AD-A012 066

ROCK CORE TESTS, PROPOSED DUPLICATE LOCK-PHASE II, STARVED ROCK LOCK AND DAM, ILLINOIS RIVER, ILLINOIS

Richard L. Stowe, et al

Army Engineer Waterways Experiment Station
Vicksburg, Mississippi

June 1975

199151

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MISCELLANEOUS PAPER C-75-9

ROCK CORE TESTS, PROPOSED DUPLICATE LOCK-PHASE II, STARVED ROCK LOCK AND DAM, ILLINOIS RIVER, ILLINOIS

by

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June 1975

Final Report

Approved For Public Release; Distribution Unlimited

Prepared for U. S. Army Engineer District, Chicago
Chicago, Illinois 60604

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER: Miscellaneous Paper C-75-9	2. GOVT ACCESSION NO.
3. RECIPIENT'S CATALOG NUMBER: AD-A012066	
4. TITLE (and Subtitle): ROCK CORE TESTS, PROPOSED DUPLICATE LOCK - PHASE II, STARVED ROCK LOCK AND DAM, ILLINOIS RIVER, ILLINOIS	5. TYPE OF REPORT & PERIOD COVERED: Final report
	6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s): Richard L. Stowe, James B. Warriner	8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS: U. S. Army Engineer Waterways Experiment Station, Concrete Laboratory, P. O. Box 631, Vicksburg, Miss. 39180	10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS: U. S. Army Engineer District, Chicago, 219 South Dearborn Street, Chicago, Illinois 60604	12. REPORT DATE: June 1975
	13. NUMBER OF PAGES: 114
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)	15. SECURITY CLASS. (of this report): Unclassified
	15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

A

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Locks (waterways)
Rock cores
Rock foundations
Rock tests (laboratory)
Starved Rock Lock and Dam

PRICES SUBJECT TO CHANGE

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A rock core testing program was conducted for the U. S. Army Engineer District, Chicago, as part of a continuing foundation investigative program. After reviewing available reports and current drilling activity, a testing program was recommended and subsequently carried out. A sufficient number of laboratory tests were conducted on representative bedrock samples and an analysis of these tests was made. Limited field work was conducted in order to obtain information about orientations of discontinuities. Laboratory tests consisted of index property tests (including unit weights, specific gravity (Continued)

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

20. ABSTRACT (Continued).

of solids, moisture content, porosity, and compressional wave velocity), unconfined compression, triaxial, tensile, and direct shear tests. Direct shear tests were conducted on intact core, precut surfaces, natural joints, and on thin shale beds contained in dolomite. A petrographic examination was conducted on selected samples to ascertain nomenclature and mineral content. A borehole televiewer was used in 7 of 19 drilled holes over the proposed site to obtain information concerning the orientations of discontinuities. Core and core logs were used to construct geologic cross sections. The geologic cross sections and the televiewer data were used to construct sections showing bedrock structural characteristics. The results of the laboratory tests are presented in the report in tables and diagrams for the different lithologies tested, i.e., dolomite, sandstone, and shale. Lower bound values are tabulated as recommended design values for each of these rocks. The foundation consists of two distinct strata: the Ordovician St. Peter sandstone and the Shakopee dolomite. Both formations are interbedded with shale having thicknesses of from 1/4 in. to about 12 in.; friable sandstone beds also occur in both formations and vary in thickness from 1/4 in. to several feet. Bedding is gently dipping towards the southeast at less than 10 degrees. Probable joint configurations were deduced from a combination of core logs and geophysical logging information. If the geometrical configuration of joint sets form possible rock wedges, then the primary consideration in foundation stability analyses will be sliding along discontinuities. No information is presented in this report relating to possible wedge sizes. This information must be developed during construction. The extremely low strength of the intact shale parallel to its bedding prescribes that, for conservative design, any shale bed be treated as a potential sliding plane parallel to its bedding. Potential failure involving wedges and horizontal sliding along shale beds should be given design consideration.

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FOREWORD

This testing program, "Rock Core Tests, Proposed Duplicate Lock Site, Starved Rock Lock and Dam, Illinois River, Illinois," was conducted for the U. S. Army Engineer District, Chicago. The work was authorized by DA Form 2544, No. 74 E-12, dated 23 May 1974.

Laboratory tests were performed at the Concrete Laboratory (CL) and the Soils and Pavements Laboratory (S&PL) of the U. S. Army Engineer Waterways Experiment Station (WES) during the period May 1974--January 1975 under the direction of Messrs. Bryant Mather and John M. Scanlon, CL. Mr. D. Banks of the S&PL supervised the field work consisting of televiewer logging, Mr. G. P. Hale supervised the laboratory testing that was conducted in the S&PL, and Mr. R. Hunt conducted the televiewer logging. Mr. J. B. Warriner, S&PL, made the analysis of the televiewer. Mr. R. L. Stowe, CL, was project officer and was assisted in performing the work at the CL by Messrs. A. D. Buck, G. S. Wong, F. S. Stewart, and J. B. Eskridge. Messrs. Stowe and Warriner prepared this report.

The Director of WES during the conduct of the program and the preparation and publication of this report was COL G. H. Hilt, CE. Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
angstrom	1.000000 E-10	metre
inches	2.540000 E-02	metre
feet	3.048000 E-01	metre
inches per day	2.939815 E-07	metre per second
pounds (mass)	4.535924 E-01	kilogram
pounds (force)	4.448222 E+00	newton
pounds (mass) per cubic foot	1.601846 E+01	kilogram per cubic metre
pounds (force) per cubic inch	6.894757 E-03	megapascals
tons (force) per square foot	9.576052 E-03	megapascals
feet per second	3.048000 E-01	metre per second

ROCK CORE TESTS, PROPOSED DUPLICATE LOCK - PHASE II

STARVED ROCK LOCK AND DAM, ILLINOIS RIVER, ILLINOIS

PART I: INTRODUCTION

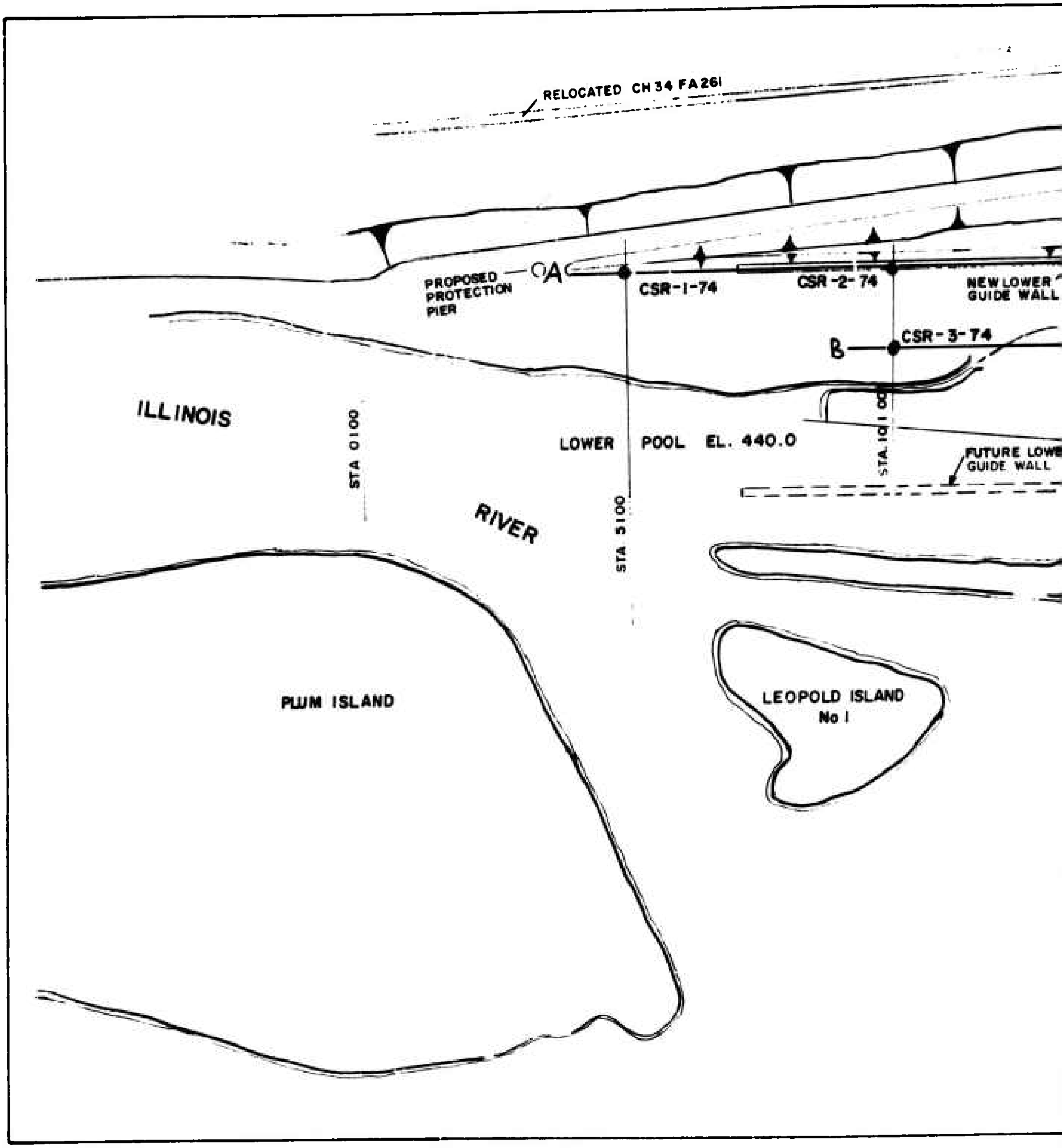
Location of Study Area

1. The Starved Rock Lock and Dam site is located in LaSalle County, Illinois, some 8 miles west of Ottawa, Illinois. The site is near mile 231 on the Illinois River; the driving distance is about 85 miles southwest of Chicago.

2. The recent foundation investigation for the proposed Duplicate Lock at Starved Rock Lock and Dam involved the drilling of 19 holes. Overburden samples and bedrock cores were recovered with the average depth being 75 ft with about 40 percent overburden. Representative overburden samples were recovered with a split-barrel and Shelby tube samplers. Nominal 6-in.-diameter rock cores were recovered from all but one hole using a diamond core bit and double-tube core barrel; 4-in.-diameter core was received in the laboratory from hole CSR-16-74. The locations of the drilled holes are presented in Fig. 1; this figure was received from the Chicago District about two months after the drilling was initiated and is similar to the location diagram presented in Reference 1. A noted difference is the boring designation. The holes are designated CSR on the copy received at the Waterways Experiment Station (WES) and CRS in Reference 1. Because the core boxes received at WES had the letters CSR, these letters were used throughout this study.

Background

3. A foundation investigation of the proposed Duplicate Lock at Starved Rock was conducted in 1972 by the Chicago District.[2] Some of the test results were considered nonconclusive and the District recommended a continuing investigative program. The WES was asked by


RELOCATED CH 34 FA 261
PROPOSED PROTECTION PIER
A
CSR-1-74
CSR-2-74
NEW LOWER GUIDE WALL
B
CSR-3-74
ILLINOIS
STA 0100
LOWER POOL EL. 440.0
STA 101 00
FUTURE LOWE
GUIDE WALL
RIVER
STA 5100
PLUM ISLAND
LEOPOLD ISLAND
No 1

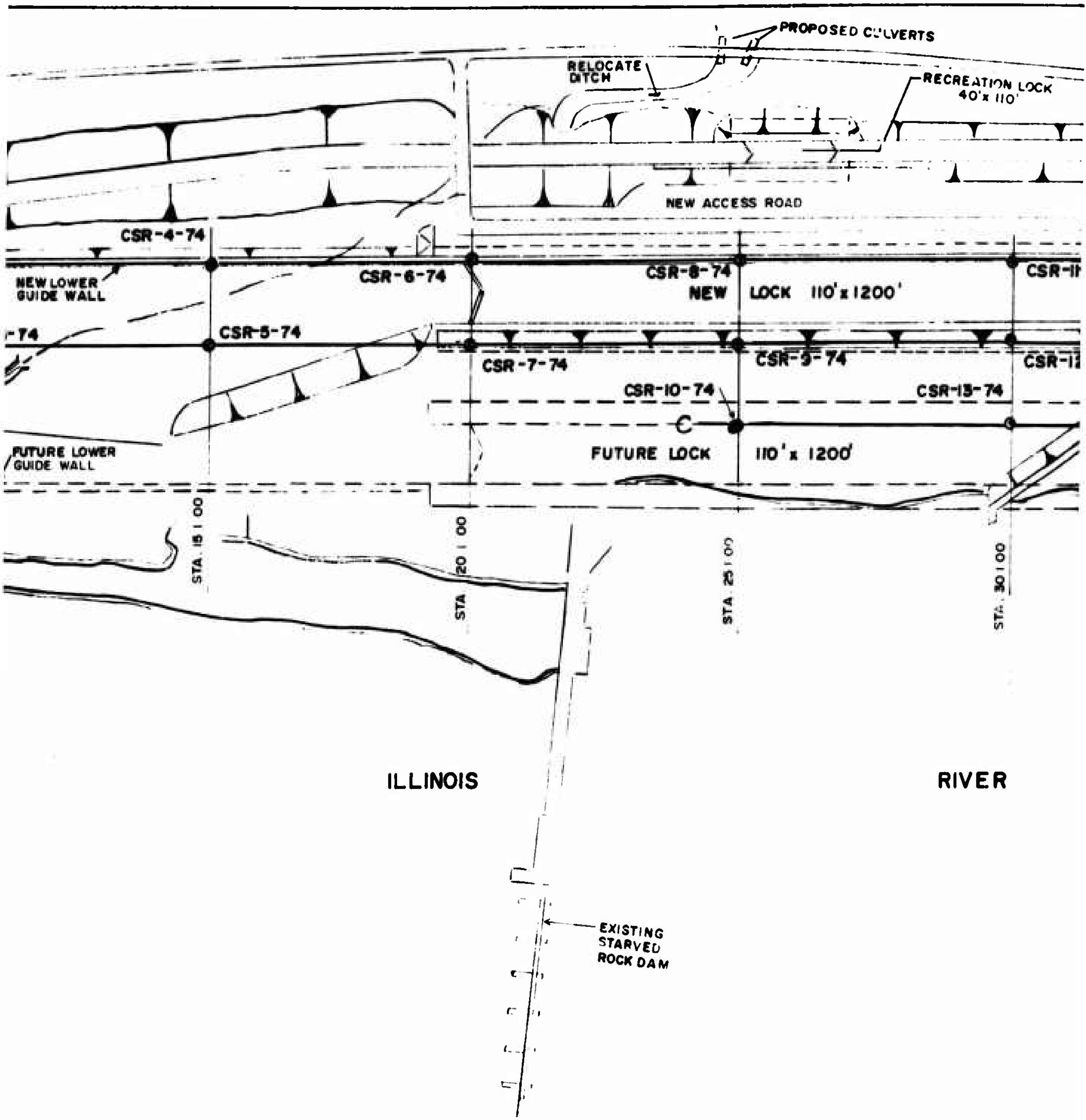
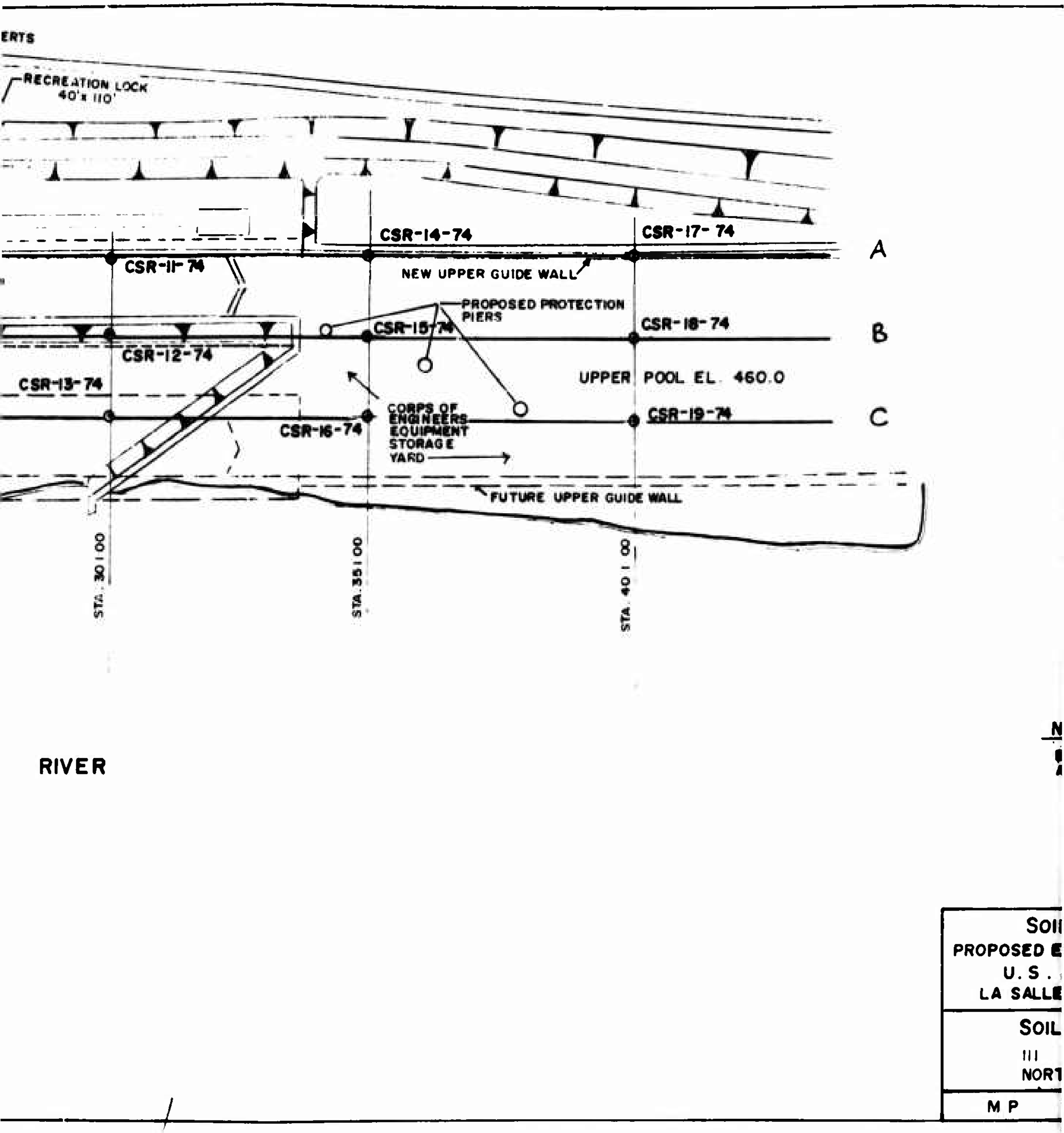


Figure 1. Drill hole location (After STS Reference 1)


ERTS
RECREATION LOCK
40'x 110'
CSR-14-74
CSR-17-74
A
CSR-11-74
NEW UPPER GUIDE WALL
PROPOSED PROTECTION
PIERS
CSR-15-74
CSR-18-74
B
CSR-12-74
UPPER POOL EL. 460.0
CSR-13-74
CORPS OF
ENGINEERS
EQUIPMENT
STORAGE
YARD
CSR-19-74
C
CSR-16-74
FUTURE UPPER GUIDE WALL
STA. 30+00
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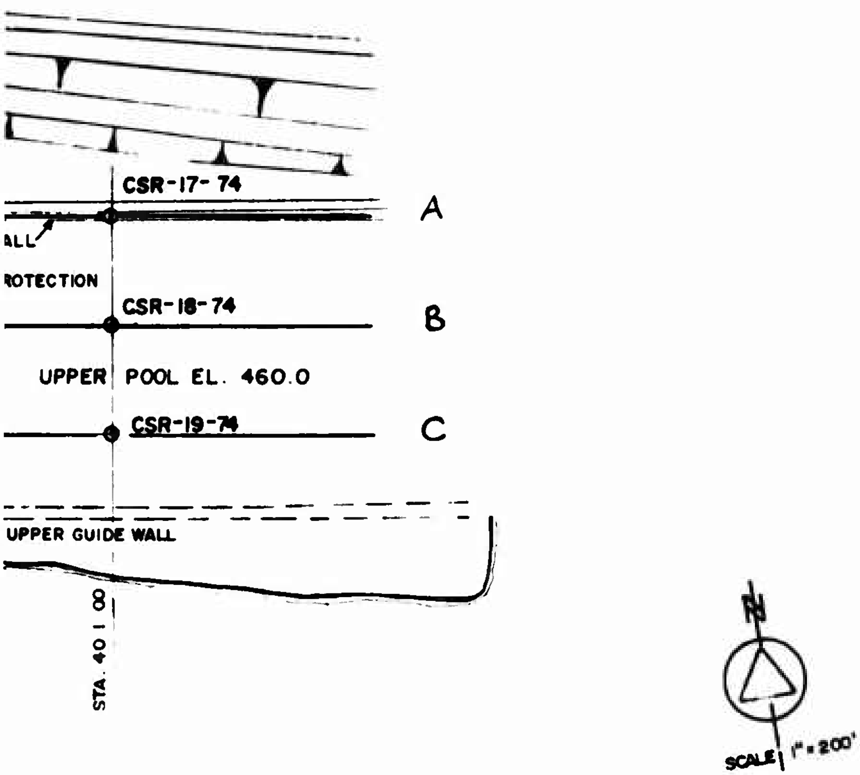

NOTE

BORING LOCATIONS & SCALE
ARE APPROXIMATE

SOIL BORING LOCATION DIAGRAM PROPOSED EXPANSION OF STARVED ROCK LOCK AND DAM U. S. ARMY CORPS OF ENGINEERS LA SALLE COUNTY ILLINOIS			
SOIL TESTING SERVICES, INC. 111 PFINGSTEN ROAD NORTHBROOK ILLINOIS			
M P	CDL	8-3-1974	17350

Chicago District (NCC) personnel to assist in the development of a rock mechanics testing program for the proposed Duplicate Lock. The request was made in early April 1974. Copies of three reports, a geological foundation report, a stability investigation report, and a stress analysis report, all pertinent to the proposed lock site, were received at WES for review and background information.[2,3,4]

4. After considering the engineering characteristics of the foundation rock and the material properties as described in the appropriate reports, a recommended laboratory testing program, as outlined in Table 1, was proposed. A minimum number of index properties on representative specimens were recommended to aid in evaluating the consistency of the foundation materials. A minimum number of unconfined compression, triaxial, and tensile tests were recommended on specimens selected to represent each lithologic unit. Strength and stress-strain relations would be obtained from these tests; various moduli and Poisson's ratio could be calculated for use in finite element analyses. Direct shear tests were recommended from which peak strength and sliding friction characteristics of portions of the foundation material could be obtained. Further, it was recommended that a cursory petrographic examination be conducted on suspected clay samples.

5. It was recommended that shear tests be run on intact specimens, precut surfaces, and natural joints. The presence of joints, shear zones, and other natural discontinuities reduces the shear strength of a rock mass to values much below those for intact rock, particularly in directions parallel to these discontinuities. When loading conditions dictate that potential failure surfaces cut across these structural features, the appropriate shear strength may approach that of intact rock. However, where loading conditions are such that the direction of loading is parallel or subparallel to the structural features, the shear strength is a function of the shearing resistance along the surfaces of the discontinuity. Also, the presence of discontinuities causes a decrease in the modulus of deformation of the mass that should be included in any proposed finite element analysis. Because of these concerns, a concentrated

effort was made to determine the orientations of discontinuities so that their effect on the proposed structure could be properly evaluated.

Objectives

6. The objectives of this study were to:

   a. Review available information from reports and current drilling activity to enable proper selection of specimens for testing.

   b. Conduct a sufficient number of laboratory tests on representative bedrock samples.

   c. Make an analysis of tests conducted and a summary of the foundation condition.

Scope

7. The foundation drilling was accomplished by contract to Soils Testing Service, Inc. (STS), and delivery of core was scheduled as work progressed. A total of five deliveries of core were made over a period of four months. The testing program was initiated after receiving two shipments of core and core logs. Geologic cross section and sections showing bedrock structural characteristics were developed from available information. These sections were updated as additional information was received. The partial sections containing information from the first two shipments and the geological information presented in Reference 2 were used in the initial selection of representative test specimens.

8. The objectives of this study were accomplished by conducting index property tests, unconfined compression, triaxial, tensile, and direct shear tests. Direct shear tests were conducted on intact core, precut surfaces, natural joints, and on thin shale beds contained in the dolomite. Several suspected clay samples were subjected to a petrographic examination to ascertain nomenclature and mineral content.

9. A borehole televiewer[5] was used in 7 of 19 drilled holes to obtain information concerning the orientations of discontinuities. The orientations of discontinuities in relation to the proposed lock were taken into account in making the foundation appraisal.

PART II: GEOLOGICAL CHARACTERISTICS OF FOUNDATION

Bedrock Stratigraphy

10. Visual observations of the core received at WES are in general agreement with descriptions of core and of stratigraphy given in References 1 and 2. Therefore, only a brief description of the foundation rock will be given in the following paragraphs. The Ordovician St. Peter sandstone and the Shakopee dolomite are the two formations sampled by drilling. Both formations are interbedded with shale having thicknesses of from 1/4 in. to about 12 in.

11. The St. Peter is a well-sorted, fine- to coarse-grained and loosely, moderately, and strongly cemented sandstone. Some of the sandstone is quite friable and can be scratched with the fingernail. Other pieces cannot be scratched with a pocketknife. The color varies from white to light gray to light brown. Minor amounts of silt and clay were noted in several hand samples from holes CSR-6 and 9. The formation contains thin beds of blue-green shale as noted in the core received at WES. Gray to black shale was described in References 1 and 2. Clay seams were described in the logs of holes CSR-9, 16, and 17. Chert bands were observed in the core received at WES.

12. The Shakopee formation underlies the St. Peter in all holes except CSR-5-74. The dolomite is fine to medium crystalline, buff to light brown and gray. The dolomitic mass contains numerous thin beds of blue-green and gray-green shale and thin lenses of sandstone. The shale beds appear to be localized, and in only two drilled holes was it possible to correlate a shale bed. Oolitic chert was observed to occur as nodules and in bands throughout the dolomite. Clay seams were observed in the core from holes CSR-5, 7, 11, and 17; results of tests on these clays will be described later. Reef characteristics were noted in localized areas but generally in the mid- to western sections of the drilling site.

Geologic Cross Sections

13. The terminology used by the four STS inspectors, who were involved in logging the core in the field, was used in most instances

while constructing the geologic and structural cross sections. Two lists of abbreviations from STS show slight differences in the definition of certain terms. In particular, SB was used to denote stylolitic bed in one list and structural breaks in another list. Certain abbreviations occurred on the log sheets which did not occur on the list of abbreviations. These differences are not considered important and do not affect our evaluation of the foundation.

14. Three cross sections were drawn to show an overview of the bedrock material as well as to assist in the selection of representative test specimens. The locations of the sections are shown in Fig. 1 as lines A-A, B-B, and C-C. The sections are aligned with the three grid lines showing drill hole location running approximately east and west. Sections A-A, B-B, and C-C are presented in Plates 1, 2, and 3, respectively.

15. The thickness of the St. Peter sandstone varies throughout the site. In the eastern and southern portions, the sandstone is relatively uniform in thickness, varying from 8 to 13 ft. The sandstone thickness in the north central and western portions varies much more, probably due to a slight depression or old river channel in the surface of the underlying dolomite. This feature has been contoured and noted to be trending in a northwest-southeast direction with a slight dip to the northwest (see Fig. 2). The maximum thickness of sandstone depicted in the logs was 33 ft in CSR-6-74. The sandstone thickness in the western portion of the site varies from 0.0 ft in CSR-5-74 to 11.5 ft in CSR-3-74. The elevation of the contact between the sandstone and the underlying dolomite drops toward the west at the western limit of the drill site.

16. The contact between the St. Peter and Shakopee formations is well defined in all three sections. A difference in the contact exists between the work WES has done and the work reported by STS.[1] This difference occurs in CSR-9-74. This report shows the contact at an elevation of 422.3 ft* (see Plate 2), while STS shows the contact at about el 404.5. The contact is shown at the higher elevation in this

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* All elevations (el) cited herein are in feet referred to mean sea level.

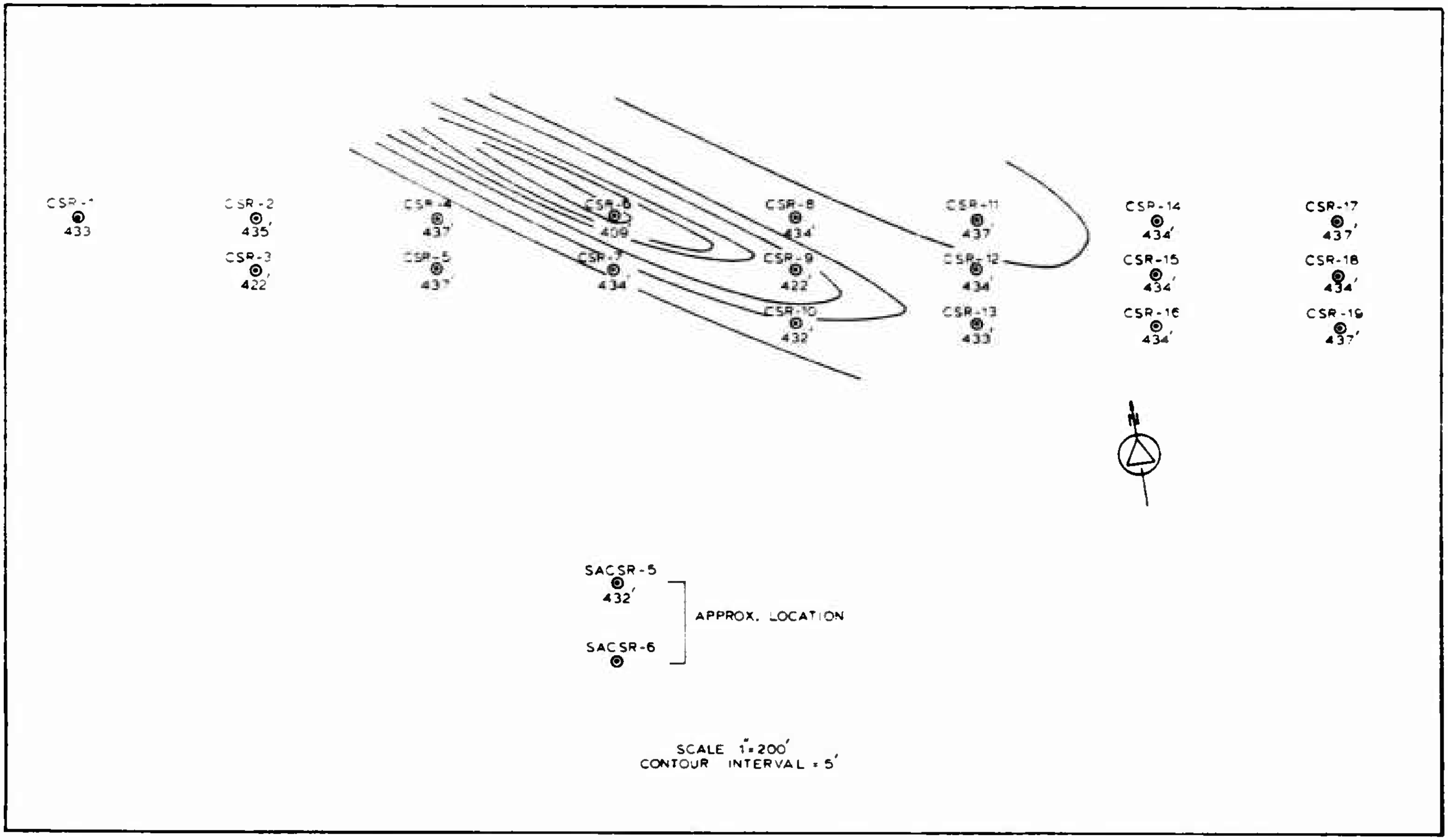

Figure 2. Contour of top of dolomite over a section of the drill site

report because of the correlation between the 4.5-ft-thick sandy dolomite bed in CSR-9-74 and a similar section of rock in CSR-12-74 at a slightly lower elevation.

17. It will be noted in Plates 1, 2, and 3 that shale is shown in several ways: first, as single dashed lines with different notations, e.g., as Bl-Grn Sh*(blue-green shale) or just Sh or as Sh Ptg (shale parting). This notation was in keeping with terms contained on the logs of cores received at WES. These shale features are considered as beds although they were not thick enough, generally between 1/8 and 1/2 in. in thickness, to be drawn as beds. Units of shale greater than 1/2 in. are presented as dashed lines bound by solid lines and denoted as beds.

18. The location of the stratification lines on the cross section is, in some cases, estimated especially in the area of the founding elevation of the proposed Duplicate Lock, el 414, which is subject to change. The sandstone lenses are well defined vertically and the possible lateral extent is shown by angular limit lines. These limits are, of course, highly questionable, and the angular lines were used to convey this fact. The two sandstone lenses near the proposed founding elevation of the lock walls (solid line drawn at el 414 in CSR-4-74 and CSR-8-74) were found to consist of the same size sand grains and to have the same cementing material. However, the sandstone at the same elevation in CSR-6-74, which is located between holes CSR-4 and CSR-8, was found to have a different sand grain size and cementing agent. This fact would further suggest the existence of the depression or the channel described earlier.

19. The stratification lines drawn in the dolomitic formation on Section B-B (Plate 2) could serve to represent the complex interbedded character of the foundation at and just below the proposed founding elevation.

Bedrock Structural Characteristics

20. Brecciated rock. Brecciated rock was described on the core logs for 14 separate depth intervals in 10 of the 19 drilled holes.

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* For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix B).

Eleven of these depth intervals were represented in the core received at WES. Pertinent information for the occurrences of brecciated rock is presented in Table 2.

21. Each of the 11 depth intervals was visually examined, and a term was applied to the appropriate interval (see Table 2). The terms used by the field inspector were correct in denoting the condition of the rock, i.e., reworked or recemented. The term "reworked" generally denotes movement of sediment after preliminary deposition, commonly resulting in transportation and sorting.[6] The term does not imply, among other things, a particle shape. The term "breccia" does denote an angular particle shape which all the "reworked" rock examined in the laboratory contained. The term "breccia," prefixed with sandstone or dolomite where appropriate, was used in this report for rock designated as reworked. The rock designated as "recemented" was called breccia as it also contained rock particles having various degrees of angularity. No attempt was made to determine how the brecciated rock was formed.

Discontinuities from Core Logs and Televiewer

22. The bedrock structural characteristics relevant to foundations are presented in Plates 4, 5, and 6 and represent the same sections (A-A, B-B, and C-C) as described under "Geologic Cross Sections" in this part of the report. These sections and the core logs from borings CSR-1-74 through CSR-19-74 were used for fracture evaluation. Because at least four separate persons logged the core samples, there is a substantial variation in the amount and type of structural information described on the separate logs. Subsequent reexamination of the core samples made available to WES personnel allowed a certain amount of information to be added, but joint frequency and descriptions are still dependent upon the subjective interpretations of the inspectors. All structural breaks (abbreviated SB), structural bedding planes (abbreviated SBP), and bedding planes (abbreviated BP) noted on the logs were listed with their respective elevations, inclinations, and associated lithologies. Where available on

the logs, the Rock Quality Designation (RQD) as defined by Deere[7] was tabulated and plotted versus depth. These plots are shown in Fig. 3. The total range exhibited in RQD was from 0 to 100 percent. The proportions of total footage for which RQD was available are classified according to Deere's system as follows:

RQD, %	Rock Quality	Proportion of Total Footage Measured, %
0-25	very poor rock	1
25-50	poor rock	3
50-75	fair rock	15
75-90	good rock	32
90-100	excellent rock	49

23. It can be seen from Fig. 3 that, while the lower RQD values are predominant above the proposed foundation founding el 414, there are some significantly low values found below that elevation. The low values are almost exclusively limited to those reaches of the borings passing through shale and sandstone beds.

24. In the absence of oriented core samples from which the strikes and dips of planar features could be determined, it was decided to use a borehole televiewer logging tool to obtain these structural parameters from the borehole walls. The televiewer contains a continuously rotating piezoelectric transducer which probes the borehole wall with bursts of acoustic energy in much the same way as sonar is used at sea. Because the tool is moved vertically up the hole simultaneously with transducer rotation, a narrow, spiral strip of the wall is probed. Vertical velocity is controlled so that the entire borehole wall is logged. The log is oriented electronically by a fluxgate magnetometer rotating with the transducer and sensing magnetic north. The amount of energy reflected by the wall and thus detected upon return to the transducer is a function of the physical properties of the surface. A smooth surface will reflect better than a rough surface, hard better than soft, and a surface perpendicular to the acoustic beam better than a skewed one. Past experience has shown that cracks and vugs as small as 1/8 to 1/4 in. are identifiable in good, competent rock. Less competent rock materials degrade this resolution somewhat because of wall roughness caused by drilling.

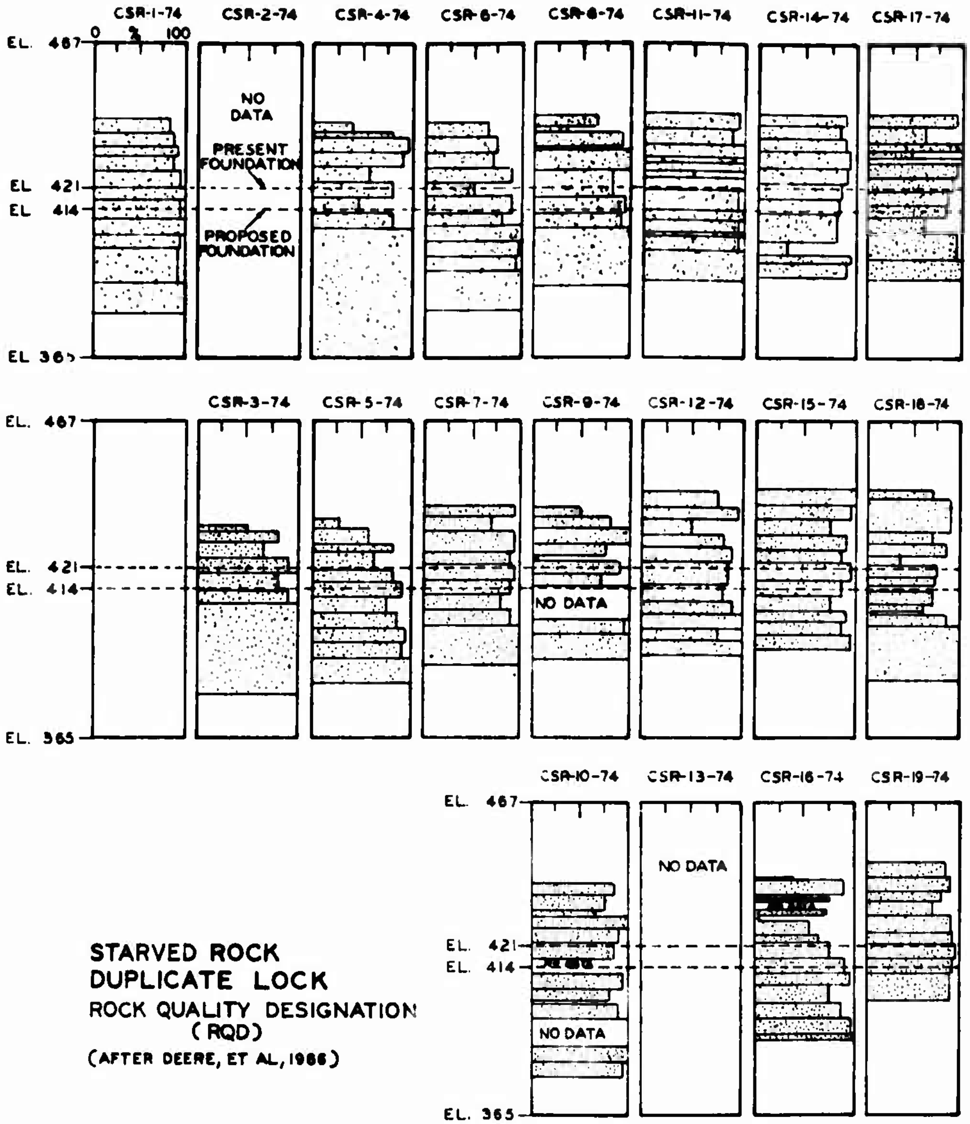

Figure 3. Rock quality designation diagrams

25. On 24-26 August 1974, seven borings at the proposed lock site were logged using the WES televiewer. These borings were CSR-3, 4, 6, 10, 13, 15, and 17. All borings were logged using the magnetometer orientation except boring CSR-4 which made strike orientations unobtainable from that log.

26. The information obtained from an evaluation of the bedrock stratigraphy and the various structural characteristics of the foundation was adequate for selecting representative test specimens.

PART III: SELECTION OF TEST SPECIMENS AND TEST PROCEDURES

Cores Received

27. Approximately 20 ft of core were received from each of 15 drilled holes, about 10 ft were received from three drilled holes, and no core was received from hole CSR-15-74. Pertinent information concerning the core received at WES is presented in Table 3. Five shipments of core were received over a 4-month period. Core boxes were held at the drilling site until enough boxes were accumulated for a full shipment.

28. Upon receipt of the core at WES, the boxes were placed in a moist curing room until the selection of test specimens was completed. About half of the core in the first shipment was inadequately wrapped. In some cases, cracks through the wax and wrapping cloth were observed, and in other cases, an inadequate coating of wax was observed. Some test specimens were selected from the portions of the core which were suitably wrapped. The cores received in the remaining shipments were adequately wrapped so as to preserve the moisture content of the core. In the initial drilling operation, some of the cores were exposed to rather warm temperatures in the out-of-doors and, as a result, were somewhat dried.

29. It was originally requested that core be sent to WES that represented the rock 10 ft above and below el 414. However, for some of the drilled holes, core was not received below this elevation and for other holes, the core received represented the rock only a few feet below el 414. Regardless, it is believed that the core received does represent the rock below el 414 over the drilled site. Assurance of this belief was obtained by correlating between the logs for which core was received and the logs for which core was not received. Visual observations of core verified the core description on the logs, and the logs indicated like materials for the missing core.

Selection of Test Specimens

30. Although two major lithologic units are present at the drilled site, the St. Peter sandstone and the Shakopee dolomite, test specimens

were not selected equally from both units. Very few specimens were obtained from the St. Peter sandstone formation because in only one hole (CSR-6-74) does the formation occur close to the proposed founding elevation (el 414). In CSR-6-74, the St. Peter extends about 4 ft below el 414. In holes CSR-3-74 and CSR-9-74, it occurs about 8 ft above el 422. The contact between the St. Peter and Shakopee is about 20 ft above el 414 in the other drilled holes. An attempt was made to select test specimens to be representative of the rock in close proximity to the proposed founding elevation. Where feasible, this was accomplished. Specimens in various physical conditions (competent, friable, loosely cemented, jointed, and interbedded) were selected for testing when available. The test assignment location for test specimens is presented in Tables 4 through 6 for sandstone, dolomite, and shale, respectively; the physical condition of the sandstone specimens is indicated in the tables.

31. The majority (32 of 57) of test specimens were obtained below el 414. It is believed that representative test specimens were obtained from both the sandstone lenses within the dolomite and the dolomitic mass. Most all the shale cores received in the laboratory were either broken during drilling or too thin for testing. The pieces suitable for testing were tested in direct shear. Limited index properties were determined and no compression or tensile tests were conducted on shale. Representative specimens of the thinly bedded shale were tested in direct shear as a filler material in the dolomite designated as filled partings.

32. Due to the extent and number of discontinuities in the foundation, attempts were made to obtain test specimens that contained a variety of these features. In selecting dolomite cores containing joints, it was observed that the surfaces were undulated such that the differential between high and low portions was too great for the direct shear apparatus. Consequently, no natural dolomite joints were tested. Two naturally jointed sandstone specimens and one shale specimen were adequate for direct shear testing.

Test Procedures

33. The index property tests were conducted in accordance with the appropriate test procedures as tabulated below.

Index Property	Test Procedure Used
Wet Unit Weight, γ_m	WES Test T-2*
Dry Unit Weight, γ_d	WES Test T-2*
Specific Gravity of Solids, G_s	WES Test T-1*
Moisture Content, w	WES Test T-2*
Porosity, n	WES Test T-4*
Degree of Saturation, S_r	WES Test T-4*
Compressional Wave Velocity, V_p	CRD-C 151 (ASTM D 2845)[8]

* See Appendix A.

The unconfined and triaxial compression test specimens were prepared according to standard method of test for triaxial strength of undrained rock core specimens, CRD-C 147.[8] Essentially, the specimens were cut with a diamond-blade saw and the cut surfaces were ground flat to 0.001 in.; specimens were checked for parallel ends and the perpendicularity of ends to the axis of the specimen. Electrical resistance strain gages were used for strain measurements, two each in the axial (vertical) and horizontal (diametral) directions. The modulus of elasticity, Poisson's ratio, and shear modulus were computed from the strain measurements. Axial specimen load was applied with a 440,000-lb-capacity universal testing machine. Confining pressure for the triaxial compression test was applied by a hand-operated electrohydraulic pump.

34. The direct shear test specimens were prepared according to applicable portions of the standard method of test for shear strength, CRD-C 90.[8] Two direct shear tests on shale were conducted in accordance with EM 1110-2-1906.[9] The concrete on rock specimens were fabricated using a general mass concrete mixture having an approximate compressive strength of 2000 psi. The concrete was wet sieved over a 1-in. sieve size screen, and the portion passing was cast on top of rock cores contained in the bottom section of 6-in.-diameter molds. Rock surfaces

onto which concrete was cast were both smooth and undulating; the smooth surfaces were prepared by sawing. Tensile test specimens were prepared according to standard method of test for splitting tensile strength of concrete specimens, CRD-C 77.[8]

Petrographic Examination

35. The hand samples listed in Part IV were examined with a stereomicroscope. Portions of some samples were ground to pass a No. 325 sieve; these powders were then backpacked and X-rayed.

36. Some of the more clayey materials were mixed with water after grinding; these slurries were placed on glass slides and allowed to dry to produce oriented clay slides. The slides were then X-rayed in the air-dry condition, and in some cases, after application of glycerol or ethylene glycol, or both. Organic liquids cause swelling clays to expand, assisting in their identification. All the X-ray patterns were made with an X-ray diffractometer using nickel-filtered copper radiation.

PART IV: TEST RESULTS AND ANALYSIS OF RESULTS

Index Properties

37. The results of the index property, triaxial, direct shear of competent intact core, and tensile strength tests are presented in Tables 7 and 8 for the dolomite and sandstone, respectively. The index property test results for shale are presented in Table 9. The average value, the range (difference in high and low values), and the number of index tests for the three rocks are tabulated below.

Summary of Index Properties

	γ_m lb/ft^3	γ_d lb/ft^3	G_s g/cc	w %	n %	S_r %	V_p ft/sec
Dolomite							
Average	164.5	160.3	2.81	4.3	8.3	76.0	14,140
Range	5.0	8.1	0.04	4.8	3.8	42.0	5,290
Number of Tests	3	3	3	16	3	3	4
Sandstone							
Competent							
Average	149.9	141.1	2.67	7.4	15.0	99.0	8,400
Range	4.3	5.6	0.01	5.6	3.7	1.0	1,560
Number of Tests	2	2	2	9	2	2	3
Friable	140.2	127.7	2.58	11.0	20.3	99.0	--
Number of Tests	1	1	1	6	1	1	--
Shale							
Average	134.4	117.3	2.71	14.8	30.7	99.0	--
Range	9.0	13.8	0.03	5.9	8.2	1.0	--
Number of Tests	2	2	2	2	2	2	--

38. An analysis of the index properties will be presented by rock type:

a. Dolomite. The unit weights and G_s are consistent and reasonable for a sound dense dolomite. The w , n , and S_r are also reasonable. However, these values should not be related to the cavernous and vuggy dolomite as described in the logs. The cavernous and vuggy dolomite represents a small percentage of the dolomite at the drilling site and, therefore, was not considered representative. If it is encountered in construction, it should be considered only moderately dense. The specific gravity reported in Reference 2 was specified as a bulk specific gravity, and was determined from calculations involving sample volume and dry weight-in-air, after the sample was dried to a constant weight at 110 C. This value should correlate to the dry unit weight (γ_d) determined during the present investigation. An average bulk specific gravity of 2.35 (146.6 lb/ft^3 using a unit weight of water = 62.4 lb/ft^3) previously reported[2] was considered to represent a less than moderately dense limestone (the term limestone is believed to indicate a dolomitic limestone). This value is in contrast to the average dry unit weight of 160.3 lb/ft^3 presented in the above summary of index properties. The V_p ranged from a low of 11,200 ft/sec to a high of 15,290 ft/sec, and in general, are in good agreement with V_p data reported earlier.[2] The previously reported V_p data ranged between 9,000 to 16,600 ft/sec. The data might be useful in correlations with in-situ seismic velocities if such were available.

b. Sandstone. The index properties are presented separately for the competent and friable sandstone. The different values are presented in Table 8. The difference between the unit weights of the competent and friable is indicative of the variability of the sandstone; although a small sampling is considered, it is believed that the values are representative. The average γ_d of the competent sandstone (141.1 lb/ft^3) presented above is somewhat lower than the previously reported value of bulk specific gravity (154.8 lb/ft^3 using a unit weight of water = 62.4 lb/ft^3). The two different values indicate the differences in unit weights of the sandstone at the drilled site. The average w , 7.4 and 11.0, indicates a small difference in w between the competent and friable sandstone. Any air-drying of the core which might have occurred would contribute to the moisture difference. The porosities measured previously and reported herein, 16.5 percent and 16.7 percent, respectively, are in good agreement. The high degree of saturation indicates at least that little or no moisture was lost from those specimens tested; it is assumed that in-situ the sandstone was nearly 100 percent saturated. The range in V_p was quite small and again serves to give a measure of the quality of the intact specimens tested. These data might also be useful in correlations with in-situ seismic velocities if measured.

c. Shale. As mentioned earlier, the pieces of shale received in the laboratory were generally unsuitable for index testing. No specimens were adequate for V_p testing, and only two specimens were tested for index properties. There was no unit weight test conducted previously[2] on shale from the proposed lock site. The unit weights presented in the above summary are considered to represent moderately dense shale. The value for bulk density recommended for use in the stability analysis is $\gamma_m = 134$ lb/ft^3 .

Triaxial

39. The stress values obtained during the triaxial tests are presented in Tables 7 and 8 for the dolomite and the sandstone, respectively. Modulus of elasticity, E , Poisson's ratio, ν , and shear modulus, G , are also presented in Tables 7 and 8. The stress-strain relations obtained for the dolomite and sandstone are presented in Plates 7 and 8, respectively, and the envelopes of failure are presented in Plates 9 and 10. An analysis of the triaxial tests will be presented by rock type.

a. Dolomite. The unconfined compressive strengths are reasonable for a sound dense dolomite. The strengths range from 4840 to 7250 psi with an average of 6300 psi. The average unconfined compressive strength previously reported[2] is 7900 psi. The mode of failure for each triaxial specimen is presented in Table 7. A single shear failure surface was noted on each specimen; the failure surfaces were inclined from the horizontal from between 57 to 63 degrees. Three separate triaxial specimens were tested using confining pressures, σ_3 , of 33, 66, and 100 psi. The strength results of these tests appear reasonable but indicate a much sounder rock than would be inferred from the unconfined compression test results. Mohr stress circles are presented in Plate 9. The angle of internal friction, ϕ , is 56 degrees, and the cohesion, c , is 1250 psi (90 tsf). The principal stress difference, $\sigma_1 - \sigma_3$, and the axial and radial strain, ε_a and ε_r , relation as presented in Plate 7 appear reasonable. A tangent E was computed for each of the curves in Plate 7. The E was calculated as an incremental value between 2000 and 3000 psi, which corresponds to the linear portion of the stress-strain curve for all dolomite specimens. The stress-strain curves presented in Plate 7 have slight differences in slopes between the initial and midportions with the exception of one specimen. If a modulus were computed for the initial or midportion of the curves, a small difference would be noted. It is, therefore, believed that the tangent modulus is representative. Poisson's ratio was calculated as an incremental value within the same stress range. The E ranged from 1.82×10^6 psi to 5.12×10^6 psi, with an average of 3.47×10^6 psi. The Poisson's ratio ranged from 0.13 to 0.32, with an average of 0.26. The average G was 1.06×10^6 psi and was a calculated value using the stress-strain test results.

b. Sandstone. The unconfined compressive strengths for the competent and friable sandstones are quite different. The strength range for the competent sandstone is 4400 to 5210 psi, and the average is 4810 psi. The strength range for the friable sandstone is 540 to 1270 psi, and the average is 910 psi. No unconfined compressive test results for sandstone were previously reported.[2] The general mode of failure was by partial shear cone associated with vertical splitting; see Table 8 for details. Only two specimens of competent sandstone were suitable for triaxial testing, and $\sigma_3 = 33$ and 100 psi were used. The triaxial strength results appear reasonable. Mohr stress circles are presented in Plate 10. The ϕ is 65° 30' and c is equal to 560 psi (40.3 tsf). The stress-strain relations for the sandstone tested are presented in Plate 8. A similar E, as computed for the dolomite, was computed for these curves and is presented in Table 8 with the exception of specimen tested from hole CSR-7-74. The E range is from 1.79×10^6 psi to 4.00×10^6 psi, and the average is 2.77×10^6 psi. The specimen from CSR-7-74 consisted of both competent and friable sandstone. Strain gages were placed on the bottom half, which was the competent sandstone. For this specimen, the stress-strain relation was linear from 0 to 1270 and E was calculated in this stress range. The E calculated for this portion of the specimen was 2.00×10^6 psi and was included in obtaining the average E just mentioned. The ν ranged from 0.12 to 0.28, and the average is 0.21. There was no E or ν previously reported.[2] The average G is 1.35×10^6 psi and is a calculated value using the stress-strain test results.

Tensile

40. The results of the tensile tests are presented in Tables 7 and 8 for the dolomite and sandstone, respectively. There were no tensile strength test results previously reported.[2] The results will be discussed by rock type.

a. Dolomite. Two competent specimens and one specimen which was sandy and shaly were tested for comparison purposes. The average strength of the competent rock is 750 psi, and the strength of the sandy and shaly rock is 110 psi. Only the average strength of the competent dolomite was used in plotting Mohr's stress circles.

b. Sandstone. Two moderately competent specimens and one friable specimen were tested for comparison purposes. The average strength of the moderately competent rock is 230 psi, and the strength of the friable is 75 psi. Only the average strength of the competent sandstone was used in plotting Mohr's stress circle.

Petrographic Examination

41. Some of the cores were inspected in the laboratory, and limited tests of selected hand samples were made to verify field descriptions. Some of the clay from several cores was X-rayed to determine if clay found in the cores was of the swelling montmorillonitic type.

42. The first shipment of core received and examined had the pieces of core numbered (DC-1 through DC-3). Cores received later (DC-4, DC-7, DC-12, DC-14) did not have the pieces numbered. The following list identifies samples that were X-rayed, either by piece number or elevations.

Drill Hole No.	CL Serial No.	Description	Elevation, ft
CSR-16-74	CHI-6 DC-1		
	Piece 1	Bluish-green shale and sandstone	
	Piece 4	Calcareous sandstone	
	Piece 6	Sandy carbonate rock	413.5-403.3
	Piece 8	Shaly sandstone	
	Piece 19	Sandstone	
CSR-5-74	CHI-6 DC-2		
	Piece 7	Bluish-green clay	
	Piece 11	Sandy-shaly carbonate rock	418.7-406.7
	Piece 13	Calcareous-shaly sandstone	
	Piece 20	Shaly calcareous sandstone	
CSR-13-74	CHI-6 DC-3		
	Piece 1	Shaly and sandy carbonate rock	
	Piece 16	Bluish-green shale	424.5-414.3
CSR-7-74	CHI-6 DC-4	Clayey material	412.5
CSR-1-74	CHI-6 DC-7	Shale	415.5
CSR-11-74	CHI-6 DC-12	Clayey material	406.0
CSR-17-74	CHI-6 DC-14	Clayey material	431.2 412.8-412.5

43. The carbonate rock that was X-rayed was either dolomite or dolomite with a small amount of calcite; the sandstone was quartz and a minor amount of potassium feldspar. The clayey material in the shale and the clayey zones in the other specimens were predominantly micaceous

in cores DC-1 through DC-4, DC-7, DC-12, and DC-14. One exception was the sample from el 412.5 in hole CSR-7-74 which was a swelling montmorillonitic type with a small amount of clay-mica (i.e., clay-sized mica).

44. The micaceous material was usually characterized by a double peak at 10-Å and about 11-Å in the air-dry condition. These two separate peaks united at about 9.9-Å with the application of glycerol. Application of ethylene glycol produced two peaks at 10-Å and about 11.8-Å. This behavior was interpreted as indicating that the micaceous material was a mixture of clay-mica and partially degraded clay-mica which is in the process of becoming vermiculite and montmorillonite.

45. The presence of montmorillonitic clay in one sample probably represents a more complete stage of alteration than that shown by the other samples.

Peak and Residual Shear Strength

46. Two types of direct shear tests were conducted to ascertain peak strength of intact specimens and sliding friction characteristics of discontinuous specimens. Peak strengths were measured for dolomite and sandstones containing a concrete-rock interface, friable sandstones, and intact shales. Sliding friction properties were measured for specimens of dolomite, sandstone, and shale along either precut surfaces, shale-filled partings, or naturally occurring joints. The direct shear test results are tabulated in Table 10 and plotted on Plates 11, 12, and 13, respectively, for the dolomite, sandstone, and shale. Table 11 lists strength parameters determined from the various tests. Also included in Plates 11 through 13 are the strength envelopes obtained as part of an earlier test program.[2] Parameters from the previous test program are listed in Table 16.

47. The first intact shale tests performed in the single plane shear device, designated MRD and used for 6-in.-diameter low-strength rock, produced an envelope indicating the test was being performed too rapidly for excess pore pressure to dissipate. At the same time, the

question arose as to the maintenance of natural moisture conditions, particularly within the shale test specimens. For these reasons, it was decided to run slow (0.0001 in. per day) shear tests on submerged samples using a 3-in.-square shear box device designated S&PL device. The resultant envelope is more reasonable in that it demonstrates both a cohesion value and an angle of internal friction even though they are small.

48. The tests performed on intact friable sandstone and one natural joint in sandstone resulted in irregular envelopes. Envelopes were not drawn for the results of tests on the concrete to sawed sandstone and concrete to jointed dolomite because when plotted the data points were concentrated over a small stress range. All other shear tests produced smooth envelopes, indicating reliability in the testing technique. The direct shear tests conducted on the more competent dolomite and sandstone cores were run in yet another shear device than the above-described tests. The device is herein designated CL and used for 6-in.-diameter high-strength rock. The apparatus was found to be malfunctioning at various stages of the test. This fact was not evident until well into the testing program. Time did not allow equipment repair and modification. The test results are presented in Table 10, but are highly questionable and the data were not presented in Plates 11 and 12. The only additional shear strength testing which might be recommended are further attempts to obtain intact sandstone strengths and verification of the intact shale strengths. The nature of the proposed structure probably does not require knowledge of residual strengths for design.

49. <u>Concrete-rock interface</u>. A total of nine direct shear tests were conducted along the interface of concrete and rock. Six dolomite specimens (three each on natural joints and on precut surfaces) and three competent sandstones were prepared and tested.

50. In general, the peak shear stresses obtained from the concrete-on-rock specimens were higher than for any of the other dolomite or sandstone specimens tested in direct shear. Specimens of concrete to shale

were not tested due to insufficient shale specimens. It is anticipated that the relatively thin shale beds would be excavated and concrete not placed on these beds; therefore, testing is probably not warranted.

Sliding Friction

51. Rock-on-rock. The rock-on-rock friction tests were of three kinds: shear of sawed rock surfaces, shear of shale-filled partings, and shear of naturally occurring joints. The respective test results are labeled and presented in Plates 11, 12, and 13 and tabulated in Table 10.

52. A brief discussion of the rock-on-rock test results is given by rock type.

a. Dolomite. From Plate 11 and Table 11, it will be noted that there is a small difference in the angle of friction for the specimens sheared along a shale-filled parting (average ϕ = 32 degrees) and a precut shear plane (average 35 degrees). A ϕ = 26 degrees and c = 0.25 tsf are recommended as design values for rock-on-rock.

b. Sandstone. The data in Plate 12 and Table 11 show a large difference in ϕ for shearing along a natural joint (ϕ = 66 and 30.5 degrees) occurring in a competent sandstone. These data suggest a wide variation in ϕ ; however, the specimen with ϕ = 66 degrees had the maximum undulation that could be accommodated in the MRD shear device. The ϕ for the two specimens tested with a precut shear plane have an average value of 32 degrees. The ϕ for the specimens of dolomite tested with shale-filled partings and precut shear surfaces are quite similar to the ϕ obtained for the sandstone along precut shear planes. Recommended design values are ϕ = 30.5 degrees and c = 1.45 tsf.

c. Shale. Data presented in Plate 13 and Table 11 show the low ϕ for the intact shale specimens, hole CRS-10, el 432.4-431.5, run in the 3-in.-square shear box device. The ϕ and c for this specimen are 14.5 degrees and 0.12 tsf, respectively, and are lower than the ϕ = 19 degrees and c = 0.45 tsf obtained from the shale specimen tested with a precut shear plane. The ϕ = 19 degrees, measured for the natural joint, is considerably lower than the ϕ measured for the sandstone. The ϕ and c obtained from the intact specimen, 14.5 degrees and 0.12 tsf, are recommended as design values for intact shale.

Load-Deformation

53. Typical shear stress versus shear and normal deformation plots are presented in Plates 14 and 15. At comparable shear stresses, the shear deformation is greatest for the intact shale specimen. Specimens that exhibited lesser amounts of shear deformation, in descending order, were:

- a. Jointed shale.
- b. Jointed competent sandstone.
- c. Precut friable sandstone.
- d. Intact friable sandstone.
- e. Shale partings in dolomite.

The precut dolomite and concrete to rock specimens exhibited the least amount of shear deformation.

PART V: SUMMARY OF FOUNDATION CONDITION, DESIGN CONSIDERATIONS, RECOMMENDED DESIGN VALUES

Foundation Condition

Bedrock stratigraphy

54. Two distinct strata were encountered at the drilled site: the Ordovician St. Peter sandstone and the Shakopee dolomite. Both formations are interbedded with shale and friable sandstone having thicknesses of from 1/4 in. to about 12 in. The proposed founding elevation of 414 is locally within thin shale beds and friable sandstone lenses. It is recommended that these materials be removed during excavation and that the sounder dolomite be considered as the founding material. It is to be noted from Table 8 that the compressive acoustic wave velocities range from 7460 to 9020 ft/sec. This is within the ranges defined as being rippable to marginally rippable by Caterpillar Tractor Company.[10] A cheap, fast seismic refraction survey could verify that this is an economical excavation method. See Fig. 4.

Geologic cross sections

55. The cross sections clearly indicate the contact between the St. Peter sandstone and the Shakopee dolomite. The cross sections give an overview of the bedrock material and show the variation in thickness of the St. Peter sandstone and the various units within the Shakopee dolomite. The location of weak shale, clay, and friable sandstone units can be readily detected which should be beneficial in the design and the excavation phases of the proposed lock.

Discontinuities from core logs and televiewer

56. Strikes and dips of prominent discontinuities were measured from the available logs. Indications from surficial geology and core logs showed that many of the discontinuities shown on the televiewer log were, in fact, bedding features since they possessed southeasterly directed dips of less than 10 degrees. These features were eliminated from further analysis, and only discontinuities showing dips of 10 degrees or greater were studied further. A histogram of joint dip frequency obtained only from the core logs is shown in Plate 16. A total of

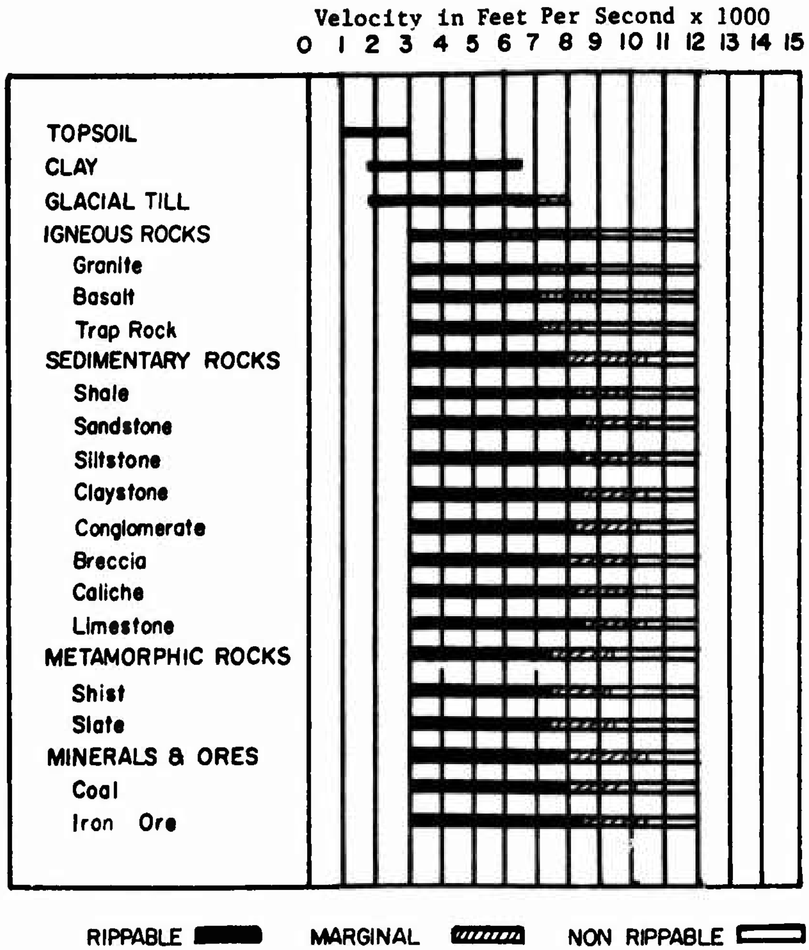

Figure 4. Rippability ranges for typical rock types - D9G caterpillar (from performance manual, Caterpillar Tractor Co., May 1970)

32 prominent discontinuities were thus measured and interpreted as joints cutting across bedding. These joints were then compared with descriptions shown on the core logs on the basis of dip depth. Of the 32 joints, 29 were found to adequately match structural breaks noted on the drill log. The standards were ± 5 degrees in dip and ± 1 ft in depth. Cursory examination of the core logs will show that 29 joints are only a small portion of all the structural breaks noted. Examination of the televiewer logs (copies of which are not included herein because of complexity of reproduction) shows that 29 joints are also a small portion of the features detected. In the absence of any other kind of data describing joint systems at the site, it will be assumed that the correlated joints are a sufficient sampling of the entire existing population insofar as orientations are concerned and, further, variations in the joint systems are only in the parameter of spatial location. In other words, the joint sets may be hypothesized to exist at any location on the project site as long as their dips and strikes are not altered. The actual location, character, and orientation parameters for the joints should be verified during construction. Figure 5 shows the traces of the prominent joints when extrapolated to a horizontal surface at el 440 (approximate top of rock). These joints all have dips ≥ 10 degrees, which correlate between the televiewer logs and the core logs. It is realized that no direct evidence justifies such extrapolation, but the overall frequency of joints for which correlation was not within the limits described above implies a high degree of probability for this type of configuration.

57. Examination of Fig. 5 indicates at least one frequent conjugate joint relationship (having strikes of azimuth 0 and 90 degrees) and certain other sets which may be conjugate in nature. The figure also shows a subparallel relationship between many of the traces and the orientation of a major regional structural feature, the Sandwich Fault Zone, located approximately 5 miles from the project. The depression or old river channel presented in Fig. 2 is also subparallel to certain joint traces and the Sandwich Fault Zone. A study of surface joint exposures over a large area would be necessary to conclusively relate the joint orientations to the fault zone orientation, but such a relationship within the structural

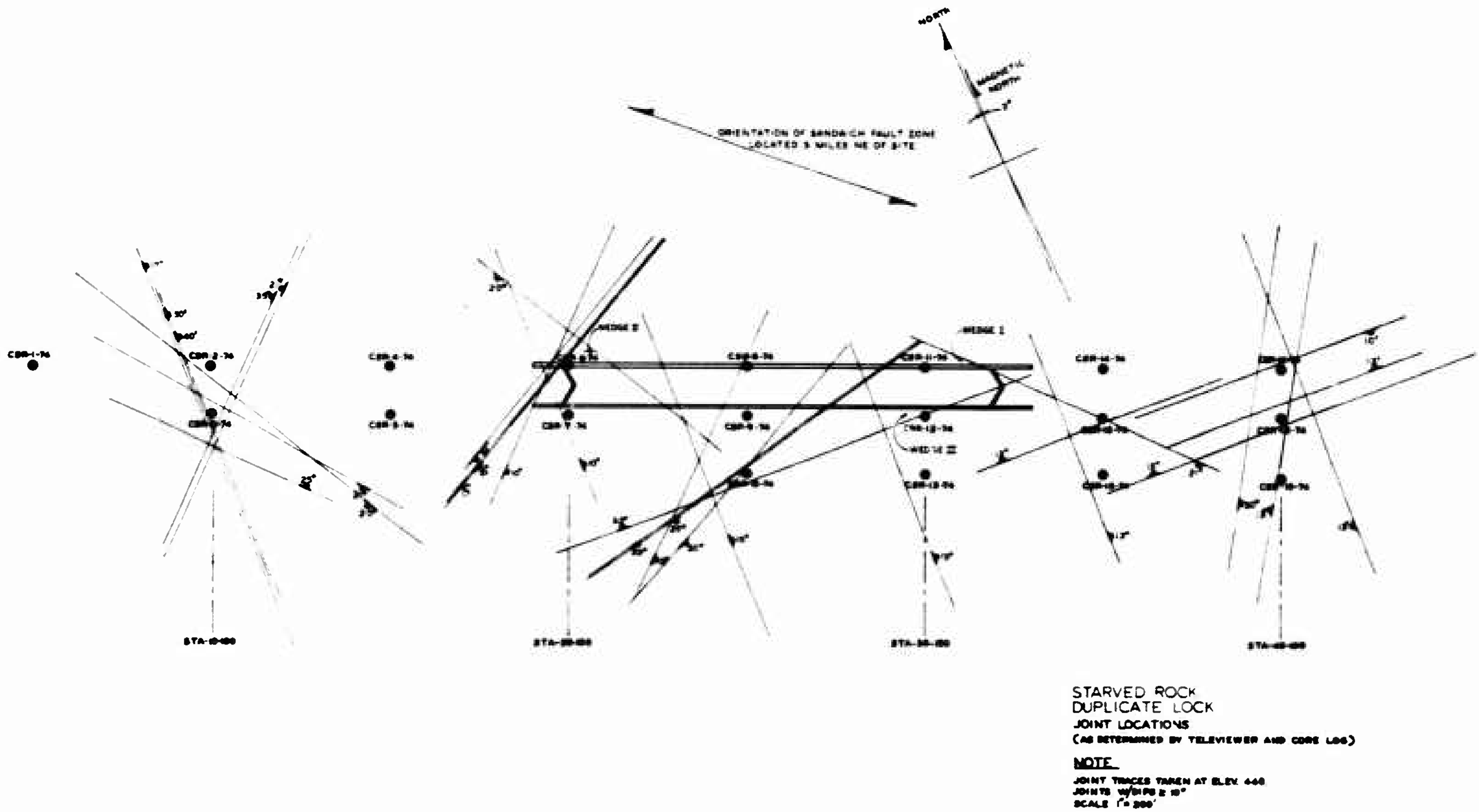

Figure 5. Plot of joint traces

fabric of the vicinity is probable. In no way is the above indicative of specific fault activity at the site.

58. In attempts to further delineate relative frequencies of joint orientation, several stereographic projections of the above data were prepared. These are shown in Plates 17 and 18. Both plates are plots of the same data but in differing forms. Plate 17 shows contoured frequencies of the unit vectors normal to the various planes (poles of the planes) located on both televiewer and core logs and is the customary presentation format for this type of data. Plate 18 shows contoured frequencies of the unit vectors parallel to the dips of the various planes. Plate 18 was made to distinguish between different low angle groups forming the high concentration near the middle of Plate 17. This separation was made necessary by the relatively low dip values of the joints located. The contoured values do not relate directly to the number of poles enclosed within any given area but are relative numbers resulting from the counting technique used. The poles for each plot were located and then a Kalsbeek counting net was superimposed on the diagrams. This net is comprised of overlapping areas, each of which is 1 percent of the total stereonet area. The number of points falling within each of these subareas was determined and the value contoured. The form of the counting net is such that each discrete pole is counted three times, thus uniformly increasing the density of points. The uniformity of the counting net form ensures a random distribution of counting areas when the basic plot is on an equal area stereographic projection.[11] Concentrations of joint orientations are as follows in order of frequency:

Dip Degrees	Dip Direction Degrees Azimuth
16	90
17	234
10	0
20	322
8	151

Design Considerations

Bearing capacity

59. The core logs were examined for indications of conditions critical to bearing capacity. These indications included thick interbedded zones of shale and sandstone, reworked or brecciated dolomite, and cavernous zones. Most of the friable sandstone observed in the laboratory fell apart under slight hand pressure. The elevation intervals tabulated below were observed. The proposed floor of most of the lock lies within a 5-ft-thick zone of interbedded shale and sandstone (in particular the riverward wall exemplified by borings CSR-3, 5, 7, 9, 12, 15, and 18 (Plate 2)).

Drill Hole No.	Elevation, ft
CSR-4-74	413.0-416.0
CSR-5-74	409.5-413.7
CSR-6-74	419.2-423.5
CSR-6-74	413.0-413.2
CSR-16-74	412.2-414.0
CSR-18-74	412.6-414.0

60. The numerous fractured zones, reworked dolomite, and brecciated dolomite described in the core logs below the proposed floor elevation are interpreted as ancient reef collapses and the results of heavy wave action. They are discontinuous over the site and, further, the fragments are of such an angular nature that they should pose no problem except localized concrete bonding flaws. The "cavities" and drill rod drop zones are too small and discontinuous to be interpreted as being critical to structural bearing capacity. No fault zones have been detected in the core logs and no indications of faulting are indicated by stratigraphic bed correlations.

61. The lowest compressive strengths obtained during this investigation for the dolomite and the sandstone are presented as bearing capacity values. The bearing capacity of the dolomite, the friable sandstone, and the competent sandstone is 350 tsf, 39 tsf, and 320 tsf, respectively. No compressive strength tests were conducted on the shale for reasons previously

mentioned. This parameter could be measured using any available shale samples stored at NCC or from samples recovered during construction.

Shear failure

62. Probable joint configurations were deduced from a combination of core logs and geophysical logging information. If the geometrical configuration of joint sets form possible rock wedges, then the primary consideration in foundation stability analyses will be sliding along discontinuities. Kinematically and geometrically, the simplest potential sliding mass is a wedge bounded by the top of rock, the excavation face, and two intersecting cross-bed joints. The formation of potential wedges is illustrated in Fig. 5 using only the 29 prominent joints. Since these observed joints are considered to be only a small sampling of the total number of joints present at the site, numerous such wedges may occur. Data concerning the orientation of joint sets that can form these potential wedges are shown in Plates 17 and 18. No information is contained in this report relating to possible wedge sizes; this information must be developed from observations of joint frequency or joint spacing within each joint set. The extremely low strength of the intact shale parallel to its bedding prescribes that, for conservative design, any shale bed be treated as a potential sliding plane parallel to its bedding. When shale beds are considered as potential sliding surfaces, the geometries of the vulnerable rock masses become more complex. In addition, since the present lock will remain operational while the proposed lock is being constructed, then horizontal sliding along shale beds, daylighted by the excavation, becomes possible because the proposed excavation will be taken below the foundation of the existing lock. Removal of the buttressing effect of intact rock during excavation can eliminate the kinematic requirement for intersecting joints, and high chamber water levels can provide the driving force.[12]

63. This discussion indicates that potential failure involving wedges and horizontal sliding along shale layers should be given design considerations. Because of the potential sliding on preexisting discontinuities, which cannot be completely described from borehole data, it is imperative that detailed geologic structure mapping be performed

during all stages of the excavation.[13] The information would be immediately available for incorporation in temporary or permanent block sliding remedial measures. Slope-indicating devices or tiltmeter installations in the intervening space between locks could give advance warning of block slippage. Tensioned rock bolts and/or tendons can stabilize block slippages.

Recommended Design Values

64. Design should consider rock type and the various bedrock structural features described herein. Guidance is presented in the following tabulation as to proper choice of design parameters.

	Dolomite	Friable Sandstone	Competent Sandstone	Shale
Index Properties:				
Dry Unit Weight, lb/ft^3	157.0(1)	127.7	138.3	110.4
Wet Unit Weight, lb/ft^3	162.0(1)	140.2	147.7	129.9
Porosity, pct	9.6	20.3	16.8	34.8
Bearing Capacity, tsf	350	39	320	--
Tensile Strength, psi	110	75	175	--
Shear Strength:				
Intact	c = 90.0 tsf(2) ϕ = 56°	--(3)	c = 40.3 tsf ϕ = 65.5°	c = 0.12 tsf ϕ = 14.5°
Natural Joint	--	--	c = 1.45 tsf ϕ = 30.5°	c = 0.0 ϕ = 38°
Shale-Filled Parting	c = 0.25 tsf ϕ = 26°	--	--	--
Precut, Rock-on-Rock	c = 0.0 ϕ = 31°	c = 0.0 ϕ = 33.5°	c = 0.0 ϕ = 31°	c = 0.45 tsf ϕ = 19°
Concrete on Rock(4)	c = 1.60 tsf ϕ = 63°	--	--	--
Modulus of Elasticity $\times 10^6$ psi	1.82	--(4)	2.00	--(6)
Poisson's Ratio	0.13	--(5)	0.12	--(7)
Shear Modulus $\times 10^6$	0.65	--	0.69	--

(1) Lower value previously reported by NCC; density γ_d = 142 lb/ft^3; γ_s = 146 lb/ft^3.
(2) Lower value previously reported by NCC; c = 72.0 tsf; ϕ = 54.5°.
(3) Lower value previously reported by NCC; c = 1.08 tsf; ϕ = 38° (moderately weak).
(4) Value previously reported by NCC; 1.40 x 10^6 psi.
(5) Value previously reported by NCC; 0.30.
(6) Value previously reported by NCC; 0.36 x 10^6 psi.
(7) Value previously reported by NCC; 0.225.

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TABLE 1

Recommended Laboratory Testing Program
Duplicate Lock at Starved Rock, Illinois

Test	Remarks
Index Properties	Wet and dry unit weights; specific gravity of solids; moisture content; porosity; compressional wave velocity.
Unconfined Compression	Stress-strain diagram, Young's modulus of elasticity, and Poisson's ratio.
Triaxial	Undrained tests at 33, 66, and 100 psi confining pressure, shear modulus and stress-strain diagrams, Young's modulus, and Poisson's ratio.
Tensile Splitting	Strength only. Stress-strain diagram for competent specimens.
Direct Shear:	
Intact Rock	Single or double plane for high-strength specimens.
Intact Shales or Friable Rock	Peak strength.
Precut and Jointed Surfaces	Sliding friction (three-stage multiloading test).
Concrete-Rock Interface	Peak strength.
Cursory Petrographic Examination	Suspected clay specimens.

TABLE 2

Brecciated Rock Described for Starved Rock Proposed Duplicate Lock Site

Drill Hole No. CSR-	Box No.	Elevation, ft	Depth, ft	Term Used In Reference 1	Description on Core Logs in Reference 1	WES Term
			Holes on Bedrock Structural Characteristics Section A-A			
2-74	6	417.9-417.6	37.6-37.9	Recemented Dol	Recemented Dol, lt gry, vuggey reef type calc pyr, glau.	Dol Breccia
2-74	3*	435.0-430.4	20.7-25.3	Recemented	Dol lt gry, recemented w/calc material & bl-grn Sh	--
4-74	6	417.6-413.2	37.9-42.3	--	Ss, lt gry, fine to crse, calc, silty, glau friable, X-bedded w/Dol fragm in Ss matrix	Dol Breccia
6-74	5	423.5-419.2	33.2-37.5	Looks like the rock was cav-erous	Ss, extensive Ch Nod w/bl-grn Sh veins, V. fri, pieces of dol 1"-4" in length intermixed w/Ss & cemented w/Sh.	Ss Breccia
6-74	6	419.2-413.7	37.5-43.0	--	Ss wh silty w/amorphous ch & bl-grn Sh veins w/pieces of Dol in Ss & Sh matrix	Ss Breccia
6-74	7	408.7-403.95	48.0-52.75	Recemented	Lt gry, pry, glau, sandy, fractured & recemented w/bl-grn Sh	Dol Breccia
8-74	6	420.0-419.8	43.6-43.8	Reworked	Reworked Dol w/glau chlorite cement	Dol Breccia
11-74	3*	436.3-431.7	30.3-34.9	Reworked Dol Zones	Dol, lt-dk gry, cherty, glauic, w/bl-grn Sh veins, A few zones have reworked Dol	--
11-74	6	421.3-416.4	45.3-50.2	Reworked Dol	Dol, lt-dk gry, clauic, slightly sandy, reworked Dol w/bl-grn Sh veins	Dol Breccia

* Box not received at WES Laboratory.

(Continued)

TABLE 2 (CONTINUED)

Drill Hole No. CSR-	Box No.	Elevation, ft	Depth, ft	Term Used In Reference 1	Description on Core Logs in Reference 1	Wes Term
			Holes on Bedrock Structural Characteristics Section B-B			
12-74	6	415.6-415.1	52.1-52.6	Reworked Dol	Dol, sandy, reworked Dol, w/Ss cement calc w/bl-grn Sh SBP's	Dol Breccia
12-74	7	414.3-411.3	53.4-56.4	Reworked Dol	Ss, lt gry, silty, glau, w/bl-grn Sh veins + ch nod w/reworked Dol	Dol Breccia
18-74	7	418.0-414.0	41.5-45.5	Reworked Dol Areas	Dol, lt-dk gry, glau, w/reworked Dol areas and bl-grn Sh beds	Dol Breccia
18-74	9	408.8-407.5	50.7-52.0	Reworked Area	Ss, med-fine grain, calc, w/pieces of Dol (reworked area)	Dol Breccia
			Holes on Bedrock Structural Characteristics Section C-C			
10-74	3*	431.3-430.4	36.1-37.0	Fault Breccia	Fault breccia glau matrix, Dol argillaceous	--
19-74	9	411.5-411.4	54.9-55.0	Reworked Areas	Dol, lt gry, slightly glau, sandy w/ reworked areas, cherty	Dol Breccia

* Not received at WES Laboratory.

TABLE 3

Core Received at WES From Proposed Duplicate Lock Site, Starved Rock Lock and Dam

WES Reference	Drill Hole No.	Date Received	Core Diam in.	Box No.	Depth, ft	Elevation: For Depth Intervals, ft	Elevation: Top of Hole, ft	Core Resealed
CHI-6 DC-1(A)	CSR-16-74	6-25-74	4	6 of 12	42.0-47.0	413.5-403.5	455.5	
CHI-6 DC-1(B)	CSR-16-74	6-25-74	4	7 of 12	47.2-52.2			
CHI-6 DC-2(A)	CSR-5-74	6-25-74	6	4 of 10	39.85-45.35	418.7-406.7	458.6	
CHI-6 DC-2(B)	CSR-5-74	6-25-74	6	5 of 10	45.35-51.9			
CHI-6 DC-3(A)	CSR-13-74	6-25-74	6	5	42.55-48.75	424.5-414.3	467.1	
CHI-6 DC-3(B)	CSR-13-74	6-25-74	6	6	48.75-52.80			
CHI-6 DC-4(A)	CSR-7-74	7-30-74	6	3 of 11	32.9-37.9	432.2-407.3	465.1	
CHI-6 DC-4(B)	CSR-7-74	7-30-74	6	4 of 11	38.8-42.5			
CHI-6 DC-4(C)	CSR-7-74	7-30-74	6	5 of 11	42.5-48.0			
CHI-6 DC-4(D)	CSR-7-74	7-30-74	6	6 of 11	48.0-53.75			
CHI-6 DC-4(E)	CSR-7-74	7-30-74	6	7 of 11	53.75-57.75			
CHI-6 DC-5(A)	CSR-9-74	7-30-74	6	3 of 11	33.7-39.1	432.8-409.3	466.5	X
CHI-6 DC-5(B)	CSR-9-74	7-30-74	6	4 of 11	39.1-43.8			X
CHI-6 DC-5(C)	CSR-9-74	7-30-74	6	5 of 11	43.8-47.8			
CHI-6 DC-5(D)	CSR-9-74	7-30-74	6	6 of 11	47.8-52.5			X
CHI-6 DC-5(E)	CSR-9-74	7-30-74	6	7 of 11	52.5-57.2			
CHI-6 DC-6(A)	CSR-10-74	7-30-74	6	3	34.16-39.4	433.2-412.1	467.4	
CHI-6 DC-6(B)	CSR-10-74	7-30-74	6	4	39.4-45.6			X
CHI-6 DC-6(C)	CSR-10-74	7-30-74	6	5	45.6-50.3			
CHI-6 DC-6(D)	CSR-10-74	7-30-74	6	6	50.3-55.3			
CHI-6 DC-7(A)	CSR-1-74	9-11-74	6	6 of 13	34.6-39.6	420.9-396.3	455.5	
CHI-6 DC-7(B)	CSR-1-74	9-11-74	6	7 of 13	39.6-44.4			
CHI-6 DC-7(C)	CSR-1-74	9-11-74	6	8 of 13	44.4-49.1			
CHI-6 DC-7(D)	CSR-1-74	8-29-74	6	9 of 13	49.1-53.9			X
CHI-6 DC-7(E)	CSR-1-74	9-11-74	6	10 of 13	53.9-59.2			

TABLE 3 (CONTINUED)

WES Reference	Drill Hole No.	Date Received	Core Diam in.	Box No.	Depth, ft	Elevation: For Depth Intervals, ft	Elevation: Top of Hole, ft	Core Resealed
CHI-6 DC-8(A)	CSR-3-74	8-29-74	6	3 of 11	31.0-36.0			X
CHI-6 DC-8(B)	CSR-3-74	8-29-74	6	4 of 11	36.0-40.8			X
CHI-6 DC-8(C)	CSR-3-74	8-29-74	6	5 of 11	40.8-45.8	423.9-399.5	454.9	X
CHI-6 DC-8(D)	CSR-3-74	8-29-74	6	6 of 11	45.8-50.7			
CHI-6 DC-8(E)	CSR-3-74	8-29-74	6	7 of 11	50.7-55.4			X
CHI-6 DC-9(A)	CSR-4-74	9-11-74	6	5 of 18	32.8-37.9			X
CHI-6 DC-9(B)	CSR-4-74	9 11-74	6	6 of 18	37.9-42.3			
CHI-6 DC-9(C)	CSR-4-74	9-11-74	6	7 of 18	42.3-47.6	422.7-398.7	455.5	X
CHI-6 DC-9(D)	CSR-4-74	9-11-74	6	8 çf 18	47.6-52.2			X
CHI-6 DC-9(E)	CSR-4-74	8-29-74	6	9 of 18	52.2-56.8			
CHI-6 DC-10(A)	CSR-6-74	8-29-74	6	5 of 13	33.2-37.5			X
CHI-6 DC-10(B)	CSR-6-74	8-29-74	6	6 of 13	37.5-43.0			X
CHI-6 DC-10(C)	CSR-6-74	8-29-74	6	7 of 13	43.0-48.0	423.5-399.1	456.7	X
CHI-6 DC-10(D)	CSR-6-74	8-29-74	6	8 of 13	48.0-52.75			X
CHI-6 DC-10(E)	CSR-6-74	8-29-74	6	9 of 13	52.75-57.6			X
CHI-6 DC-11(A)	CSR-2-74	9-11-74	6	6 of 14	32.8-37.8			
CHI-6 DC-11(B)	CSR-2-74	9-11-74	6	7 of 14	37.8-42.7			X
CHI-6 DC-11(C)	CSR-2-74	9-11-74	6	8 of 14	42.7-47.75	422.9-399.2	455.7	X
CHI-6 DC-11(D)	CSR-2-74	9-11-74	6	9 of 14	47.75-52.6			
CHI-6 DC-11(E)	CSR-2-74	9-11-74	6	10 of 14	52.6-56.5			X
CHI-6 DC-12(A)	CSR-11-74	9-11-74	6	4 of 11	34.9-40.7			X
CHI-6 DC-12(B)	CSR-11-74	9-11-74	6	5 of 11	40.7-45.3	431.7-411.3	466.6	X
CHI-6 DC-12(C)	CSR-11-74	9-11-74	6	6 of 11	45.3-50.2			X
CHI-6 DC-12(D)	CSR-11-74	9-11-74	6	7 of 11	50.2-55.3			X

(Continued)

TABLE 3 (CONTINUED)

WES Reference	Drill Hole No.	Date Received	Core Diam in.	Box No.	Depth, ft	Elevation: For Depth Intervals, ft	Elevation: Top of Hole, ft	Core Resealed
CHI-6 DC-13(A)	CSR-12-74	9-11-74	6	3 of 11	30.6-35.8			X
CHI-6 DC-13(B)	CSR-12-74	9-11-74	6	4 of 11	35.8-44.7			X
CHI-6 DC-13(C)	CSR-12-74	9-11-74	6	5 of 11	44.7-48.5	437.1-410.5	467.7	X
CHI-6 DC-13(D)	CSR-12-74	9-11-74	6	6 of 11	48.5-53.3			X
CHI-6 DC-13(E)	CSR-12-74	9-11-74	6	7 of 11	53.3-57.2			
CHI-6 DC-14(A)	CSR-17-74	9-11-74	6	4 of 10	37.3-42.8			
CHI-6 DC-14(B)	CSR-17-74	9-11-74	6	5 of 10	42.8-47.75	428.9-412.6	466.2	
CHI-6 DC-14(C)	CSR-17-74	9-11-74	6	6 of 10	47.75-53.6			
CHI-6 DC-15(A)	CSR-8-74	9-24-74	6	4 of 12	32.3-36.6			X
CHI-6 DC-15(B)	CSR-8-74	9-24-74	6	5 of 12	36.6-41.3			X
CHI-6 DC-15(C)	CSR-8-74	9-24-74	6	6 of 12	41.3-45.9	431.3-408.4	463.6	X
CHI-6 DC-15(D)	CSR-8-74	9-24-74	6	7 of 12	45.9-50.7			X
CHI-6 DC-15(E)	CSR-8-74	9-24-74	6	8 of 12	50.7-55.2			X
CHI-6 DC-16(A)	CSR-14-74	9-24-74	6	4 of 12	33.3-38.2			
CHI-6 DC-16(B)	CSR-14-74	9-24-74	6	5 of 12	38.2-44.3			X
CHI-6 DC-16(C)	CSR-14-74	9-24-74	6	6 of 12	44.3-48.7	433.8-408.6	467.1	X
CHI-6 DC-16(D)	CSR-14-74	9-24-74	6	7 of 12	48.8-53.4			X
CHI-6 DC-16(E)	CSR-14-74	9-24-74	6	8 of 12	53.4-58.5			X
CHI-6 DC-17(A)	CSR-18-74	10-25-74	6	5 of 13	34.2-37.2			X
CHI-6 DC-17(B)	CSR-18-74	10-25-74	6	6 of 13	37.2-41.5			X
CHI-6 DC-17(C)	CSR-18-74	10-25-74	6	7 of 13	41.5-45.5	425.3-403.8	459.5	
CHI-6 DC-17(D)	CSR-18-74	10-25-74	6	8 of 13	45.5-50.75			X
CHI-6 DC·17(E)	CSR-18-74	10-25-74	6	9 of 13	50.75-55.7			X
CHI-6 DC-18(A)	CSR-19-74	10-25-74	6	5 of 10	35.1-40.1			X
CHI-6 DC-18(B)	CSR-19-74	10-25-74	6	6 of 10	40.1-44.3			X
CHI-6 DC-18(C)	CSR-19-74	10-25-74	6	7 of 10	44.3-49.3	431.4-407.6	466.5	X
CHI-6 DC-18(D)	CSR-19-74	10-25-74	6	8 of 10	49.3-54.0			X
CHI-6 DC-18(E)	CSR-19-74	10-25-74	6	9 of 10	54.0-58.9			

TABLE 4

Test Assignment Locations for Sandstone

Drill Hole No.	Elevation ft	Physical Condition	Index Property	Unconfined Compression	Triaxial	Tensile	Direct Shear: Concrete to Rock	Direct Shear: Intact	Direct Shear: Precut Surface	Direct Shear: Jointed
CSR-1-74	420.3-420.3	C				X				
CSR-1-74	420.0-419.0	C	X		X					
CSR-1-74	415.3-414.3	C		X						
CSR-1-74	412.8-412.1	C		X						
CSR-1-74	406.3-405.6	C	X					X		
CSR-2-74	416.4-415.7	C								X
CSR-2-74	413.9-413.4	C							X	X
CSR-2-74	407.8-407.2	C	X					X		
CSR-2-74	407.2-406.8	fri						X	X	
CSR-5-74	415.0-414.6	fri						X		
CSR-5-74	412.2-411.9	C							X	
CSR-5-74	411.9-411.5	mod fri	X							
CSR-6-74	419.2-418.7	fri						X		
CSR-6-74	417.2-417.0	fri					X			
CSR-7-74	414.6-413.6	fri		X						
CSR-7-74	413.3-413.1	fri					X			
CSR-8-74	424.5-424.1	mod C				X				
CSR-8-74	417.7-417.4	fri						X		
CSR-9-74	432.0-431.1	mod C	X							
CSR-9-74	431.0-430.0	C	X							
CSR-10-74	412.8-412.4	C	X							
CSR-12-74	414.0-413.2	V fri		X						
CSR-12-74	413.2-413.0	fri					X			
CSR-14-74	410.3-409.7		X					X		
CSR-18-74	410.5-409.9	fri				X				

Note: C = Competent, mod = moderately, fri = friable, V = Very.

TABLE 5

Test Assignment Location for Dolosite

Drill Hole No.	Elevation ft	Index Property	Unconfined Compression	Triaxial	Tensile	Direct Shear: Concrete to Rock	Direct Shear: Intact	Direct Shear: Precut Surface	Direct Shear: Filled Partings	Petrographic Examination
CSR-1-74	417.4-416.7	X					X			
CSR-1-74	398.1-394.4	X					X			
CSR-2-74	417.4									X
CSR-2-74	405.1-404.7							X		
CSR-2-74	401.8-401.2								X	
CSR-4-74	410.1-409.3				X					
CSR-4-74	408.8-406.9		X							
CSR-4-74	404.5-403.7	X					X			
CSR-4-74	402.9									X
CSR-5-74	418.8-418.3	X								
CSR-5-74	410.9-409.9			X						
CSR-5-74	408.7-407.9			X						
CSR-5-74	418.7-406.7									X
CSR-6-74	408.7-408.5					X				
CSR-6-74	407.9-407.6					X				
CSR-7-74	430.3-429.3		X							
CSR-7-74	418.1-418.0							X	X	
CSR-8-74	428.8-428.2								X	
CSR-8-74	409.6-408.6	X								
CSR-9-74	422.1-421.7	X								
CSR-10-74	421.8-421.6	X								
CSR-10-74	421.6-420.8			X						
CSR-12-74	411.0-410.8					X				
CSR-12-74	410.7-410.5					X				
CSR-12-74	407.1									X

(Continued)

TABLE 5 (Continued)

Drill Hole No.	Elevation ft	Index Property	Unconfined Compression	Triaxial	Tensile	Direct Shear: Concrete to Rock	Direct Shear: Intact	Direct Shear: Precut Surface	Direct Shear: Filled Partings	Petrographic Examination
CSR-13-74	422.1-421.3				X					
CSR-13-74	415.1-414.7					X				
CSR-13-74	414.7-414.5					X				
CSR-13-74	424.5-414.3									X
CSR-14-74	432.1									X
CSR-14-74	413.7-413.4									X
CSR-16-74	413.5-403.3									X
CSR-17-74	426.8-425.8		X							
CSR-17-74	420.3-419.4				X					
CSR-19-74	408.8-408.1	X								

TABLE 6

Test Assignment Locations for Shale

Drill Hole No.	Elevation ft	Index Property	Direct Shear Intact	Direct Shear Precut Surface	Direct Shear Jointed
CSR-3-74	405.0-405.7			X	
CSR-8-74	410.1-409.7		X		
CSR-10-74	432.3-431.5		XX		
CSR-14-74	432.1				X
CSR-19-74	422.8	X			
CSR-19-74	414.8	X			

TABLE 7

Test Results of Dolomite Cores, Starved Rock Lock and Dam

Drill Hole No.	Elevation ft	Index Properties for Dolomite						
		Wet Unit Weight γ_m, lb/ft^3	Dry Unit Weight γ_d, lb/ft^3	Specific Gravity of Solids G_s, g/cc	Moisture Content w, pct	Porosity n, pct	Degree of Saturation S_r, pct	Comp Wave Velocity Vp, ft/sec
CSR-1-74	417.4-416.7				3.7			
CSR-1-74	398.1-397.4				6.0			
CSR-4-74	410.1-409.3				8.6			
CSR-4-74	407.8-406.9				2.5			
CSR-4-74	404.5-403.7				2.5			
CSR-5-74	418.8-418.3	162.0	157.0	2.79	3.3	9.6	83	
CSR-5-74	410.9-409.9				5.6			13,560
CSR-5-74	408.7-407.9				5.8			15,290
CSR-7-74	430.3-429.3				8.2			
CSR-8-74	409.6-408.6							11,200
CSR-9-74	422.1-421.7	164.5	158.9	2.83	3.7	9.6	94	
CSR-10-74	421.8-421.6	167.0	165.1	2.81	1.2	5.8	52	
CSR-10-74	421.6-420.8				4.4			16,490
CSR-13-74	422.1-421.3				3.4			
CSR-17-74	426.8-425.8				6.0			
CSR-17-74	420.3-419.4				1.4			
CSR-19-74	408.8-408.1				3.1			
Average		164.5	160.3	2.81	4.3	8.3	76	14,140
Range		5.0	8.1	0.04	4.8	3.8	42	5,290

TABLE 7 (Continued)

Drill Hole No.	Elevation ft	Triaxial: Minor Prin Stress σ_3, psi	Major Prin Stress σ_1, psi	Prin Stress Difference $\sigma_1 - \sigma_3$, psi	Modulus of Elasticity E, x 10^6 psi	Poisson's Ratio, ν	Shear Modulus G, x 10^6 psi	Tensile Strength T_o, psi	Remarks
CSR-1-74	417.4-416.7								
CSR-1-74	398.1-397.4								
CSR-4-74	410.1-409.3							110	Sandy, shaley, and vuggy
CSR-4-74	407.8-406.9	0	7250	7250	4.00	0.13			Shear failure at 60° from horiz
CSR-4-74	404.5-403.7								
CSR-5-74	418.8-418.3								
CSR-5-74	410.9-409.9	33	9540	9510	1.82	0.26	0.65		Shear failure at 63° from horiz
CSR-5-74	408.7-407.9	66	9260	9190	3.33	0.29	1.29		Shear failure at 57° from horiz
CSR-7-74	430.3-429.3	0	6800	6800	3.33	0.27			Shear failure at 62° from horiz
CSR-8-74	409.6-408.6								
CSR-9-74	422.1-421.7								
CSR-10-74	421.8-421.6								
CSR-10-74	421.6-420.8	100	9730	9630	3.20	0.28	1.25		Shear failure at 57° from horiz
CSR-13-74	422.1-421.3							830	
CSR-17-74	426.8-425.8	0	4840		5.12	0.32			Shear failure at 63° from horiz
CSR-17-74	420.3-419.4							660	
CSR-19-74	408.8-408.1								
Average		0	6300		3.47	0.26	1.06		
Range			2410		3.30	0.19	0.64		
							Competent:	750	
							Sandy & Shaley:	110	

TABLE 8

Test Results of Sandstone Cores, Starved Rock Lock and Dam

Drill Hole No.	Elevation ft	Index Properties: Wet Unit Weight γ_m, lb/ft^3	Dry Unit Weight γ_d, lb/ft^3	Specific Gravity of Solids G_s, g/cc	Moisture Content w, pct	Porosity n, pct	Degree of Saturation S_r, pct	Comp Wave Velocity V_p, ft/sec	Remarks
CSR-1-74	420.3-420.0				7.1				
CSR-1-74	420.0-419.0				6.5			9020	
CSR-1-74	415.3-414.3				6.2				
CSR-1-74	412.8-412.1				6.8				
CSR-1-74	406.3-405.6				7.0				
CSR-2-74	407.8-407.2				11.0				Friable
CSR-5-74	438.9-438.5	147.7	138.3	2.66	6.9	16.8	89.0		Calcareous matrix
CSR-7-74	414.6-413.6				12.8				Friable
CSR-8-74	424.5-424.1				11.0				
CSR-9-74	432.0-431.1	140.2	127.7	2.58	9.5	20.3	99.0		Slightly to moderately friable
CSR-9-74	431.0-430.0				9.7			7460	
CSR-10-74	412.8-412.4	152.0	143.9	2.67	5.4	13.1	99.0	8730	Calcareous matrix
CSR-12-74	414.0-413.2				14.4				Friable
CSR-14-74	410.3-409.7				14.0				Very friable
CSR-18-74	410.5-409.9				13.9				Friable, Ss, silty and shaly
Competent:	Average	149.9	141.1	2.67	7.4	15.0	99.0	8400	
	Range	4.30	5.6	0.01	5.6	3.7	1.0	1560	
Friable:	Average	140.2	127.7	2.58	11.0	20.3	99.0		
	Range	--	--	--	6.0	--	--		

(Continued)

TABLE 8 (Continued)

Drill Hole No.	Elevation ft	Triaxial: Minor Prin Stress σ_3, psi	Triaxial: Major Prin Stress σ_1, psi	Triaxial: Prin Stress Difference $\sigma_1-\sigma_3$, psi	Triaxial: Modulus of Elasticity $E, \times 10^6$ psi	Triaxial: Poisson's Ratio, ν	Triaxial: Shear Modulus $G, \times 10^6$ psi	Tensile Strength T_o, psi	Remarks
CSR-1-74	420.3–420.0							280	Fine grained calc and X-bedded.
CSR-1-74	420.0–419.0	100	8460	8360	3.30	0.12	2.00		Partial shear failure w/vertical splitting.
CSR-1-74	415.3–414.3	0	4400	4400	4.00	0.25			Shear failure at 62° from horizontal.
CSR-1-74	412.8–412.1	0	5210	5210					Fine to coarse calc.
CSR-1-74	406.3–405.6								
CSR-2-74	407.8–407.2								
CSR-5-74	438.9–438.5								
CSR-7-74	414.6–413.6	0	1270	1270	2.00	0.2			Top half specimen fri; tested w/open horz BP; shear cone in top; strain gages on bottom half of core which was moderately competent.
CSR-8-74	424.5–424.1	0						175	Interbedded with shale.
CSR-9-74	432.0–431.1								
CSR-9-74	431.0–430.0	33	7240	7210	1.79	0.28	0.69		Partial shear cone w/vertical splitting.
CSR-10-74	412.8–412.4								
CSR-12-74	414.0–413.2	0	540	540	--	--			Very fri; partial shear failure w/vertical splitting; strain gages inoperative.
CSR-14-74	410.3–409.7								
CSR-18-74	410.5–409.9							75	Fri, Ss, silty and shaley.
Average:	Competent:	0	4810				Competent:	230	
	Friable:	0	910				Friable:	79	

TABLE 9

Test Results of Shale Cores, Starved Rock Lock and Dam

Drill Hole No.	Elevation ft	Index Properties: Wet Unit Weight γ_m, lb/ft^3	Dry Unit Weight γ_d, lb/ft^3	Specific Gravity of Solids G_s, g/cc	Moisture Content w, pct	Porosity n, pct	Degree of Saturation S_r, pct
CSR-19	422.8	129.9	110.4	2.72	17.7	34.8	90
CSR-19	414.8	138.9	124.2	2.69	11.8	26.6	89
Average		134.4	117.3	2.71	14.8	30.7	89
Range		9.0	13.8	0.03	5.9	8.2	1

TABLE 10

Laboratory Test Results - Starved Rock Replacement Lock
Single Plane Shear Tests

Lithology	Drill Hole No. CSR-	Elevation ft	Test	Normal Stress tsf	Peak Shear Stress tsf
Dolomite	2-74	401.2-401.8	Filled parting[1]	2.37	2.56
				4.75	4.28
				7.20	6.03
Dolomite	7-74	418.0-418.1	Filled parting	2.37	1.61
				4.75	2.56
				7.20	3.72
Dolomite	8-74	428.2-428.8	Filled parting	2.37	2.83
				4.75	3.69
				7.20	5.22
Dolomite	7-74	417.8-418.1	Precut	2.37	1.42
				4.75	2.77
				7.20	4.35
Dolomite	2-74	404.7-405.1	Precut	2.37	1.80
				4.75	3.46
				7.20	5.55
Concrete to Dolomite	6-74	408.5-408.7	Natural joint	2.37	7.53
Concrete to Dolomite	13-74	414.7-415.1	Natural joint	4.75	10.80
Concrete to Dolomite	12-74	410.5-410.7	Natural joint	7.20	20.33
Concrete to Dolomite	6-74	407.6-407.9	Precut	2.37	24.28
Concrete to Dolomite	13-75	414.5-414.7	Precut	4.75	20.13
Concrete to Dolomite	12-74	410.8-411.0	Precut	7.20	26.33

(Continued)

TABLE 10 (Continued)

Lithology	Drill Hole No. CSR-	Elevation ft	Test	Normal Stress tsf	Peak Shear Stress tsf
Dolomite[2]	1-74	416.7-417.4	Intact	2.37	39.96
Dolomite[2]	1-74	397.4-398.1	Intact	2.37	44.26
Dolomite[2]	4-74	403.7-404.5	Intact	4.75	60.12
Dolomite[2]	19-74	408.1-408.8	Intact	7.20	63.35
Sandstone, C[2]	2-74	407.2-407.8	Intact	0.0	7.92
Sandstone, C[2]	1-74	405.6-406.3	Intact	2.37	25.20
Sandstone, C[2]	14-74	409.7-410.5	Intact	7.20	49.17
Sandstone, Fri[3]	6-74	418.7-419.2	Intact	2.37	4.23
Sandstone, Fri	5-74	415.0-414.6	Intact	4.75	12.35
Sandstone, Fri	8-74	417.7-417.4	Intact	7.20	12.86
Sandstone, C[4]	2-74	413.4-413.9	Natural joint	2.37	8.49
				4.75	10.18
				7.20	16.97
Sandstone, C	2-74	415.7-416.4	Natural joint	2.37	2.84
				4.75	4.35
				7.20	5.63
Sandstone, Fri	2-74	406.8-407.2	Precut	2.37	1.72
				4.75	3.01
				7.20	4.90
Sandstone, C	2-74	413.4-413.9	Precut	2.37	1.40
				4.75	2.89
				7.20	4.76
Sandstone, C	5-74	411.9-412.2	Precut	2.37	1.33
				4.75	2.90
				7.20	4.72

(Continued)

TABLE 10 (Continued)

Lithology	Drill Hole No. CSR-	Elevation ft	Test	Normal Stress tsf	Peak Shear Stress tsf
Concrete to Fri Sandstone	6-74	417.0-417.2	Precut	2.37	19.62
Concrete to Fri Sandstone	7-74	413.1-413.3	Precut	4.20	--[5]
Concrete to Fri Sandstone	12-74	402.5-402.3	Precut	7.20	18.62
Shale	10-74	432.2-432.4	Intact	2.37	3.42
				4.75	3.18
				7.20	3.09
Shale	10-74	431.5-432.4	Intact Slow	2.38	0.72
				7.20	1.92
Shale	14-74	432.1	Natural joint	2.37	1.99
				4.75	3.56
				7.20	5.60
Shale	3-74	405.7-406.0	Precut	2.37	1.39
				4.75	2.06
				7.20	2.88

1. Filling material in parting is shale.
2. Run with a malfunctioning single plane shear rig; data highly questionable.
3. Fri = Friable.
4. C = Competent.
5. Specimen was eccentrically loaded.

TABLE 11

Laboratory Test Results - Starved Rock Replacement Lock
Rock Shear Test Envelopes

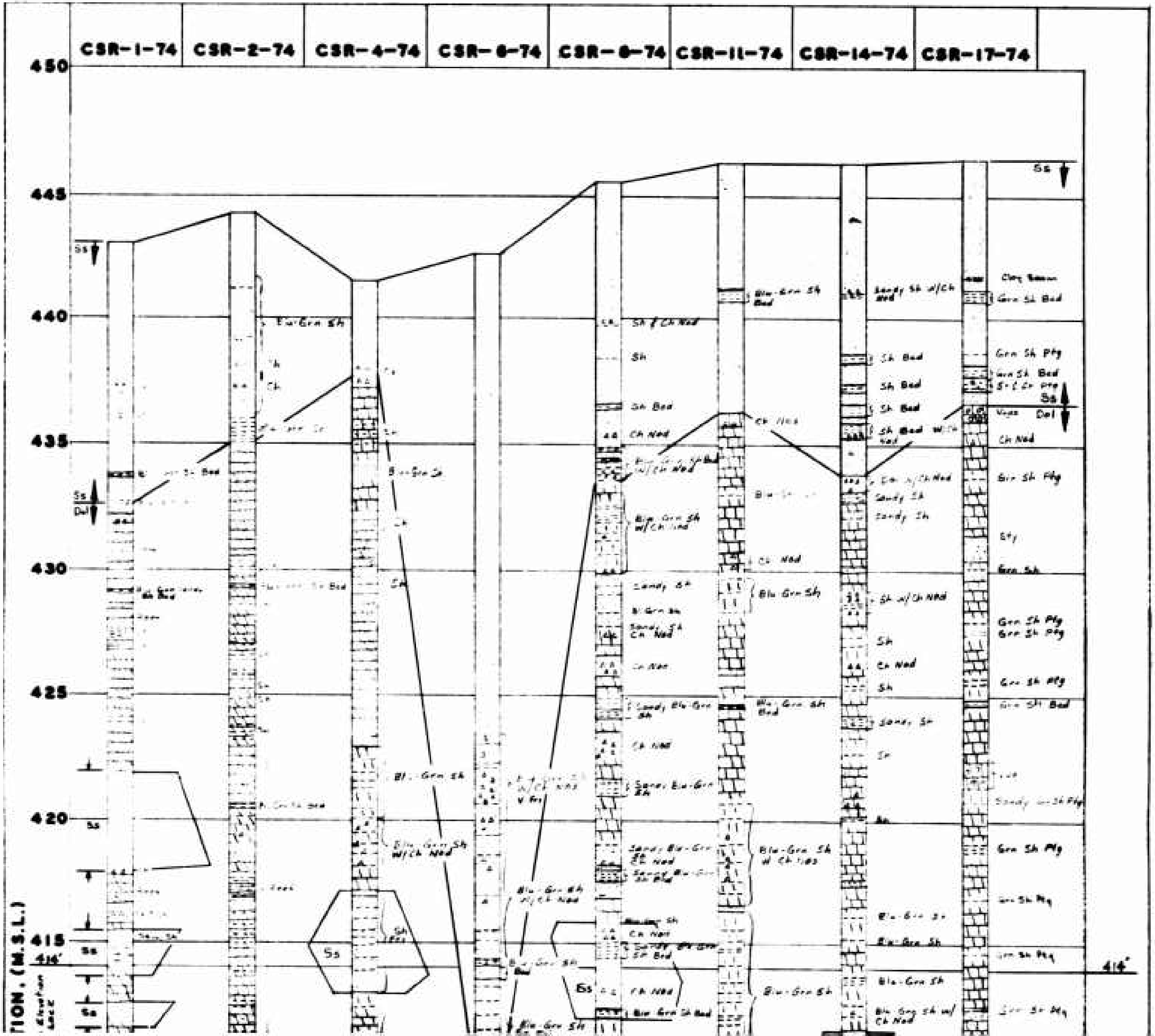
Lithology	Drill Hole No. CSR-	Elevation ft	Test	Cohesion tsf	Angle of Friction deg
Dolomite	2-74	401.2-401.8	Filled parting	0.8	36
Dolomite	7-74	418.0-418.1	Filled parting	0.25	26
Dolomite	8-74	428.2-428.8	Filled parting	0.65	33
Dolomite	7-74	417.8-418.1	Precut	0.0	31
Dolomite	2-74	404.7-405.1	Precut	0.0	38
Concrete to Dolomite	--	--	Natural joint	--	--
Concrete to Dolomite	--	--	Precut	1.60	63
Sandstone, Fri*	--	--	Intact	0.0	66
Sandstone, C*	2-74	413.4-413.9	Natural joint	0.0	66
Sandstone, C	2-74	415.7-416.4	Natural joint	1.45	30.5
Sandstone, Fri	2-74	406.8-407.2	Precut	0.0	33.5
Sandstone, C	2-74	413.4-413.9	Precut	0.0	31
Sandstone, C	5-74	411.9-412.2	Precut	0.0	33
Concrete to Fri Sandstone	--	--	Precut	--	--
Shale	10-74	432.2-432.4	Intact	--	--
Shale	10-74	431.5-432.4	Intact slow	0.12	14.5
Shale	14-74	432.1	Natural joint	0.0	38
Shale	3-74	405.7-406.0	Precut	0.45	19

* Fri = Friable; C = Competent.

TABLE 12

From "Summary of Rock and Soil Design Parameters," Plate No. 7, Appendix A, Stability Analysis, 1973

Physical Properties	Dolomite	Shale	Sandstone
Compressive strength	8500 psi		moderately weak
Shear strength (along shear planes)			
a. Peak (horizontal)	c = 18.5 psi ϕ = 50 deg	c = 32 psi ϕ = 49.5 deg	c = 12 psi ϕ = 33 deg
b. Residual (horizontal)	c = 0 ϕ = 35 deg	c = 0 ϕ = 49.5 deg	c = 0 ϕ = 33 deg
Shear, intact rock cross-bedded	c = 1000 psi ϕ = 54.5 deg	c = 47.5 psi ϕ = 60 deg	c = 15 psi ϕ = 38 deg
Young's Modulus Static (0-1000 psi)	3.9×10^6 psi	0.36×10^6 psi	1.4×10^6 psi
Poisson's Ratio (0-1000 psi)	0.128	0.225	0.300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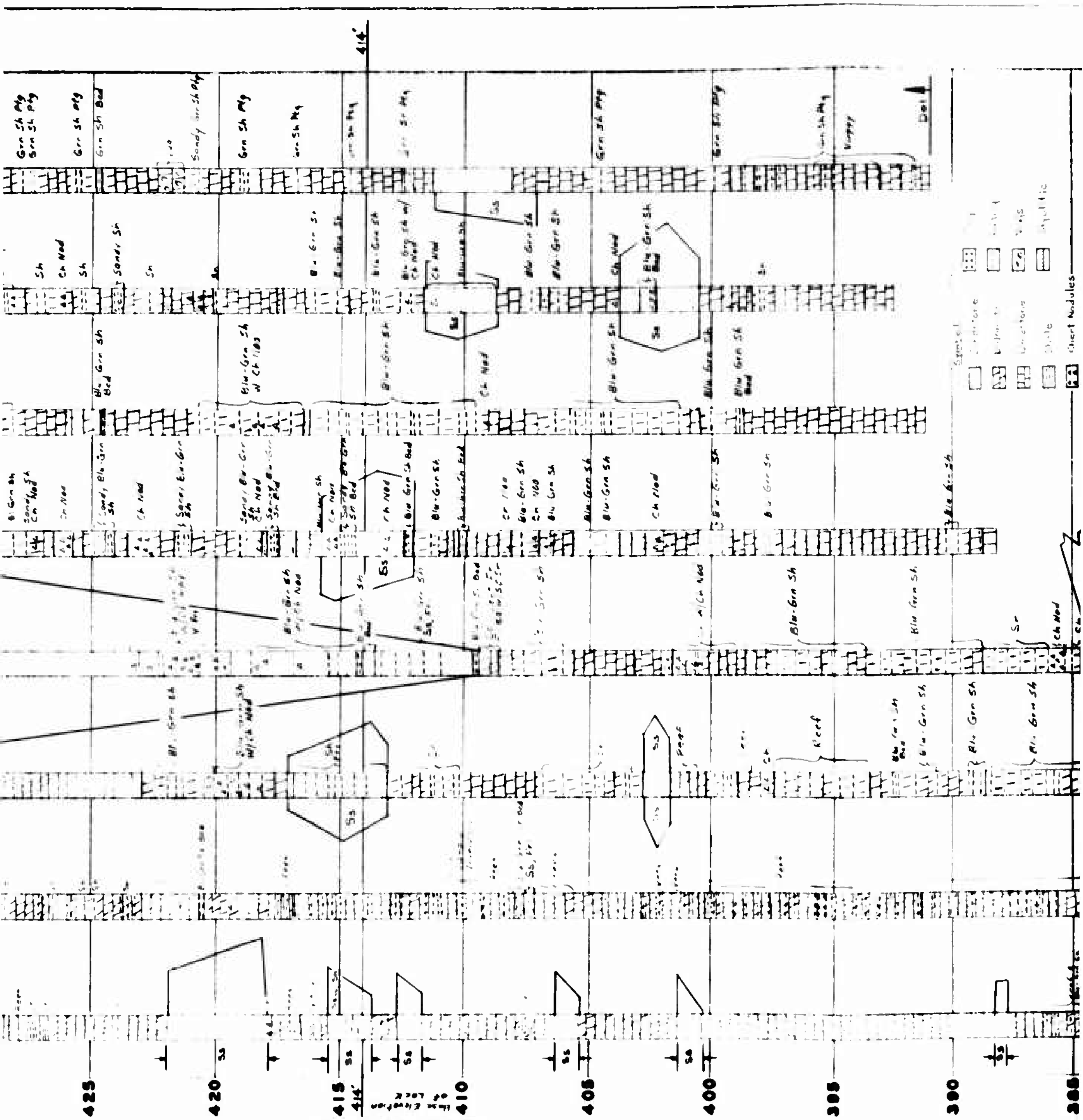
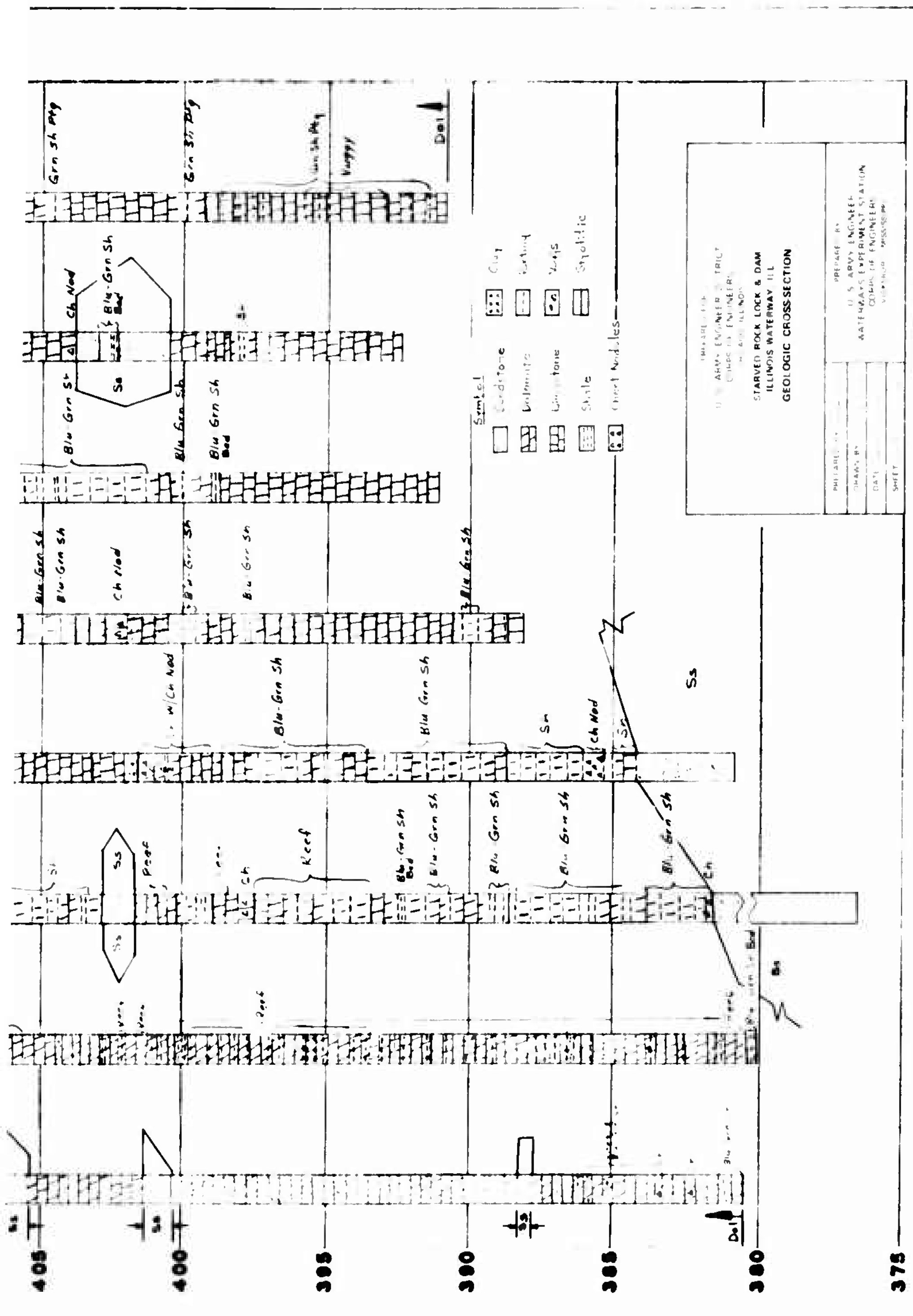
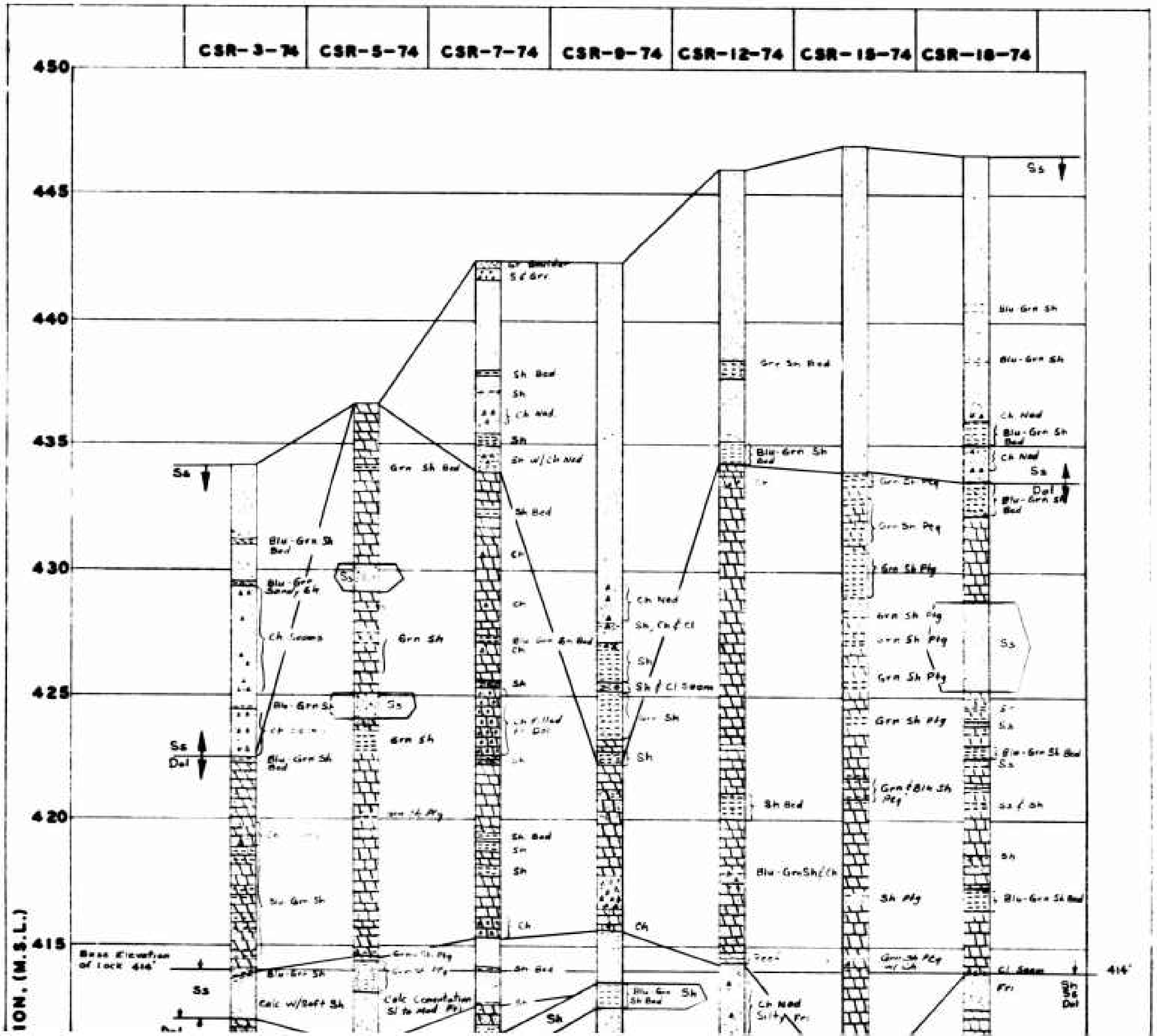
CSR-2-74
CSR-4-74
CSR-6-74
CSR-8-74
CSR-11-74
CSR-14-74
CSR-17-74
450
445
440
435
430
425
420
415
414'
414'
TION, (M.S.L.)
Ss
Dol
Blu-Grn Sh
Bl-Grn Sh
Sh Bed
Ch Nod
Sh & Ch Nod
Sandy Sh
Blu-Grn Sh w/Ch Nod
Sandy Sh w/Ch Nod
Gra Sh Bed
Grn Sh Pty
Grn Sh Bed
Ch Nod
Sandy Sh
Grn Sh
Grn Sh Pty
Grn Sh Bed
Grn Sh Pty
Blu-Grn Sh w/Ch Nod
Blu-Grn Sh
Blu-Grn Sh w/Ch Nod


ELEVATION, (M.S.L.)
425
420
415
414'
410
405
400
395
390
385
Base Elevation of Lock
Chert Nodules

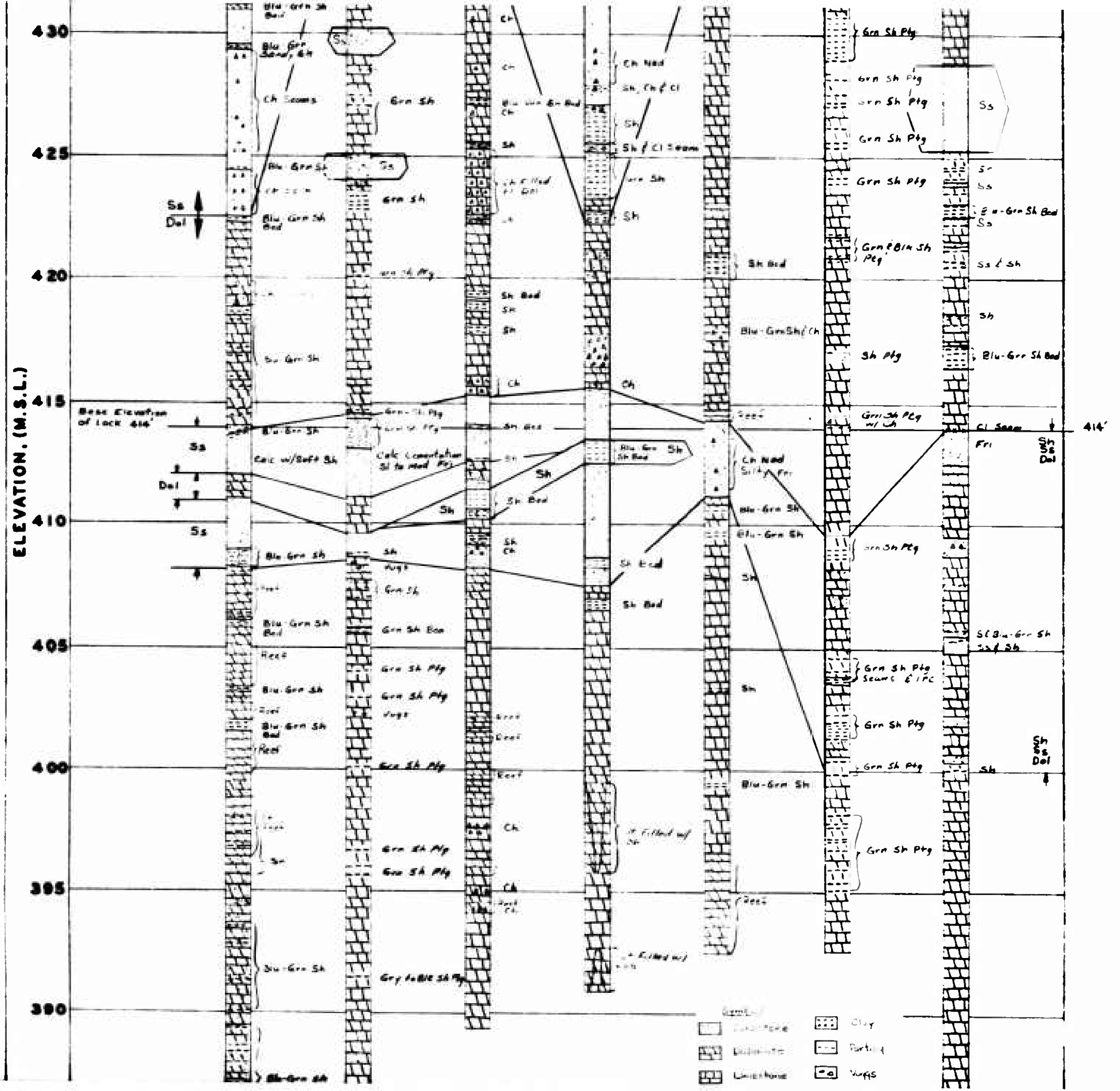


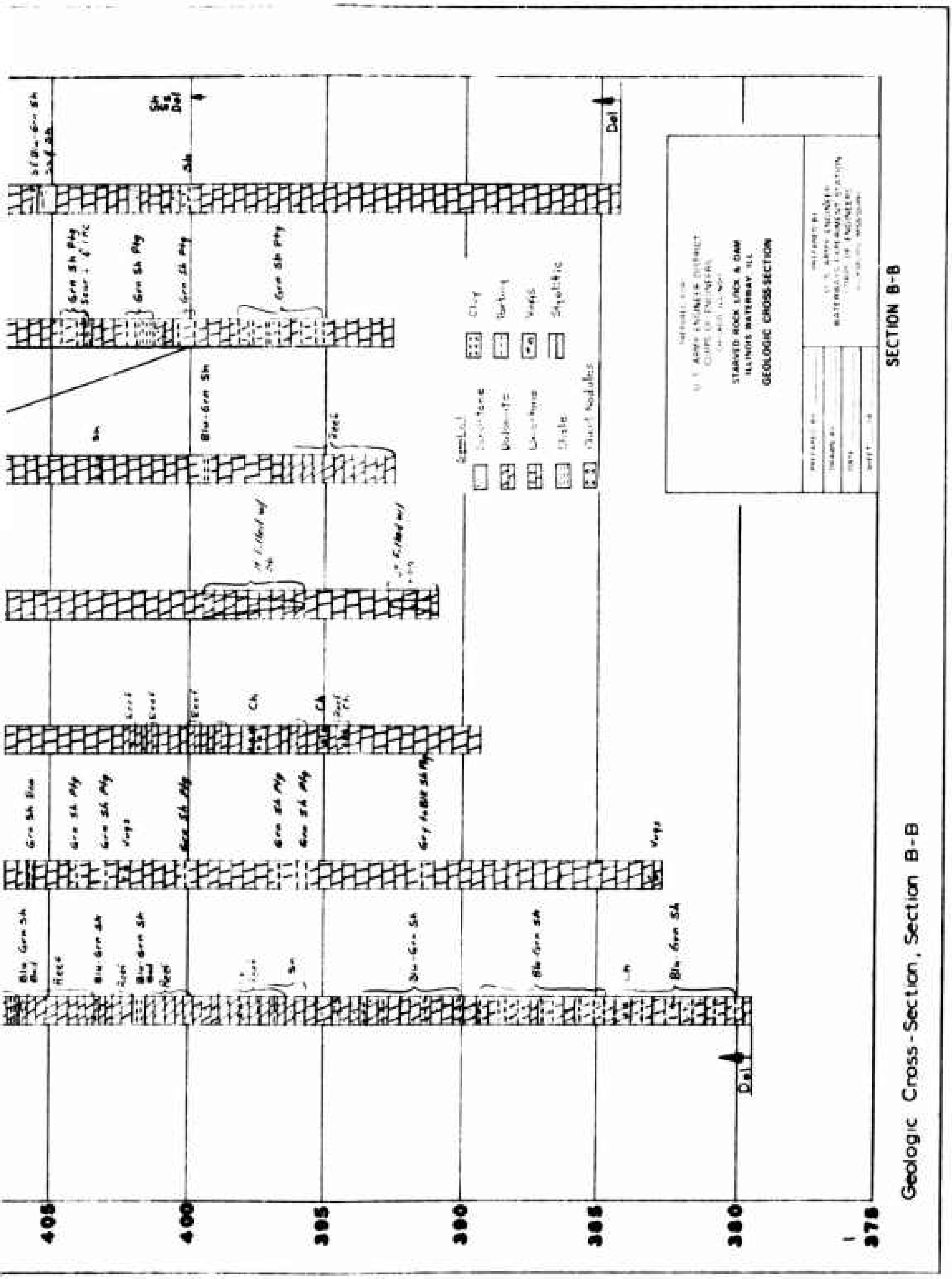
Geologic Cross-Section, Section A-A

PLATE 1



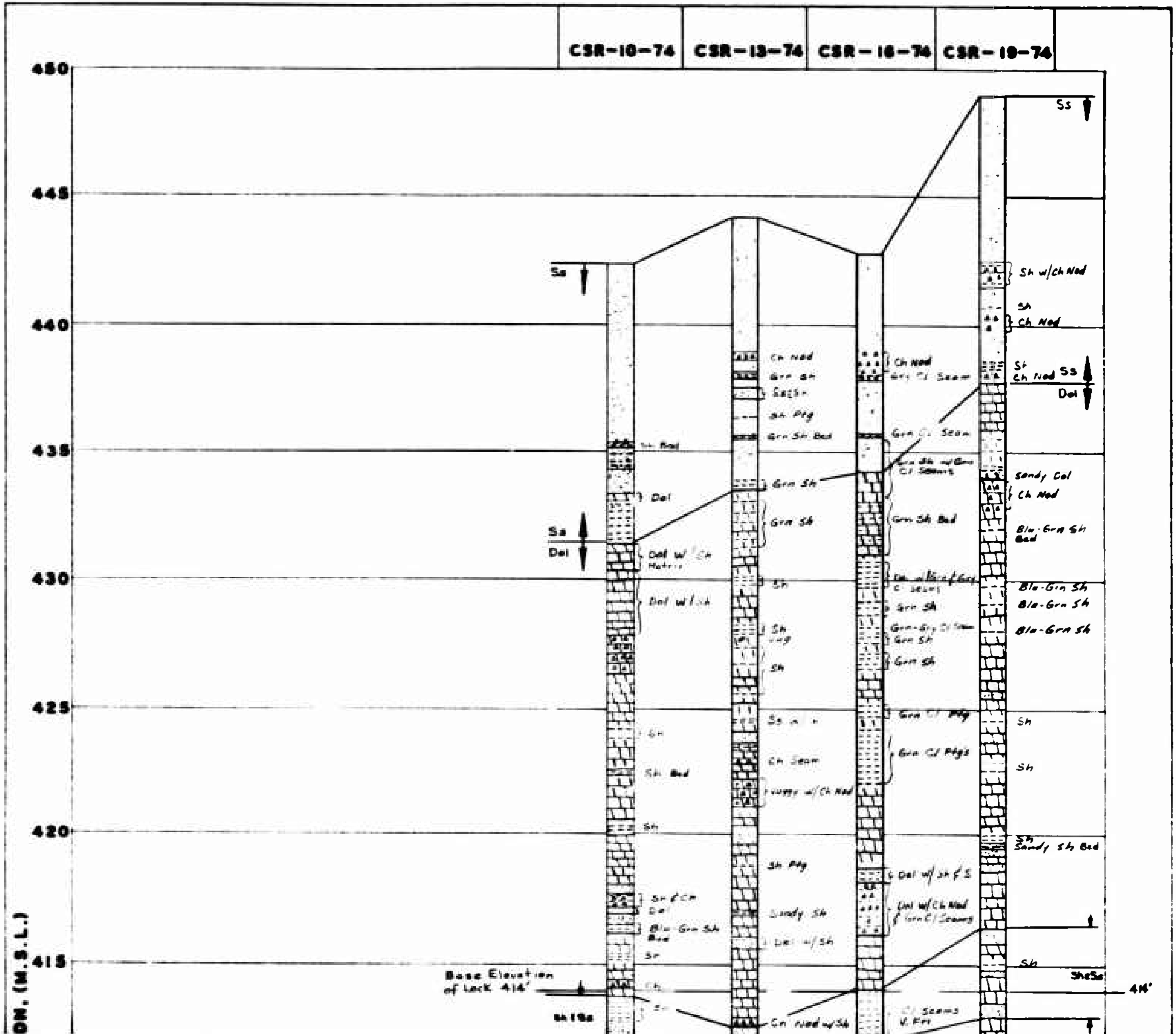
CSR-3-74
CSR-5-74
CSR-7-74
CSR-8-74
CSR-12-74
CSR-15-74
CSR-18-74
450
445
440
435
430
425
420
415
ION. (M.S.L.)
Base Elevation of Lock 414'
414'
Ss
Dol
Sh
Blu-Grn Sh Bed
Blu-Grn Sand/Ch
Ch Seams
Blu-Grn Sh
Calc w/Soft Sh
Grn Sh Bed
Grn Sh
Calc Cementation Sl to Mod Fri
Sh Bed
Ch Nod
Sh w/Ch Nod
Ch
Sh, Ch & Cl
Sh & Cl Seam
Grn Sh Bed
Blu-Grn Sh/Ch
Ch Nod
Grn Sh Ptg
Sh Ptg
Blu-Grn Sh
Cl Seam
Fri


ELEVATION, (M.S.L.)
430
425
420
415
410
405
400
395
390
Base Elevation of Lock 414'
414'
Ss
Dol
Sh
Ss
Dol
Blu-Grn Sh
Calc w/Soft Sh
Calc Cementation Sl to Mod Ptg
Ch Seams
Grn Sh
Grn Sh Ptg
Blu-Grn Sh Bed
Sh Bed
Ch Nod
Cl Seam
Fri
Reef
Vugs
Gry to Blk Sh Ptg
Clay
Parting
Vugs
Limestone

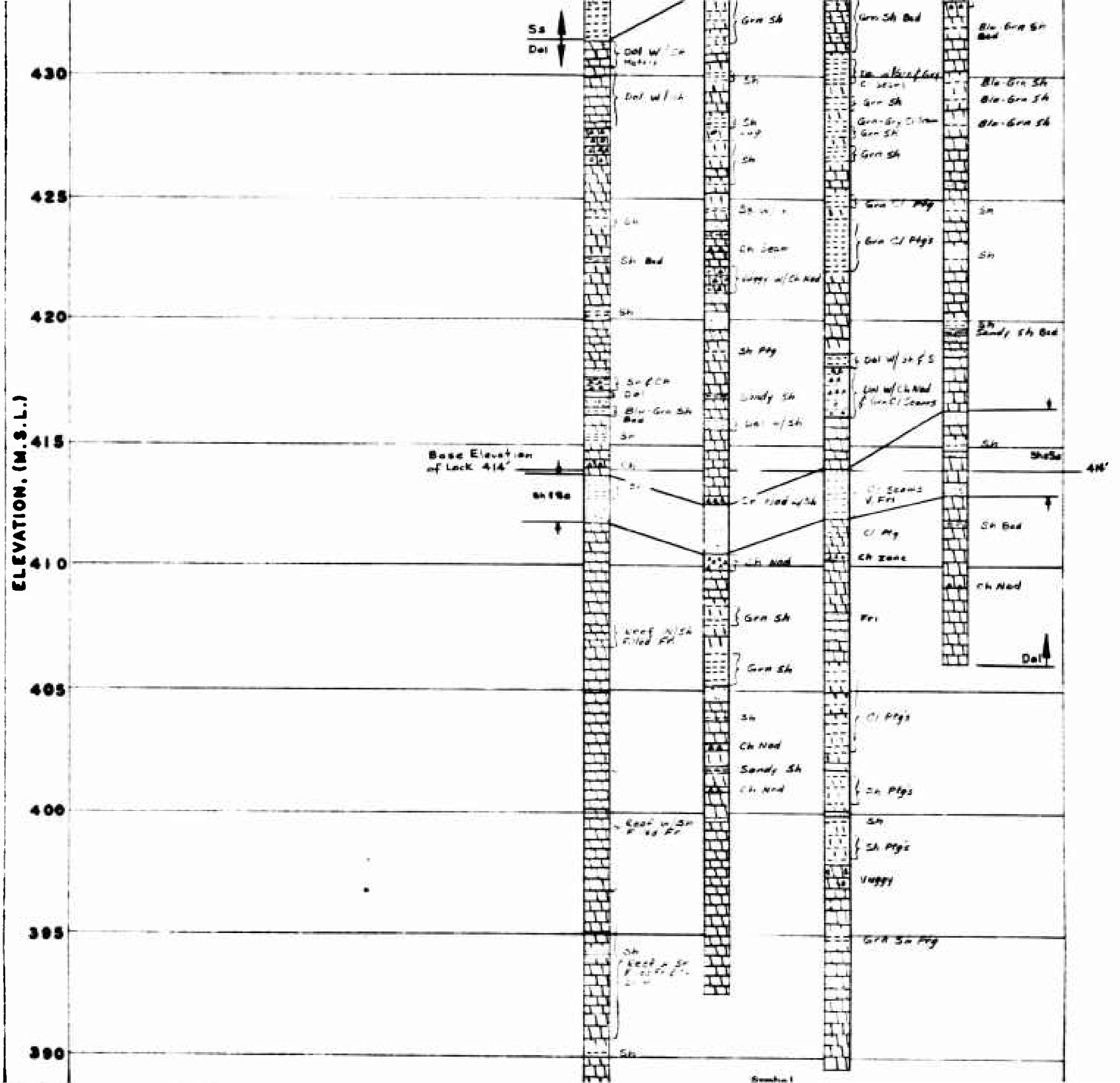


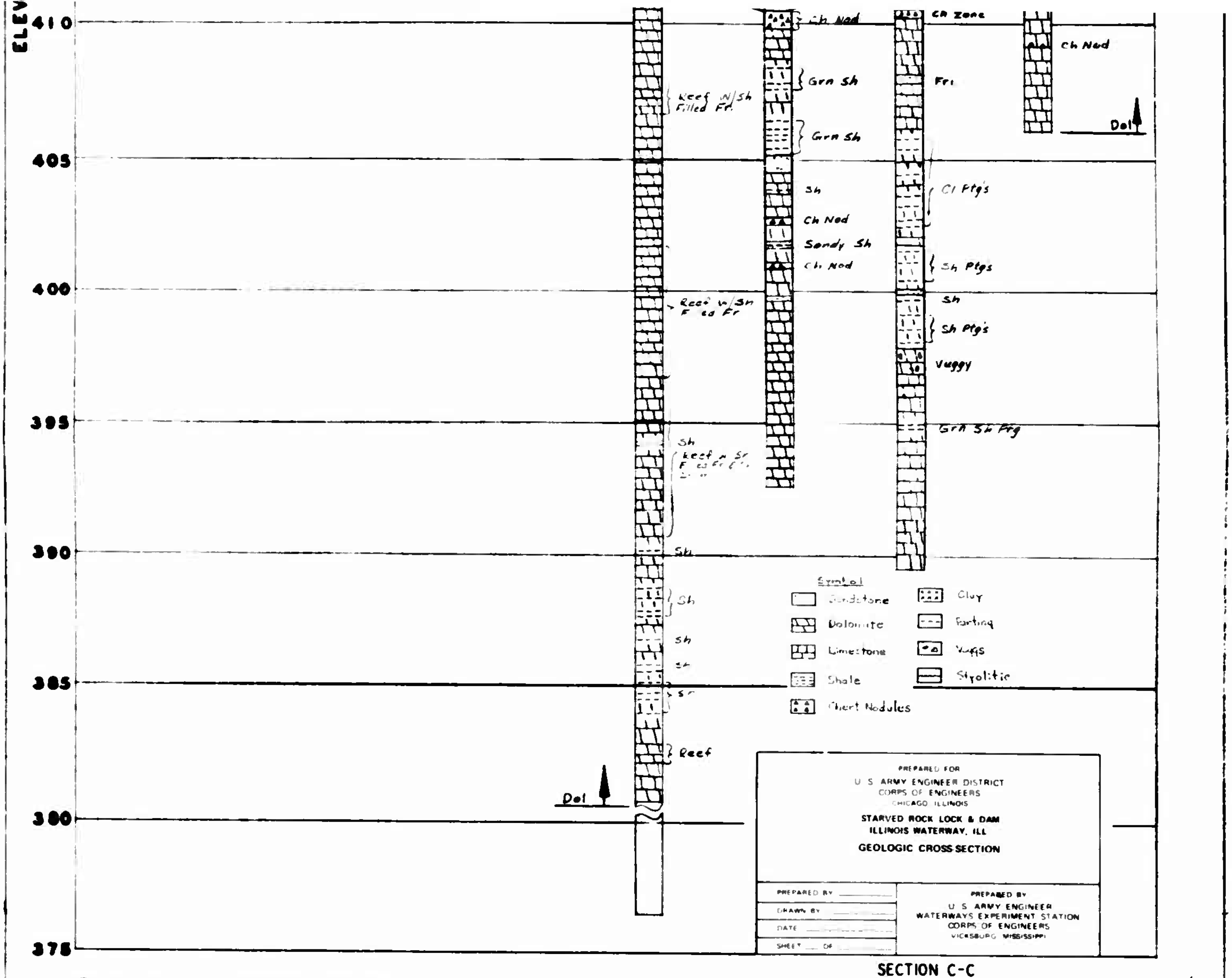
Geologic Cross-Section, Section B-B

PLATE 2

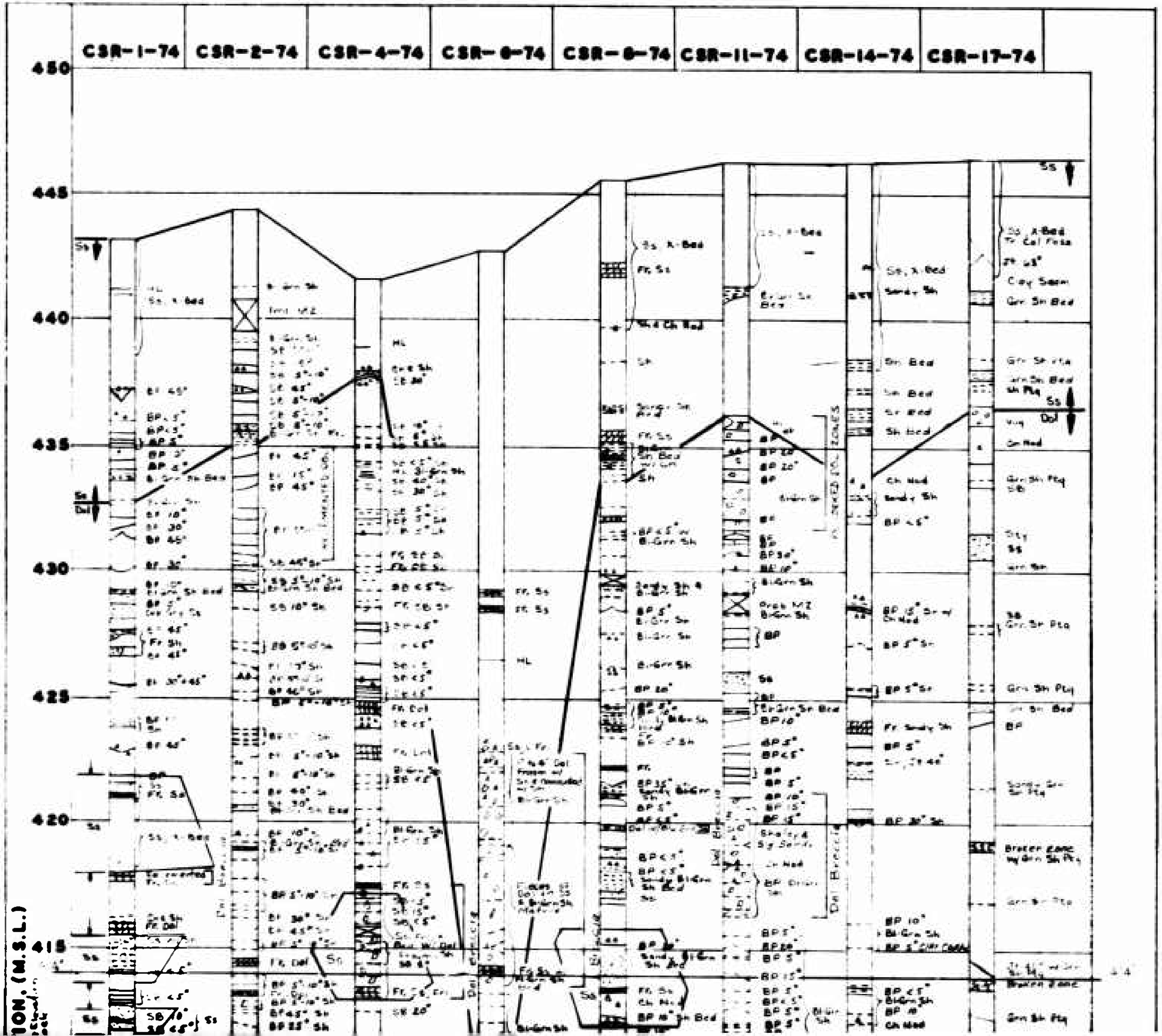


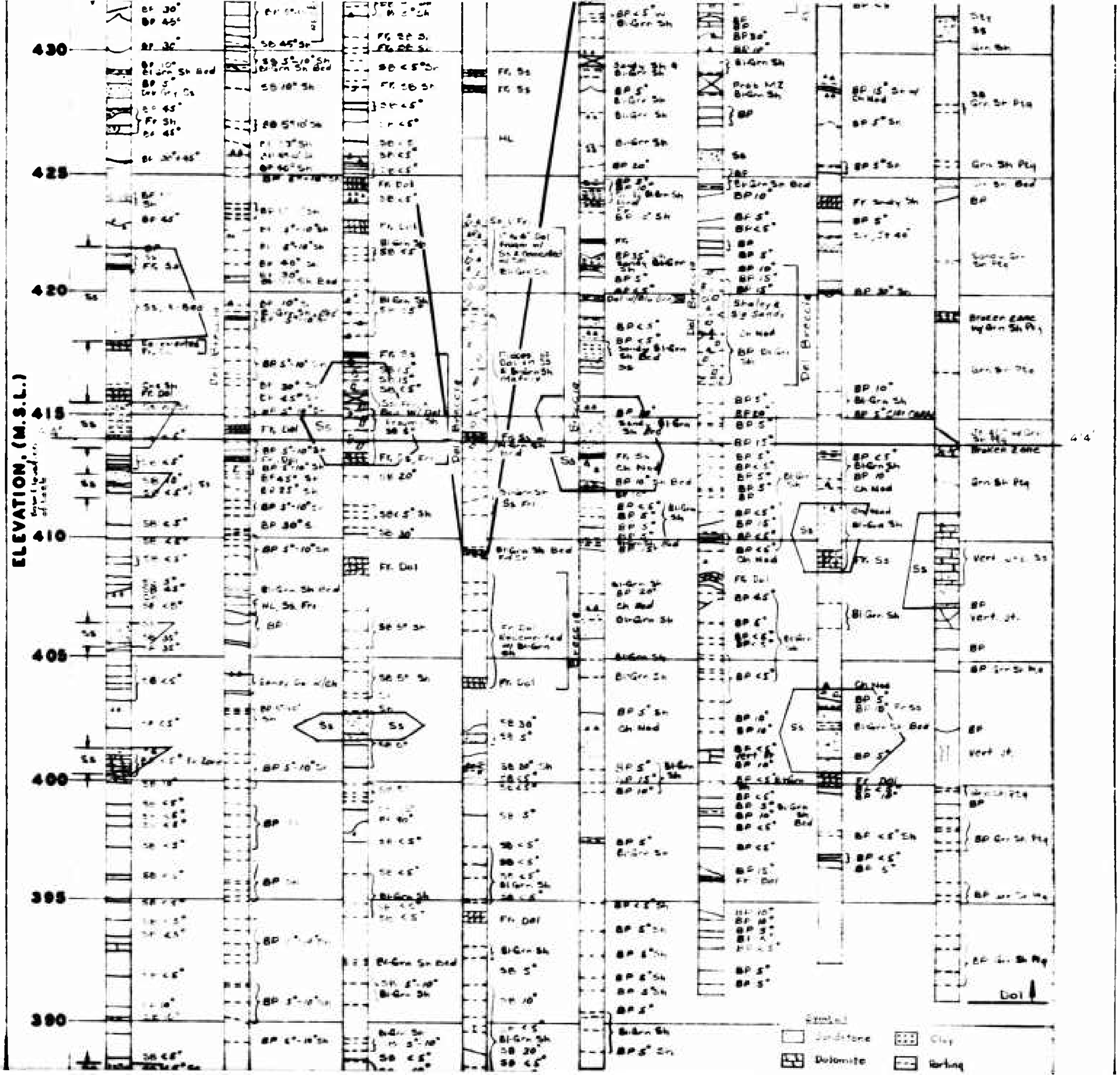
CSR-10-74
CSR-13-74
CSR-16-74
CSR-19-74
450
445
440
435
430
425
420
415
ON. (M.S.L.)
Ss
Dol
Sh w/Ch Nod
Ch Nod
Grn Sh
Sh Ptg
Grn Sh Bed
Grn Sh
Gry Cl Seam
Grn Sh Bed
Sandy Dol
Blu-Grn Sh
Dol w/Sh
Sh Bed
Sh
Ch Seam
Sandy Sh
Sandy Sh Bed
Grn Cl Ptgs
Base Elevation of Lock 414'
414'

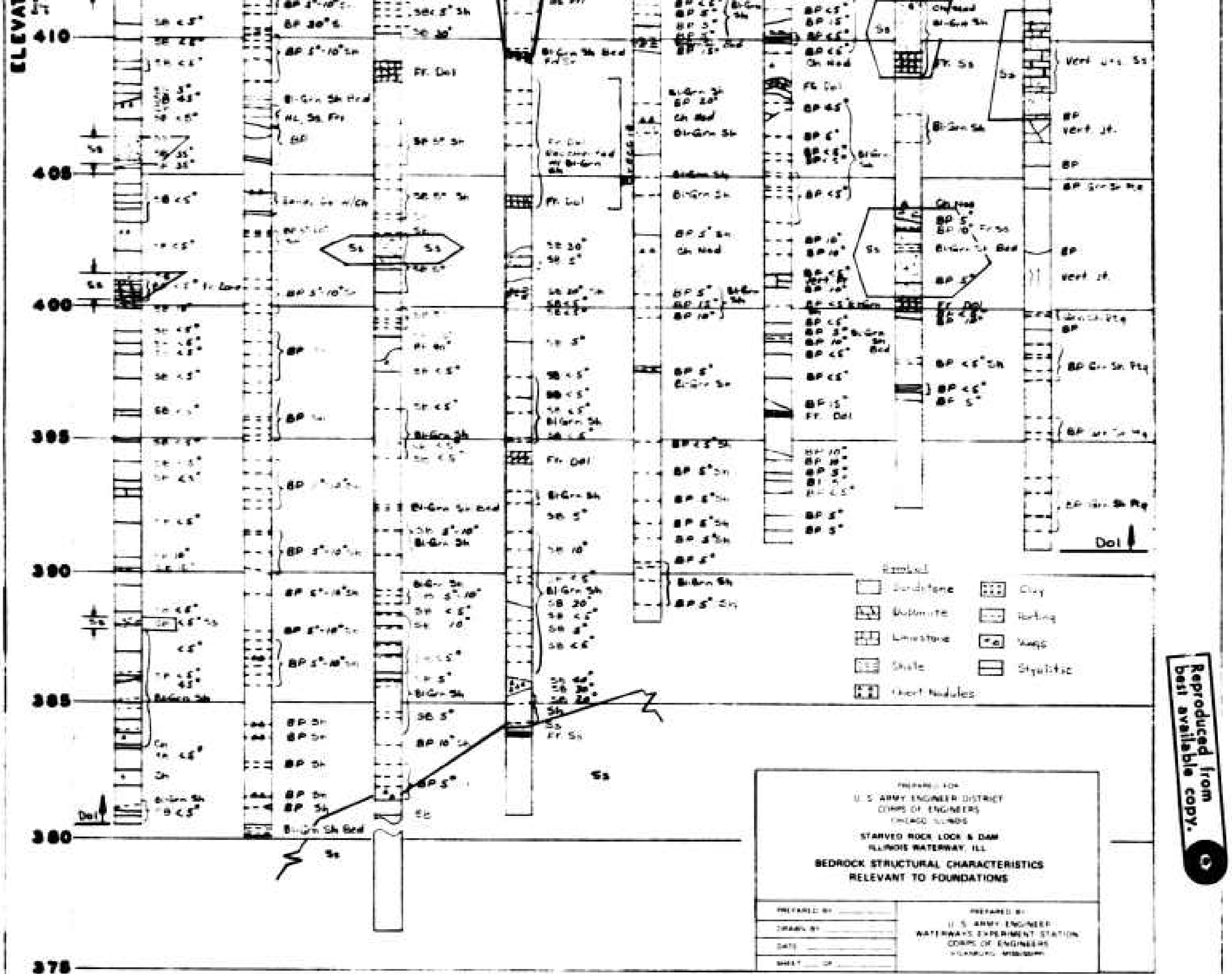

ELEVATION, (M.S.L.)
430
425
420
415
410
405
400
395
390
Ss
Dol
Base Elevation of Lock 414'
416'
Dol
Grn Sh
Sh
Sh Bed
Sh Pkg
Sandy Sh
Ch Nod
Grn Sh Bed
Grn Cl Pkgs
Cl Pkg
Ch Zone
Fri
Sh Pkgs
Vuggy
Grn Sh Pkg
Blu-Grn Sh
Sandy Sh Bed



Geologic Cross-Section, Section C-C

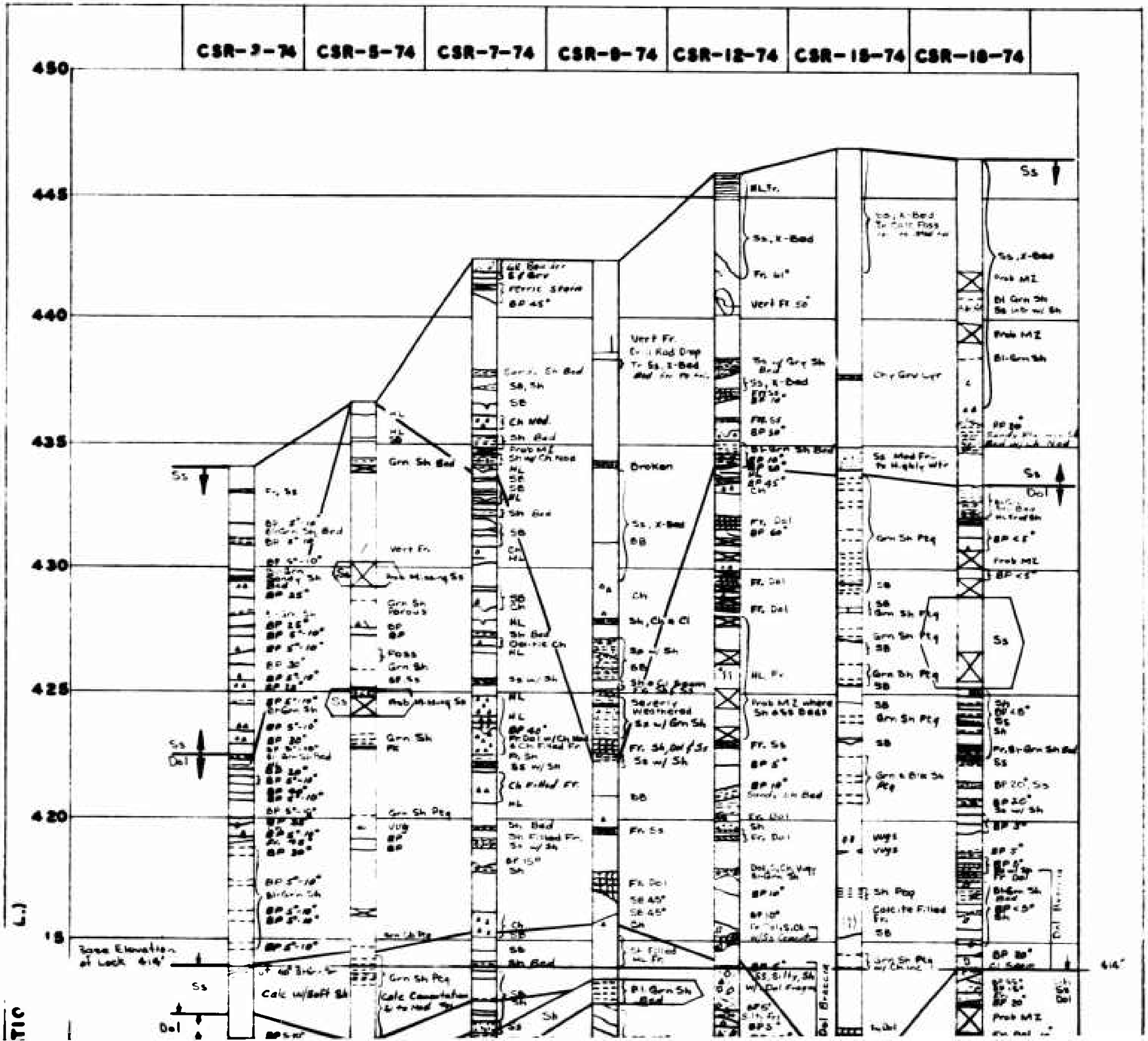

CSR-1-74
CSR-2-74
CSR-4-74
CSR-6-74
CSR-8-74
CSR-11-74
CSR-14-74
CSR-17-74
450
445
440
435
430
425
420
415
...ION, (M.S.L.)

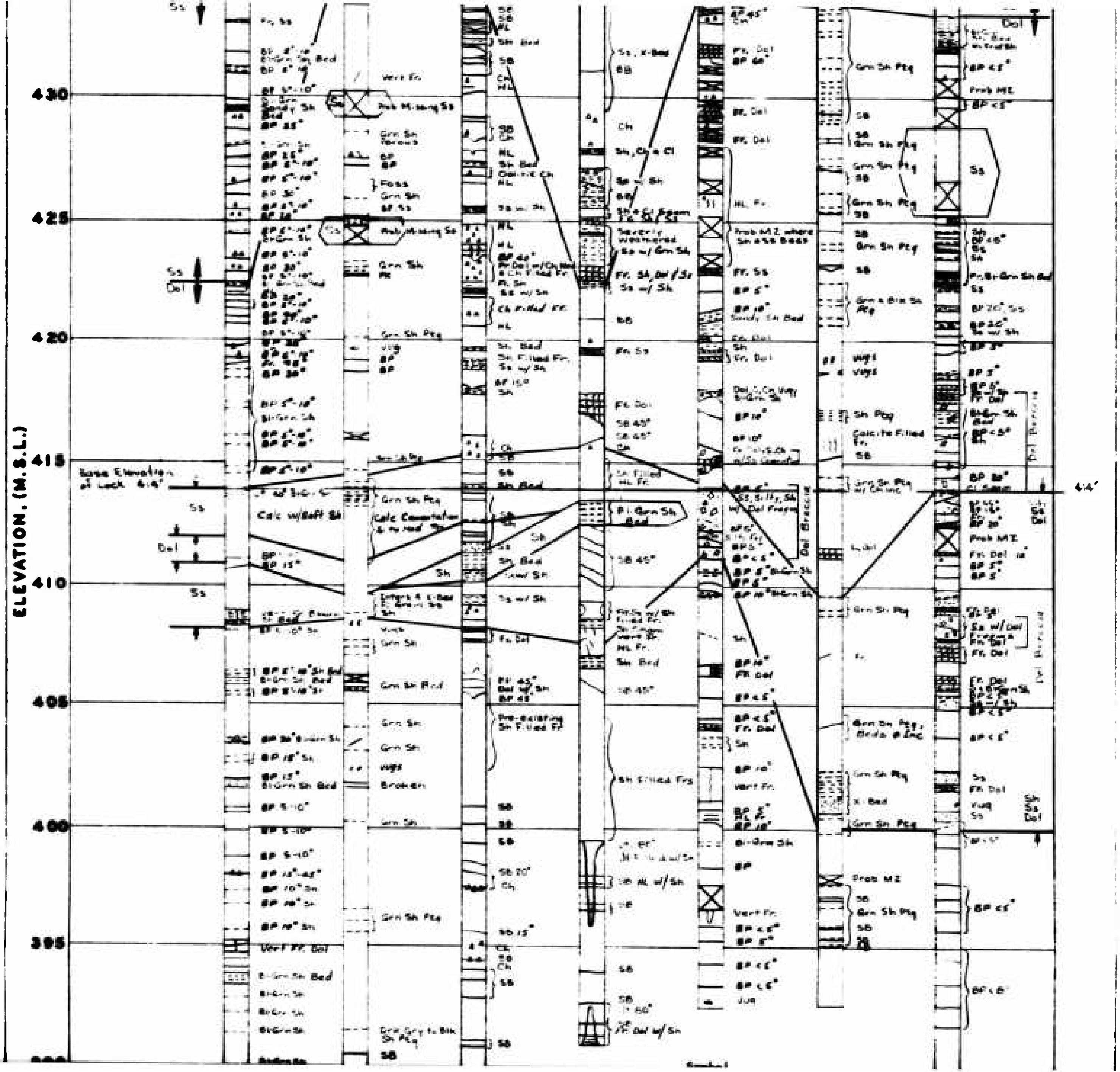

ELEVATION, (M.S.L.)
430
425
420
415
410
405
400
395
390
Dol
Symbol
Sandstone
Clay
Dolomite
Sh
Ss
Del Breccia
Broken Zone



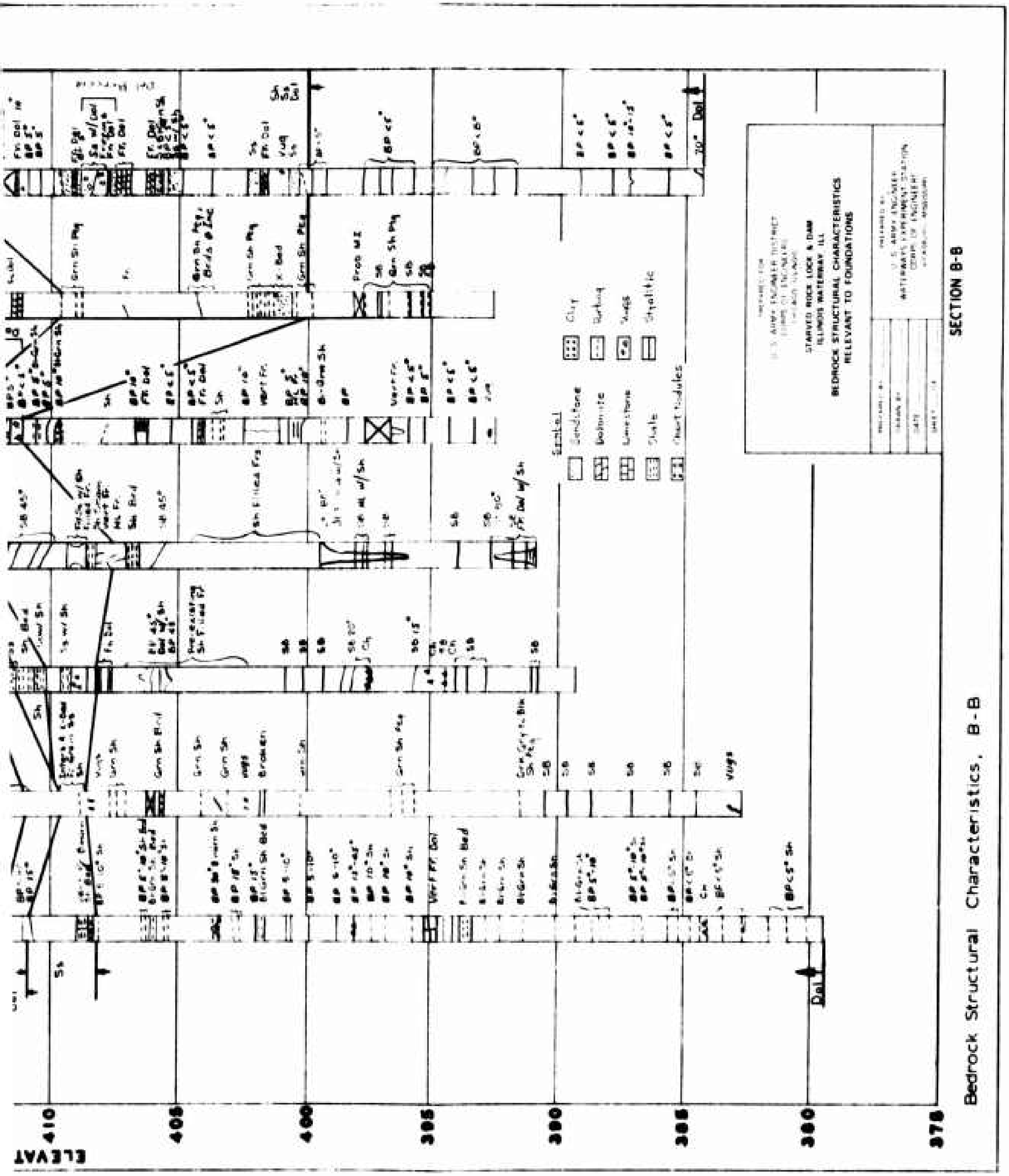
Bedrock Structural Characteristics, Section A-A

PLATE 4


CSR-3-74
CSR-5-74
CSR-7-74
CSR-9-74
CSR-12-74
CSR-15-74
CSR-18-74
450
445
440
435
430
425
420
15
Base Elevation of Lock 414'
414'
Ss
Dol
Broken
Vert Fr.
Grn Sh Bed
Prob Missing Ss
Grn Sh Peg
Ss, X-Bed
Prob MZ
Bl-Grn Sh
Fr. Dol
Ch
Sh, Ch & Cl
Severely Weathered
Ss w/ Grn Sh
Fr. Ss
Calcite Filled Fr.
Dol Breccia

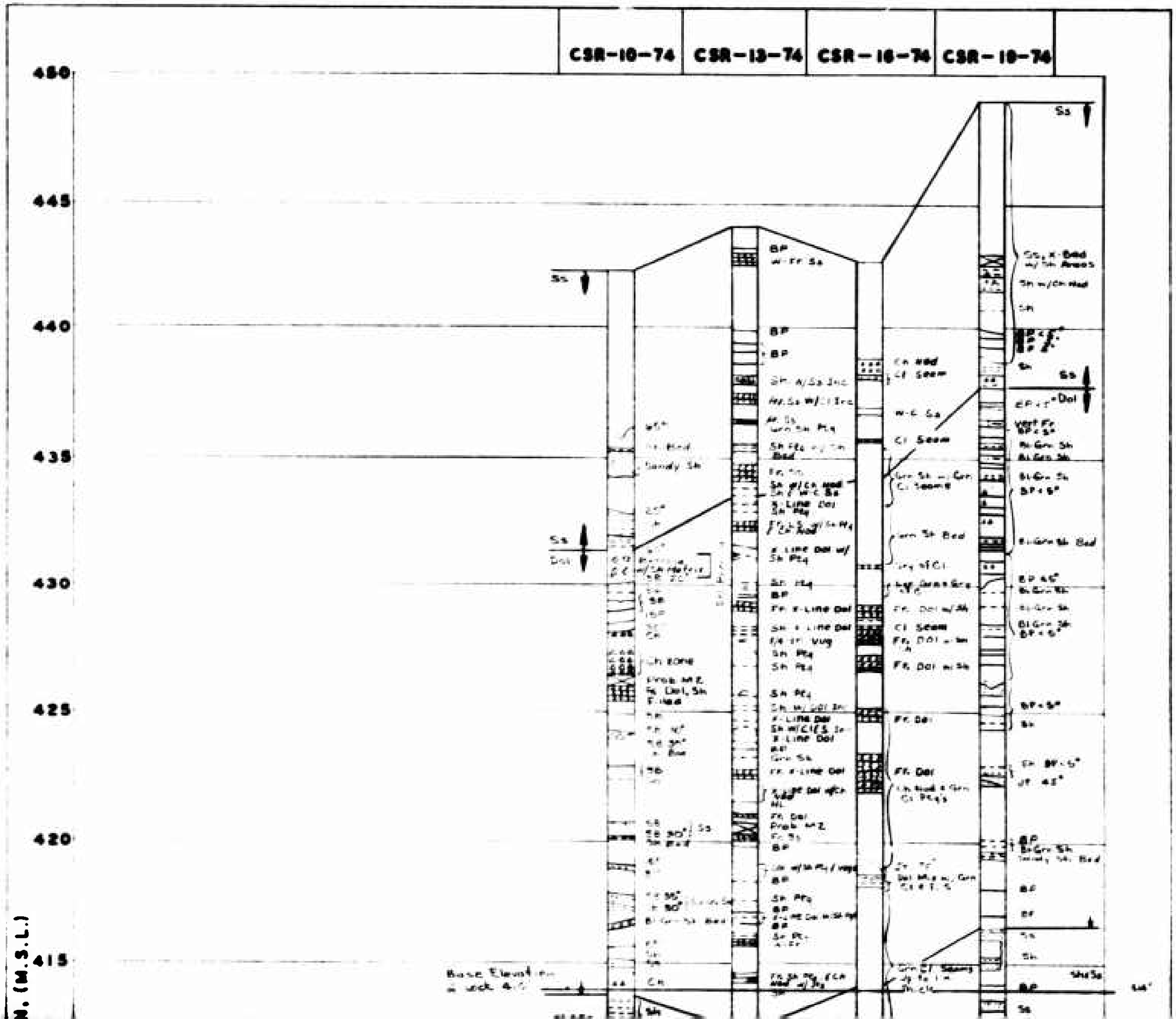


ELEVATION, (M.S.L.)
430
425
420
415
410
405
400
395
Base Elevation of Lock 414'
414'
Dol Breccia

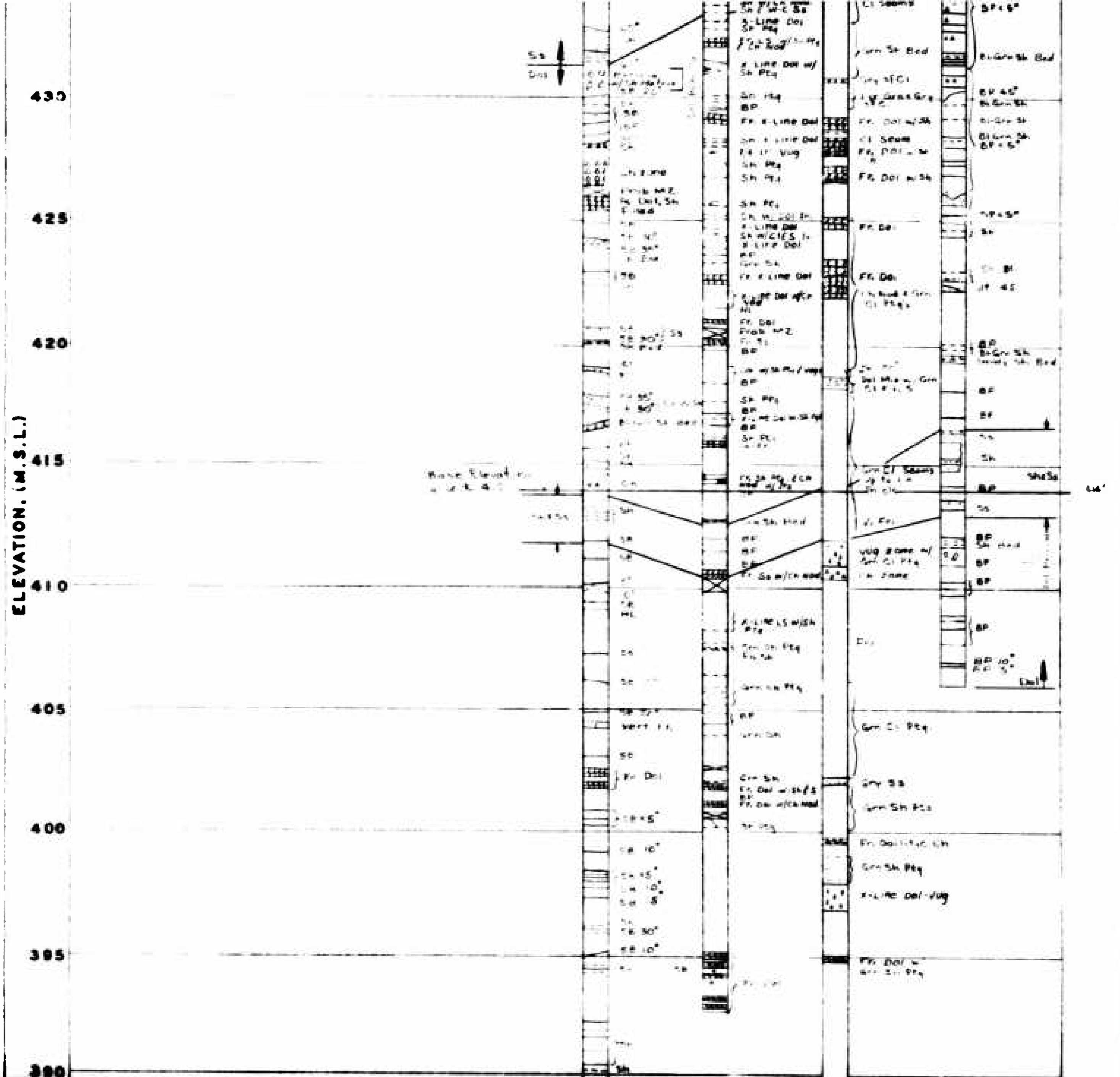


Bedrock Structural Characteristics, B-B

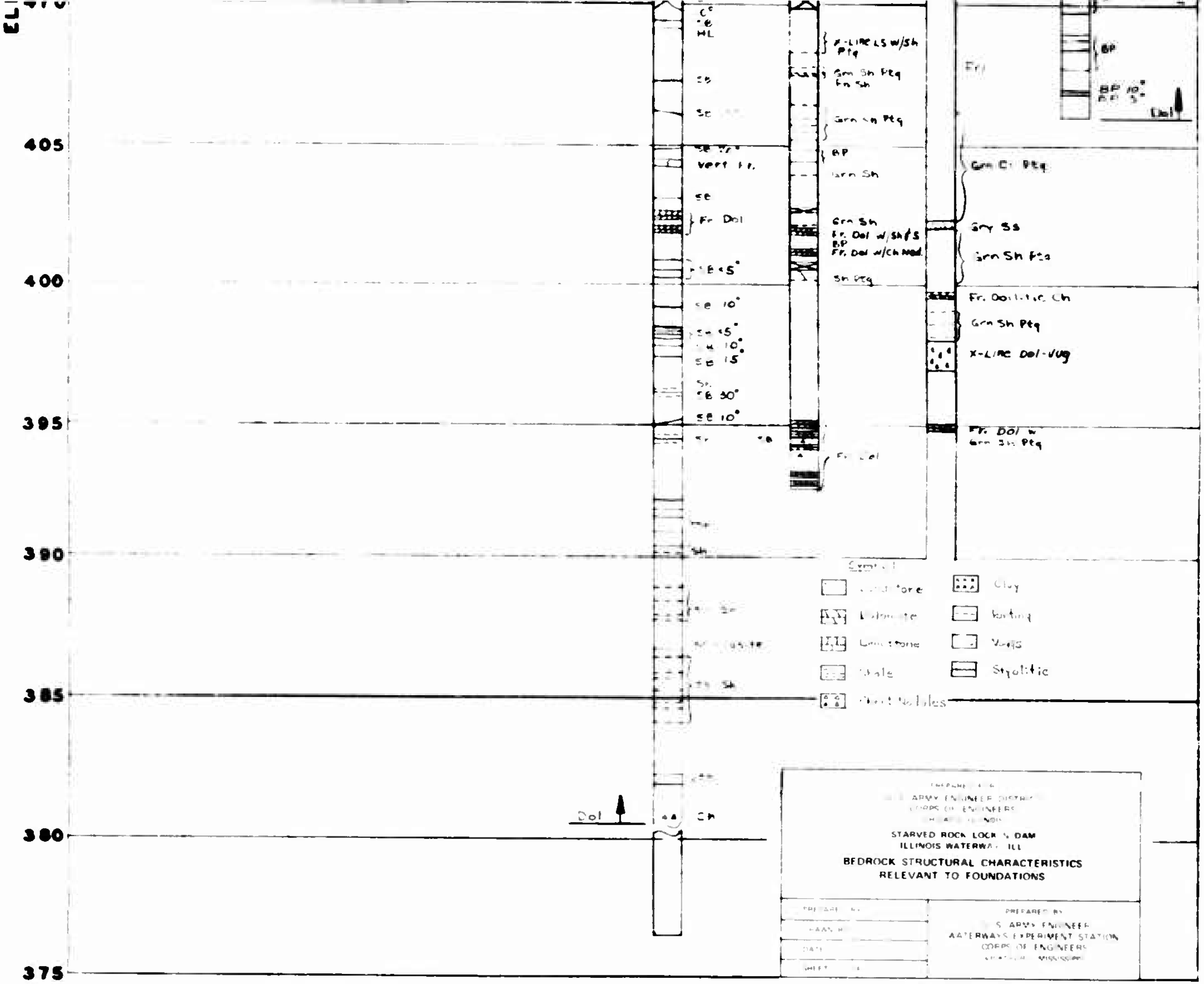
PLATE 5



CSR-10-74
CSR-13-74
CSR-16-74
CSR-19-74
450
445
440
435
430
425
420
415
N. (M.S.L.)
Base Elevation
Ss
Dol



ELEVATION, (M.S.L.)
430
425
420
415
410
405
400
395
390
Base Elevation
Fr. Dol w/ Sh
Cl Seam
Fr. Dol
Fr. X-Line Dol
Sh Pty
BP
Dol
Grn Cl Ptg
Gry Ss
Grn Sh Ptg
Grn Sh Ptg
X-Line Dol-Vug
Vug Zone w/ Grn Cl Ptg
Grn Cl Seams
Gr-Grn Sh Bed
Grn Sh Bed
Sh
Gry Sh
Fr. Dol



Bedrock Structural Characteristics, Section C-C

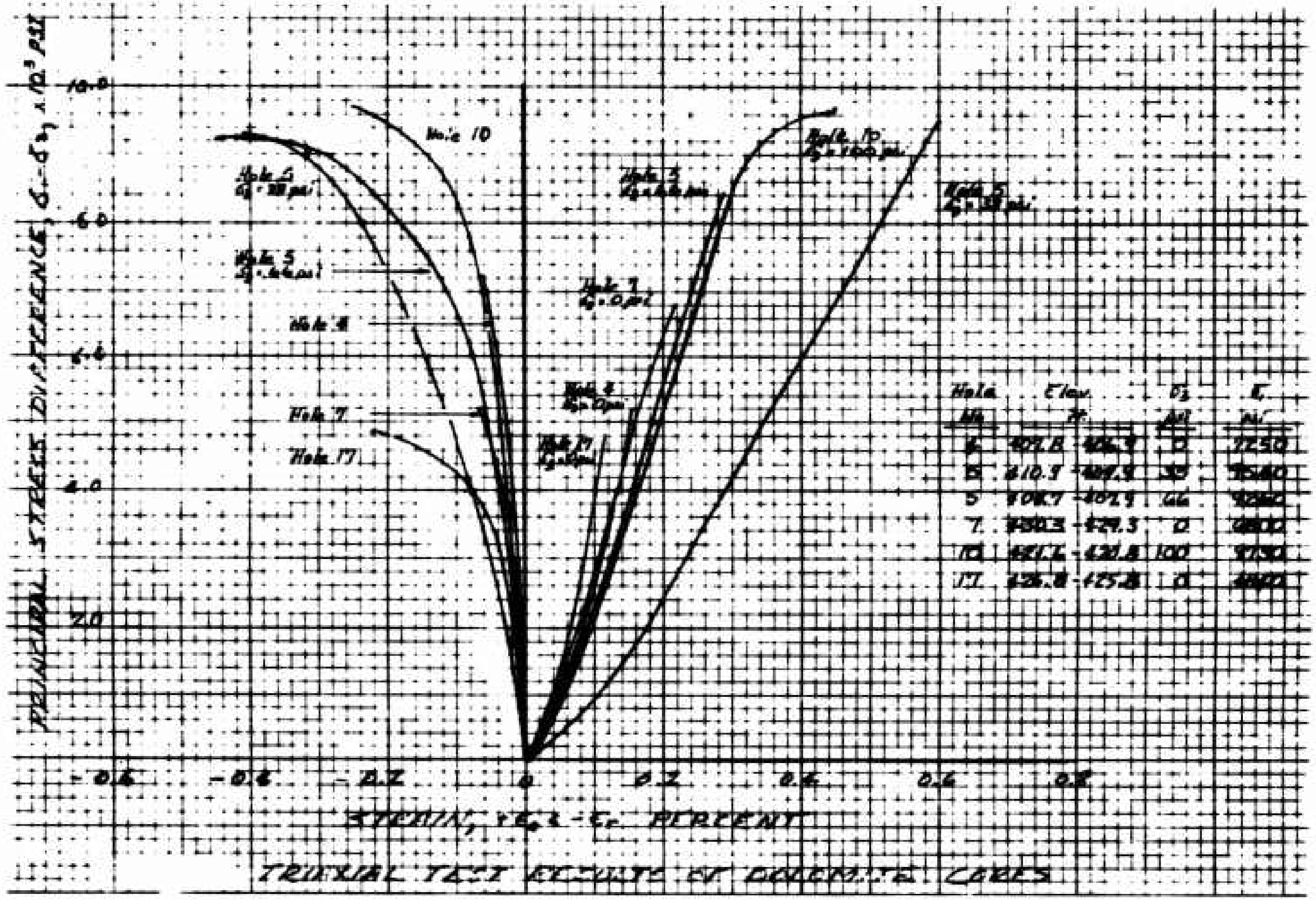


PLATE 7

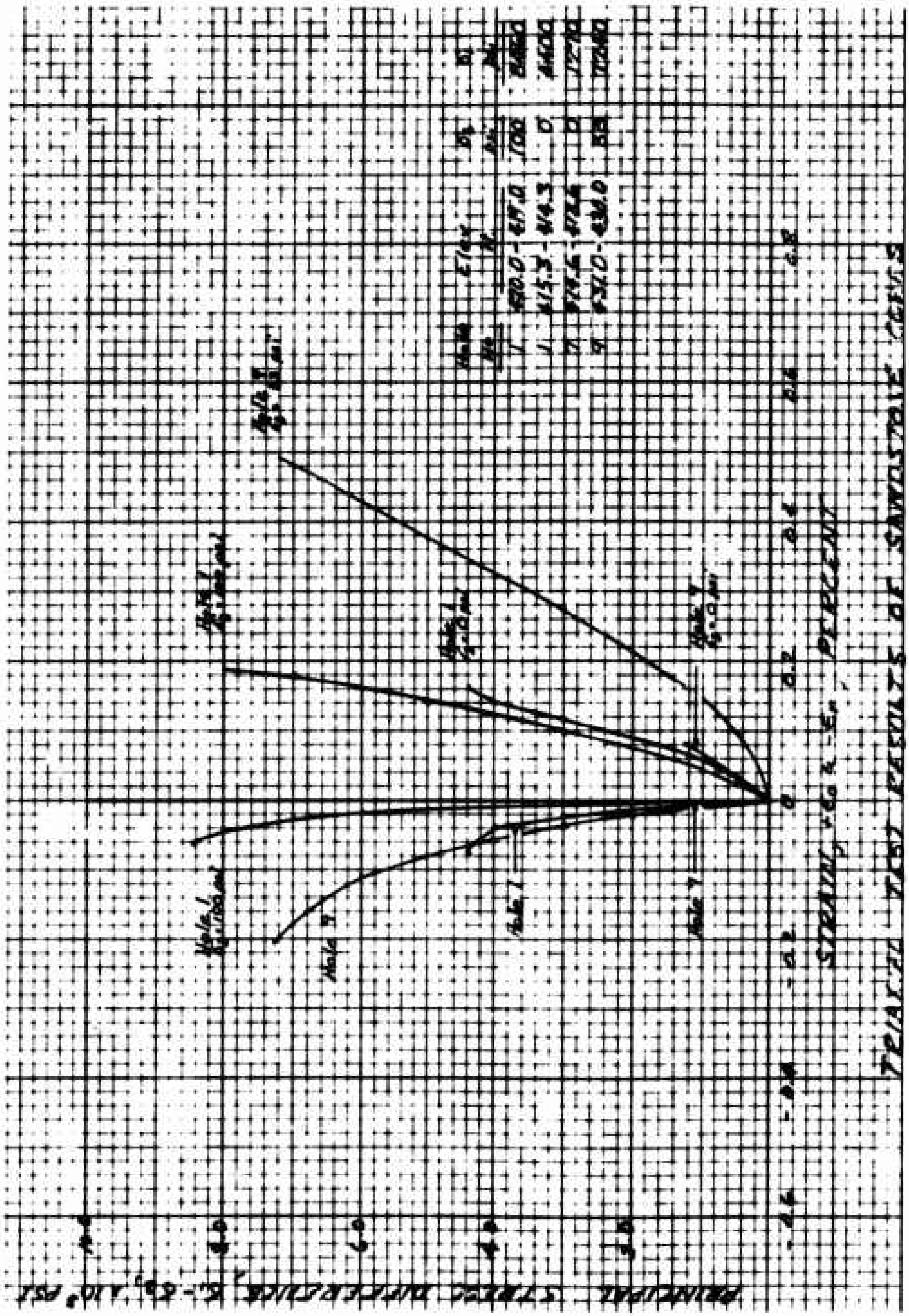

TRIAXIAL TEST RESULTS OF SANDSTONE CORES
PERCENT
Hole 9
Hole 1
Hole 7
-0.2
0
0.2
0.4
0.6
0.8
2.0
4.0
6.0
8.0
10.0

PLATE 8

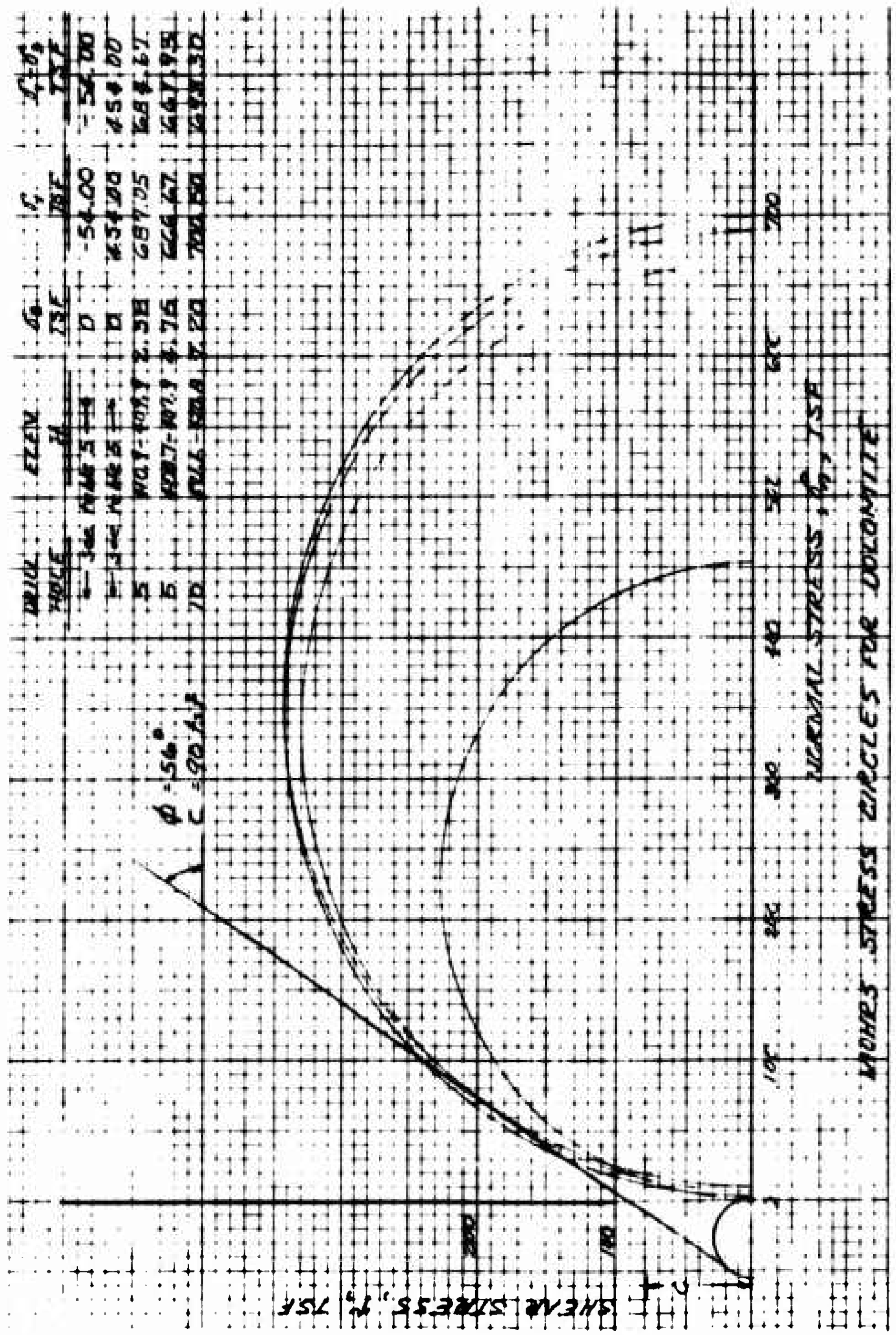

DRILL HOLE
ELEV ft
σ3 TSF
σ1 TSF
σ1-σ3 TSF
-54.00
454.00
687.05
φ = 56°
C = 90 TSF
SHEAR STRESS, τ, TSF
NORMAL STRESS, σn, TSF
MOHRS STRESS CIRCLES FOR DOLOMITE

PLATE 9

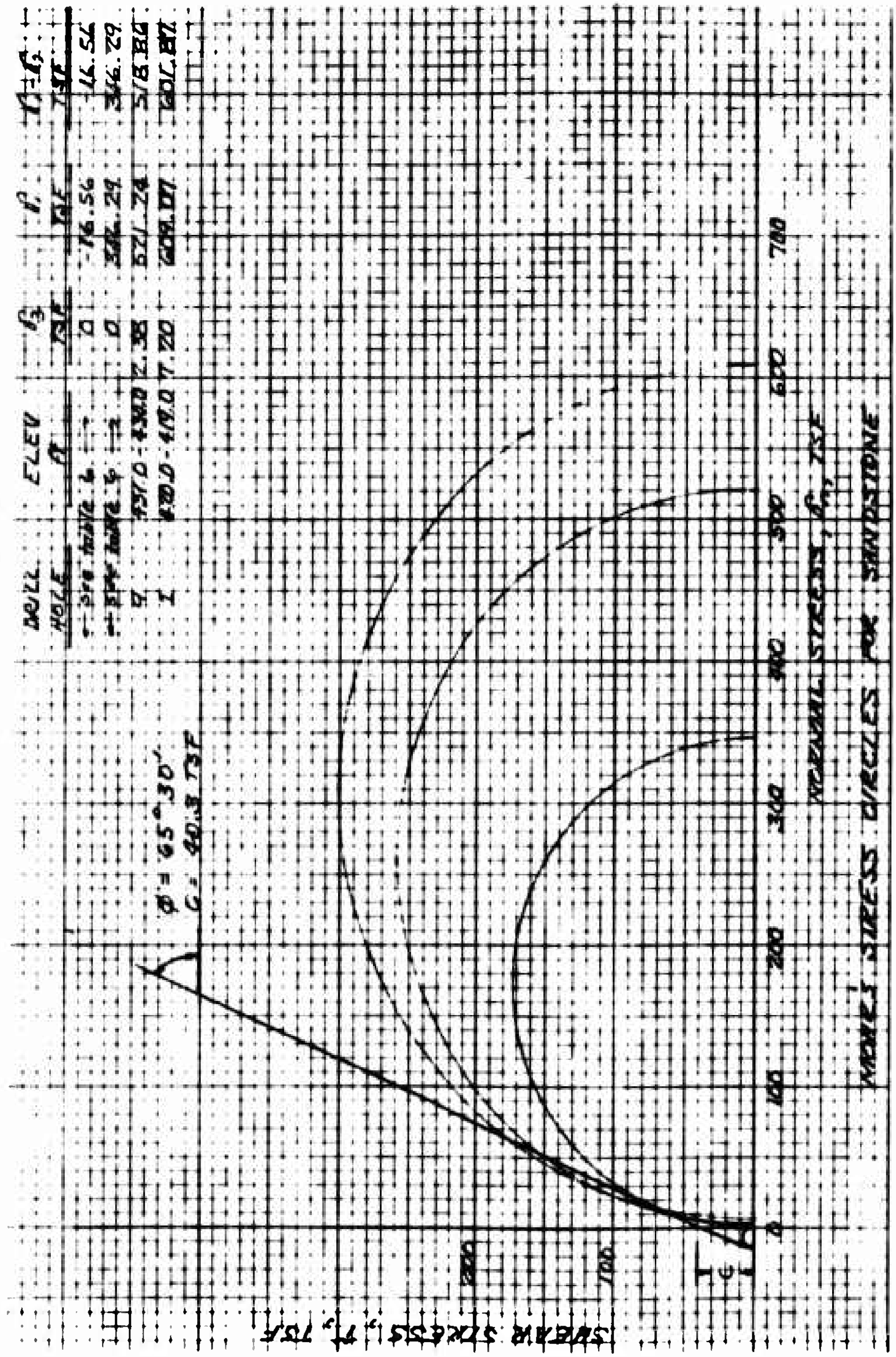
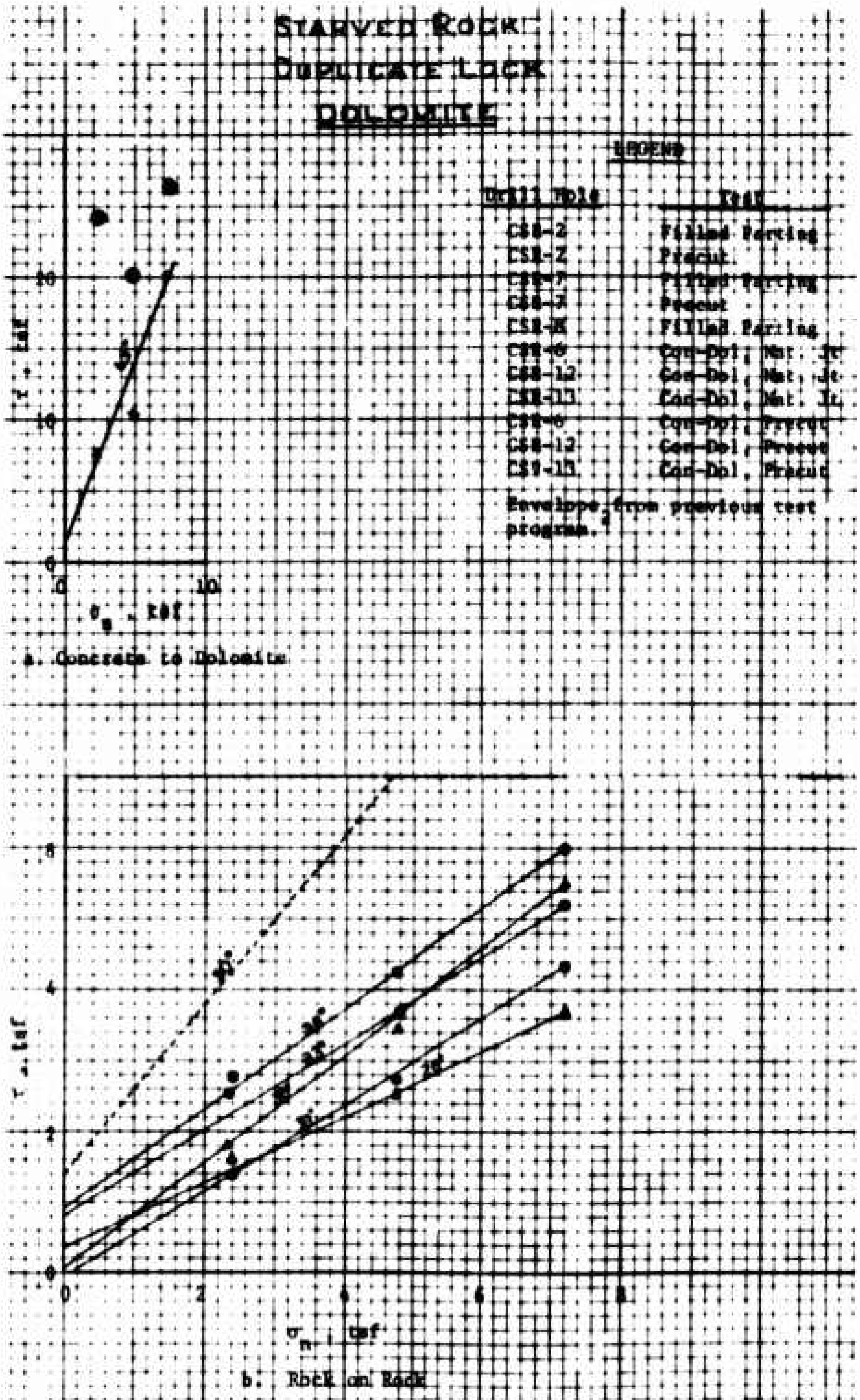
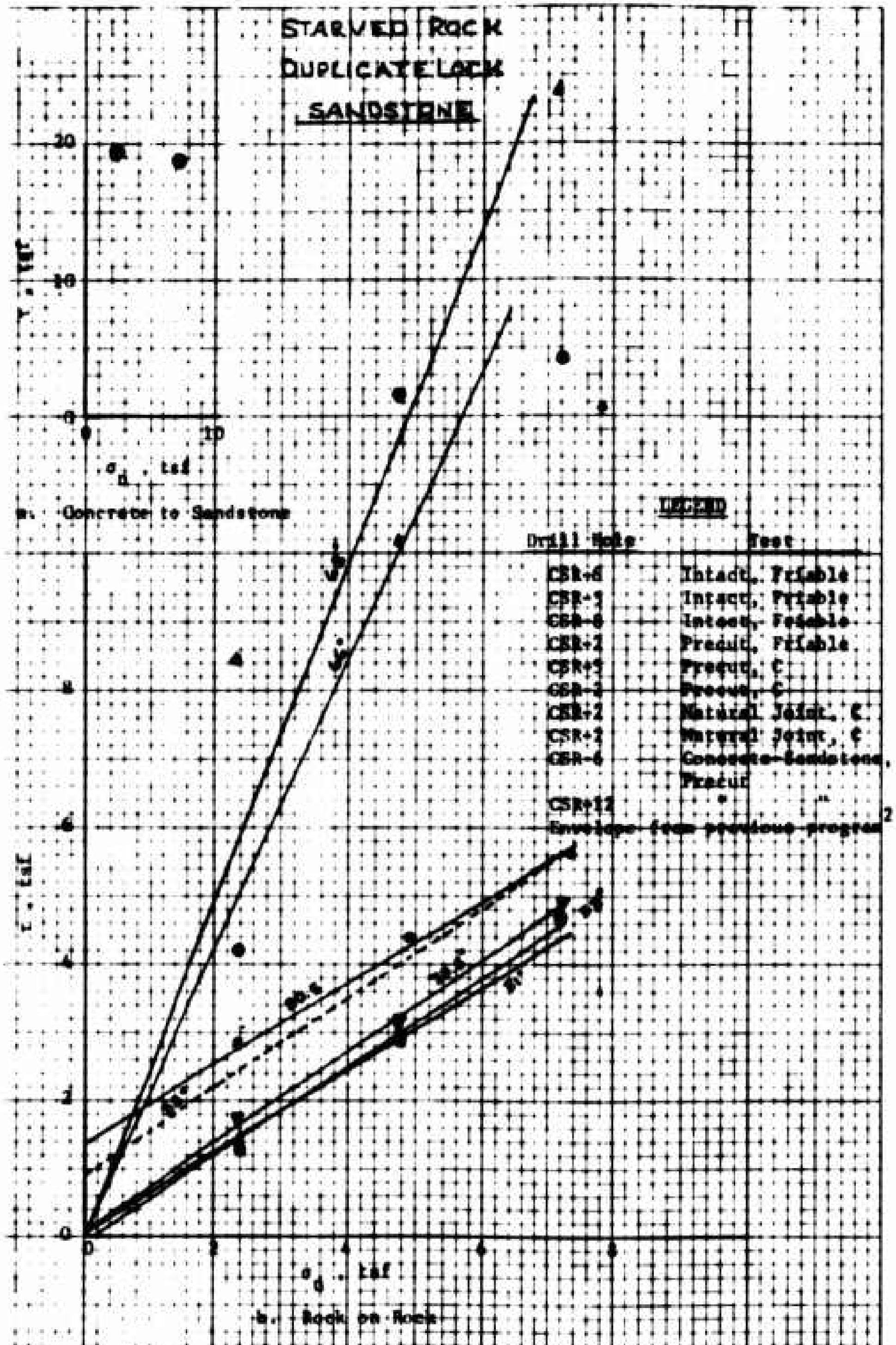

MOHR'S STRESS CIRCLES FOR SANDSTONE
NORMAL STRESS, TSF
DRILL HOLE
ELEV FT
TSF
0
100
200
300
400
500
600
700

PLATE 10



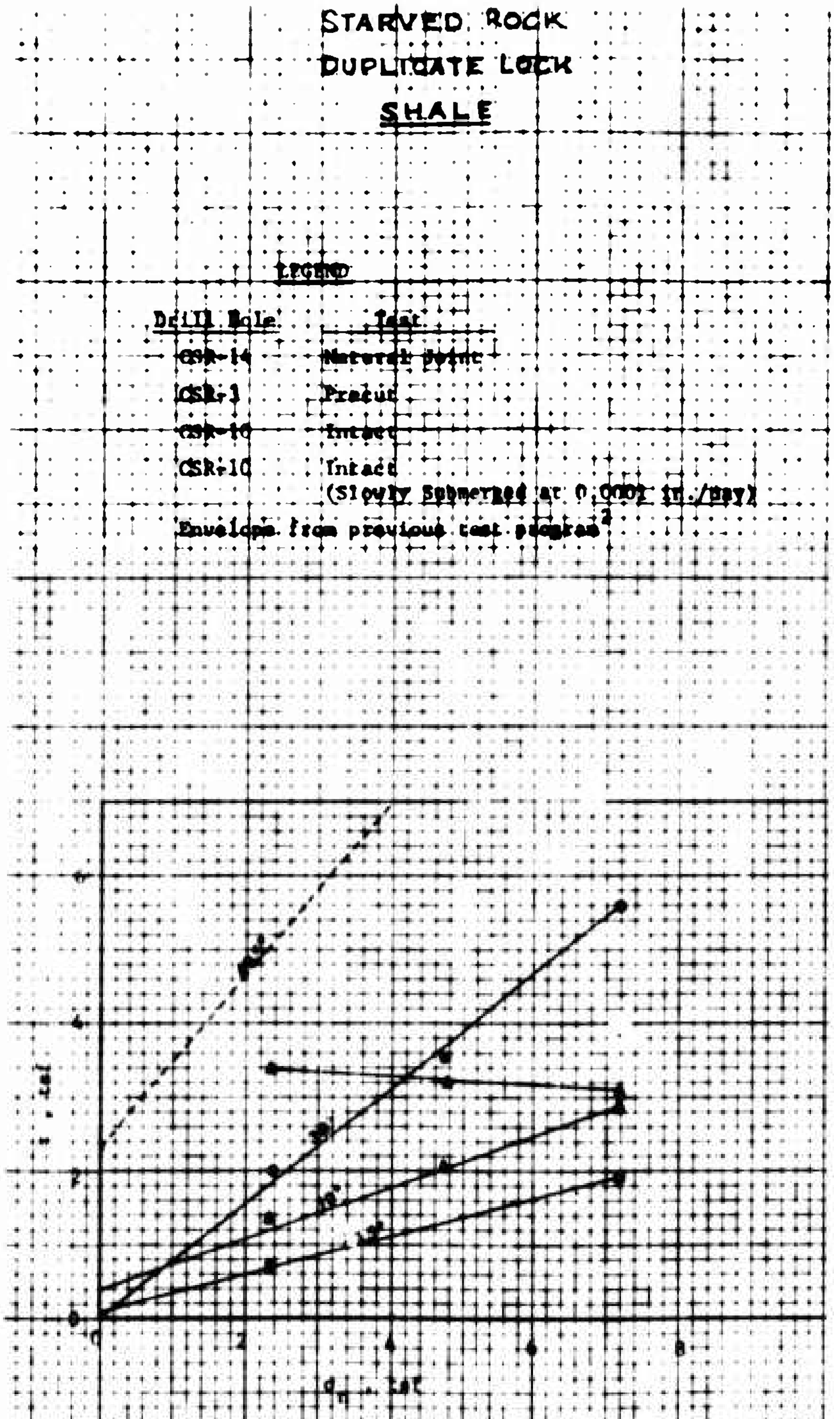
Failure Envelopes for Dolomite Tested in Direct Shear

PLATE 11



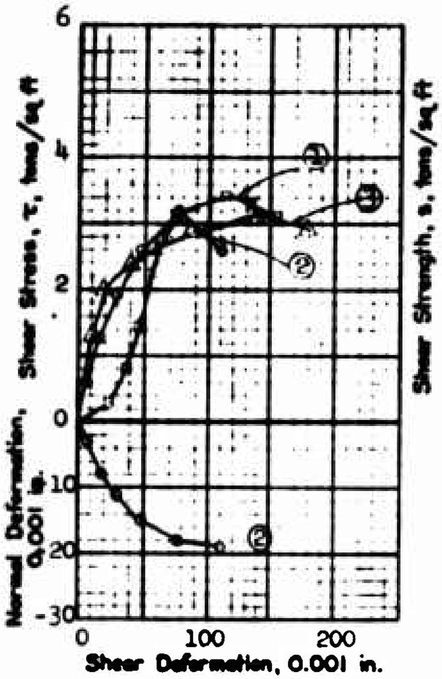
Failure Envelopes for Sandstone Tested in Direct Shear

PLATE 12

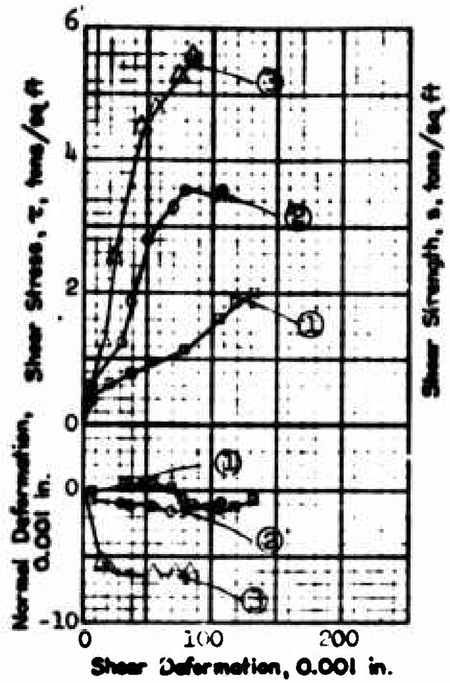


Failure Envelope for Shale Tested in Direct Shear

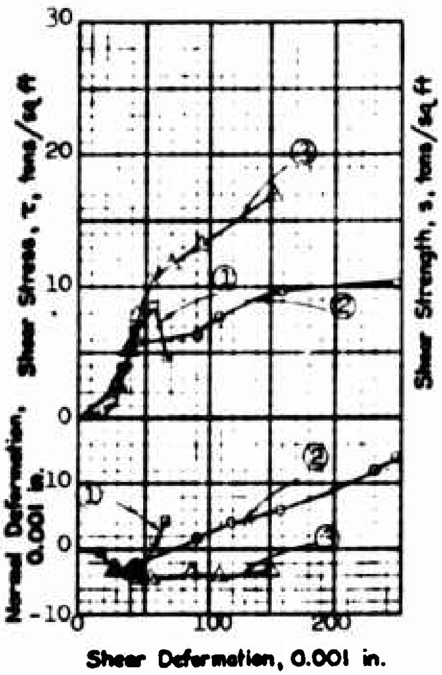
PLATE 13



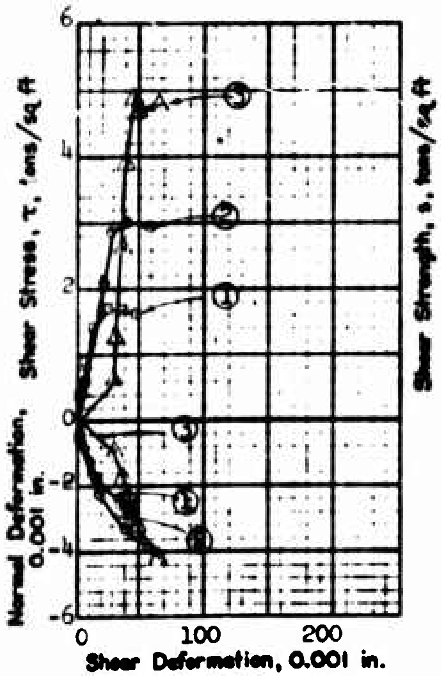
a. Intact shale.



b. Natural joint shale.

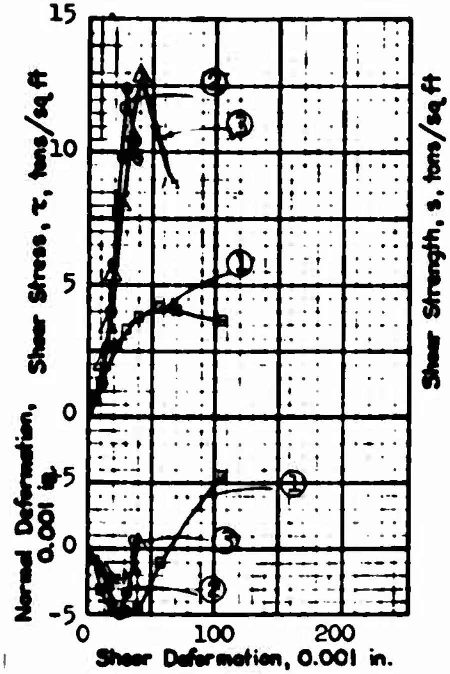


c. Jointed competent sandstone.

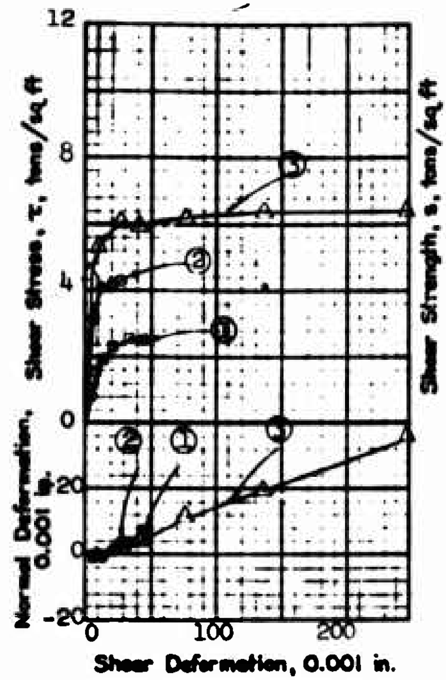


d. Precut friable sandstone.

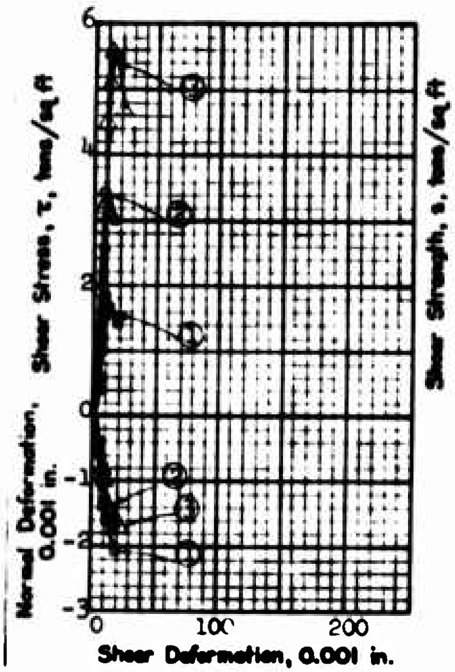
Typical Shear Stress Versus Shear and Normal Deformation



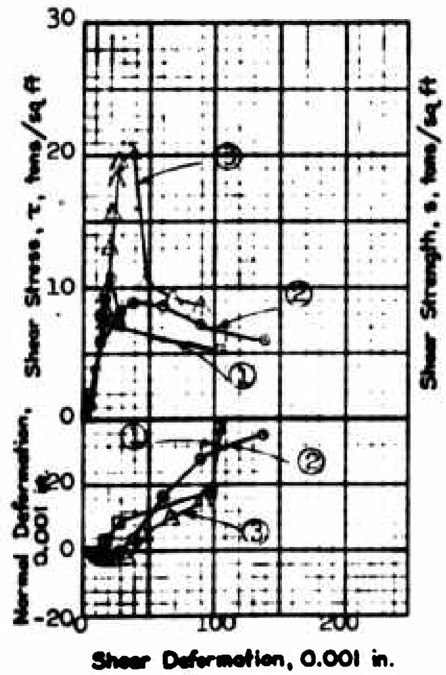
a. Intact friable sandstone.



b. Shale parting in dolomite.



c. Precut dolomite.



d. Concrete to dolomite.

Typical Shear Stress Versus Shear and Normal Deformation

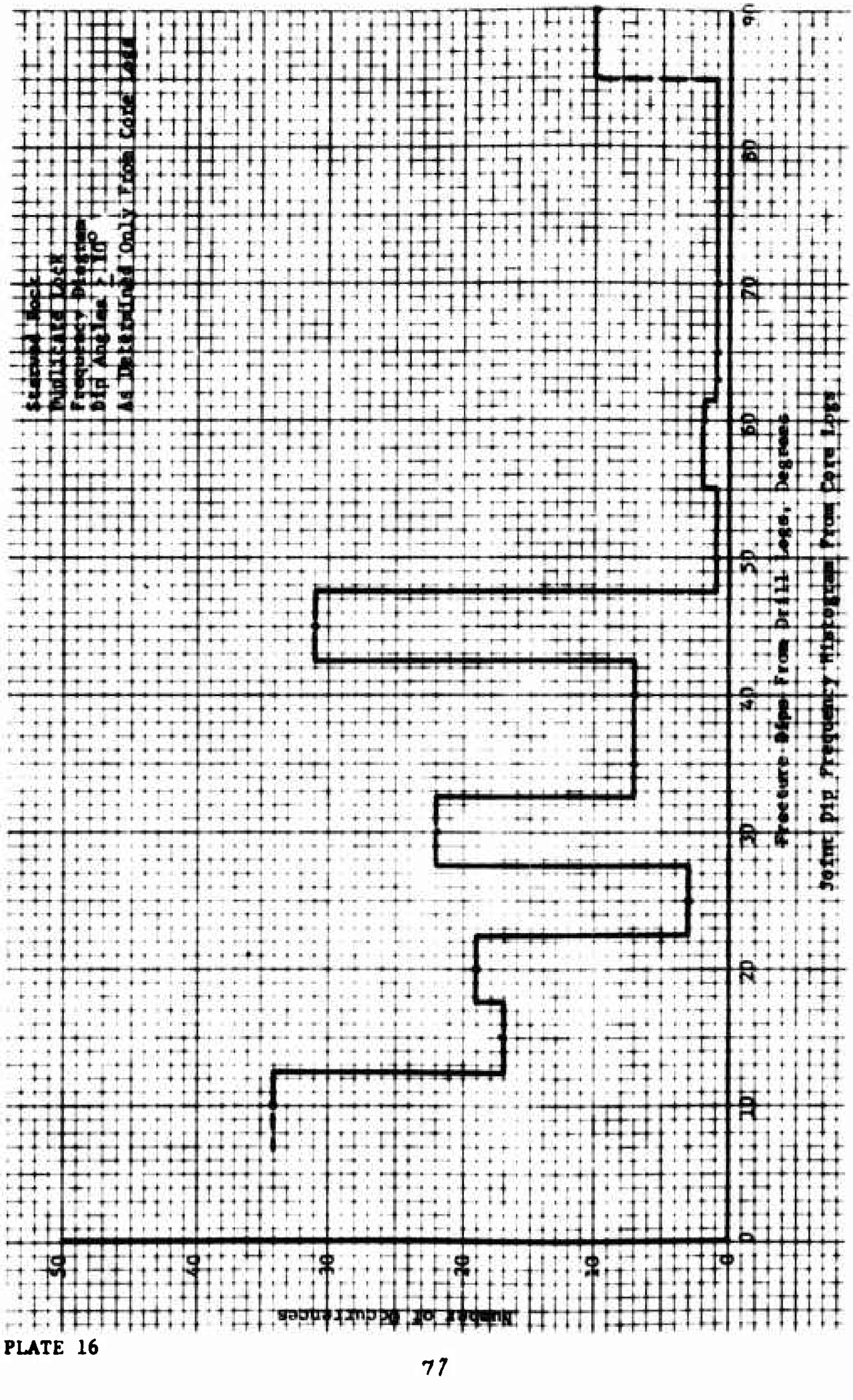
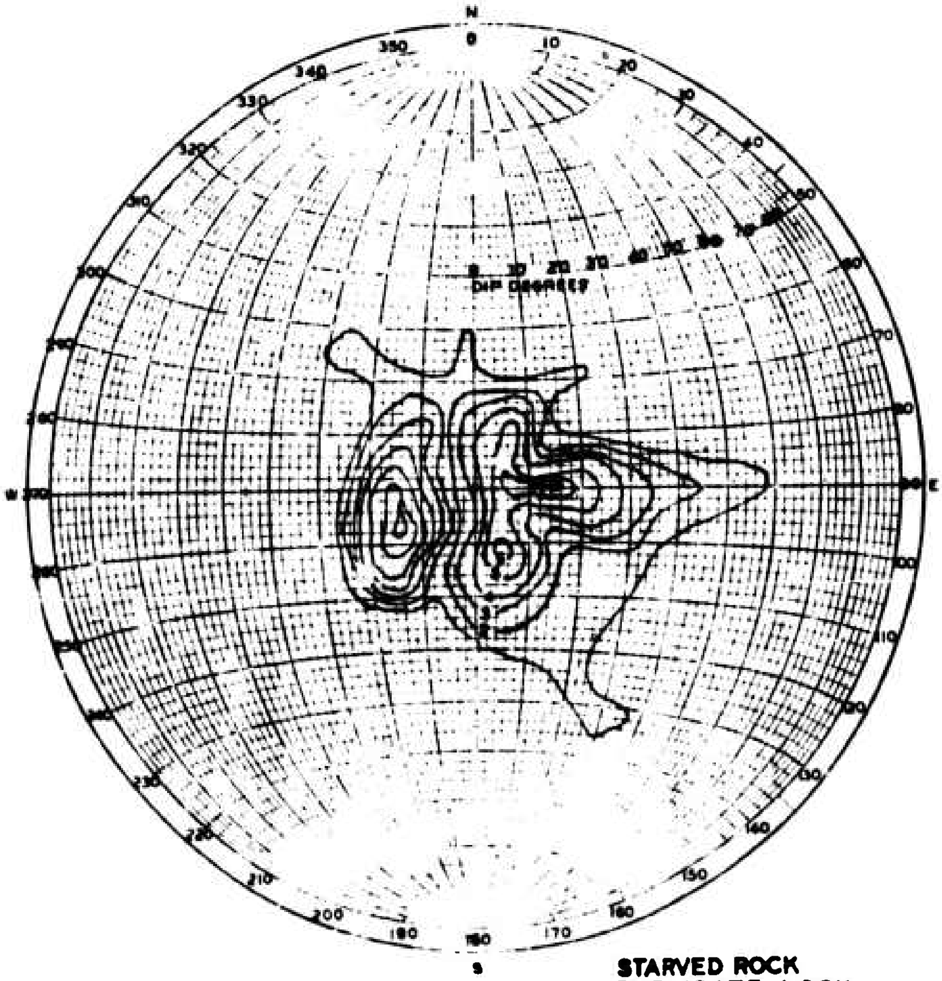
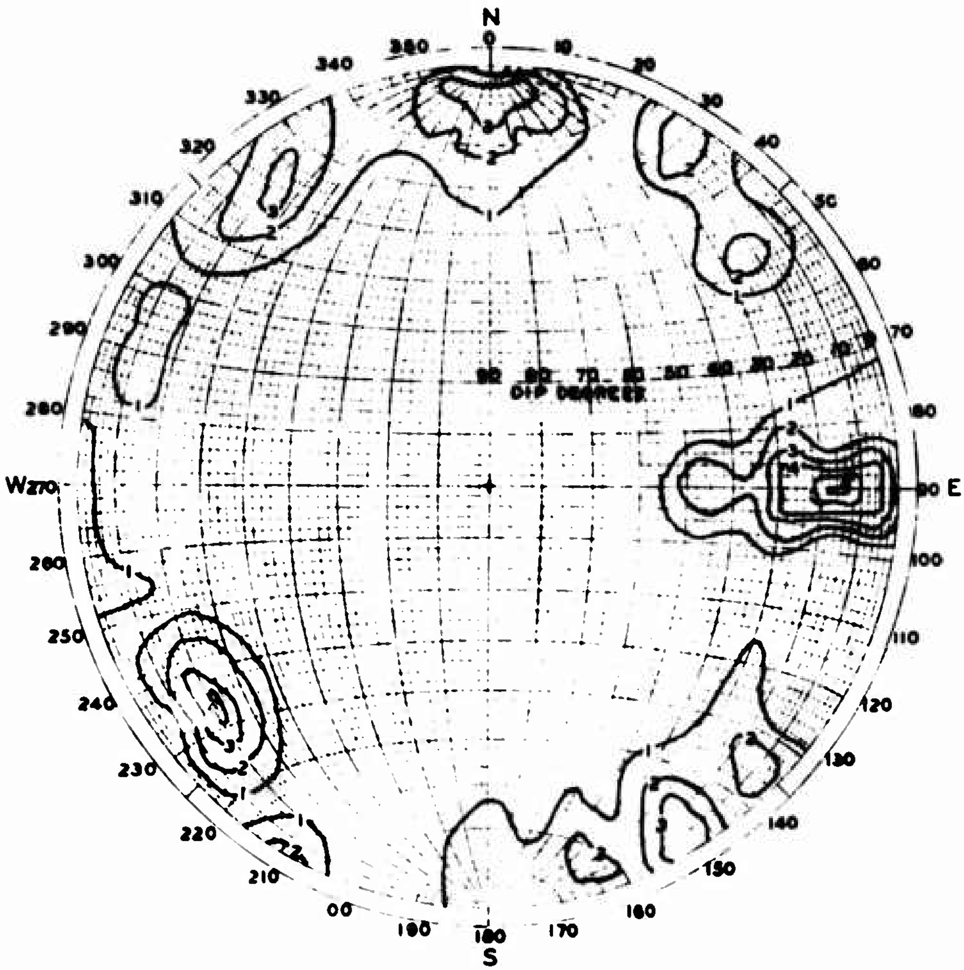

Frequency Diagram
As Determined Only From Core Logs
Fracture Dips From Drill Logs, Degrees
Joint Dip Frequency Histogram From Core Logs
Number of Occurrences
0
10
20
30
40
50
60
70
80
90

PLATE 16



STARVED ROCK
DUPLICATE LOCK
POLAR DIAGRAM
NORMAL VECTORS
(AS DETERMINED BY TE EVIEWER
AND CORE LOG)
NORMAL VECTORS ≥ 10°

Contoured Frequency of Normal Vectors



STARVED ROCK
DUPLICATE LOCK
POLAR DIAGRAM
DIP VECTORS
(AS DETERMINED BY TELEVIEWER AND CORE LOG)
DIP VECTORS ≥ 10°

Contoured Frequency of Dip Vectors

PLATE 18

APPENDIX A

TEST PROCEDURES

Revised
5 May 1975

T-1 Method of Determining Specific Gravity of Solids

Scope and Definition

1. This method covers procedures for determining the specific gravity of solids of rock. The specific gravity of solids of rock is the ratio of the weight in air of a given volume of crushed solids at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature.

Apparatus

2. The apparatus shall consist of the following:

   a. Volumetric flask, 500-cc capacity.

   b. Vacuum pump or aspirator connected to vacuum line.

   c. Oven of the forced draft type, automatically controlled to maintain a uniform temperature of 110 $\pm$ 5 C throughout the oven.

   d. Balance, sensitive to 0.01 g, capacity 500 g or more.

   e. Thermometer, range 0 to 50 C, graduated in 0.1 deg.

   f. Evaporating dish.

   g. Water bath.

   h. Sieves, U. S. Standard No. 4 and No. 30 conforming to ASTM Designation E-11, Standard Specifications for Sieves for Testing Purposes.

   i. Sample splitter suitable for splitting material passing No. 4 and No. 30 sieves.

Calibration of Volumetric Flask

3. The volumetric flask shall be calibrated for the weight of the flask and water at various temperatures. The flask and water are calibrated by direct weighing at the range of temperatures likely to be encountered in the laboratory. The calibration procedure is as follows.

> Fill the flask with deaired, distilled, or demineralized water to slightly below the calibration mark and place in a water bath which is at a temperature between 30 and 50 C. Allow the flask to remain in the bath until the water in the flask reaches the temperature of the water bath. This may take several hours. Remove the flask from the water bath and adjust the water level in the flask so that the bottom of the meniscus is even with the calibration mark on the neck of the flask. Thoroughly dry the outside of the flask and remove any water adhering to the inside of the neck above the graduation; then weigh the flask and water to the nearest 0.01 g. Immediately after weighing, shake the flask gently and determine the temperature of the water to the nearest 0.1 C by immersing a thermometer to the mid-depth of the flask. Repeat the procedure outlined above at approximately the same temperature, then make two more determinations, one at room temperature and the other at approximately 5 deg less than room temperature. Draw a calibration curve showing the relation between temperature and corresponding weights of the flask plus water. Prepare a calibration curve for each flask used for specific gravity determination and maintain the curves as a permanent record. A typical calibration curve is shown in Fig. A1.

Sample

4. Crush the rock core until it all passes a No. 4 sieve. With a sample splitter, separate out 120 to 150 g of representative crushed material and pulverize using a marble mortar and pestle until all of the material will pass a No. 30 sieve. Oven-dry the crushed material to constant weight, weigh to the nearest 0.01 g, and record the weight.

Procedure

5. After weighing, transfer the crushed rock to a volumetric flask, taking care not to lose any material during this operation. To avoid possible loss of preweighed material, the sample may be weighed after transfer to the flask. Fill the flask approximately half full with deaired distilled water. Shake the mixture well and allow it to stand overnight.

6. Then, connect the flask to the vacuum line and apply a vacuum of approximately 29.5 in. mercury for approximately 4 to 6 hr, agitating the flask gently at intervals during the evacuation process. Again allow the flask to stand overnight. Finally, fill the flask with deaired, distilled water to about 3/4 in. below the 500-cc graduation and again apply a vacuum to the flask until the suspension is deaired; slowly and carefully remove the stopper from the flask, and observe the lowering of the water surface in the neck. If the water surface is lowered less than 1/8 in., the suspension can be considered sufficiently dea'-ed. Fill the flask until the bottom of the meniscus is coincident with the calibration line of the neck of the flask. Thoroughly dry the outside of the flask and revmoe the moisture on the inside of the neck by wiping with a paper towel.

Weigh the flask and contents to the nearest 0.01 g. Immediately after weighing, stir the suspension to assure uniform temperature, and determine the temperature of the suspension to the nearest 0.1 C by immersing a thermometer to the mid-depth of the flask. Record the weight and temperature.

7. Compute the specific gravity of the rock solids, G_s , from the following formula:

$$G_s = \frac{W_s \gamma_w}{W_s + W_{bw} - W_{bws}}$$

where: W_s =

W_s = the oven dry weight of the crushed rock sample in grams,

γ_w = unit weight of water at test temperature, g/cc (Table A1),

W_{bw} = weight of flask plus water at test temperature in grams (from calibration curve, Fig. A1), and

W_{bws} = weight of glask plus water plus solids at test temperature in grams.

Report

8. The report shall include the following:

a. The specific gravity of solids of the rock.

b. Oven-dry weight of test sample.

c. Water temperature during test.

Reference Method Appendix IV, EM 1110-2-1906, Laboratory Soils Testing, 30 November 1970.

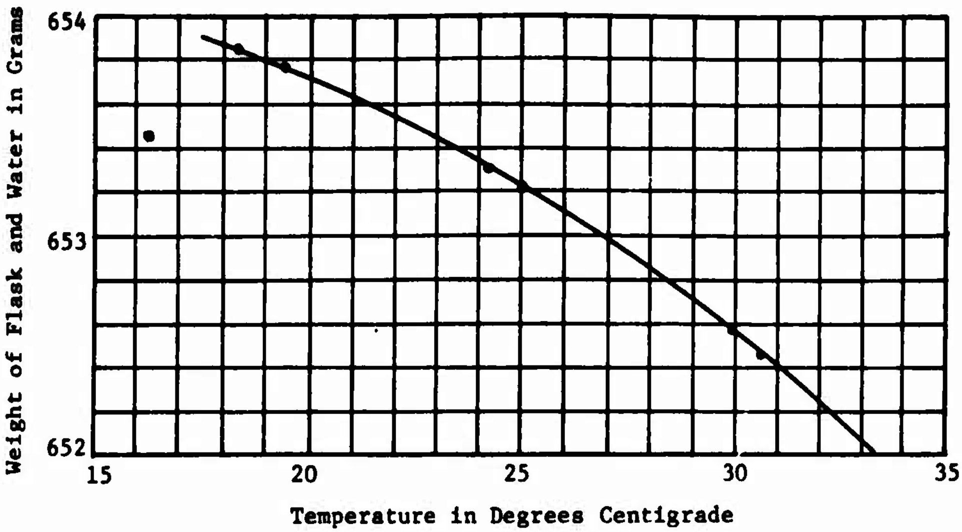


Figure A1. Typical calibration curve of volumetric flask

Revised
5 May 1975

T-2 Method of Determining Unit Weights, Wet, Dry, and Saturated-Surface-Dry, and Moisture Contents of Rock Cores

Scope and Definition

1. This method covers the procedure for determining the unit weight and moisture content of rock cores. The unit weight of a rock core is the weight in air of a unit volume of permeable material (including both permeable and impermeable voids normal to the material), at a stated temperature. The unit weight may be determined for a core in an oven-dry condition (dry unit weight), in the condition in which it was received (wet unit weight irrespective of the degree of saturation), or in a saturated-surface-dry condition (saturated unit weight).

Apparatus

2. The apparatus shall consist of the following:

   a. Balance having a capacity of 5 kg or more and accurate to 1 g.

   b. Wire basket of No. 4 mesh, diameter at least 2 in. greater than that of the core to be tested, walls at least one-half the height of the cylinder, and bail clearing the top of the core by at least 1 in. at all points.

   c. Watertight container in which the wire basket may be suspended with a constant level overflow spout at such a height that the wire basket, when suspended below the spout, will be at least 1 in. from the bottom of the container.

d. Suspending apparatus suitable for suspending the wire basket in the container from the center of the balance platform or pan so that the basket will hang completely below the overflow spout and not be less than 1 in. from the bottom of the container.

e. Thermometer, range 0 to 50 C, graduated to 0.1 deg.

f. Caliper or suitable measuring device capable of measuring lengths and diameters of test cores to the nearest 0.001 in. (or 0.002 cm).

g. Oven of the forced draft type, automatically controlled to maintain a uniform temperature of 110 ± 5 C throughout.

Sample

3. The test core diameter shall not exceed 6 in., and the length shall not exceed 15 in.

Wet Unit Weight

4. The test procedure for determining the wet unit weight of rock cores in the "as-received" condition shall consist of the following steps:

a. Weigh the core as-received to the nearest gram (0.1 g for 3-in. and smaller cores) (W_a) and observe the temperatures in the working area near the core surface.

b. Determine the volume of the core in cubic centimetres by one of the following two methods:

(1) Determine the average length and diameter of the core from measurements of each of these dimensions at evenly spaced intervals covering the surface of the specimen. These measurements should be made to the nearest 0.002 cm or to the nearest 0.001 in. and converted to the nearest 0.002 cm. Calculate the volume using the formula: $V = \frac{\pi}{4} d^2 L$, where V_o = volume in cubic centimeters, d = diameter of the core in centimetres, obtained by averaging the 12 measurements taken as shown in Fig. Method T-3, and L = length of the core in centimetres, obtained by averaging the 10 dimensions shown in Fig. Method T-3.

(2) Coat the surface of the core with wax or other suitable coating until it is watertight, making sure that the coating material does not measurably penetrate the pores of the core. Weigh the specimen, after coating, to the nearest gram (or 0.1 g). The density of the coating material shall be determined. The volume of the coating on the core shall be determined by dividing the weight of coating by the density of the coating. Determine the volume of the coated core by liquid displacement in cubic centimetres. Subtract the volume of the coating material from the volume of the coated core and obtain the volume of the core (V_o) in cubic centimetres.

c. Calculate the wet unit weight of the core in the "as-received" condition from the following formula:

$$\gamma_m = \frac{W_a}{V_o}$$

where:

γ_m = wet unit weight of the core, as-received,

W_a = weight of core in grams, as-received, and

V_o = volume of the core in cubic centimetres.

Moisture Content

5. Moisture content (water content) is the weight of water in the core expressed as a percent of the dry weight of the core. It can be determined using the following procedure.

a. If used, remove the coating used to waterproof the specimen in para 4b(2), and if applicable, brush to remove dust or elements of the coating and weigh the core (W_b).

NOTE: If there is no weight loss in stripping or loss or gain in moisture, $W_a = W_b$.

b. Oven-dry the core to constant weight at 110 ± 5 C (constant weight is achieved when the weight does not change as much as 0.1 percent during any 4-hr drying period) (W_o), cool it to room temperature and weigh it, then record all weights and room temperature in the area of the test core on the data sheet.

c. Calculate the moisture content in percent of dry weight as follows:

$$w = \frac{W_b - W_o}{W_o}$$

where:

w = moisture content in percent,

W_b = as-received weight of undried core with coating removed,

W_o = oven-dry weight.

Dry Unit Weight

6. The dry unit weight (oven-dry) can be calculated using the following formula:

$$\gamma_d = \frac{\gamma_m}{1 + w}$$

where:

γ_d = dry unit weight of the oven-dry core,

γ_m = wet unit weight of the core as-received, and

w = moisture content (para 5c).

Saturated Unit Weight

7. The test procedure for determining the saturated unit weight of the rock cores in the saturated-surface-dry condition shall consist of the following:

a. Brush the core thoroughly to remove dust or other coatings from the surface and weigh.

b. Immerse the core in water at 15 to 25 C for a period of 24 hr.

c. Remove the specimen from the water and roll it in a large absorbent cloth until all visible film of water is removed, although the surface may appear to be damp. If weighing cannot be accomplished immediately, keep the core covered with a damp cloth to prevent evaporation of absorbed water.

d. Weigh the core in air to the nearest gram (0.1 g for 3-in. or smaller cores).

e. Immediately after weighing in air, place the core in the wire basket, adjust the water level,* and determine its weight in water. Take the temperature of the water. Suspend the empty basket in the water* and weigh. Weight of the core and basket in water minus weight of the basket in water is the weight of the core in water.

f. Calculate the saturated unit weight of the core in the saturated-surface-dry condition from the following formula:

$$\gamma_G = \frac{W_s \gamma_w}{W_s - W_w}$$

where:

γ_G = saturated unit weight of the core, saturated-surface-dry,

W_s = weight in air of the saturated-surface-dry core in grams,

W_w = weight in water of the saturated-surface-dry core, and

γ_w = unit weight of water at test temperature in grams per cubic centimetre (Table A1).

---

* Before each weighing in water, bring the level of the water in the container up to the bottom of the overflow spout.

Report

8. The report shall include the weight of the core as-received, its oven-dry weight, its saturated-surface-dry weight in air, its saturated-surface-drv weight in water, the length and diameter of the core, the unit weights, the moisture content, and the test temperature in the area of the core surface during testing.

Revised
5 May 1975

T-4 Method of Calculating Various Material Properties of Rock Core

Scope and Definition

1. This method covers the procedure for calculating various material properties of rock core using parameters determined by Methods T-1, T-2, and T-3.

2. Formulas for calculations:

a. Volume of solids (V_s) = $\frac{\gamma_d}{G_s}$.

b. Volume of voids (V_v) = $(1 - V_s)$.

c. Volume of water (V_w) = $(\gamma_d \times w)$.

d. Volume of air (V_a) = $(V_v - V_w)$.

e. Degree of saturation (S_r) = $(\frac{V_w}{V_v}) \times 100$.

f. Degree of saturation (S_r) = $(\frac{\gamma_m - \gamma_d}{n \times \gamma_w}) \times 100$.

g. Void ratio (e) = $(\frac{V_v}{V_s})$.

h. Percent porosity (n) = $(\frac{e}{1 + e}) \times 100$.

i. Percent porosity (n) = $(\frac{G_s - \gamma_d}{G_s}) \times 100$.

j. Percent air voids $1 - (V_s + V_w) \times 100$.

k. Percent absorption (after saturation) (by weight) = $\frac{W_w - W_o}{W_o} \times 100$.

l. Percent absorption (after saturation) (by volume) = $\frac{W_w - W_o}{V_o} \times 100$.

TABLE 1

Unit Weight of Water at Various Temperatures

Temperature C	Unit Weight g/cc	Temperature C	Unit Weight g/cc	Temperature C	Unit Weight g/cc
0	0.99987	16	0.99897	32	0.99505
1	0.99993	17	0.99880	33	0.99473
2	0.99997	18	0.99862	34	0.99440
3	0.99999	19	0.99843	35	0.99406
4	1.00000	20	0.99823	36	0.99371
5	0.99999	21	0.99802	37	0.99336
6	0.99997	22	0.99780	38	0.99299
7	0.99993	23	0.99756	39	0.99262
8	0.99988	24	0.99732	40	0.99224
9	0.99981	25	0.99707	41	0.99186
10	0.99973	26	0.99681	42	0.99147
11	0.99963	27	0.99654	43	0.99107
12	0.99952	28	0.99626	44	0.99066
13	0.99940	29	0.99597		
14	0.99927	30	0.99567		
15	0.99913	31	0.99537		

APPENDIX B

NOTATION

APPENDIX B

Dol - Dolomite
Ls - Limestone
Ss - Sandstone
Sh - Shale
Ch - Chert
Gr - Granite
S - Sand
Gr - Gravel
Cl - Clay
Chy - Cherty
Calc - Calcareous
X-Line - Crystalline
Foss - Fossiliferrous
Sty - Styolitic Bed
Fragm - Fragmented
Tr - Trace
Interb - Interbedded
W-C - Well-Cemented
Sf - Soft
Fri - Friable
Inc - Inclusion
Wtr - weathered
Lyr - Layer
Nod - Nodule
W/ - With
V - Very
Vert - Vertical
Slg - Slightly
Mod - Moderately
Fi - Fine
Blu - Blue
Bl - Blue
Blk - Black
Gry - Gray
Grn - Green
Drk - Dark
Fr - Fracture
Ptg - Parting
Jt - Joint
Hl - Hairline
SB - Structural Break
BP - Bedding Plane
X-Bed - Cross-Bedded
Prob MZ - Probable Missing Zone