device or a pressure gauge if the water level is higher than the ground surface.

c. Wellpoint piezometer. The instrument consists of a perforated tip (well screen, epoxied sand filter, well strainer, etc.) connected to a standpipe. Water level is measured by lowering a sounding device into the standpipe.

d. Hydraulic piezometer. Two designs are in general use: the USBR device and the Bishop device. This type of piezometer consists of two tubes leading to the tip, which contains a porous element common to the two tubes. The tip must be de-aired for proper operation; de-airing is accomplished by flushing water through one tube and bleeding through the other until saturation is achieved. One tube is then shut off and the other connected to a vacuum/pressure gauge which reads fluid level directly.

e. Diaphragm piezometers. Two designs are in general use: the Warlan device and the Gloetzl device. The diaphragm piezometer consists of two tubes leading to the piezometric porous tip. A membrane is forced against the end of one of the tubes by in situ water pressure. To make pore pressure measurement, from the observation station, air pressure is introduced into the tube that has the membrane against it until the pore pressure acting on the opposite side is slightly exceeded, allowing air past the membrane flapper. This air, escaping through the opposite tube, is detected with a bubble chamber at the observation station. The air pressure is then reduced until bubbling stops, at which time the air pressure in the line is assumed to equal the pore water pressure.

f. Electronic strain gauge piezometers. The Carlson piezometer and electronic pressure transducers are examples of this design of piezometer. The principal of operation is that water pressure deflects a diaphragm which has been strain-gauged or otherwise fitted for electronic displacement measurement. Pressure versus strain meter reading is established by calibration so that pore water pressure is determined directly by a meter reading.

g. Vibrating wire piezometers. Mechanical resonance or vibrating wire transducers are sometimes used to measure water pore pressure. In these instruments, a wire under tension is connected to (the center of) a diaphragm which deflects as the result of water pressure. Deflection of the membrane changes the tension in the wire and therefore the resonant frequency of the structure. The system is configured so that the wire under tension may be excited to resonance by a magnetic coil and the resonant frequency measured electronically. The relationship between resonant frequency and pressure is established by calibration. Vibrating wire piezometers are essentially like electronic strain gauge transducers except that the internal displacement, or strain associated with pressure change is measured using electromechanical means.

E-5. Flow Measurement

Flow measurement associated with dam construction may be achieved using two basic devices, weirs and impeller flow transducers. Each will be briefly described.

a. Weirs. A weir is an obstruction in a channel that causes water to back up behind it and to flow over or through it. By measuring the height of the upstream water surface, the rate of flow is determined. Weirs constructed from a sheet of metal or other material such that the jet, or nappe, springs free as it leaves the upstream face are called sharp-crested weirs. For example, the V-notch weir is a very effective and widely used sharp-crested weir which may be calibrated quite precisely and reliably for use in flow measurement. Other weir types, such as the broad-crested weir, support water flow in a longitudinal direction. The relationship between flow rate versus height above the crest of a particular weir is established by calibration against a standard with known volume discharge characteristics.

b. Impeller flow transducers. The impeller flow transducer consists of a flow chamber around an impeller shaft and rotor. As fluid flows through the chamber, it impinges on the blades to cause rotation of the impeller shaft, the speed of which is measured electronically with an encoder. A relationship between quantity of flow and shaft rotation speed/electronic output may be established by calibration. Impeller flow transducers are generally used to measure relatively small rates of flow which must be obtained precisely. However, impeller flow transducers do not work well when used with water containing sediment, as particles of grit tend to jam the mechanism, causing it to seize.

E-6. Temperature Measurement

One possible benefit of temperature measurement connected with dam construction is that it may aid in determining the source of seepage or leakage water. Temperature measurement sensors may be permanently installed in a compacted earth structure during construction or may be mobile and simply lowered into boreholes for spot temperature checks. Two electronic devices are generally used for temperature measurement, the thermocouple and the thermistor. The mercury thermometer is also useful. Each will be briefly described.

a. Thermocouples. A thermocouple is an electronic circuit consisting of two dissimilar metals in which a voltage is produced when two junctions of the metals are at different temperatures. For example, the temperature of ice water is typically used as a reference junction temperature and the voltage produced by the opposite junction calibrated versus temperature (of that junction). In many commercial thermocouple instruments, the function of the reference junction is simulated electronically. For the temperature range expected in earth dam construction, copper/constantan or iron/constantan thermocouples (commonly called ISA[1] J-type or T-type thermocouples, respectively) will be most appropriate and useful.

b. Thermistors. A thermistor is a composite semiconductor that has a large negative temperature coefficient of resistance and, as such, can be used for temperature measurement. The electronic circuit associated with a thermistor is designed to measure the resistance of the thermistor and, therefore, the temperature-resistance characteristics of the device must be established by calibration. The electronic circuitry associated with thermistors is often designed to produce readings directly in engineering temperature units.

c. Mercury thermometer. A mercury thermometer is a closed evacuated glass tube containing a quantity of mercury. Mercury has a relatively high coefficient of temperature expansion, and when the tube is placed in a temperature environment, the mercury will expand to fill a given portion of the tube. The tube is graduated, and the relationship between temperature and the expansion of the mercury in the tube as quantified by graduations on the tube is established by calibration. The mercury thermometer is very easy to use and relatively precise; however, because of the fast time response of thermal expansion of mercury, the device must be used only in applications when the tube may be observed directly (that is, it may not be lowered into boreholes).

E-7. Strong Motion Monitoring

Strong motion monitoring is used to measure the response of an embankment dam to seismic activity. The most important benefit obtained is to guide decisions on inspection and repair after the structure has been subjected to a seismic event. The information may be used to determine if the event was larger or smaller than the design earthquake and to decide what repair or strengthening is needed. Instruments for strong motion monitoring are called strong motion accelographs of seismographs. The key element of the instrument is an accelerometer, which consists of a mass suspended in a case. The case itself is securely fastened to the dam. During an earthquake, relative movement between the mass and the case is converted to an electrical signal which is converted to either the acceleration or the velocity of the ground motion. An accelograph also contains signal amplifiers, a recording device such as paper, photographic film, or magnetic tape, along with a rechargeable battery power supply, a very accurate clock, and a motion trigger to turn on the instrument when a predetermined level of ground motion is exceeded. An important consideration in the design and installation of such instruments is that they be sensitive enough to give an accurate account of the motion, yet be protected so that they are not damaged during the event.

[1] Instrument Society of America.

Index