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CONCRETE AND ROCK TESTS REHABILITATION WORK, BRANDON ROAD DAM, ILLINOIS WATERWAY CHICAGO DISTRICT

by

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May 1978

Final Report

Approved For Public Release; Distribution Unlimited

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

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Drilling for field testing and laboratory testing was carried out for the U. S. Army Engineer District, Chicago, as part of a stabilization program at the Brandon Road Dam on the Illinois Waterway. A previous stability investigation concluded that all sections of the dam failed to meet current overturning criteria. It was recommended that the dam be stabilized by installation of grouted, prestressed tendons. This report presents physical property data of concrete and foundation rock for use in a stability analysis and the design of

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an anchorage system involving grouted steel tendons. A down hole teleview was used to obtain orientation of natural discontinuities in the foundation rock. Pressure transducer measurements were taken in the field in order to monitor uplift pressures at the base of the dam. Laboratory testing included the determination of characterization properties (compressive strength, unit weight, tensile strength, compressional wave velocities) and engineering design properties (elastic moduli, triaxial strength including multistage loading, direct shear of intact and discontinuous rock samples, and consolidated-undrained (R) and drained (S) triaxial strength of overburden samples).

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PREFACE

This testing program, "Concrete and Rock Tests, Rehabilitation Work, Brandon Road Dam, Illinois Waterway, Chicago District," was conducted for the U.S. Army Engineer District, Chicago. The work was authorized by DA Form 2544, NCC-IA-77-31, dated 8 October 1976.

Drilling was conducted by personnel of the Explorations Branch of the Soils and Pavements Laboratory (S&PL) of the U.S. Army Engineer Waterways Experiment Station (WES) during the period October 1976-December 1976 under the direction of Mr. Mark Vispi. Laboratory tests were performed at the Concrete Laboratory (CL) and the S&PL of the WES during the period January 1977-March 1977 under the direction of Messrs. Bryant Mather and J. M. Scanlon, both of CL. Mr. W. B. Steinriede, S&PL, conducted the televiewer logging, Mr. D. L. Ainsworth, CL, conducted the pressure transducer tests, Mr. G. P. Hale supervised the laboratory testing that was conducted in the S&PL, and Mr. G. S. Wong, CL, conducted the petrographic examination. The stability analysis and design of an anchorage system referenced in this report were conducted by Dr. C. E. Pace, CL. Mr. R. L. Stowe was Project Leader and was assisted in performing laboratory work at the CL by Messrs. F. S. Stewart and J. B. Eskride. Mr. Stowe prepared this report.

The Commander and Director of WES during the conduct of the program and the preparation and publication of this report was COL J. L. Cannon, 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
inches	0.0254	metres
feet	0.3048	metres
pounds (mass)	0.4535924	kilograms
pounds (force)	4.448222	newton
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (force) per square inch	0.006894757	megapascals
tons (force) per square foot	0.09576052	megapascals
feet per second	0.3048	metres per second
miles	1.609	kilometers

CONCRETE AND ROCK TESTS, REHABILITATION WORK, BRANDON ROAD DAM, ILLINOIS WATERWAY CHICAGO DISTRICT

PART I: INTRODUCTION

Location of Study Area

1. The Brandon Road Lock and Dam is located near Rockdale, Will County, Illinois. It is on the Des Plaines River at river mile 286. It is about a 35-mile* drive from the southwest city limits of Chicago.

2. The 1976 foundation investigation for the rehabilitation work of the dam involved the drilling of eight holes. Three of the eight holes were drilled into the river silt (sludge) in the upstream (US) pool and samples recovered represented an average depth of 9.3 ft. The locations of the drilled holes are presented in Figure 1. The borings are designated BR WES-1 through 8-76; the letter and numbers stand for Brandon Road, Waterways Experiment Station (WES), number of hole, and year hole was drilled (1976).

Background

3. At a meeting held at the offices of the Chicago District (CDO) on 15 October 1976, representatives of the Concrete Laboratory (CL) and the Soils and Pavements Laboratory (S&PL), WES, were requested to submit a proposal for work to assist CDO in the rehabilitation of Brandon Road Dam. The names and organizations of the attendees at the 15 October 1976 meeting are tabulated below.

ATTENDEES

Fred Paterson	NCD
Terrence Smith	NCD

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* A table of factors for converting U.S. customary to metric (SI) units of measurement is given on page 4.

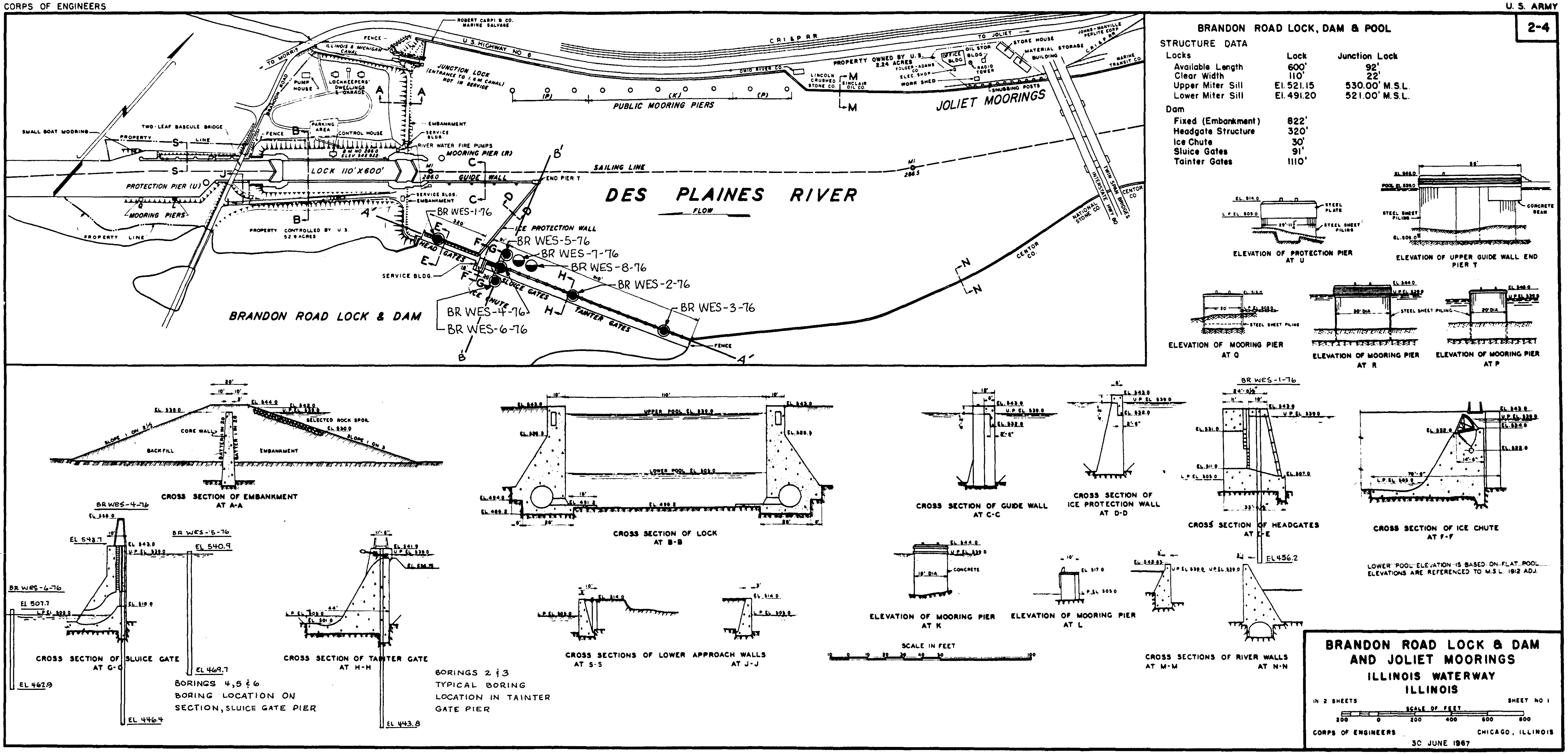


Figure 1. Drill hole locations.

Terry Soupos	NCD
Carl Pace	WES
Richard Stowe	WES
Mark A. Vispi	WES
I. Juzenas	CDO
Geroge Sanborn	CDO
C. Ruiter	CDO
CPT James R. Van Epps	CDO

Note: NCD - North Central Division

WES - Waterways Experiment Station

CDO - Chicago District Office

4. As a result of a previous investigation of the Brandon Road Dam conducted in early 1973,[1] it was concluded that all sections of the dam failed to meet current overturning criteria. The CDO recommended a plan of proposed rehabilitation of the dam. It was recommended that the dam be stabilized by installation of grouted, prestressed tendons. The proposed method is presented in Plates 1 through 4 of this report (Plates 83 through 86, respectively, of a handout distributed at the 15 October 1976 meeting at CDO). The installation of the tendons is designed to enable the structure to meet current overturning requirements under all loading conditions.

5. WES was asked by the CDO staff to review four pertinent reports and then conduct field drilling; recommend field and laboratory testing of concrete, rock, and silt; validate a previous stability analysis; determine, present, and analyze a prestressed anchoring system. The results of structures work will be presented under a separate cover.[1] Copies of four reports, a stability investigation report, a periodic inspection report, a stability analysis report, and a geological foundation report (all pertinent to the lock and dam) were received at the October meeting for review and background information.[2-5] The reports were reviewed and after assessing the extent and difficulty of drilling, input requirements for finite-element codes, and the proposed stabilization method to be used at Brandon Road Dam, a proposal was prepared, sent to CDO, and approved by CDO.

6. After considering the engineering characteristics of the foundation rock and the material properties as described in the appropriate reports, a recommended in situ and laboratory testing program, as outlined in Table 1, was proposed. Pressure transducer tests were recommended in order to evaluate the uplift pressures near the borehole. Televiewer logging was proposed to determine the presence and orientations of discontinuities in the foundation in order that their effect on foundation stability and on the proposed rehabilitation plan could be properly evaluated.

7. A minimum number of characterization 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. Furthermore, it was recommended that a detailed petrographic examination be conducted on suspected clay samples.

8. 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 in order to properly evaluate their effect on the structure.

Objectives

9. The objectives of this study were as follows:

   a. Review available information from reports to enable proper selection of specimens for testing.

   b. Conduct drilling for field and laboratory testing of concrete, rock, and silt.

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

   d. Prepare a concrete and rock data appendix for the rehabilitation design memorandum.

Scope

10. The drilling was accomplished using a WES drilling crew, plant, and supplies. A geologist from WES logged the core and preserved it for laboratory testing. Delivery of the core and silt samples to WES was made in two shipments, one in mid-November 1976 and one in early December 1976. The laboratory testing program was initiated after receiving the first shipment 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 shipment and the geological information presented in Reference 5 were used in the initial selection of representative test specimens.

11. The objectives of this study were accomplished by drilling concrete and rock core and sampling silt, and by conducting characterization 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.

12. A borehole pressure transducer was used in four of the eight drilled holes to determine uplift pressure. A borehole televiewer[6] was used in five of the eight boreholes to obtain information concerning the

orientations of discontinuities. The orientations of discontinuities in relation to the dam were considered in making the foundation appraisal.

PART II: FOUNDATION EXPLORATIONS

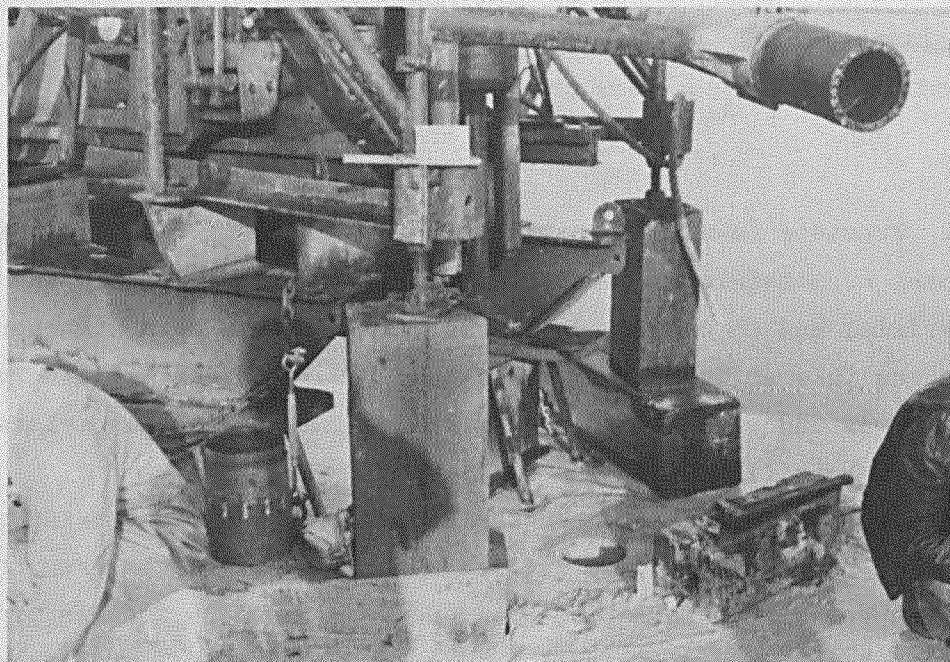
Previous Explorations

13. Previous foundation explorations (1970 and 1971) were completed for purposes of obtaining a foundation appraisal of the bedrock and backfill materials, and to provide design criteria for use in a structural stability analysis at Brandon Road Lock and Dam. Another drilling program (1972) was undertaken for the purpose of installing piezometers in the lock wall backfill.

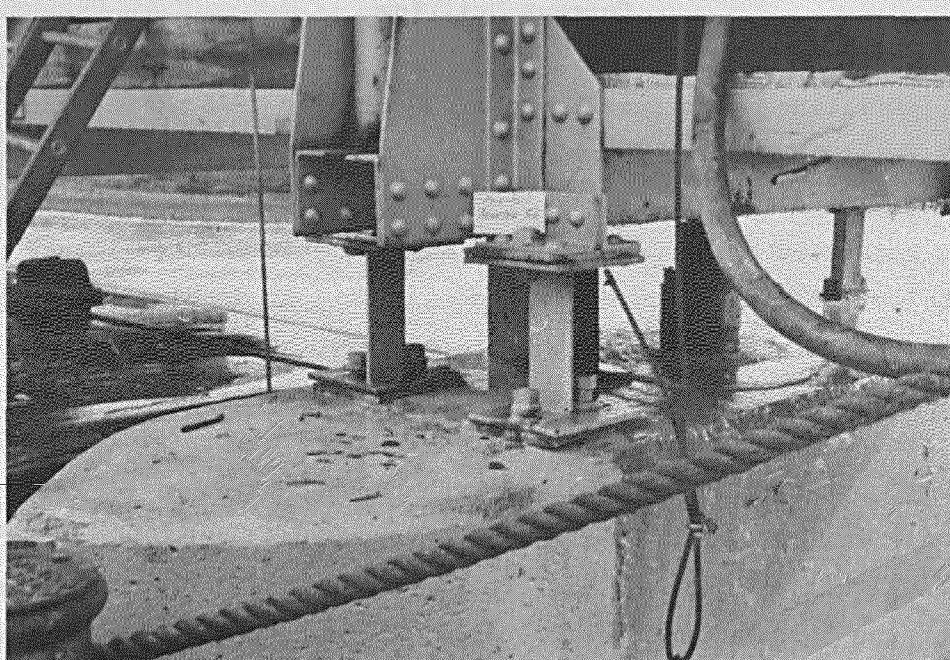
14. Five borings penetrated the bedrock with only one being drilled near the dam. Hole SACBR-6 was drilled about 14 ft upstream from tainter gate 9. The geologic information obtained from these borings is presented in Reference 5 and served as a valuable reference for the work accomplished during this investigation.

Current Drilling

15. Drilling operations began on 30 October 1976 with boring number BR WES-1-76 and were completed on 9 December 1976 with boring number BR WES-8-76 (see Figure 1 for drill hole locations). Drilling equipment consisted of an Acker Toredo Mark II skid-mounted rotary drill rig. Six-inch ID diamond core bits and a 5-ft long double-tube barrel were used to drill the concrete and the bedrock. The upstream silt was sampled using a 3-in.-diam by 2-1/2-ft long Hvorslev fixed piston sampler driven by the hydraulic system of the drill rig. Access to the drill holes was by a marine floating plant with the exception of BR WES-1-76; the skid rig was pulled to this location in the head-gate section. Typical drill rig setups are shown in Figures 2 and 3. Pertinent information about the borings drilled at Brandon Road Dam is tabulated below.

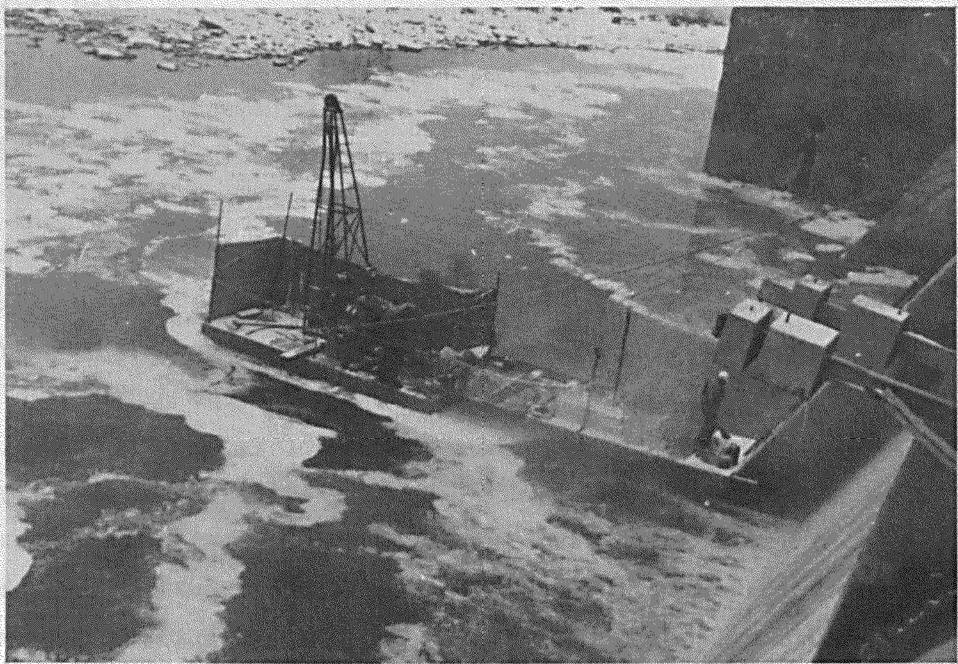


a. Drilling on hole BR WES-1-76, headgate section



b. Typical drill rig setup over holes BR WES-2 and 3-76, tainter-gate section

Figure 2. Drill rig setup for holes BR WES-1, 2, and 3-76



Drill rig setup over hole BR WES-6-76, downstream
of concrete apron and sluice-gate pier

Figure 3. Drill rig setup over holes BR WES-4 and 6-76

Boring No. BR WES-76	Section Hole Located	Elevation, Top of Hole, ft	Depth of Hole, ft	Material Drilled
1	Head-gate pier	543.0	86.8	41.3 ft concrete; 45.5 ft bedrock
2	Tainter-gate pier between gates 8 and 9	541.9	98.3	52.9 ft concrete; 45.6 ft bedrock
3	Tainter-gate pier between gates 18 and 19	541.9	97.9	52.5 ft concrete; 45.5 ft bedrock
4	Sluice-gate pier	543.7	97.3	51.7 ft concrete; 45.6 ft bedrock
5	US pool	540.9	71.2	13.0 ft silt; 6.5 ft gravel and boulders; 31.7 bedrock
6	DS pool	507.7	42.8	4.5 ft gravel and boulders; 30.3 ft bedrock
7	US pool	540.9	17.5	8.5 ft silt
8	US pool	540.9	17.5	8.5 ft silt

16. Continuous samples were obtained except in the first portion of the US and downstream (DS) holes. Some of the overburden sediments containing organic materials, gravels, and boulders could not be sampled. Four- and eight-inch casing was set in the silt and gravels, respectively, when mecessary to keep the borings open. The casing was set to competent bedrock.

17. Total footage pushed and drilled was 30.0 and 453.5 ft, respectively; 198.4 ft of concrete and 255.1 ft of rock were drilled. Representative samples of concrete and all of the bedrock samples were preserved for laboratory testing. The procedure for preserving the samples was as follows: As the core came out of the core barrel it was laid out and pieced together; a cursory log was constructed; a photograph was made; then the core was wrapped in plastic and waxed with a lukewarm mixture of 50:50 microcrystalline and paraffin wax. The average length of time the core was exposed to the air was 4 ± 1 min. The core was wetted during the exposed period. Core recovery was very good, ranging from 99 to 100 percent.

The bedrock is apparently tight as evidenced by little or no loss of drilling water.

18. The four drill holes across the dam were plugged at the top with wood but left open for the remaining depth. The US and DS holes that were drilled into bedrock were grouted the full depth with Sack-Crete, a commercially available prepackaged concrete mixture.

Pressure Transducer Measurements

19. Measurements of the uplift pressure at the base of the dam were taken in the four holes along the crest of the dam. The drilling rig was used to place a packer at a predetermined elevation. After measurements were taken, the drilling was continued.

20. These measurements were made immediately after the concrete-rock interface had been reached. The technique consisted of a strain-gaged diaphragm pressure transducer mounted in the bottom of a packer with the signal leads connected to required instrumentation on the top of the dam. With the gage and packer in place, the packer was inflated to seal the hole and pressure measurements were made continuously for a sufficient period of time to determine the uplift pressure. Approximately one hour was required before a constant pressure was recorded. The uplift pressure can only be associated with the area immediately adjacent to the borehole.

Televiewer Logging

21. 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 WES televiewer was run in the four borings across the crest of the dam and in the DS hole, BR WES-6-76. The full depth of the holes was logged, hence information on fractures in the concrete and bedrock was obtained. The televiewer was run in the holes from a geophysical truck that was placed on a floating barge (Figure 4). Cable to the downstream hole was fed through a series of pullies.

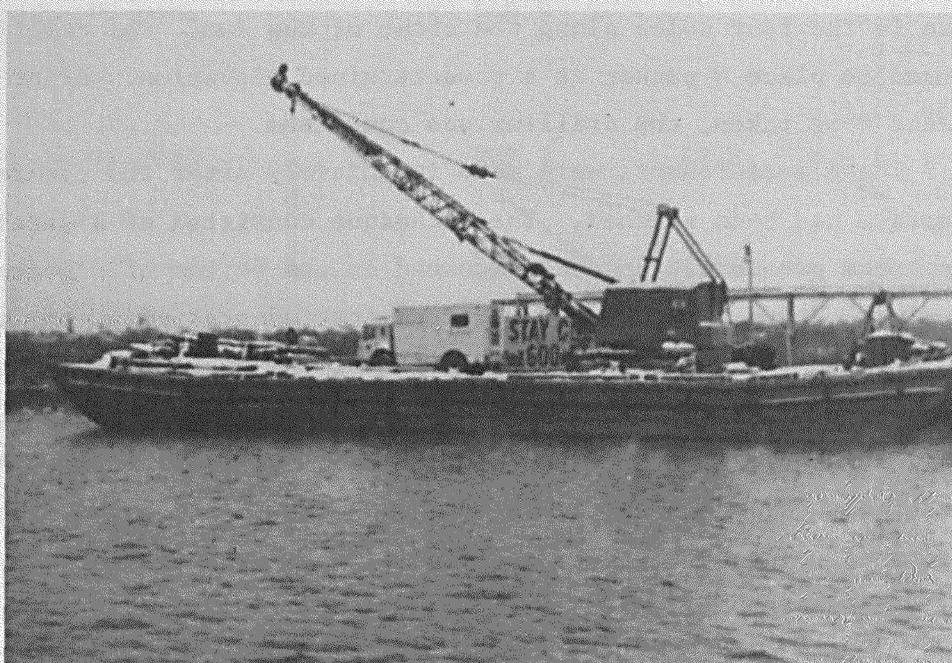


Figure 4. Barge-mounted geophysical truck

22. The televiewer contains a continuously rotating piezoelectric transducer which probes the borehole wall with bursts of acoustic energy in a manner similar to sonar. 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, a hard surface better than a soft one, 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.

PART III: GEOLOGICAL CHARACTERISTICS OF FOUNDATION

Bedrock Stratigraphy

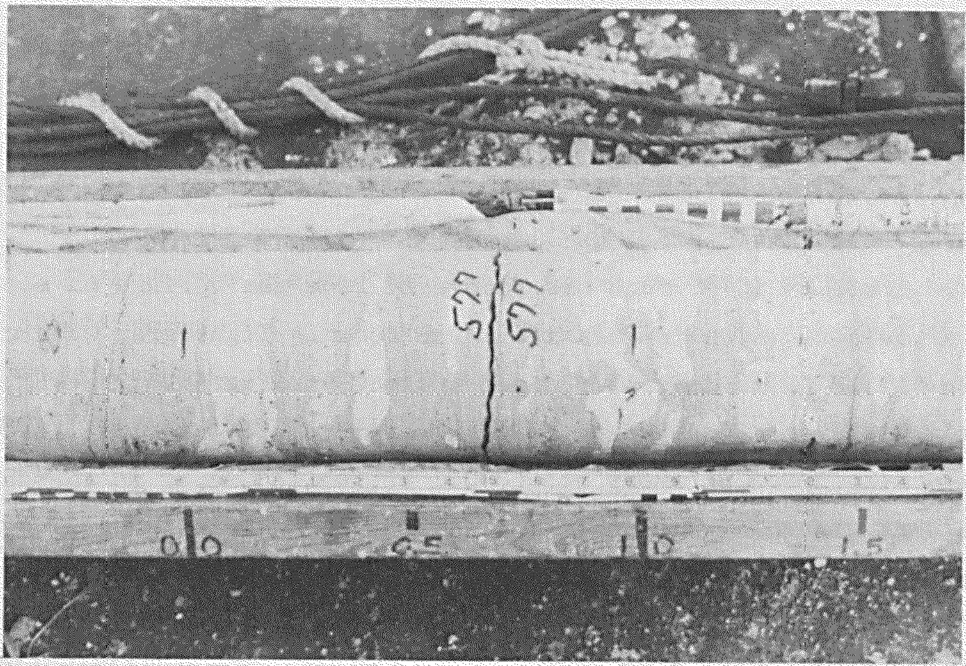
23. Visual observations of the core received at WES are in general agreement with descriptions of core and of stratigraphy given in Reference 5. Therefore, only a brief description of the foundation rock will be given in the following paragraphs.

24. The bedrock beneath and adjacent to the dam is dolomite of the Brandon Bridge member of the Joliet Formation which is part of the Niagaran series of Silurian age. The dolomite is a light gray, fine-grained rock. The upper portion contains blue-green, clay-filled bedding partings (BP) which give the rock a light blue-green appearance. The rock lower in the section contains numerous dark gray, shale-filled BP. The rock becomes increasingly shaly with depth and the shale-filled BP become less distinctive. Numerous chert nodules and bands occur throughout the rock. Figure 5 presents typical photographs made of the upper and lower sections of bedrock.

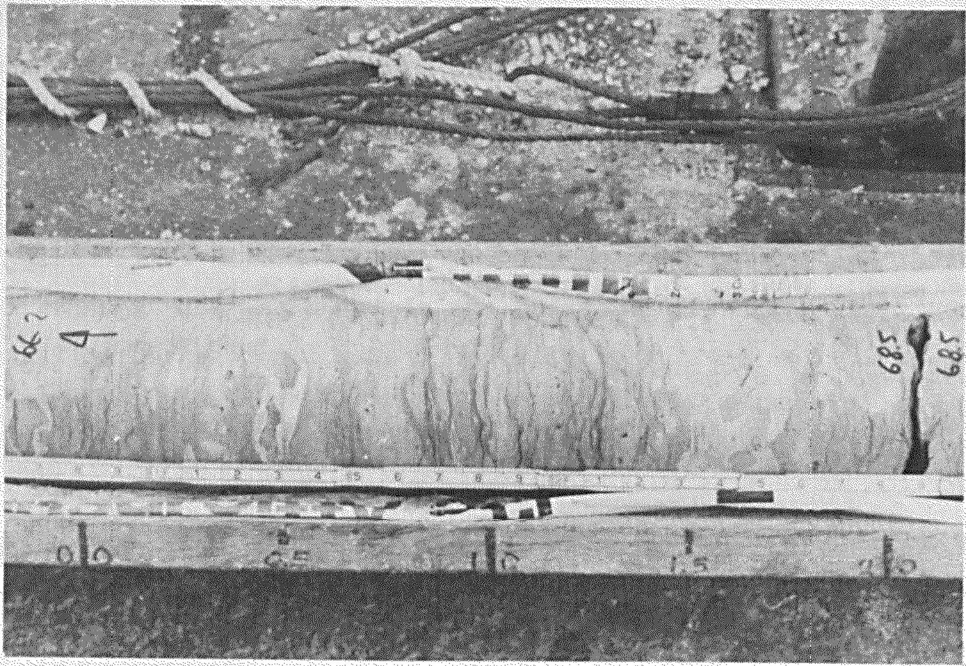
Geologic Cross Sections

25. A list of the abbreviations used on the geologic and structural cross sections is presented in Appendix A. Two 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 location of the sections is shown in Figure 1 as lines A'-A' and B'-B' with the A' line parallel, and the B' line perpendicular to the axis of the dam. Sections A'-A' and B'-B' are presented in Plates 5 and 6, respectively.

26. Section A'-A' shows the thickness of concrete and the depth of the bedrock recovered from each borehole. The concrete is considered to be in good condition having been adequately consolidated during placement. The top 4 ft of concrete in the tainter-gate piers section shows the results of frost action; however, compressive strengths within this zone are in excess of 3550 psi. Both tight and loose horizontal construction joints were encountered during drilling. Portions of borings BR WES 2- and



a. Sample of upper rock



b. Sample of lower rock showing numerous bedding plains

Figure 5. Typical bedrock samples

3-76 were drilled parallel to vertical pier construction joint; some portions were tight while others were loose. Both of these holes were drilled in tainter-gate piers. A tight contact between the concrete and the bedrock was observed in hole BR WES-1-76 (head-gate section) while in the other three holes the contact was loose.

27. About 45 ft of rock was recovered from the borings on section A'-A', all of which is dolomite. The rock is light gray with clay- and shale-filled BP as described under Bedrock Stratigraphy; the BP are as thick as 1/16 in. Individual BP will vary from 1/64 to 1/16 in. The blue-green clay BP occur every 0.1 to 0.2 ft, while deeper in the section the dark gray clay and shale-filled BP occur more frequently at about every 0.05 to 0.2 ft. Occasional clay-filled seams with apparent uniform thickness of 1/16 in. occurred in the upper dolomite containing the blue-green clay-filled BP.

28. The bedrock in sections A'-A' and B'-B' has been subdivided into three units as seen in Plates 5 and 6. The subdivisions were made based upon the color and composition of the filled BP. The units are continuous across the dam foundation and consist of dolomite with the blue-green clay BP, dolomite with blue-green shale BP, and dolomite with dark gray shale BP. The overburden abutting the tainter gates of the dam consists of about 9 ft of organic silt with large amounts of fibrous organic matter and about 6.5 ft of gravel-to-boulder size rock; these depths apply to the area just behind the sluice-gate pier adjacent to tainter-gate 1. The section of silt and gravel is presented in Plate 6.

Bedrock Structural Characteristics

29. The bedrock structural characteristics relevant to foundations are presented in Plates 7 and 8 and represent the same sections (A'-A' and B'-B') as described in paragraphs 25-28. These sections and the core logs from borings BR WES-1-76 through BR WES-6-76 were used for fracture evaluation.

30. The bedrock is considered competent and massive. No appreciable dip could be measured on the bedding surfaces, therefore bedding is assumed to be horizontal over the foundation. The bedding surfaces are irregular with a nominal 1-in. differential between the peaks and valleys both of which are generally rounded (Figure 5).

31. Reference 5 states that there is no evidence of major faulting in the Joliet formation in and around the dam site. No evidence of sheared or brecciated rock was detected in the core. The reference cites geological literature that indicates an intersecting vertical-joint pattern trending NE-SW and SE-NW. Five high-angle joints were detected in four drill holes with an average dip of 75 degrees (Plates 7 and 8). These joints are trending NE-SW and SE-NW. The joints occurring in the boreholes are too scattered to establish the existence of joint sets or set orientations.

32. Reference 5 uses the term stylolitic planes to describe about 95 percent of the bedding planes observed in core recovered from a number of borings. After a detailed petrographic examination it was decided to use the term "bedding planes with irregular surfaces" for nearly all the BP instead of stylolitic planes. Pettijohn's definition[7] of stylolitic seams was used as the criterion.

> A stylolitic seam is a surface of contact marked by interlocking or mutual interpretation of the two sides. The teeth-like projections of one side fit into sockets of like dimension on the other.
>
> The stylolitic surface may be one of minute and, in some cases, of microscopic irregularities, or it may be grossly uneven with a relief of several centimetres (up to 30 cm). The stylolitic projections commonly have a columnar aspect and may even be striated or grooved in a longitudinal direction perpendicular to the stylolitic surface as a whole.

33. A small number of macro- and microscopic stylolites were observed on the core drilled during this study. A moderate number of the filled irregular BP had pieces of fractured dolomite contained in

the filling material. This was evidence of a depositional features as opposed to a postdepositional feature (as stylolites are described in Pettijohn).[7]

34. The irregular clay-filled BP are not considered continuous over the foundation; they are very tight and the rock gives a solid ring when struck with a hammer. The 1/16-in. uniformly thick clay-filled BP observed in borings BR WES-1, 2, and 3 are not considered continuous over the foundation.

PART IV: SELECTION OF TEST SPECIMENS AND TEST PROCEDURES

Cores Received

35. Approximately 10 ft of concrete and 45 ft of rock were received from each of the four borings along the crest of the dam; about 14 ft of silt and about 30 ft of rock core were received from the upstream and downstream holes. Pertinent information concerning the core received at WES is presented in Table 2. Two shipments were received during the drilling operation. Eleven boxes of concrete, 58 boxes of rock, and 7 tubes of silt were received at WES. Upon receipt of the core at WES, the boxes were placed in a moist curing room until the selection of test specimens was completed.

Selection of Test Specimens

36. The petrographic examination indicated that the deepest concrete deterioration occurred at 4.3 ft. Therefore, concrete test specimens were selected from the first 4 ft of core. For comparison purposes, concrete from the mid and bottom portion of the dam (tainter-gate piers) was selected for testing. Characterization properties, effective (wet) unit weight (γ m), compressional wave velocity (V_p), compressive strength (UC), Young's modulus (E), and Poisson's ratio (ν) were determined or calculated. Only V_p's could be obtained from the concrete at the mid and bottom portions of the hole because the core contained a vertical construction joint.

37. Since the bedrock at Brandon Road Dam was considered competent, it was suggested that only the first 25 ft of bedrock be tested. It was anticipated that the proposed anchorages would be located no deeper than 25 ft. An attempt was made to select test specimens representative of the rock in close proximity to the concrete-rock contact. Where feasible, this was accomplished. The test assignment locations can be obtained from the tables of test results.

38. There were six types of specimens tested in direct shear: concrete on rock, intact, cross-bedded, precut, filled parting, and natural jointed. The majority of test specimens selected for direct shear was obtained within the first 5 ft below the concrete-rock contact. Representative specimens of the clay-filled BP were tested in direct shear and designated as filled partings. In selecting dolomite cores containing natural joints, it was observed that the surfaces were undulated to such an extent that the differential between high and low portions was too great in most cases for the direct shear apparatus. Consequently, only two specimens were found that could be tested.

39. Three UC specimens from each boring were selected to represent each 10 ft of core for the first 30 ft below the concrete-rock contact. Specimens containing the 1/16-in. uniformly thick clay-filled BP were tested in UC; a total of five specimens was obtained. All other test specimens contained a number of the paper-thin filled BP. Specimens for direct tensile strength (T_s) were selected near the 25 ft depth below the concrete-rock contact where the greatest tensile stress would be imparted to the bedrock by the anchors. Three of the silt samples were selected for S triaxial testing and two for R triaxial testing.

Test Procedures

40. The characterization property tests were conducted in accordance with the appropriate test methods as tabulated below.

Property	Test Method
Effective Unit Weight (As Received), γ_m	RTM 109*
Dry Unit Weight, γ_d	RTM 109
Water Content, w	RTM 106
Compressional Wave Velocity, V_p	RTM 110 (ASTM D 2845)[8]
Compressive Strength, UC	RTM 111 (ASTM D 2938)
Direct Tensile Strength, T_s	RTM 112 (ASTM D 2936)

* Proposed Rock Test Method, Corps of Engineers, in review prior to publication.

41. The engineering design tests were conducted in accordance with the appropriate test methods as presented below.

Property	Test Method
Elastic Moduli	RTM 201 (ASTM D 3148)
Triaxial Strength	RTM 202 (ASTM D 2664)
Multistage Triaxial Strength	RTM 204
Direct Shear Strength	RTM 203
R and S Triaxial Strength	EM 1110-2-1906[9]

42. For the compression and triaxial compression test, 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 were employed in the axial (vertical) and horizontal (circumferential) 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.

43. The concrete-on-rock specimens were fabricated using a general mass concrete mixture having an approximate compressive strength of 2000 psi at 28 days age. 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.-diam molds. Rock surfaces onto which the concrete was cast were gently undulating. Rock cores used for these tests were taken from within 6 in. of the dam concrete-rock contact.

Petrographic Examination

44. The concrete cores were examined for signs of deterioration and general physical condition. The near-surface pieces were sawed in half parallel to the long axis of the core to allow better examination of the effects of frost damage and other deleterious reaction. An immersion mount of apparent gel from the core was examined and gel was identified.

45. The rock portion of each boring was examined more carefully to supplement the information obtained during the field logging. A comparative visual examination was made of the rock at various depths within each boring and between the borings.

46. Pieces of core representing the different rock types from each boring were examined with a stereomicroscope. Some of the pieces were broken, their surfaces etched with dilute hydrochloric acid and then reexamined.

47. A sample of each rock type was ground to pass a 45-μm sieve and then examined by X-ray diffraction. Samples of clay partings were selectively removed and ground in distilled water. The resulting paste was put on a glass slide, air dried, and then examined by X-ray diffraction. The X-ray examinations were made with an X-ray diffractometer using nickel-filtered copper radiation.

48. The photographs taken in the field have been assembled in a loose-leaf notebook. One set is on file at CDO and one set at WES.

PART V: TEST RESULTS AND ANALYSIS

Pressure Transducer Measurements

49. The results of the pressure transducer tests are tabulated below. The drill-hole numbers are not consecutive, but rather in sequence as the holes were drilled across the dam (Figure 1).

Drill Hole No.	Gage Depth Below US Head ft	Tailwater Depth ft	Tailwater ft	Pressure Measured psi	Drill Hole Water Head ft	Uplift Pressure psi
BR WES-1-76	41.20	33.60	7.60	4.73	10.92	1.43
BR WES-4-76	51.80	31.00	20.80	8.37	19.32	0.64
BR WES-2-76	52.90	33.80	19.10	16.04	37.00	7.76*
BR WES-3-76	52.50	31.00	21.50	10.40	24.01	1.09*

* Note: Hole drilled on vertical construction joint, assumed open to upstream head.

Date measured: 1-76, 11-3-76, 2-76, 11-9-76; 3 and 4 76, 11-16-76.

50. If the drainage system under the dam is effective, the uplift at the toe of the dam will generally be tailwater pressure. "The uplift pressure at any point under the structure will be tailwater pressure plus the pressure measured as an ordinate from tailwater to the hydraulic gradient between upper and lower pool."[10] The head in borings BR WES-1 and 4-76 indicates that slight uplift pressure exists above that expected from tailwater. This additional pressure is assumed to be caused by the hydraulic gradient between upper and lower pool. The measurement of the gradient ~~was beyond the scope of this study.~~ The head in borings BR WES-2 and 3-76 is meaningless in terms of uplift pressure because these borings are open to the upstream head through an open vertical construction joint. It was noted during the drilling operation that drill water was lost in boring BR WES-3-76 at a depth of about 40 ft. The loss probably occurred through the construction joint. As mentioned earlier, the uplift pressure measurements were conducted as a matter of interest; however, they indicate improper drainage at specific locations under the dam.

Discontinuities from Core Logs and Televiewer

51. Borehole televiewer surveys were run in borings BR WES-1, 2, 3, 4, and 6. The televiewer logs provide records of the five borings surveyed at Brandon Road and are presented as Figures 6 and 7; these figures are drawn with the same format as the geologic cross sections described earlier. Each log strip is divided vertically into eight segments, each representing a 45-deg segment of the compass. North is at the left margin and again at the right margin. South is represented by the center line, east the midpoint left of the center line, and west the midpoint right of the center line. Features of the borehole wall appear on the film in black and white tones in much the same shape and dimensions in which they occur on the bore wall. Size and shape of openings and width, strike, and dip of planar discontinuities such as bedding planes, fractures, and joints can be measured on the televiewer log.

52. Prior to analysis of the televiewer logs, core logs and core photos were consulted with the result that very little geologic structure was indicated in the subsurface to the depths explored in this investigation. The geologic structure considered important for purposes of stability analysis includes joints or bedding planes dipping greater than 10 degrees. Only 15 fractures in rock, most of which were short discontinuous fractures, were described in the core logs of the five borings. Of those 15, only 4 were sufficiently continuous and distinct in the core to be considered joints. These four were also discernible on the televiewer logs and are tabulated below.

Drill Hole No.	El, ft	Strike and Dip
BR WES-1-76	491.0 to 488.8	N 50° E, 75° NW
BR WES-3-76	476.1 to 473.3	N 45° W, Vert.
BR WES-3-76	458.9 to 457.4	N 45° W, Vert.
BR WES-6-76	489.6 to 486.6	N 90° E, 78° S

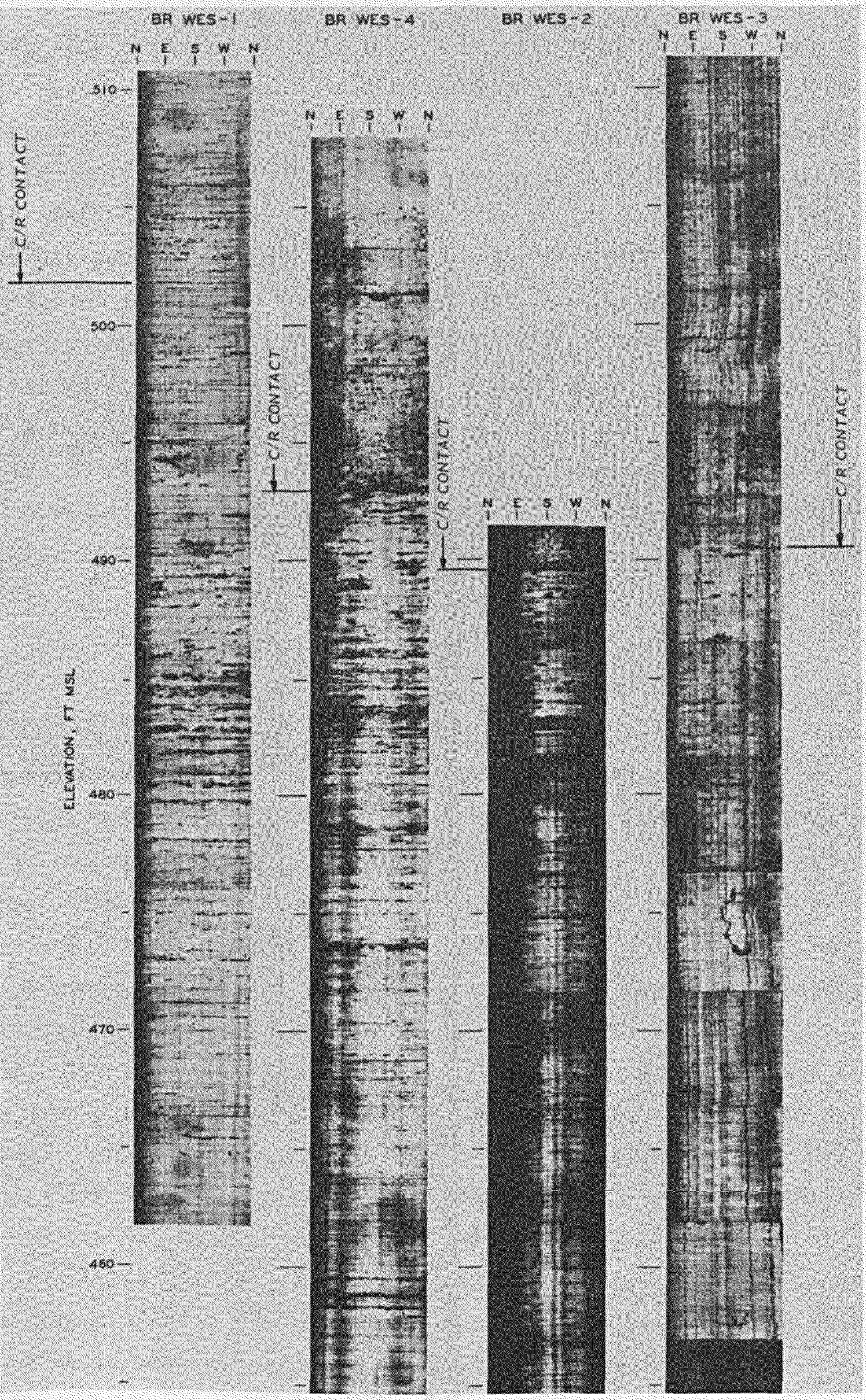


Figure 6. Televiewer logs, section A'-A'

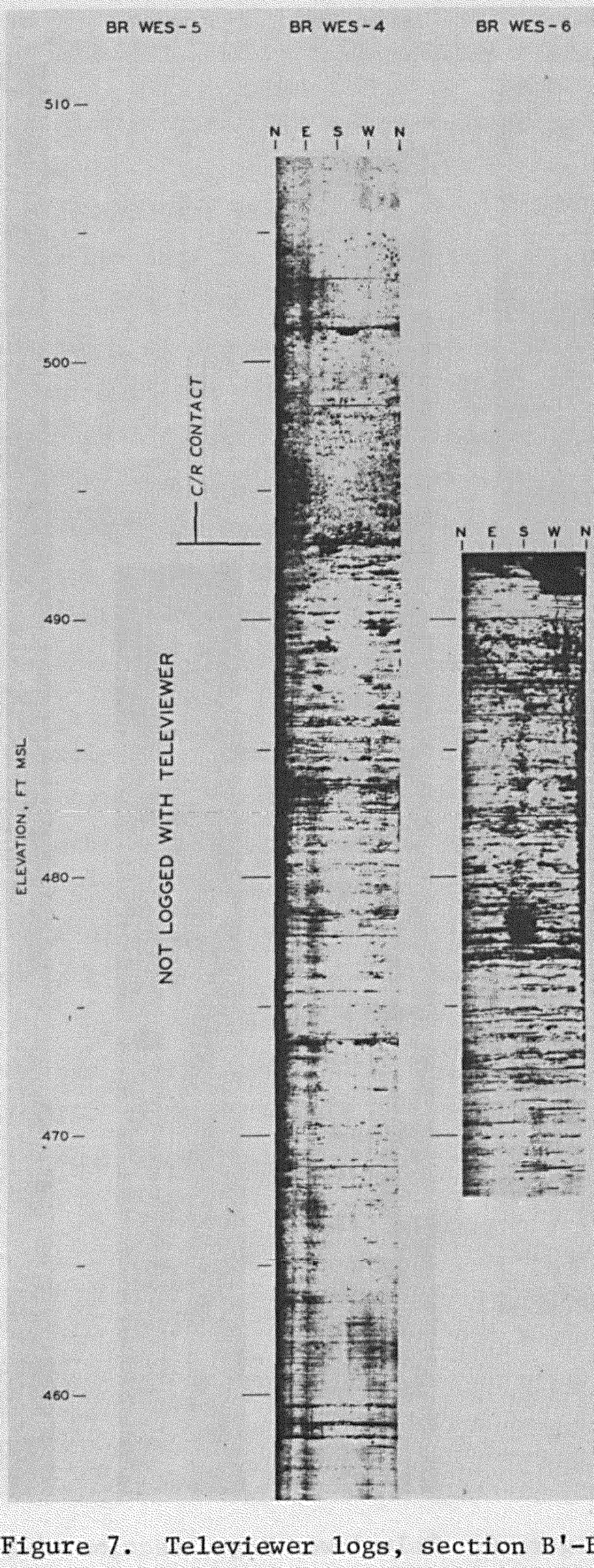


Figure 7. Televiewer logs, section B'-B'

53. The majority of the other features visible on the televiewer logs are probably horizontal bedding-related structures such as vugs, primarily solution cavities, zones of soft material such as clay and shale that washed out during drilling, and light and dark banding by clay and shale interbeds. The fractures occurring in the boreholes were too scattered to establish the existence of joint sets or set orientations. Deere's rock quality designation[11] (RQD) classifies rock as excellent if the RQD is between 90 and 100 percent. On this basis, the rock at Brandon Road would be rated excellent because all breaks in the core were classified as mechanical.

54. In three of the four borings across the dam, the concrete-bedrock contact was loose. The headgate boring shows a very tight contact. This fact should be seriously considered when making the stability analysis.

Petrographic Examination

55. The concrete was generally in good condition. Frost damage was generally confined to the first 4 ft of concrete measured from the surface. Frost action has caused parallel to subparallel cracking through the paste and aggregates.

56. Some alkali-silica gel reaction product was observed in the concrete. The occurrence of this reaction product was not believed to be extensive enough to affect the concrete. The description of the cores are given in the geologic cross section A'-A' (Plate 5).

57. The foundation material was essentially made up of one rock type with only a variation in color and some texture differences within each hole. Very little variation between the holes was noted. The near-surface rock contained soft blue-green clay. The clay was easily dug out from the BP and the BP could be separated with little or no effort. The extent of this feature and the color differentiation are noted in geologic cross sections A'-A' and B'-B' (Plates 5 and 6). The remaining rock contained shale with occasional clay-filled partings.

58. The rock was essentially a light gray to dark gray dolomite with some quartz. The color change was caused by shale impurities but in all of the cases examined, the shale was not present in large enough quantities to be detected by X-ray diffraction of a composite of the rock. The bluish-green clay and shale causing different colors in the rock consisted of clay-mica and kaolinite.

59. There were only a few structural features and they are shown in structural cross sections on Plates 5 and 6. The most notable feature was the high-angle fracture in holes BR WES-1, 3, 5 and 6-76 at elevations 490 ft, 475 ft, 488 ft, and 487 ft, respectively.

60. The rock was similar in all of the holes and should have the same general physical properties. The area near the rock surface containing the clay seams and the fractured areas should give different physical test results.

Characterization Properties

Concrete

61. The results of the characterization property tests of the bedrock are presented in Table 3. The average value, the range (difference between highest and lowest values), and the number of tests for the concrete are tabluated below.

	γm lb/ft^3	γd lb/ft^3	w %	V_p fps	UC psi	T_s psi
Concrete, <4 ft depth						
Average	147.9	135.8	9.0	13,891	4320	455
Range	4.3	4.3	1.5	2,665	1520	--
No. of tests	3	3	3	3	3	1
Concrete, >4 ft depth						
Average	151.3	141.8	6.7	15,682	5760	542
Range	6.2	8.5	1.2	3,491	3020	105
No. of tests	5	5	5	10	5	2

62. Most of the characterization properties of the top 4 ft of concrete are considerably different from the properties of the concrete below. There is a 3 and 4 percent difference in unit weights for the γ_m and γ_d, respectively, with the deeper concrete having the greater unit weight. The w for the deteriorated concrete is 26 percent higher than the w for the deeper concrete. The V_p for the deteriorated concrete is 11 percent lower than the V_p for the deeper concrete. The UC for the top 4 ft of concrete is 25 percent lower than the UC for the deeper concrete, while the tensile strength is 16 percent lower. The marked difference between these two zones of concrete is due to damage incurred by the top 4 ft of concrete as it underwent freezing and thawing.

63. Selected physical properties are located by elevation on cross section sheets having the same format as the geologic cross section described earlier (Plates 9 and 10). The plots give a visual comparison of property data for the length of concrete and bedrock core drilled and tested. In general the γ_m, V_p, and UC of the concrete increases with depth. The greatest concrete strength and V_p are 8140 psi and 17,895 fps, respectively, which represent concrete about one-third of the way down the sluice-gate pier.

64. Except for the top 4 ft of concrete in the sluice- and tainter-gate piers, the concrete is considered sound and structurally adequate for its intended purpose. An analysis of the top 4 ft of concrete will be given with the results of the petrographic examination.

Bedrock

65. The results of the characterization property tests of the bedrock are presented in Table 4. The average value, the range, and the number of tests are tabulated below. The UC and E for the dolomite containing the uniform 1/16-in. thick clay seam are quite different from the UC and E for dolomite without the seam. The dolomite is tabulated in two groups to emphasize this fact.

	γ_m lb/ft^3	γ_d lb/ft^3	w %	V_p fps	UC psi	T_s psi
Dolomite with 1/16-in.-thick Cl seams						
Average	164.9	159.9	3.3	17,691	7,430	--
Range	9.8	15.3	4.8	3,336	4,620	--
No. of tests	5	5	5	5	5	--
Dolomite without seam						
Average	164.3	159.0	3.3	17,295	10,560	123
Range	11.7	17.4	4.1	5,181	7,460	110
No. of tests	19	18	18	16	13	3

66. The unit weights of the dolomite are consistent and reasonable for a sound dense dolomite. The average γ_m is 164.4 lb/ft^3 which compares quite well with the 166 lb/ft^3 reported in Reference 5. The range in γ_m is about 13 lb/ft^3 which is not considered great in view of the variations in number of filled BP between test specimens. The w for the dolomite varied considerably. The average is 3.3 percent with a range of 4.8 percent. Again, the difference in the number of filled BP likely caused the wide range in w.

67. The V_p ranged from a low of 14.7 x 10^3 ft/sec to a high of 19.9 x 10^3 ft/sec and, in general, are in good agreement with the velocity data reported earlier.[5] The previously reported V_p data ranged between 14.3 and 20.0 x 10^3 ft/sec. The data might be useful in correlations with in situ seismic velocities if such were available.

68. In the above summary of characterization properties the dolomite was divided into two units, dolomite with a 1/16-in. uniformly thick clay seam and dolomite without such a uniformly thick clay seam. All specimens tested in compression contained the paper-thin irregular BP, most of which had small amounts of clay or shale filling. The five test specimens with a 1/16-in. seam have an average strength of 7430 psi; 13 specimens without this seam have an average strength of 10,560 psi.

69. Because only five specimens were found to contain the uniform clay seam and these specimens represented a small portion of the bedrock, the suggestion was made that the strengths of the specimens with the seam be averaged with the other strengths. It would have been difficult to

include the 1/16-in. seam in the finite element analysis of the prestressed system design, i.e., in a grid of the foundation.

70. The average direct tensile strength of the dolomite located about 25 ft below the concrete-rock interface is 123 psi. The controlling factor in the tensile-strength tests was the filled-bedding planes. The approximate force of 3500 lb required to separate the BP of a 6-in.-diam rock core indicates the relatively good bond along the bedding planes. As indicated in Reference 5 the filler material is well cemented to the dolomite.

Engineering Design Properties

Modulus of elasticity and Poisson's ratio

71. Results of the modulus of elasticity and Poisson's ratio tests are presented in Table 3 for the concrete and in Table 5 for the bedrock. The values are also presented in Plates 9 and 10 at the emblematic location from which the test specimens were taken. The stress-strain relation recorded for the concrete and dolomite cores is presented in Plate 11 and Plates 12 through 29, respectively. The E was calculated as an incremental value between 0 and 1000 psi; in most cases the 0 - 1000 increment corresponded to the initial linear portion of the stress-strain curve for the concrete and dolomite.

72. The E and ν of the deteriorated concrete is quite different from similar values for the sound concrete. The average E is 2.39×10^6 psi for the deteriorated concrete compared with 4.86×10^6 psi for the sound concrete. The deteriorated concrete and the sound concrete have an average ν of 0.19 and 0.23, respectively. Both the high and low values of E and ν were suggested for use in the anchorage system design if the top 4 ft of concrete was not going to be removed prior to placing bearing plates at the top of the dam. If the deteriorated concrete is removed to a depth of 4 ft, then the higher values should be used.

73. The E and ν for the specimens containing the uniform 1/16-in. clay seam average 1.12×10^6 psi and 0.18, respectively. The specimens

without the clay seam have an average E that is 2.5 times greater than the specimens containing the seam. The range in E for the dolomite without the uniform seam is 2.50×10^6 to 3.33×10^6 psi with an average of 2.75×10^6 psi. The average ν for the dolomite without the uniform clay seam is 0.17. It is recommended that the difference in deformation between the bedrock containing the clay seam and the remaining bedrock be taken into account in the anchorage system design.

Bearing capacity

74. The core logs were examined for indications of conditions critical to bearing capacity; none were detected. The lowest compressive strengths obtained during this investigation for the concrete and dolomite are presented as bearing capacity values. The bearing capacity of the first 4.3 ft and lower concrete and of the dolomite is 310, 368, and 423 tons per sq ft (tsf), respectively. Bearing capacity of the foundation rock is not considered critical; failure within the bedrock would be controlled by sliding rather than by bearing.

Triaxial

75. The stress values obtained during the triaxial tests are presented in Table 6 for the dolomite. The stress values are plotted on Plate 30 using the p-q diagram.

76. There are two common ways to find values of the cohesion intercept (c) and the angle of shearing resistance (ϕ): (a) construct Mohr circles and draw the Mohr envelope; or (b) plot values of p and q, draw the K-line, and then compute c and ϕ. "The choice between these two methods is largely one of personal preference. However, when there are many tests in the series, it will usually be less confusing to plot the results on a p-q diagram, and, further, it will be easier to fit a line through a series of data points than to attempt to pass a line tangent to many circles."[12]

77. The equations to convert from the standard p-q parameters (a and α) to the conventional c and ϕ are as follows:

$$c = \frac{a}{\cos \phi} \tag{1}$$

$$\phi = \sin^{-1} \tan \alpha \qquad (2)$$

where

α = inclination angle with respect to the horizontal of the least square best fit line for the individual p-q test plots, and

a = the least square best fit line q intercept.

78. All three triaxial (TX) specimens had a single shear failure surface inclined at 60 degrees from the horizontal. The confining pressures were 100, 500, and 1000 psi, respectively. The strength results of these tests appear reasonable. The c is 1033 psi (74.4 tsf) and ϕ is 55.5 degrees. A similar value (56 degrees) was obtained on the dolomite taken from the proposed Starved Rock Duplicate Lock site.[13]

79. A multistage triaxial test was conducted using a piece of concrete and dolomite from hole BR WES-1-76. The results are given in Table 6. The test method used is quite similar to a standard triaxial test method (described in Reference 14).

80. The results of the multistage test can be used to calculate the shearing stress (τ) across an established surface for various values of the normal stress (σ_n). The preselected surface was 45 degrees from the horizontal. With this information, the coefficient of friction (ϕ_j) on the surface can then be determined. When the principal stresses are known and the τ and the σ_n on a surface at an angle θ with respect to the principal plane are required, the following equations can be used to calculate these stresses:

$$\sigma_n = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\theta \qquad (3)$$

$$\tau = \frac{\sigma_1 - \sigma_3}{2} \sin 2\theta \qquad (4)$$

The values of σ_n and τ are then plotted, and values of ϕ_j and c are determined.

81. Results of the multistage test are presented in Plate 31 where the stress circles for the seven loading stages are plotted. The τ and σ_n were plotted on the stress circles and connected to form the strength envelope for the sawed surface. The equation of the strength envelope is as follows:

$$\tau = 0.0 + 0.4900\ \sigma_n$$

The equation yields an angle of shearing resistance of 26.1 degrees and a zero cohesion. The coefficient of friction calculated from stresses on the concrete-rock sawed surface is nearly equal to the coefficient of friction obtained for the precut rock specimens tested in direct shear. These results will be discussed later under Sliding Friction. Due to the loose contact between the concrete and rock described earlier, the sliding friction value from the multistage triaxial test is recommended for computing structural stability for that portion directly under the dam.

Peak shear strength

82. The stress values for the direct-shear tests are presented in Table 7 and plotted on Plate 32. The design parameters suggested in Reference 5 are presented in Table 8 and the direct-shear envelope is plotted on Plate 32.

83. 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 the dolomite containing a concrete-rock interface, intact dolomites, and cross-bedded intact dolomites. Specimens containing filled partings were scheduled to be tested for sliding friction properties, however the specimens were inadvertently tested for peak strengths rather than for sliding friction. The precut and natural jointed specimens were tested for sliding friction. The question that needs answering is, "Is the ϕ obtained for the precut specimens and the multistage specimen a lower value than the ϕ that would have been obtained had the filled-parting specimens been tested for sliding friction?" The author believes so.

84. The average ϕ values presented in Reference 5 for horizontal shear along stylolitic bedding (sty; referred to in this report as filled partings) are 52 and 39 degrees, respectively, for peak and residual shear. These data show a 25 percent decrease in ϕ from peak to residual; the residual value is 15 degrees higher than the ϕ value for the precut specimens tested during this study. It is reasonable to expect that the ϕ for filled BP tested for sliding friction would be greater than the value for sawed rock-on-rock or sawed concrete-on-rock.

85. All specimens were tested in the single-plane shear device designated the MRD device.* The tests performed on intact concrete-on-rock, and cross-bedded specimens resulted in a moderate amount of scatter. The tests of the clay-filled partings and the precut specimens had a very small amount of scatter. All envelopes were calculated using a linear regression analysis.

86. A total of three concrete-on-rock tests were conducted along the interface of concrete and rock; all specimens contained the natural rock-bedding planes. The peak shear stresses obtained from the concrete-on-rock specimens were about 30 percent lower than the peak stress measured on the intact specimens. The c and ϕ for the concrete-on-rock is 8.04 tsf and 78 degrees, respectively. In contrast the c and ϕ for the intact specimens is 33.5 tsf and 77 degrees. The asperities as described in Reference 5, pages 38-39, contribute to the relatively high peak shear strength.

87. The cross-bedded specimens were tested at an angle of 45 degrees to bedding. The peak shear stresses approach those obtained on the intact specimens. The c and ϕ is 29 tsf and 68 degrees. These values from the cross-bedded shear tests were recommended for use in the stability analysis for that portion of the foundation rock at the toe of the dam.

88. The results of the peak strength tests on the specimens containing clay-filled parting are reasonable. The average c and ϕ is 1.22 tsf and 47 degrees. These results are reasonably close to the

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* MRD, Missouri River Division, CE. The MRD device is used for 6-in.-diam low-strength rock.

results reported in Reference 5 for horizontal shear along stylolitic bedding; the average ϕ = 52 degrees and c = 1.17 tsf.

Sliding friction

89. The sliding friction tests were of two kinds: shear of sawed rock surfaces and shear of naturally occurring joints. The respective test results are labeled and presented in Table 6 and Plate 32.

90. From Plate 32 and Table 6, it will be noted that there is a large difference in the angle of friction for the specimens sheared along a natural joint (average ϕ = 51 degrees) and those sheared along the precut shear plane (average ϕ = 24 degrees). The surface roughness, about 1/4 in. between peaks and valleys for the specimens tested, accounts in large part for the relatively high ϕ for the natural jointed rock.

R and S triaxial of overburden

91. The peak stress values obtained from the R and S triaxial tests are presented in Table 8. The stress circles, stress-strain plots, and characterization properties are presented in Plates 33 through 38 for the R tests and in Plates 39 through 42 for the S tests.

92. The saturated unit weight and the coefficient of earth pressure of the gravel was assumed from the literature. The samples from hole BR WES-5-76 consisted of rather large pieces of rock, therefore an accurate measure of the unit weight could not be made. The design parameters recommended for the silt and gravel adjacent to the tainter gates are tabulated below:

	Saturated Unit Weight lb/ft^3, γs	Shear Strength	Coefficient Earth Pressure, K_r
Silt	82.8	c = 0.0 ϕ = 37°	0.5
Gravel	140.0*		0.45*

* Commonly accepted design value.

CDO suggested that a K_r = 0.5 be used in the stability analysis instead of the 0.4 value that would be obtained using the ϕ = 37 degrees.

Therefore the $K_r = 0.5$ is recommended. The main reason for using 0.5 is that the K_r for the silt should reasonably be higher than a K_r value for gravel. The value selected for the gravel is a commonly accepted figure.

PART VI: SUMMARY OF CONCRETE AND FOUNDATION CONDITION AND RECOMMENDED STABILITY AND DESIGN VALUES

Concrete Condition

93. The one boring in the head-gate section did not show evidences of frost damage. Minor scabbing on the top surface of the head-gate section has occurred. Resurfacing of these areas is necessary to stop further deterioration.

94. The concrete within the first 4.3 ft of the tainter-gate piers, which has deteriorated due to frost action, should be removed before any anchorage system is started. Continual exposure to frost action will cause additional deterioration at an increasing rate if the concrete is left in place. With the small number of borings, four along 1521 ft of dam section, a statement concerning possible limits of concrete removal along the entire dam alignment cannot be made. It is suggested that a random sampling of the concrete along the crest of the tainter-gate overflow sections be made and the cores tested for extent of deterioration. The concrete in the top portion of the overflow sections is suspected of being damaged to some degree because of its exposure to the same weather conditions as those to which the concrete in the piers has been subjected.

95. The concrete below 4.3 ft is considered to be of good quality with compressive strengths in excess of 5000 psi and moduli of elasticity greater than 4.4×10^6 psi. The concrete appears to have been well consolidated at the time of placement as only a few areas show slight honeycombing. The contact of the concrete with the bedrock is assumed loose across the base of the sluice- and tainter-gate sections. This assumption is based on the fact that the contact was loose in the three borings in these two sections.

Foundation Condition

Bedrock stratigraphy

96. Only one stratum was encountered at the drilled site; the dolomite of the Brandon Bridge member of the Joliet Formation. The dolomite is a light-gray, fine-grained rock containing numerous paper-thin to 1/16-in.-thick clay- and shale-filled irregular bedding planes; these features are referred to as stylolitic planes in Reference 5. The rock becomes increasingly shaly with depth and the filled partings become less distinctive. The clay-filled bedding planes occur every 0.1 to 0.2 ft while the shale-filled bedding planes occur every 0.05 to 0.2 ft. Chert modules and bands occur throughout the depth of rock drilled (about elevation 445).

Geologic cross sections

97. The cross sections clearly indicate the extent of deteriorated concrete, the contact between concrete and bedrock, and the occurrence of the clay- and shale-filled bedding planes in the bedrock. The cross sections give an overview of the bedrock material and the correlation of the color change of the filler material within the irregular bedding planes. The bedrock was correlated across the foundation by the color of the bedding planes. This was the only feature that could be so correlated since all the bedrock consisted of light-gray dolomite.

Bedrock structural characterization

98. The bedrock is considered massive. Bedding is assumed to be horizontal over the foundation; bedding surfaces are generally smooth but not planar, rather irregular with a nominal 1-in. differential between the peaks and valleys. These features are referred to as stylolitic planes in Reference 5. Numerous partings which contain a thin film of clay or shale, occur along the horizontal bedding planes. They are the major structural features in the foundation. If a shear failure occurs within the foundation, it is likely that it will be along the filled partings. The filled partings are considered not to be continuous over the foundation.

99. The loose contact between the concrete and bedrock is considered the probable potential sliding plane beneath the dam.

100. The core logs and televiewer logs show no indications of large voids, weak zones, or fracture areas; however local fracture areas do exist. The high-angle joints do not occur regularly and no joint sets could be identified in the foundation. The dam is aligned at about N 70 E. One joint was measured to be striking N 50 E and dipping to the NW at 78 degrees. This joint could participate in the shear failure involving a rock wedge in front of the dam. However, with only one joint observed dipping to the NW, the probability that the joint would participate in the shear failure seems low.

101. There was no evidence of prior movement in the samples observed in the field and laboratory. Also, no evidence of sheared or brecciated rock was detected. These facts tend to support the statement made in Reference 5 that there is no evidence of major faulting at the Brandon Road Dam site.

Recommended Stability and Design Values

102. The interior concrete of the dam is of high quality and would not be overstressed using the proposed prestress loads. Maximum prestress load in the tainter-gate monoliths is 1555 kips.[1] Small localized honeycombing would not be affected by the prestressing. Any frost damaged concrete should be removed before setting the top bearing plates of any anchorage system through the tainter-gate piers. The additional drilling mentioned in paragraph 94 will indicate where concrete removal is necessary in the overflow sections of the tainter-gate dam.

103. The contact between the concrete and foundation rock is considered loose under the tainter-gate section of the dam. The prestressing should not affect the interface other than by tightening it. The foundation is massive and is considered quite suitable for setting grouted tendon anchors. It is recommended that the measured deformation obtained from test specimens representing portions of bedrock containing clay seams be considered in the design of an anchorage system.

104. Concrete, rock type, and the various structural features described herein should be considered when formulating the stability analysis

and design parameters of the anchorage system. Guidance is presented in the following tabulation for proper choice of design parameters.

	Concrete <4.3 ft Depth	Concrete >4.3 ft Depth	Dolomite	Silt	Gravel
Characterization Properties					
Effective (wet) unit weight, lb/ft^3	147.1	151.5	164.4*	82.8*	140.0
Dry unit weight, lb/ft^3	135.8	141.8	159.0*	--	--
Bearing capacity, tsf	310	368	432.0*	--	--
Tensile strength, tsf	32	39	8.8	--	--
Shear strength:					
Concrete-on-rock	--	--	c=8.0 tsf $\phi=78^o$	--	--
Intact	--	--	c=33.5 tsf $\phi=77^o$	c=0.0 $\phi=30^o$**	--
Cross bed, 45^o	--	--	c=29 tsf* $\phi=68^o$	--	--
Clay-filled parting	--	--	c=0.43 tsf* $\phi=45^o$	--	--
Precut, rock-on-rock	--	--	c=0.4 tsf $\phi=22^o$	--	--
Precut, concrete-on-rock	--	--	c=0.0* $\phi=26^o$	--	--
Natural joint	--	--	c=5.93 tsf $\phi=46^o$	--	--
Modulus of elasticity psi x 10^6	2.39	4.86	2.30*	--	--
Poisson's ratio	0.19	0.21	0.17*	--	--
Shear modulus, psi x 10^6	1.00	1.98	0.98	--	--
Coefficient earth	--	--	--	0.50	0.45

* See Table 7 for values previously reported by CDO.

** $\phi=30$ suggested by CDO.

REFERENCES

1. Pace, Carl E., "Stability Analysis, Brandon Road Dam, Illinois Waterway, Chicago District," US Army Engineer Waterways Experiment Station, Vicksburg, MS, in preparation.

2. US Army Engineer District, Chicago, "Stability Investigation, Existing Brandon Road Lock and Dam, Illinois Waterway," Feb 1973.

3. US Army Engineer District, Chicago, "Periodic Inspection, Report No. 2, Brandon Road Lock and Dam, Illinois Waterway," Jun 1973.

4. US Army Engineer District, Chicago, "Appendix E, Stability Analysis of Dam Masonry, Brandon Road Lock and Dam, Feb 1973.

5. US Army Engineer District, Chicago, "Appendix A, Soils and Geology for Structural Stability Analysis, Brandon Road Lock and Dam, Illinois Waterway," Sep 1972, Revised Jan 1973.

6. Zemanek, J., et al, "The Borehole Televiewer - A new Logging Concept for Fracture Location and Other Types of Borehole Inspection," Reprinted, <u>Journal of Petroleum Technology</u>, Jun 1969.

7. Pettijohn, F. J., "Sedimentary Rocks," Harper and Brothers, Pub., New York, 1949, pp 156-157.

8. American Society for Testing and Materials, <u>1977 Book of ASTM Standards</u>, Part 19, 1977, Philadelphia.

9. US Army, Office, Chief of Engineers, "Engineering and Design: Laboratory Soils Testing," EM 1110-2-1906, 30 Nov 1970, US Government Printing Office, Washington, DC.

10. US Army, Office, Chief of Engineers," Gravity Dam Design," EM 1110-2-2200, 25 Sep 1958.

11. Stagg, K. G. and Zenkiewicz, O. C, ed., "Rock Mechanics and Engineering Practice," Wiley, London, 1968.

12. Lambe, T. W., and Whitman, R. V., "Soil Mechanics," John Wiley & Sons, Inc., New York, 1969, pp 137-143.

13. Stowe, R. L., and Warriner, J. B., "Rock Core Tests, Proposed Duplicate Lock - Phase II, Starved Rock Lock and Dam, Illinois River, Illinois," MP C-75-9, US Army Engineer Waterways Experiment Station, CE, Vicksburg, MS, Jun 1975.

14. Pace, C. E., Stowe, R. L., Buck, A. D., "Engineering Condition Survey and Structural Investigation of Locks and Dam 3, Monongahela River," Final Report, MP C-76-9, Final Report, US Army Engineer Waterways Experiment Station, CE, Vicksburg, MS Aug 1976.

Table 1

Recommended In Situ and Laboratory Testing Program

Rehabilitation Work, Brandon Road

Illinois Waterway, Chicago District

Field

Test	Remarks
Downhole Pressure Transducer	Ascertain uplift pressure
Downhole Televiewer	Determine orientation of discontinuities

Laboratory

Test	Remarks
Index Properties	Wet and dry unit weights; moisture content; compressional wave velocity
Unconfined Compression	Stress-strain diagram, Young's modulus of elasticity, and Poisson's ratio
Standard & Multistage Triaxial	Undrained tests at 100, 500, and 1000 psi confining pressure, shear modulus and stress-strain diagrams, Young's modulus, and Poisson's ratio
Direct Tensile	Strength only
Direct Shear:	
Intact Rock	Peak strength
Intact Shales or Friable Rock	Peak strength
Filled-Partings	Sliding friction (three-stage multi-loading test)
Precut and Jointed Surfaces	Sliding friction
Concrete-Rock Interface	Peak strength
Detailed Petrographic Examination	Suspected clay specimens

Table 2

Core Received at WES From Brandon Road 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	Remarks
CHI-10 DC-1 (A)	BR-WES 1	Nov 76	6	1 of 14	0 - 2.1	543.0 - 540.9	543.0	Concrete
CHI-10 DC-1 (A)			"	1 of 14	4.0 - 6.1	539.0 - 536.9		
CHI-10 DC-1 (B)			"	2 "	21.6 - 24.3	521.4 - 518.7		
CHI-10 DC-1 (C)			"	3 "	33.0 - 35.0	510.0 - 508.0		
CHI-10 DC-1 (D)			"	4 "	40.8 - 45.5	502.2		Concrete & rock
CHI-10 DC-1 (E)			"	5 "	45.5 - 49.8			Rock
CHI-10 DC-1 (F)			"	6 "	49.8 - 54.4			
CHI-10 DC-1 (G)			"	7 "	54.4 - 59.2			
CHI-10 DC-1 (H)			"	8 "	59.2 - 63.1			
CHI-10 DC-1 (I)			"	9 "	63.1 - 67.2			
CHI-10 DC-1 (J)			"	10 "	67.2 - 71.8			
CHI-10 DC-1 (K)			"	11 "	71.8 - 75.1			
CHI-10 DC-1 (L)			"	12 "	75.1 - 79.4			
CHI-10 DC-1 (M)			"	13 "	79.4 - 83.0			
CHI-10 DC-1 (N)			"	14 of 14	83.0 - 86.8	456.2		
CHI-10 DC-2 (A)	BR-WES 2	Nov 76	"	1 of 14	2.6 - 6.0	539.3 - 535.9	541.9	Concrete
CHI-10 DC-2 (B)			"	2 "	40.8 - 45.2	501.1 - 496.7		Concrete
CHI-10 DC-2 (C)			"	3 "	51.5 - 55.5	490.4		Concrete & rock
CHI-10 DC-2 (D)			"	4 "	55.5 - 59.0			Rock
CHI-10 DC-2 (E)			"	5 "	59.0 - 63.4			
CHI-10 DC-2 (F)			"	6 "	63.4 - 66.2			
CHI-10 DC-2 (G)			"	7 "	66.2 - 71.1			
CHI-10 DC-2 (H)			"	8 "	71.7 - 75.8			
CHI-10 DC-2 (I)			"	9 "	75.8 - 79.1			
CHI-10 DC-2 (J)			"	10 "	79.1 - 83.0			
CHI-10 DC-2 (K)			"	11 "	83.0 - 86.9			
CHI-10 DC-2 (L)			"	12 "	86.9 - 91.4			
CHI-10 DC-2 (M)			"	13 "	91.4 - 94.3			
CHI-10 DC-2 (N)			"	14 of 14	94.3 - 98.3	443.6		

(Continued)

Table 2 (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	Remarks
CHI-10 DC-3 (A)	BR-WES 3	Nov 76	6	1 of 14	1.2 - 5.8	540.7 - 536.1	541.9	Concrete
CHI-10 DC-3 (B)			"	2 "	21.0 - 25.9	520.9 - 516.0		
CHI-10 DC-3 (C)			"	3 "	43.6 - 47.7	498.3 - 494.2		
CHI-10 DC-3 (D)			"	4 "	50.4 - 53.6	491.5		Concrete & rock
CHI-10 DC-3 (E)			"	5 "	53.6 - 58.2			Rock
CHI-10 DC-3 (F)			"	6 "	58.2 - 62.8			
CHI-10 DC-3 (G)			"	7 "	62.8 - 67.2			
CHI-10 DC-3 (H)			"	8 "	67.2 - 70.8			
CHI-10 DC-3 (I)			"	9 "	70.8 - 75.5			
CHI-10 DC-3 (J)			"	10 "	75.5 - 80.2			
CHI-10 DC-3 (K)			"	11 "	80.2 - 84.7			
CHI-10 DC-3 (L)			"	12 "	84.7 - 88.5			
CHI-10 DC-3 (M)			"	13 "	88.5 - 93.6			
CHI-10 DC-3 (N)			"	14 of 14	93.6 - 97.9	444.0		
CHI-10 DC-4 (A)	BR-WES 4	Dec 76	"	1 of 14	0 - 2.1	543.7 - 541.6	543.7	Concrete
CHI-10 DC-4 (A)			"	1 of 14	2.5 - 5.2	541.2 - 538.5		
CHI-10 DC-4 (B)			"	2 "	16.8 - 20.8	526.9 - 522.9		
CHI-10 DC-4 (C)			"	3 "	49.2 - 53.1	494.5		Concrete & rock
CHI-10 DC-4 (D)			"	4 "	53.1 - 57.9			Rock
CHI-10 DC-4 (E)			"	5 "	57.9 - 62.1			
CHI-10 DC-4 (F)			"	6 "	62.1 - 65.0			
CHI-10 DC-4 (G)			"	7 "	65.0 - 68.8			
CHI-10 DC-4 (H)			"	8 "	68.8 - 73.3			
CHI-10 DC-4 (I)			"	9 "	73.3 - 77.8			
CHI-10 DC-4 (J)			"	10 "	77.8 - 82.5			
CHI-10 DC-4 (K)			"	11 "	82.5 - 87.2			
CHI-10 DC-4 (L)			"	12 "	87.2 - 91.1			
CHI-10 DC-4 (M)			"	13 "	91.1 - 94.3			
CHI-10 DC-4 (N)			"	14 of 14	94.3 - 97.5	446.2		

(Continued)

Table 2 (Concluded)

WES Reference	Drill Hole No.	Date Received	Core Diam in.	Box No.	Depth, ft	Elevation: For Depth Intervals, ft	Elevation: Top of Hole, ft	Remarks
CHI-10 DC-5 (A)	BR-WES 5	Dec 76	Silt	--	21.9 - 24.4	519.0 - 516.5	540.9	Silt
CHI-10 DC-5 (B)			Silt	--	24.9 - 27.4	516.0 - 513.5		
CHI-10 DC-5 (C)			Silt	--	27.9 - 30.4	513.0 - 510.5		
CHI-10 DC-5 (D)			Silt	--	30.9 - 33.0	510.0 - 507.9		
CHI-10 DC-5 (E)			6	1 of 8	39.1 - 42.5	519.0		Rock
CHI-10 DC-5 (F)			"	2 "	42.5 - 46.7			
CHI-10 DC-5 (G)			"	3 "	46.7 - 51.3			
CHI-10 DC-5 (H)			"	4 "	51.3 - 55.8			
CHI-10 DC-5 (I)			"	5 "	55.8 - 60.2			
CHI-10 DC-5 (J)			"	6 "	60.2 - 64.3			
CHI-10 DC-5 (K)			"	7 "	64.3 - 69.8			
CHI-10 DC-5 (L)			"	8 of 8	69.8 - 71.2	469.7		
CHI-10 DC-6 (A)	BR-WES 6	Dec 76	"	1 of 7	15.2 - 17.6	492.5	507.7	
CHI-10 DC-6 (B)			"	2 "	17.6 - 22.2			
CHI-10 DC-6 (C)			"	3 "	22.2 - 26.7			
CHI-10 DC-6 (D)			"	4 "	26.7 - 31.5			
CHI-10 DC-6 (E)			"	5 "	31.5 - 35.5			
CHI-10 DC-6 (F)			"	6 "	35.5 - 39.8			
CHI-10 DC-6 (G)			"	7 of 7	39.8 - 45.0	462.7		
CHI-10 DC-7 (A)	BR-WES 7	Dec 76	Silt	--	10.0 - 12.5	530.9 - 528.4	540.9	Silt
CHI-10 DC-7 (B)			Silt	--	15.0 - 17.5	525.9 - 523.4		
CHI-10 DC-8 (A)	BR-WES 8	Dec 76	Silt	--	10.0 - 12.5	530.9 - 528.4	540.9	
CHI-10 DC-8 (B)			Silt	--	15.0 - 17.5	525.9 - 523.4		

Table 3

Concrete Test Results, Brandon Road Dam

	Characterization Tests							Engineering Design Tests	
Drill Hole No. BR WES- -76	Elevation ft	Effective Unit Wt γ_m, lb/ft^3	Dry Unit Wt γ_d, lb/ft^3	Water Content w, %	Comp. Wave Velocity V_p, ft/sec	Comp. Strength UC, psi	Tensile Splitting T_s, psi	Elastic Modulus X 10^6 psi	Poisson's Ratio
1	538.1	152.3	141.8	7.4	16,665	5120			
1	520.5	148.0	139.0	6.5	15,875	5900		4.49	0.21
1	519.9						595		
1	509.5	149.2	139.2	7.2	16,950	7010			
2	538.7	148.6	137.5	8.1	13,880	5070		2.67	0.17
2	500.2				14,404				
2	489.5				14,628				
3	539.8	145.5	133.2	9.2	12,565	3550		2.10	0.20
3	539.1						455		
3	520.1				15,384				
3	497.4				14,772				
3	490.9				15,625				
4	539.0	149.8	136.7	9.6	15,230	4340			
4	526.3	153.0	143.9	6.3	17,895	8140			
4	493.2	154.2	145.2	6.2	14,625	5630		5.23	0.24
4	492.4						490		
Average		150.1	139.6	7.6	15,269				
Range		8.7	12.0	3.0	4385				
No. of Tests		8	8	8	13				

Table 4

Characterization Test Results of Dolomite Cores, Brandon Road Dam

Drill Hole No. BR WES- -76	Elevation ft	Effective Unit Wt γ_m, lb/ft^3	Dry Unit Wt γ_d, lb/ft^3	Water Content w, %	Comp. Wave Velocity V_p, ft/sec	Comp. Strength UC, psi	Direct Tensile T_d, psi
1	495.8	169.3	166.5	1.7	18,578	10,500*	--
1	485.2	166.6	163.8	1.7	18,545	6720*	--
1	480.6	159.5	151.2	5.5	15,507	6080**	--
2	483.7	161.7	152.4	6.1	16,983	5880†	--
2	479.8	164.5	--	--	16,949	--	--
2	475.2	165.0	157.0	5.1	16,533	10,690	--
2	474.2	160.8	156.4	2.8	--	--	50
2	467.8	160.8	152.6	5.4	14,705	10,120	--
3	487.3	167.6	165.4	1.3	18,843	7970*	--
3	482.7	161.0	152.9	5.3	16,393	--	--
3	477.5	166.0	160.7	3.3	17,821	13,060	--
3	465.5	161.2	157.0	2.7	--	--	160
3	462.7	160.9	154.0	4.5	14,925	12,120	--
4	487.3	168.4	164.9	2.1	19,686	7520	--
4	482.2	161.7	155.3	4.1	15,873	--	--
4	479.3	162.9	156.8	3.9	16,800	8560	--
4	470.9	162.5	156.0	4.2	15,446	10,390	--
4	468.2	160.5	156.4	2.6	--	--	160
5	491.7	167.6	164.6	1.8	19,077	9310	--
5	486.8	172.2	169.1	1.8	19,647	9580	--
5	477.4	163.4	158.3	3.2	16,944	11,900	--
6	490.5	172.2	170.0	1.3	19,886	14,870	--
6	484.5	167.8	164.3	2.1	19,387	7410	--
6	468.8	162.6	156.3	4.0	16,666	11,690	--
	Average	164.4	159.2	3.3	17,390		
	Range	12.7	18.8	4.8	5181		
	No. of tests	24	23	23	21		

*Specimen contained 1-1/16" thick Cl seam.
**Specimen contained 1-1/16" thick Cl seam and Ch bands.
†Specimen contained 1-1/16" thick Cl seam and Ch nod.

Table 5

Engineering Design Test Results of Dolomite Cores, Brandon Road Dam

Drill Hole No. BR WES- -76	Elevation ft	Elastic Modulus x 10^6 psi	Poisson's Ratio	Triaxial: Minor Prin. Stress σ_3, psi	Triaxial: Major Prin. Stress σ_1, psi
1	495.8	1.06	0.15	--	--
1	485.2	1.69	0.30	--	--
1	480.6	0.91	0.10	--	--
2	483.7	1.23	0.15	--	--
2	479.8	--	--	500	14,960
2	475.2	2.86	0.14	--	--
2	474.2	--	--	--	--
2	467.8	2.44	0.11	--	--
3	487.3	0.7	--	--	--
3	482.7	--	--	100	7,020
3	477.5	2.94	0.16	--	--
3	465.5	--	--	--	--
3	462.7	2.50	0.10	--	--
4	487.3	2.86	0.42	--	--
4	482.2	--	--	1000	12,750
4	479.3	2.79	0.13	--	--
4	470.9	3.23	0.19	--	--
4	468.2	--	--	--	--
5	491.7	2.38	0.12	--	--
5	486.8	2.50	0.15	--	--
5	477.4	2.50	0.10	--	--
6	490.5	2.50	0.15	--	--
6	484.5	2.94	0.25	--	--
6	468.8	3.33	0.16	--	--

Multistage Triaxial (45 deg. Surface)

Drill Hole No. BR WES- -76	Elevation ft	Material	Minor Prin. Stress σ_3, psi	Major Prin. Stress σ_1, psi
1	508.5	Concrete	10	30
1	487.3	Dolomite	35	109
			65	196
			100	291
			150	441
			200	586
			250	736

Table 6

Laboratory Test Results - Brandon Road Dam

Single Plane Shear Tests & Envelopes

Lithology	Drill Hole No. BR WES- -76	Elevation ft	Test	Normal Stress tsf	Peak Shear Stress tsf	Cohesion tsf	Angle of Friction deg
Dolomite	1	501.7	Concrete to rock	2.0	4.29	8.04	78
	2	488.6		4.0	46.46		
	3	489.4		8.0	38.96		
Dolomite	2	476.5	Intact	2.0	33.05	33.49	77
	2	485.4		4.0	64.44		
	4	491.1		8.0	63.56		
Dolomite	5	490.4	Cross bed, 45°	2.0	34.6	29.0	68
	5	496.3		4.0	61.4		
	6	482.5		8.0	49.52		
Dolomite	2	498.2	Precut	2.0	2.87	1.97	24
				4.0	3.68		
				8.0	5.47		
Dolomite	2	487.0	Precut	2.0	3.48	3.11	22
				4.0	5.37		
				8.0	6.11		
Dolomite	4	489.7	Precut	2.0	1.40	0.4	26
				4.0	2.30		
				8.0	4.30		
Dolomite	1	499.4	Filled parting	2.0	2.50	0.43	45
	1	494.3		4.0	4.41		
	5	495.6		8.0	8.55		
Dolomite	1	500.0	Filled parting	2.0	4.77	2.01	49
	3	482.1		4.0	5.89		
	6	489.3		8.0	11.42		
Dolomite	2	487.6	Natural joint	2.0	8.78	5.8	56
				4.0	11.59		
				8.0	17.55		
Dolomite	3	486.6	Natural joint	2.0	6.45	5.93	46
				4.0	12.31		
				8.0	13.36		

Table 7

"Summary of Rock and Soil Design Parameters"

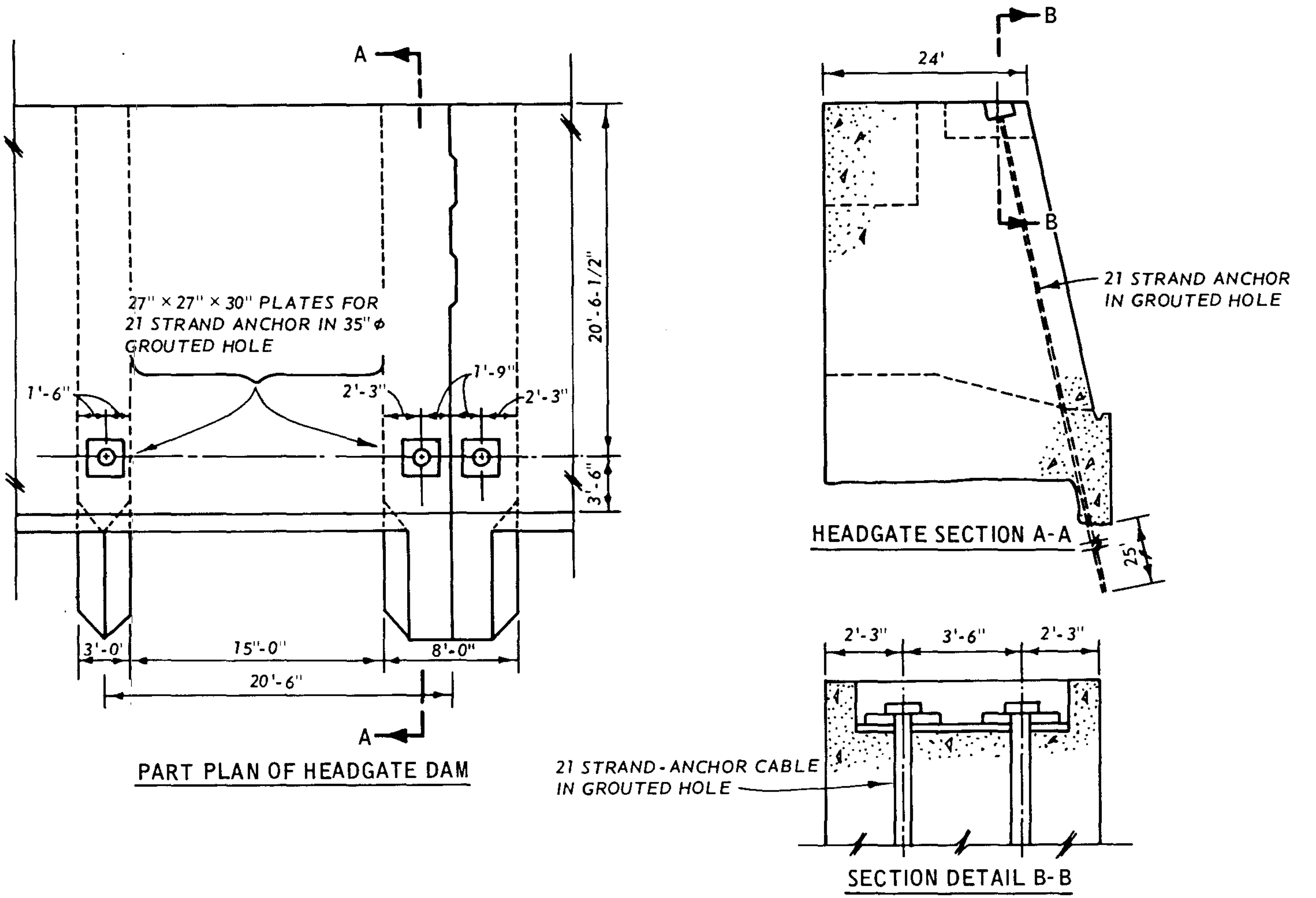
(From Reference 5)

Physical Properties	Dolomite
Compressive strength	10,000 psi
Bearin capacity (U1T.)	100 tsf
Shear strength (Along stylolitic bedding) (a) Peak (horizontal) (b) Residual (horizontal)	 $c=16.3$ psi $\phi=52^{o}$ $c=0$, $\phi=39^{o}$
Sliding Friction (coefficient) (interface rock-concrete)	 0.7
Shear, intact rock cross-bedded	 $c=1500$ psi $\phi=54^{o}$
Poisson's ratio	0.225
Modulus of Elasticity (E_i , 0-1000 psi)	 4.95×10^{6}
Density γ_d γ_s	 162 pcf 166 pcf

Table 8

Engineering Design Tests Results, Silt, Brandon Road Dam

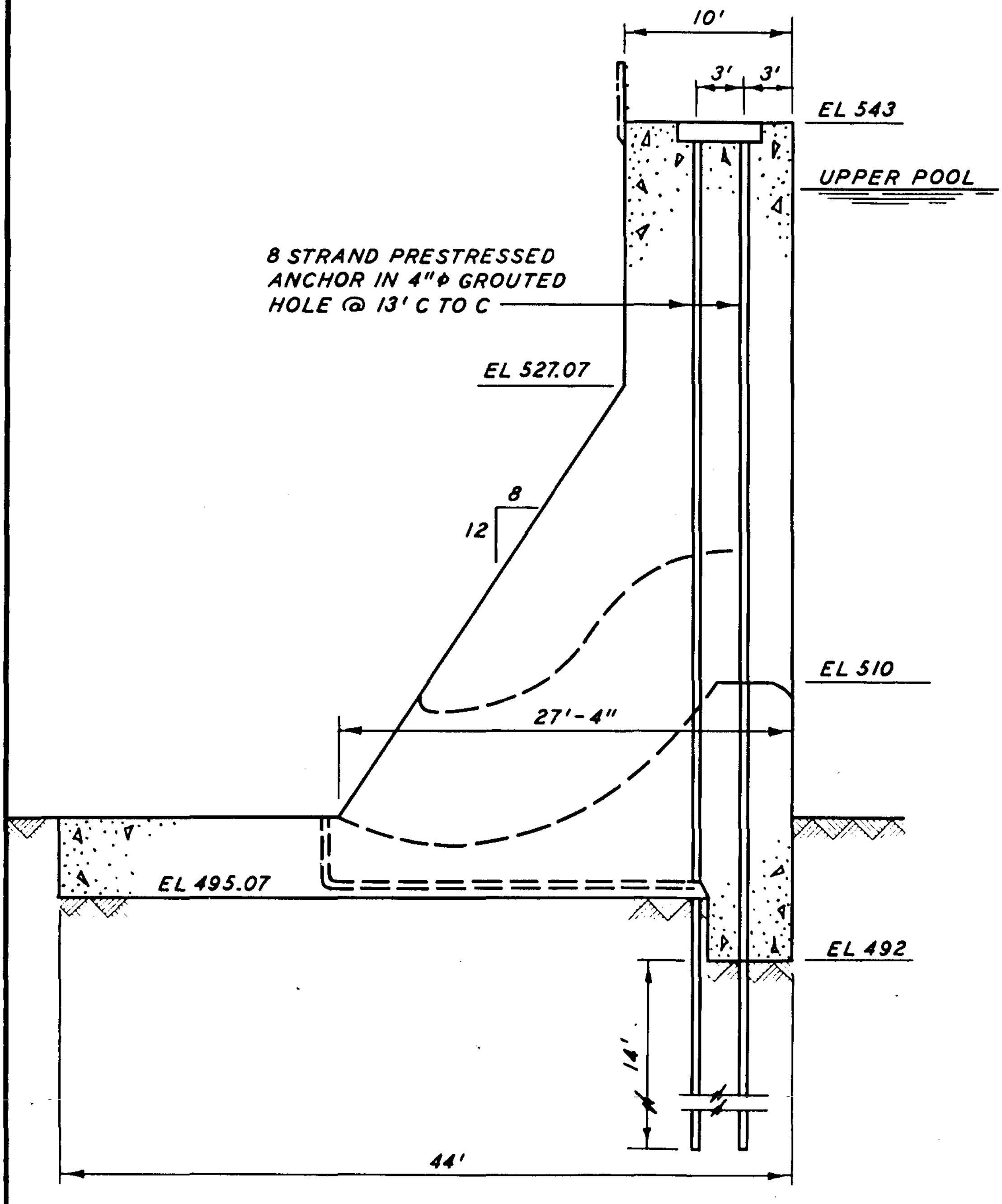
Drill Hole No. BR WES- -76	Elevation ft	Triaxial Total Stress ϕ	Triaxial Total Stress Cohesion tons/ft^2	Triaxial Effective Stress ϕ	Triaxial Effective Stress Cohesion tons/ft^2	Type Test
5	519.0-516.5	15^{o}	0.20	38^{o}	0.00	R
5	516.0-513.5	17^{o}	0.10	37^{o}	0.00	R
5	510.0-507.9	20^{o}	0.00	45^{o}	0.00	R
5	519.0-516.5			23^{o}	0.00	S
5	516.0-513.5			24^{o}	0.00	S



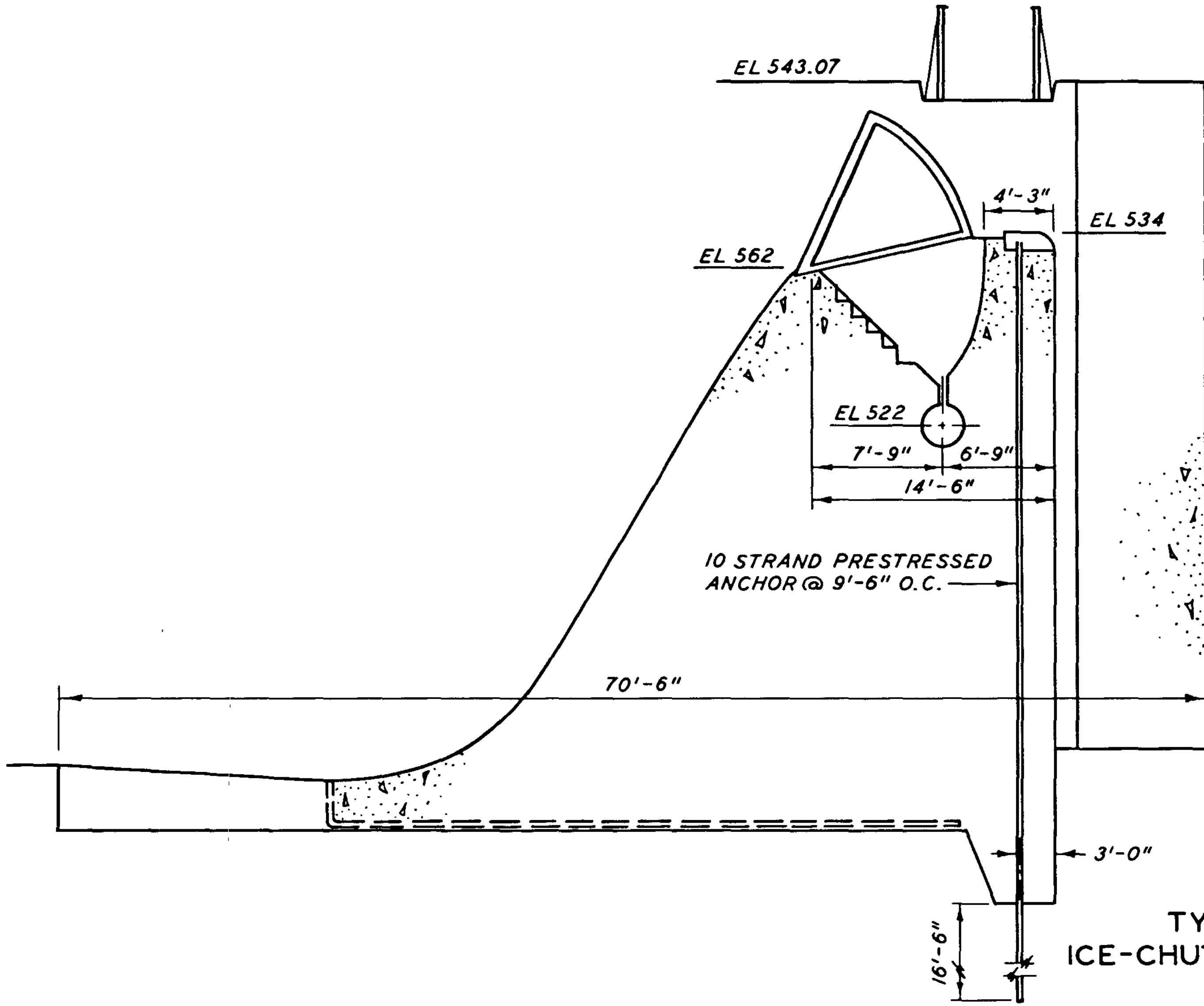
BRANDON ROAD DAM HEADGATES

PROPOSED STABILIZATION

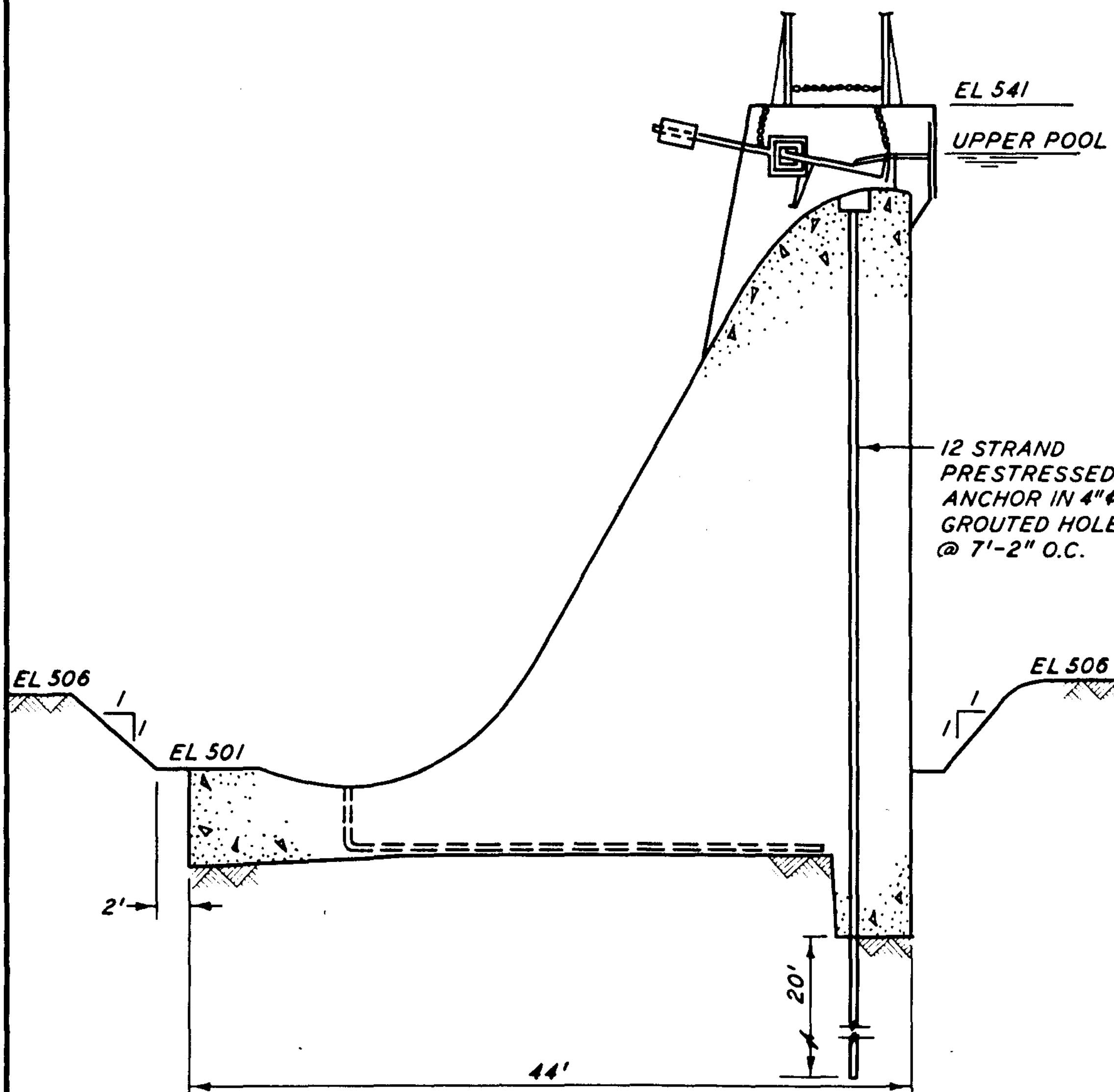
PLATE 1



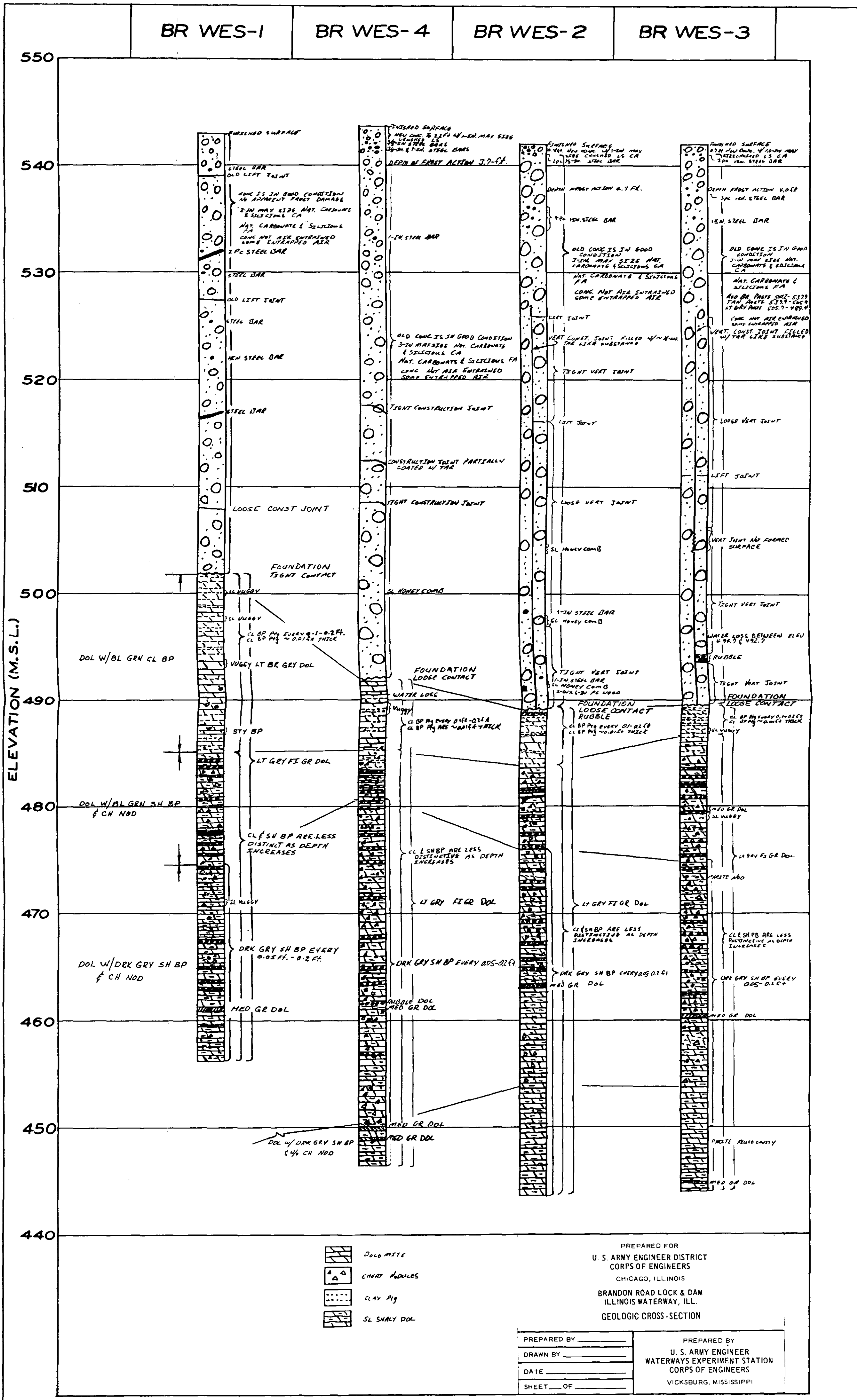
TYPICAL SLUICE GATE SECTION



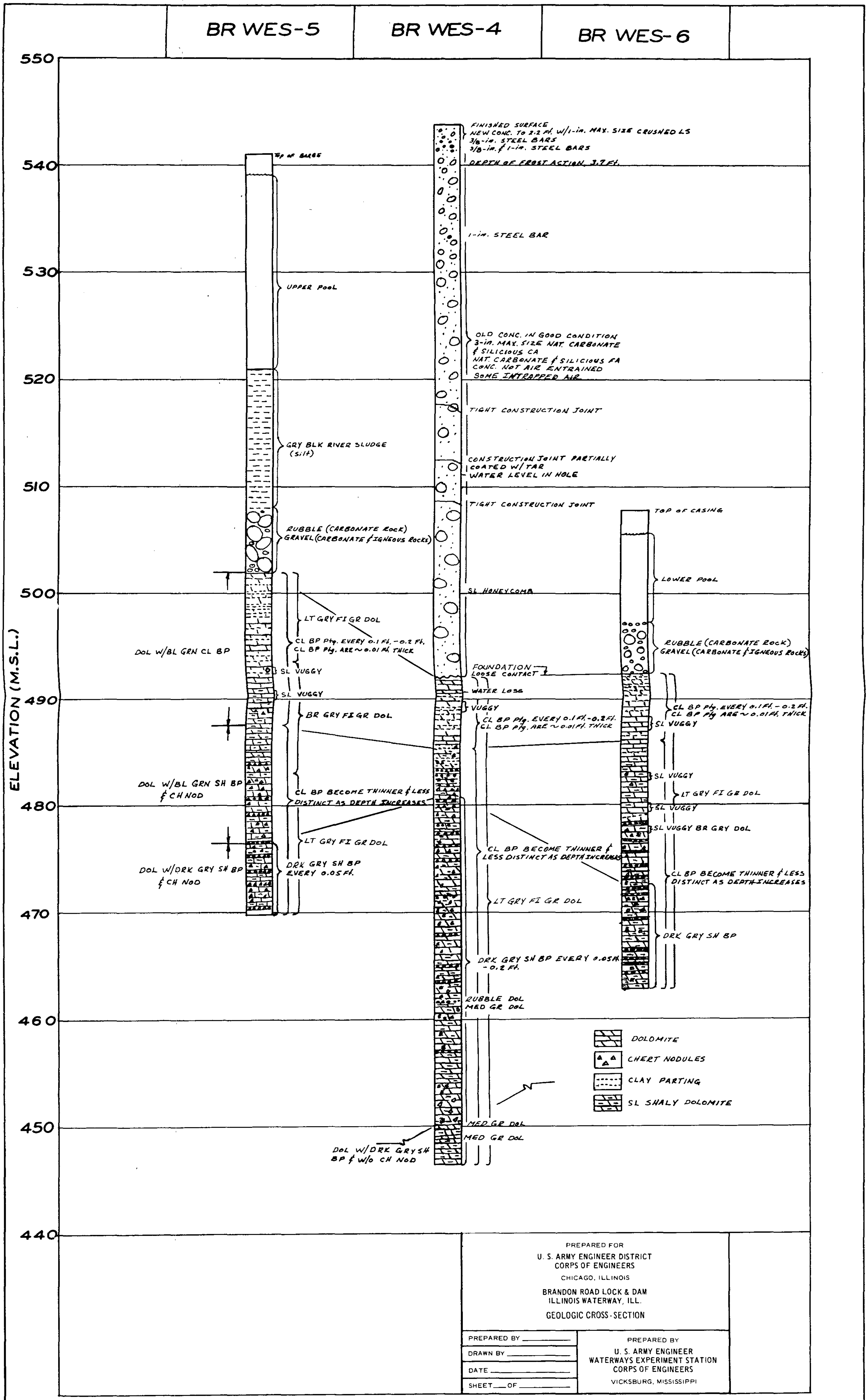
TYPICAL
ICE-CHUTE SECTION



TYPICAL TAINTOR GATE SECTION

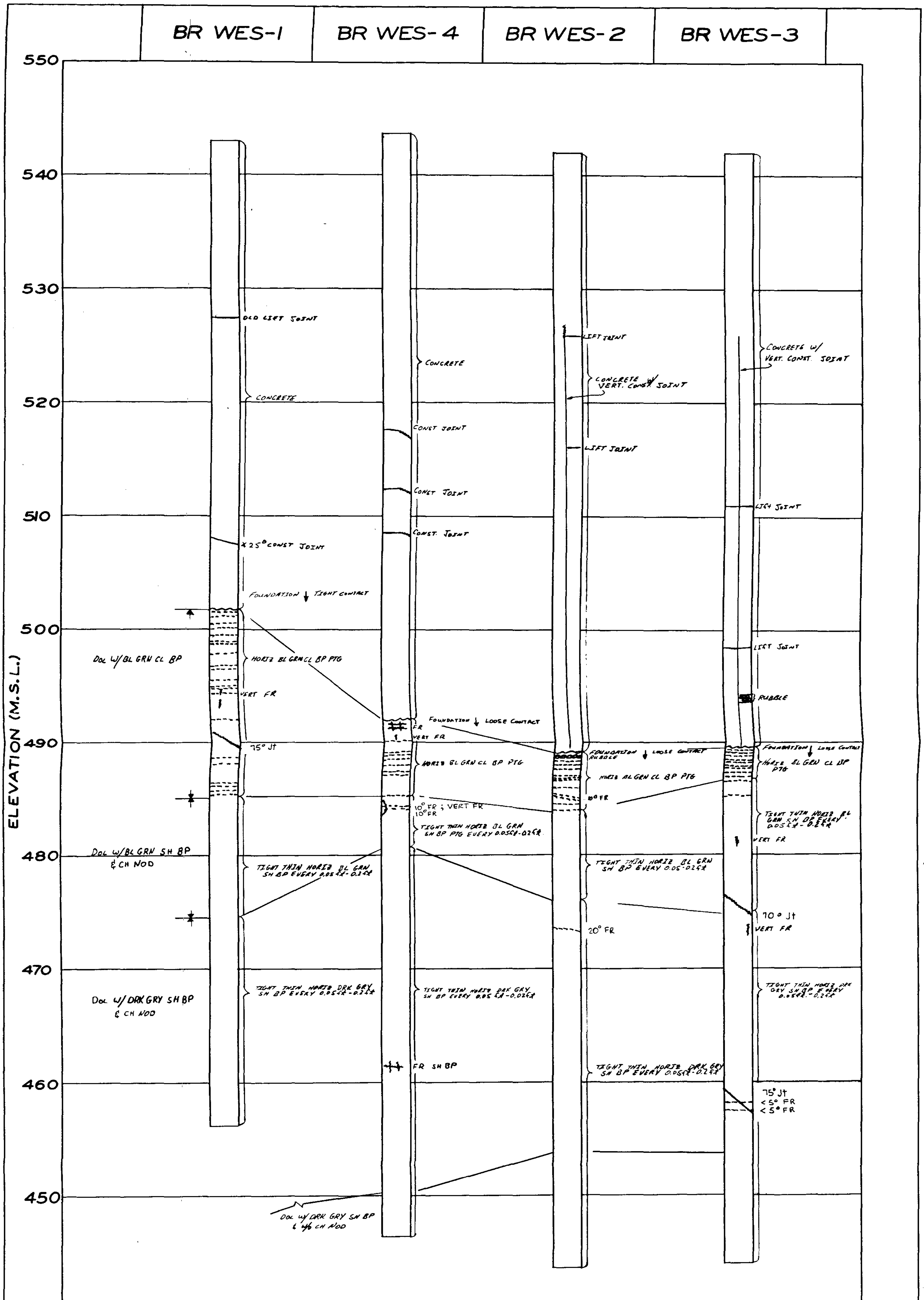


Geologic Cross-Section, Section A'-A'



Geologic Cross-Section, Section B'-B'

PLATE 6



PREPARED FOR
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
CHICAGO, ILLINOIS

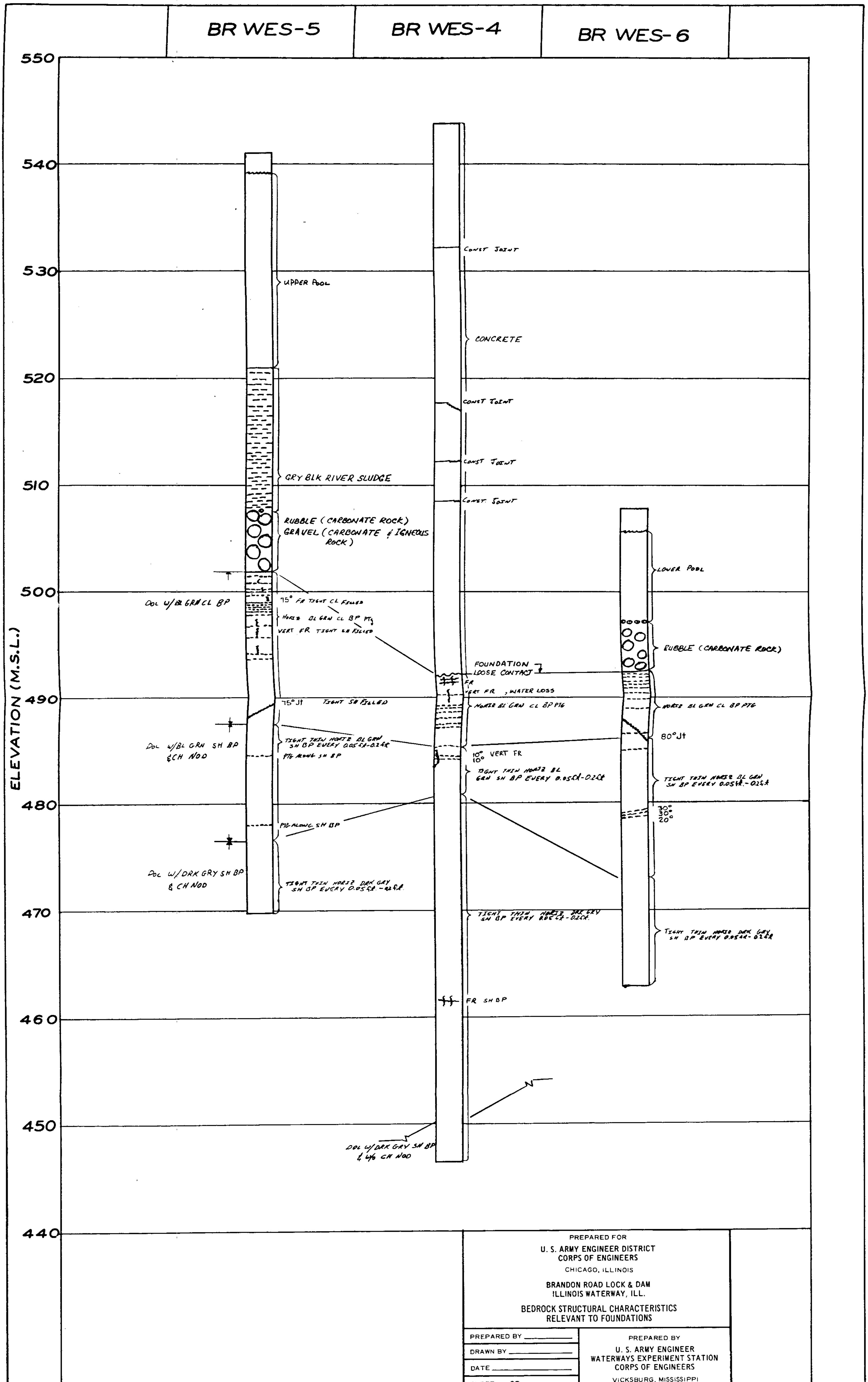
BRANDON ROAD LOCK & DAM
ILLINOIS WATERWAY, ILL.

BEDROCK STRUCTURAL CHARACTERISTICS
RELEVANT TO FOUNDATIONS

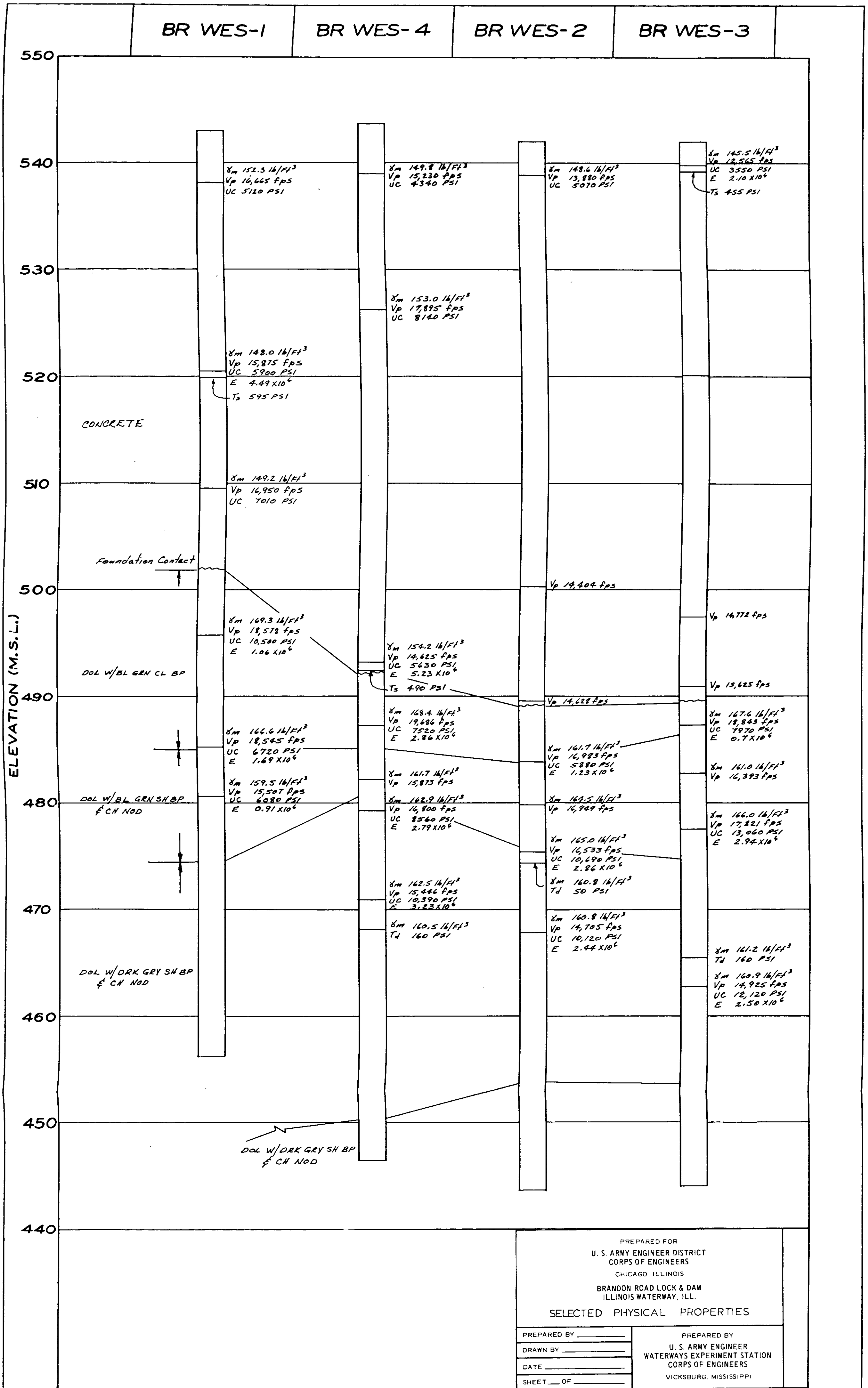
PREPARED BY ______
DRAWN BY ______
DATE ______
SHEET ___ OF ______

PREPARED BY
U. S. ARMY ENGINEER
WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
VICKSBURG, MISSISSIPPI

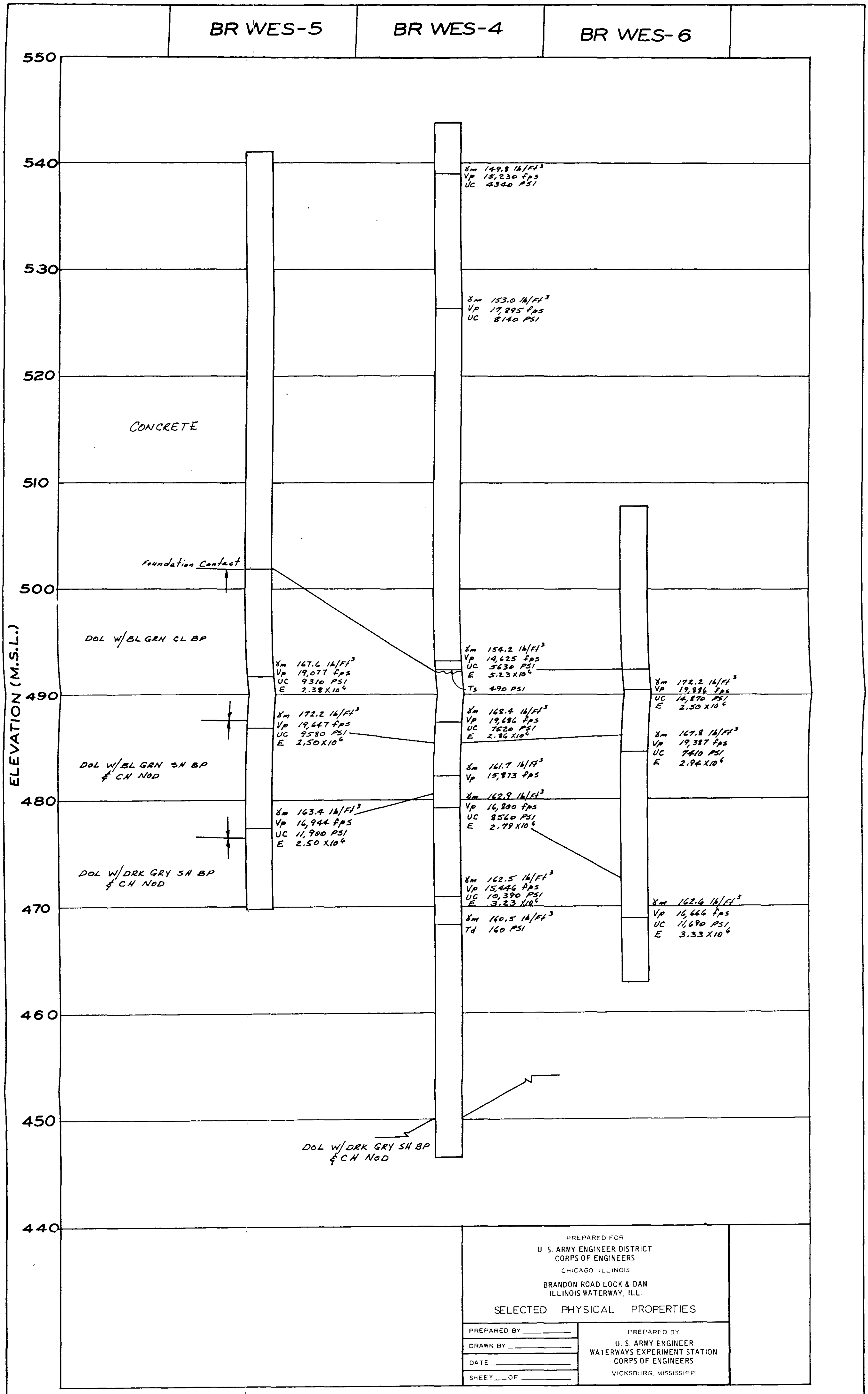
Bedrock Structural Characteristics, Section A'-A'



Bedrock Structural Characteristics, Section B'-B'



Selective Physical Properties



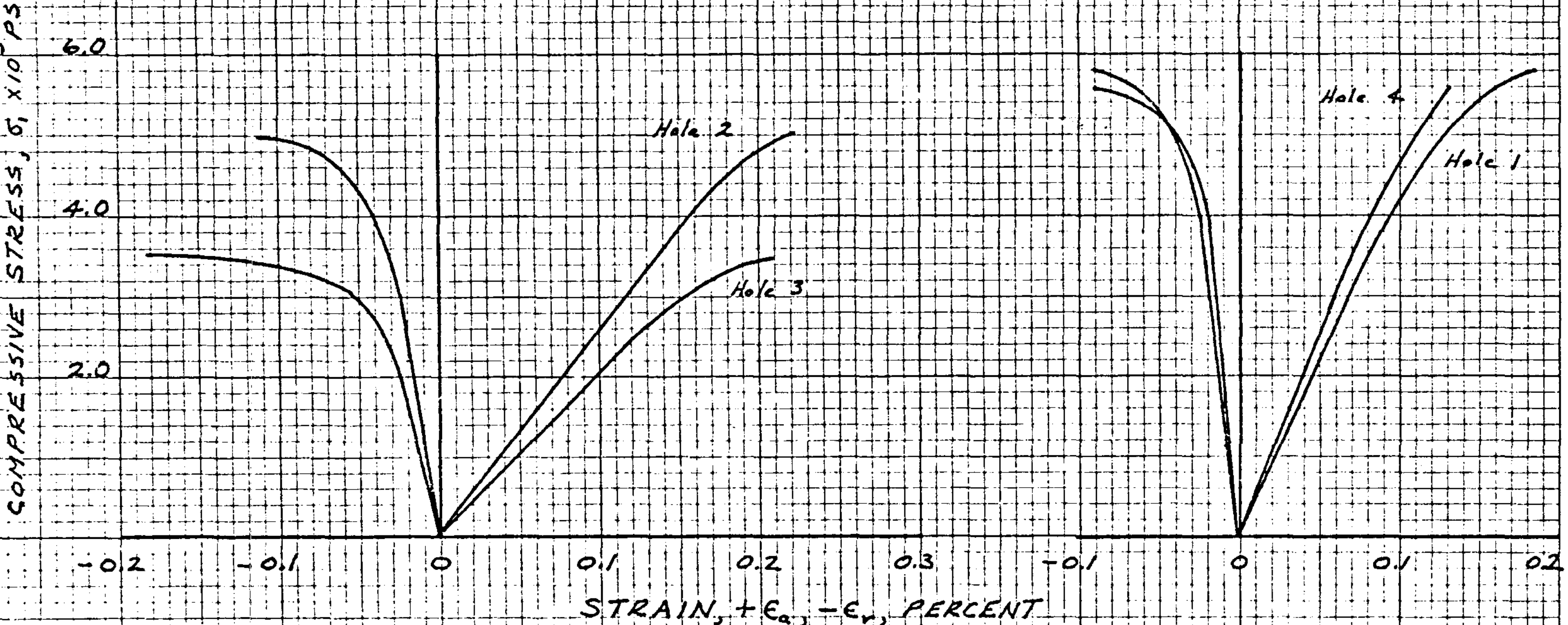
Selective Physical Properties

Concrete < 4.3'

Hole No.	Elev. Ft.	σ_1 psi	Modulus $E \times 10^6$	Poisson's Ratio	Water Content Percent
2	538.7	5070	2.67	0.17	8.1
3	539.8	3550	2.10	0.20	9.2

Concrete > 4.3'

Hole No.	Elev. Ft.	σ_1 psi	Modulus $E \times 10^6$	Poisson's Ratio	Water Content Percent
1	520.5	5900	4.49	0.21	6.5
4	493.2	5650	5.23	0.24	6.2



STRESS - STRAIN RELATION FOR CONCRETE, BRANDON ROAD DAM

PLATE 11

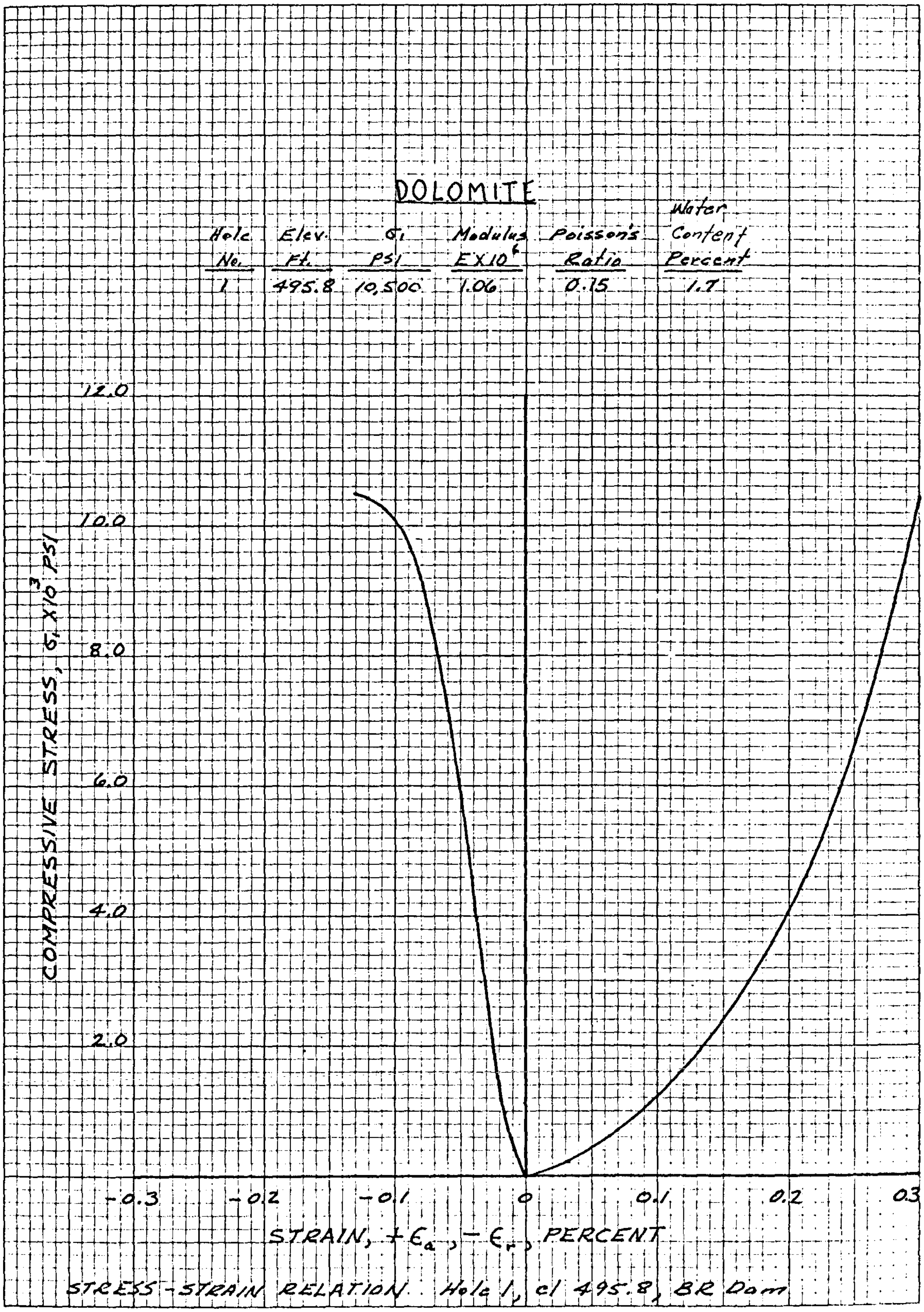

DOLOMITE
Hole No.
1
Elev. Ft.
495.8
σ1 PSI
10,500
Modulus E X 10^6
1.06
Poisson's Ratio
0.15
Water Content Percent
1.7
12.0
10.0
8.0
6.0
4.0
2.0
COMPRESSIVE STRESS, σ1 X10^3 PSI
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, +εa, -εr, PERCENT
STRESS-STRAIN RELATION Hole 1, el 495.8, BR Dam

PLATE 12

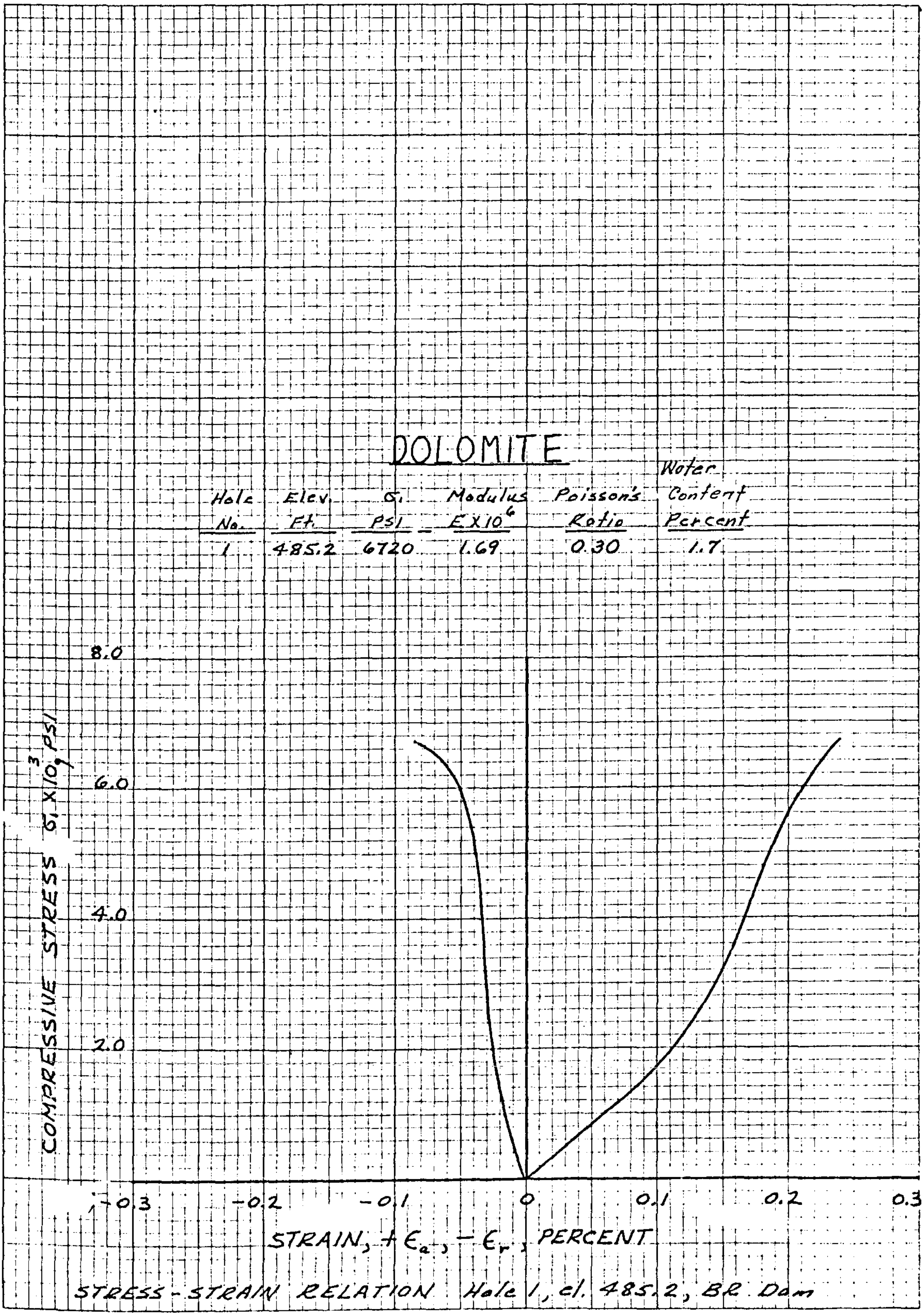

DOLOMITE
Hole No. | Elev. Ft. | σ_1 PSI | Modulus $E \times 10^6$ | Poisson's Ratio | Water Content Percent
1 | 485.2 | 6720 | 1.69 | 0.30 | 1.7
8.0
6.0
4.0
2.0
COMPRESSIVE STRESS $\sigma_1 \times 10^3$, PSI
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, $+\epsilon_a$, $-\epsilon_r$, PERCENT
STRESS-STRAIN RELATION Hole 1, el. 485.2, BR Dam

PLATE 13

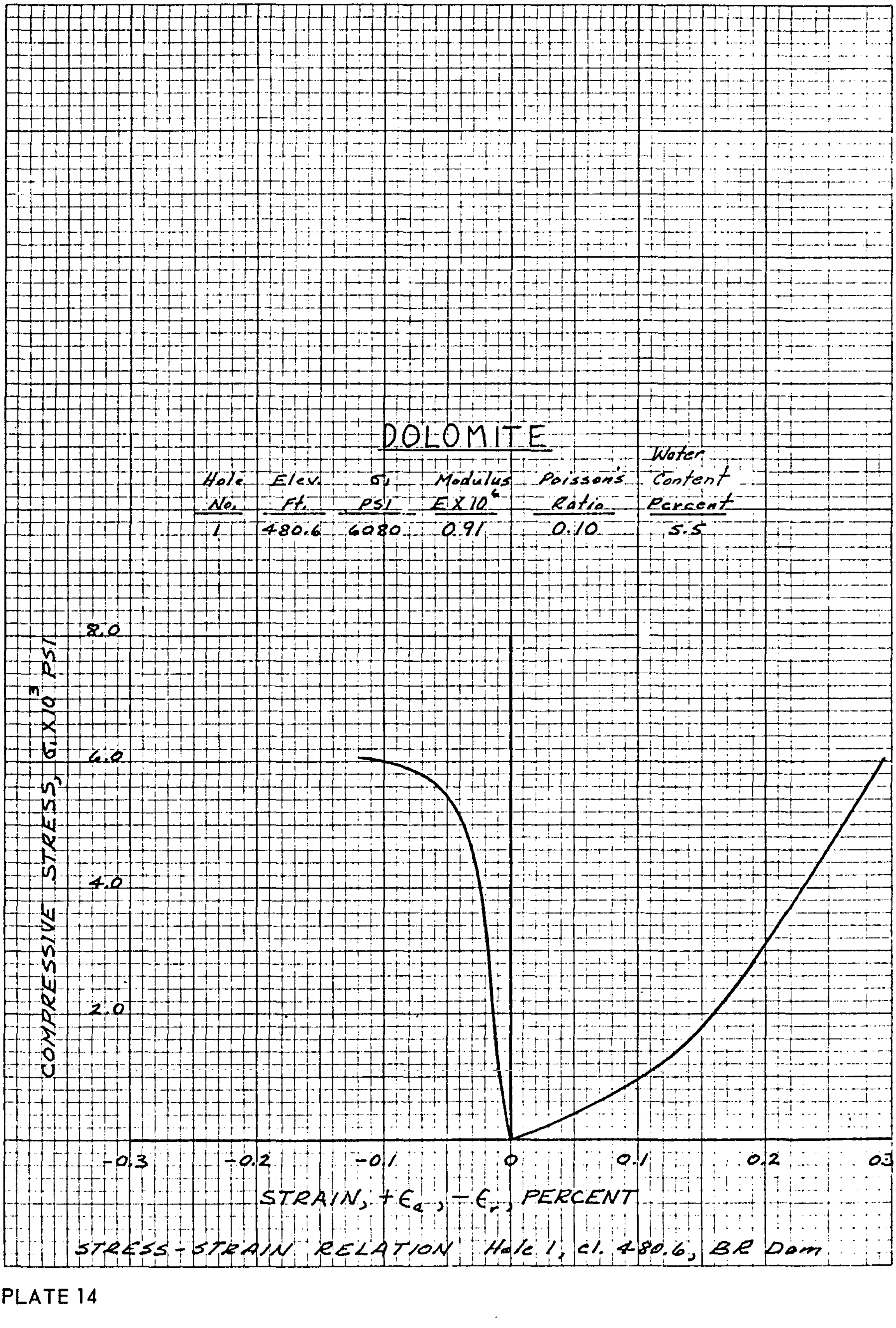

DOLOMITE
Hole No. | Elev. Ft. | σ_1 PSI | Modulus $E \times 10^6$ | Poisson's Ratio | Water Content Percent
1 | 480.6 | 6080 | 0.91 | 0.10 | 5.5
8.0
6.0
4.0
2.0
COMPRESSIVE STRESS, $\sigma_1 \times 10^3$ PSI
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, $+\epsilon_a$, $-\epsilon_r$, PERCENT
STRESS-STRAIN RELATION Hole 1, el. 480.6, BR Dam

PLATE 14

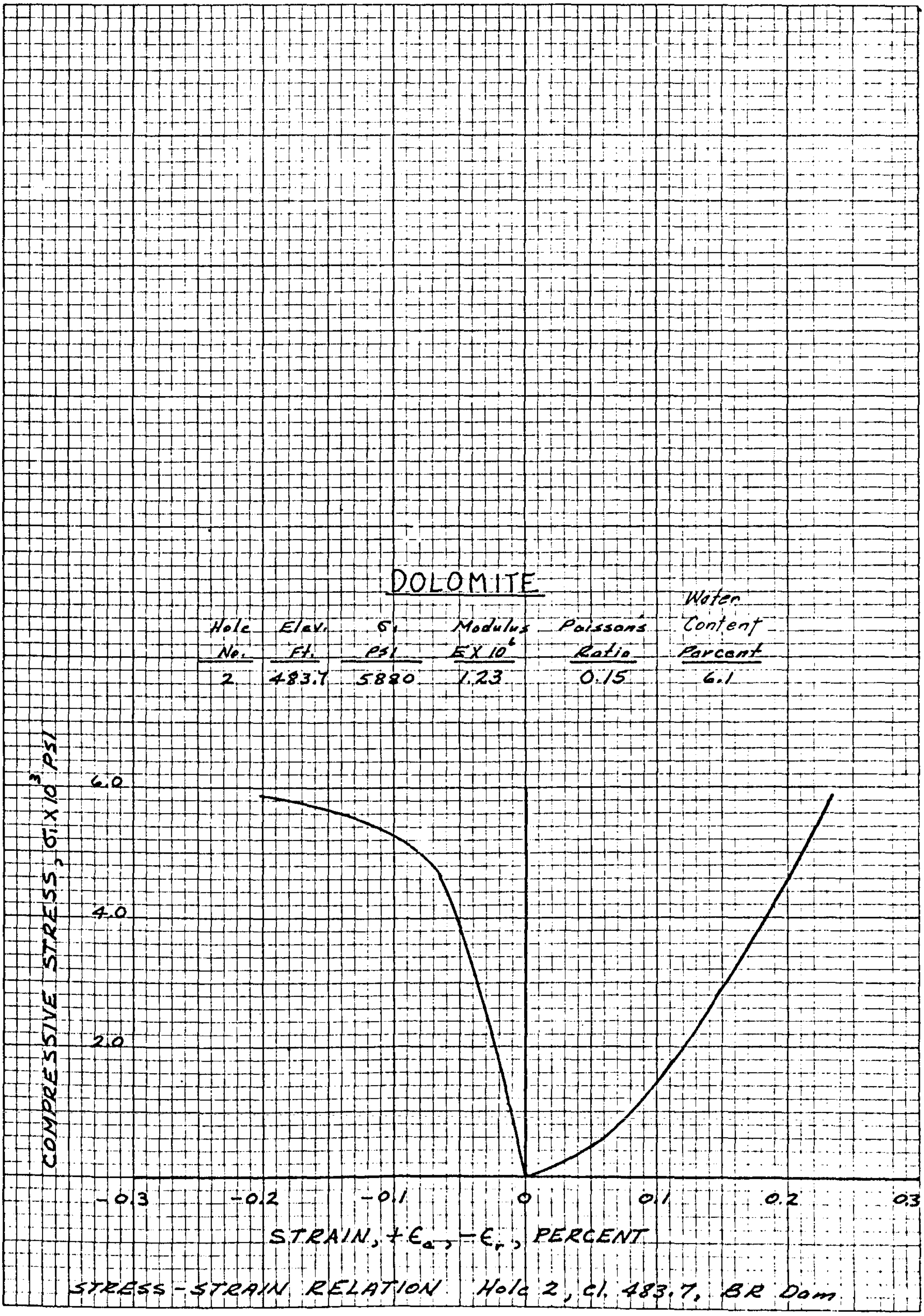
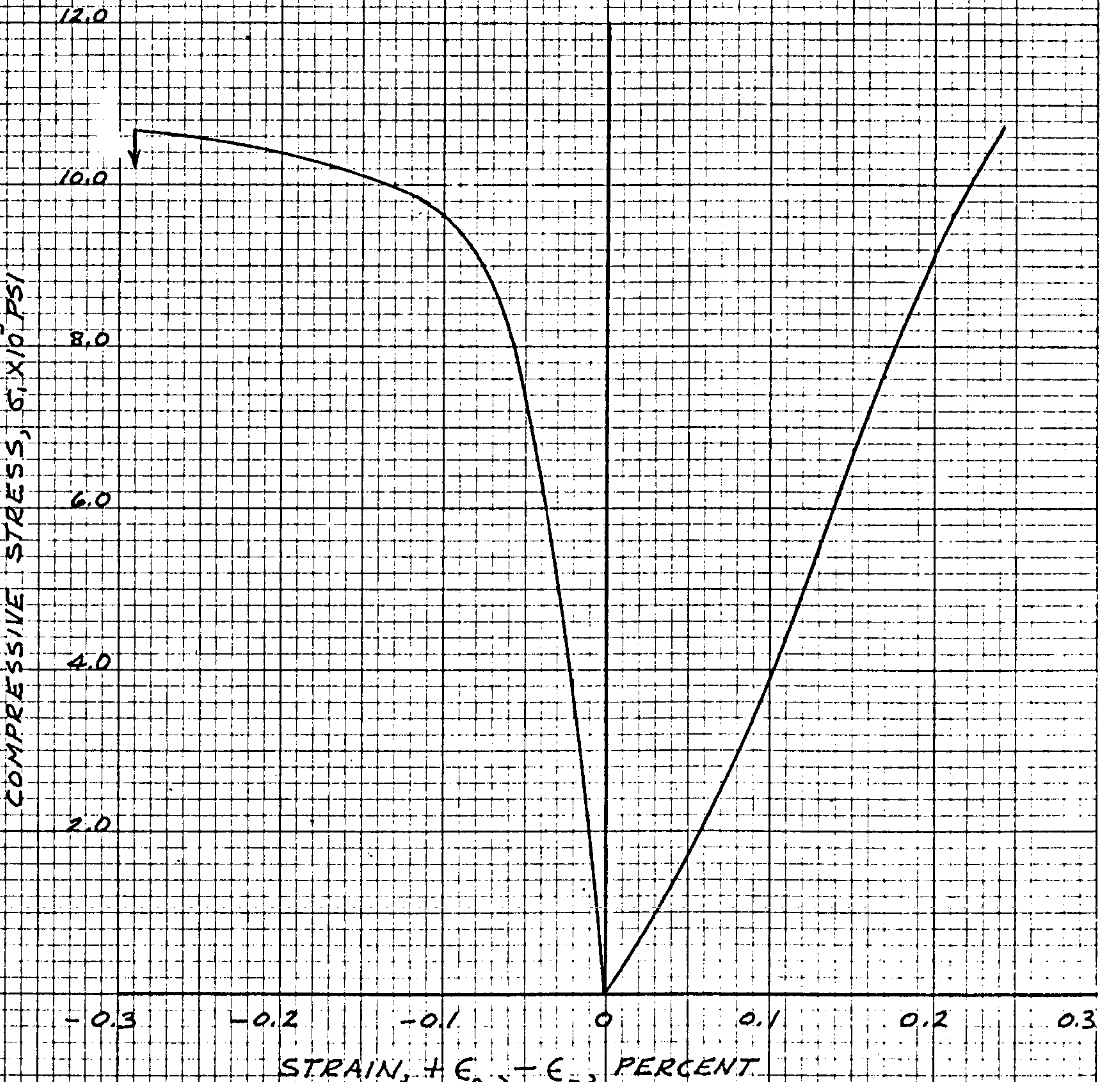

DOLOMITE
Hole No. | Elev. Ft. | σ_1 psi | Modulus $E \times 10^6$ | Poissons Ratio | Water Content Percent
2 | 483.7 | 5880 | 1.23 | 0.15 | 6.1
6.0
4.0
2.0
COMPRESSIVE STRESS, $\sigma_1 \times 10^3$ PSI
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, $+\epsilon_a$, $-\epsilon_r$, PERCENT
STRESS-STRAIN RELATION Hole 2, el. 483.7, BR Dom

PLATE 15

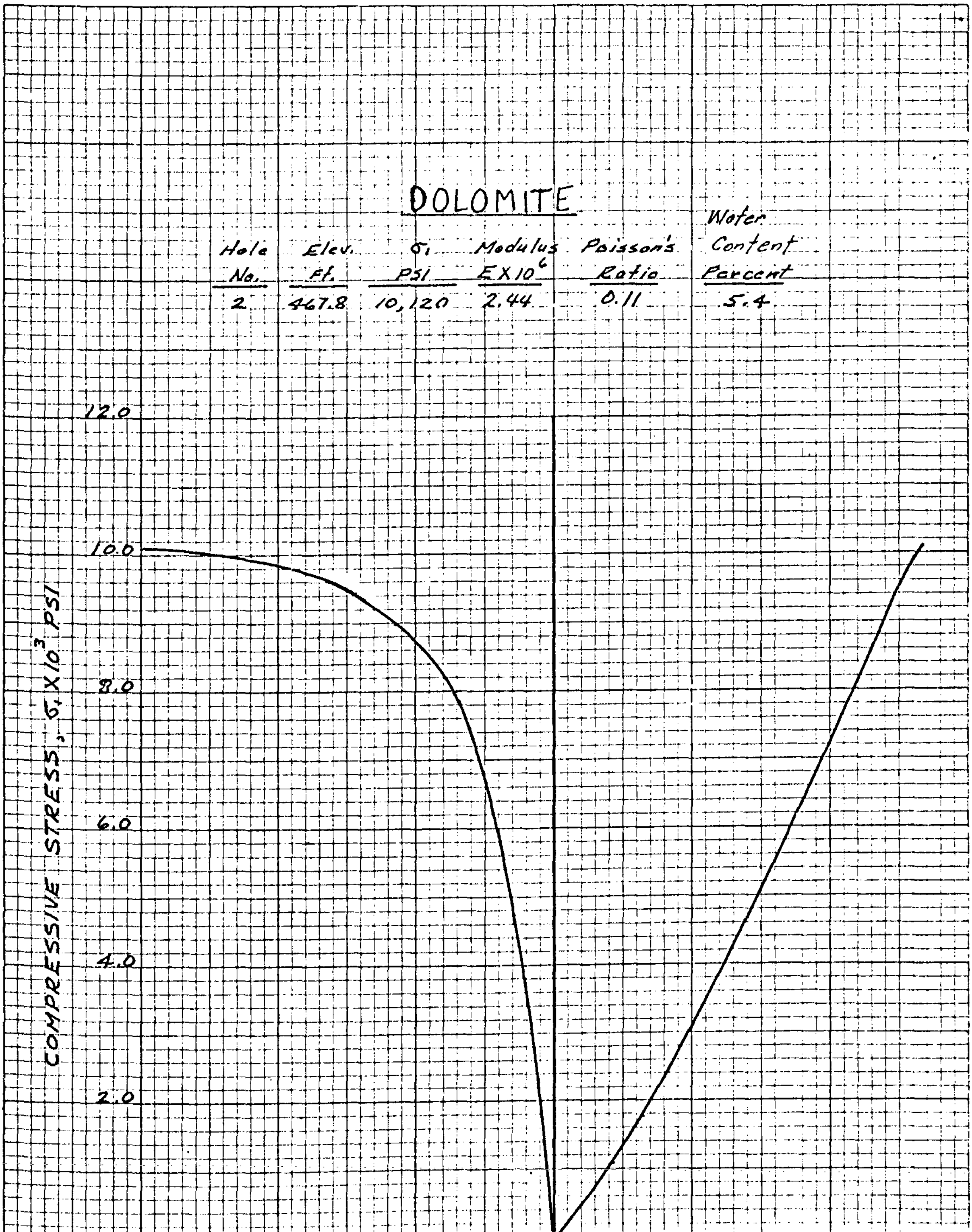
DOLOMITE

Hole No.	Elev. Ft.	σ_1 PSI	Modulus $E \times 10^6$	Poissons Ratio	Water Content Percent
2	475.2	10,690	2.86	0.14	2.8



STRESS-STRAIN RELATION Hole 2, el. 475.2, BR Dam

PLATE 16


DOLOMITE
Hole No. | Elev. Ft. | σ1 PSI | Modulus E X 10^6 | Poisson's Ratio | Water Content Percent
2 | 467.8 | 10,120 | 2.44 | 0.11 | 5.4
12.0
10.0
8.0
6.0
4.0
2.0
COMPRESSIVE STRESS, σ1 X 10^3 PSI
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, +εa, -εr, PERCENT
STRESS-STRAIN RELATION Hole 2, el. 467.8, BR Dam

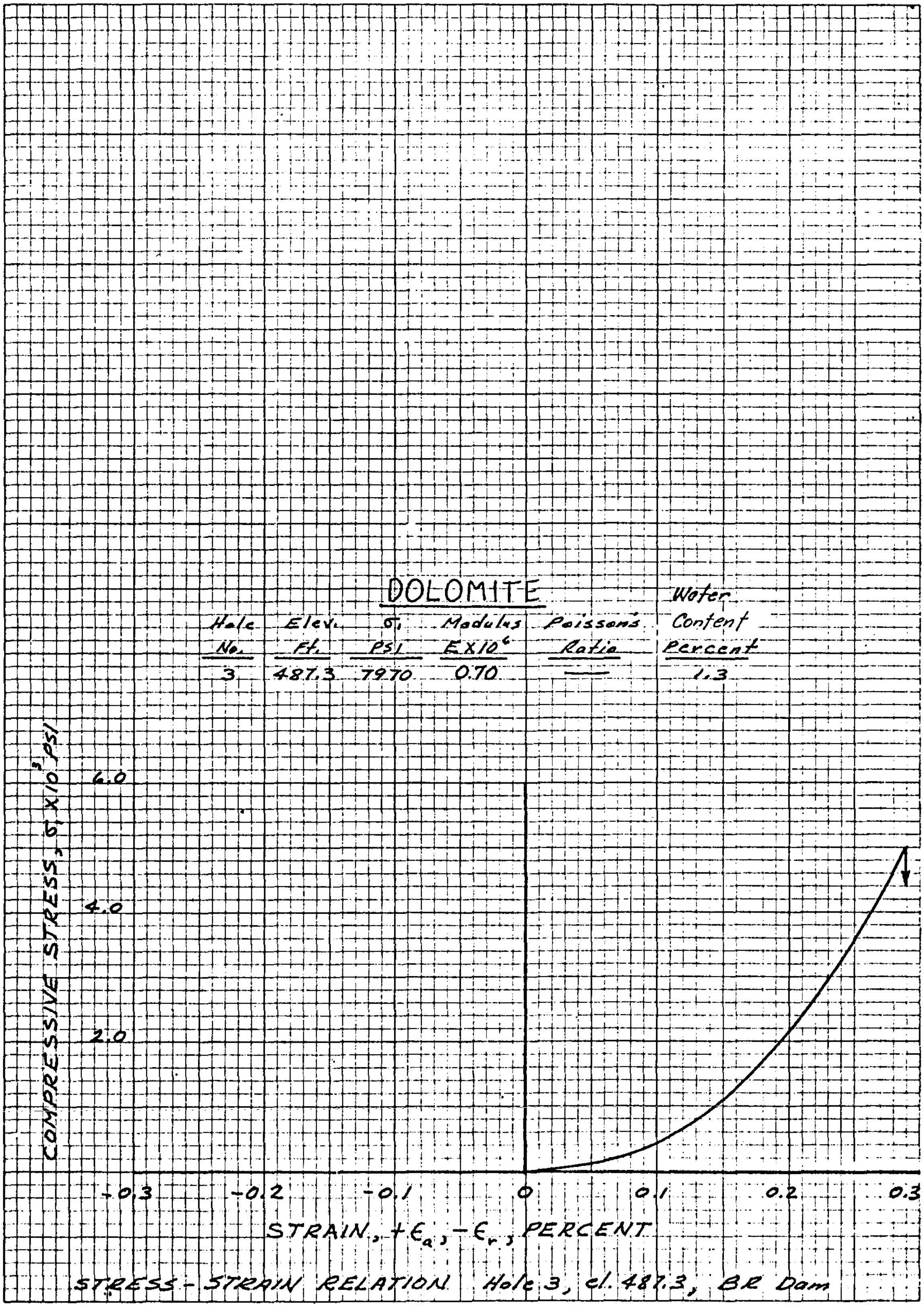

DOLOMITE
Hole No. | Elev. Ft. | σ_1 PSI | Modulus $E \times 10^6$ | Poisson's Ratio | Water Content Percent
3 | 487.3 | 7970 | 0.70 | — | 1.3
COMPRESSIVE STRESS, $\sigma_1 \times 10^3$ PSI
6.0
4.0
2.0
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, $+\epsilon_a$, $-\epsilon_r$, PERCENT
STRESS-STRAIN RELATION Hole 3, el. 487.3, BR Dam

PLATE 18

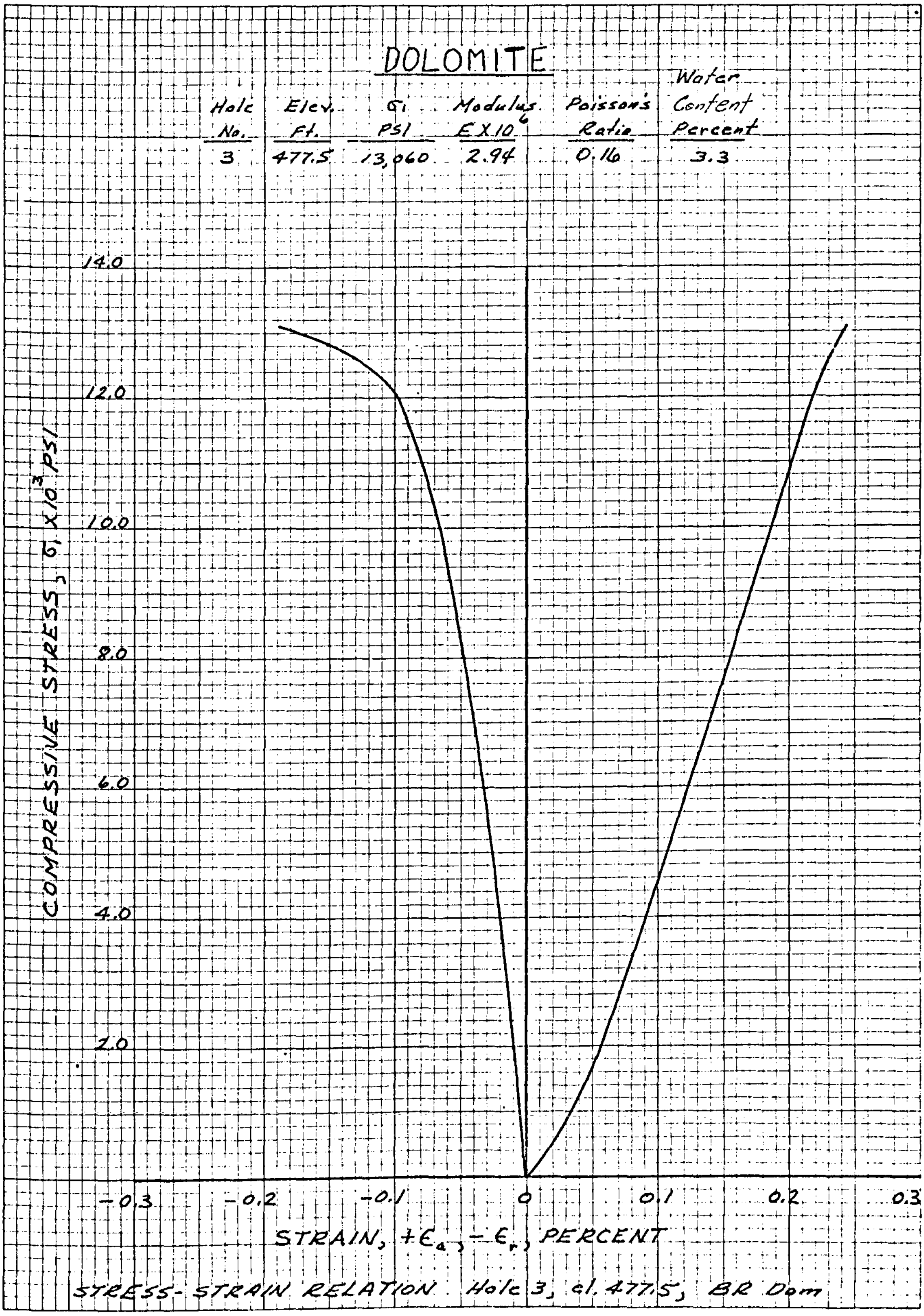

DOLOMITE
Hole No. | Elev. Ft. | σ1 PSI | Modulus E X 10^6 | Poisson's Ratio | Water Content Percent
3 | 477.5 | 13,060 | 2.94 | 0.16 | 3.3
14.0
12.0
10.0
8.0
6.0
4.0
2.0
COMPRESSIVE STRESS, σ1 X 10^3 PSI
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, +ε_a, -ε_r, PERCENT
STRESS-STRAIN RELATION Hole 3, el. 477.5, BR Dom

PLATE 19

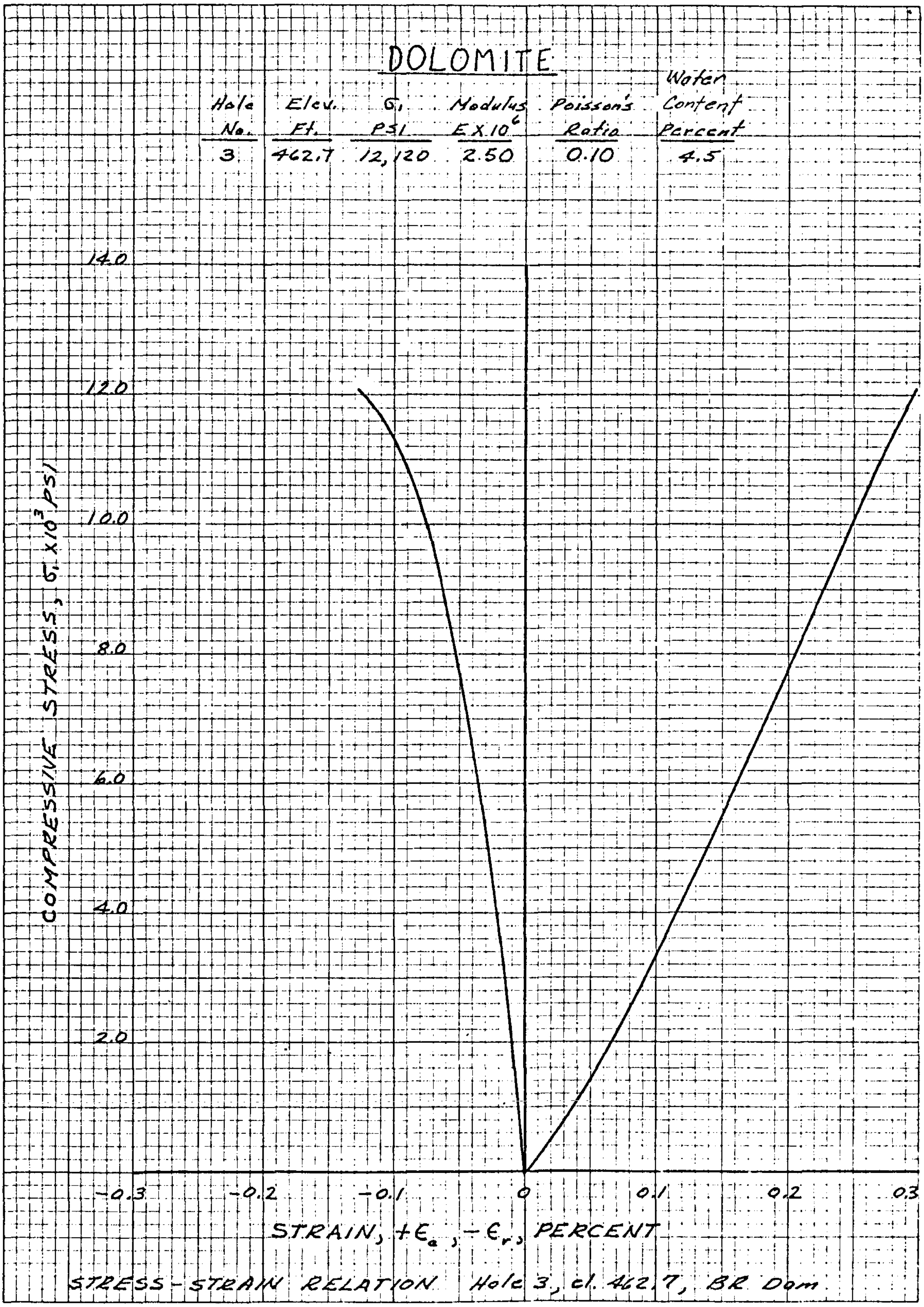
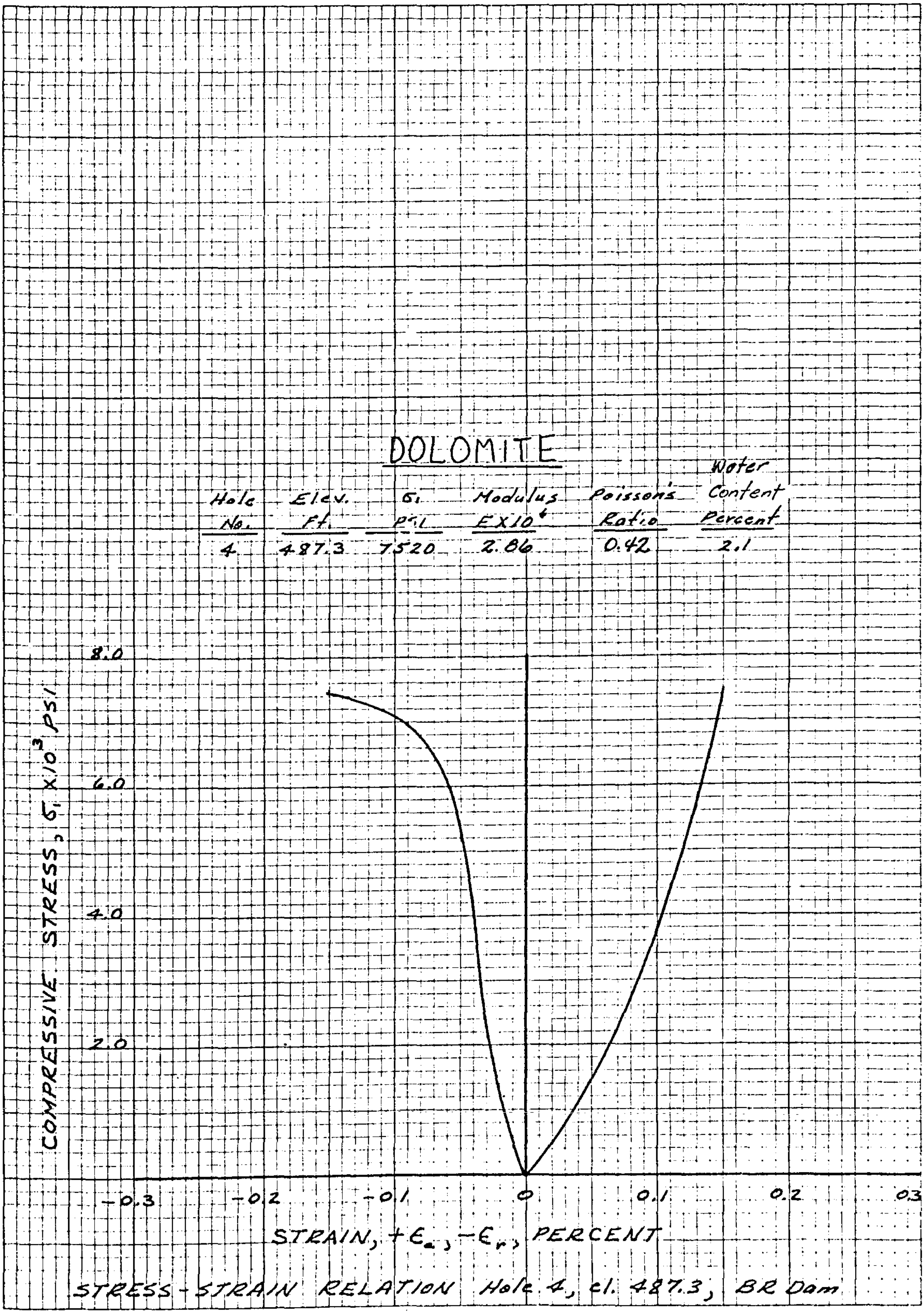

DOLOMITE
Hole No.
3
Elev. Ft.
462.7
σ_1 PSI
12,120
Modulus $E \times 10^6$
2.50
Poisson's Ratio
0.10
Water Content Percent
4.5
COMPRESSIVE STRESS, $\sigma_1 \times 10^3$ PSI)
14.0
12.0
10.0
8.0
6.0
4.0
2.0
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, $+\epsilon_a$, $-\epsilon_r$, PERCENT
STRESS-STRAIN RELATION Hole 3, el. 462.7, BR Dam

PLATE 20


DOLOMITE
Hole No. | Elev. Ft. | σ1 psi | Modulus E X 10^6 | Poisson's Ratio | Water Content Percent
4 | 487.3 | 7520 | 2.86 | 0.42 | 2.1
8.0
6.0
4.0
2.0
COMPRESSIVE STRESS, σ1 X 10^3 psi
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, +εa, -εr, PERCENT
STRESS-STRAIN RELATION Hole 4, el. 487.3, BR Dam

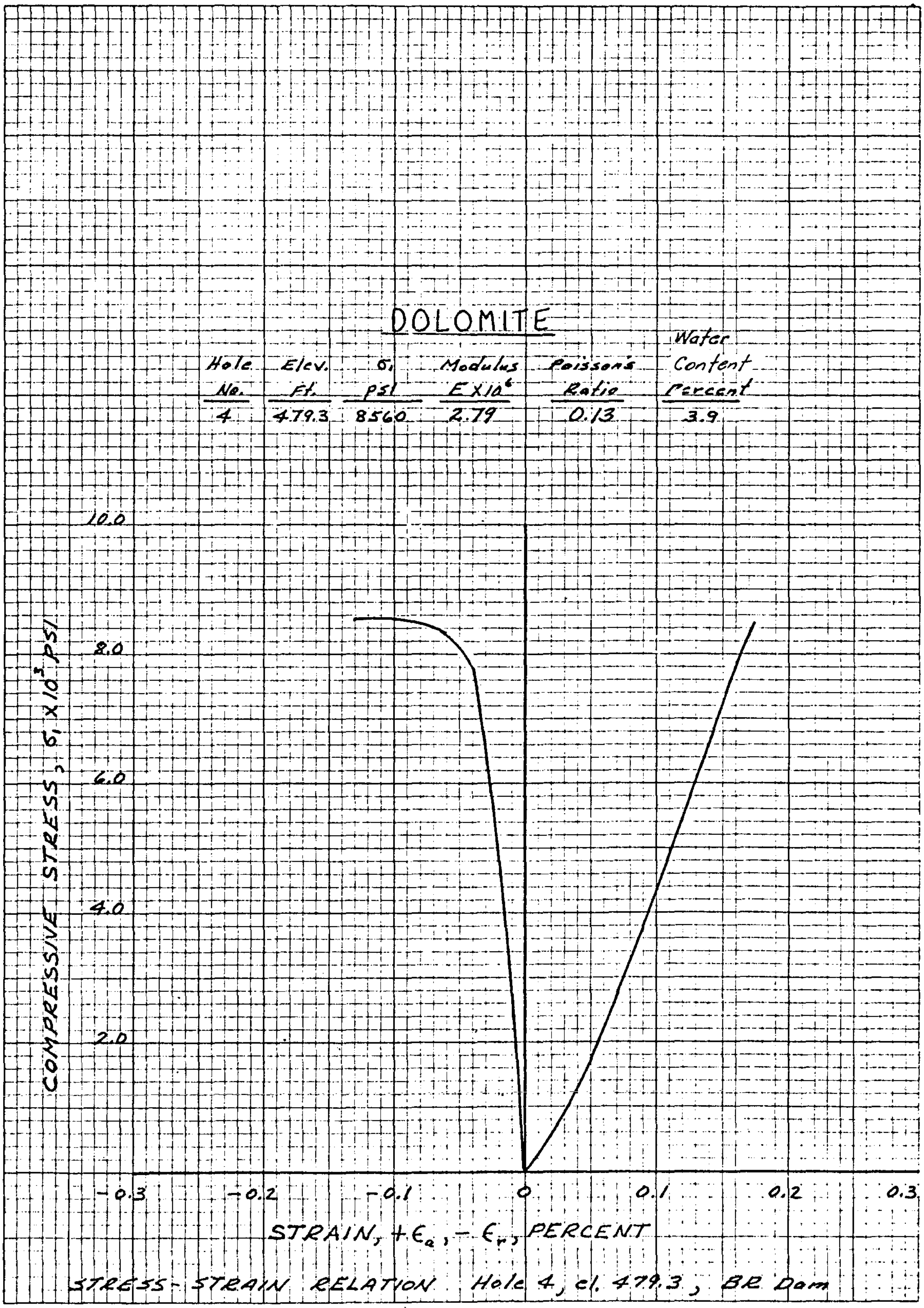

DOLOMITE
Hole No. | Elev. Ft. | σ₁ psi | Modulus E×10⁶ | Poisson's Ratio | Water Content Percent
4 | 479.3 | 8560 | 2.79 | 0.13 | 3.9
COMPRESSIVE STRESS, σ₁ ×10³ PSI
10.0
8.0
6.0
4.0
2.0
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, +ε_a, -ε_r, PERCENT
STRESS-STRAIN RELATION Hole 4, el. 479.3, BR Dam

PLATE 22

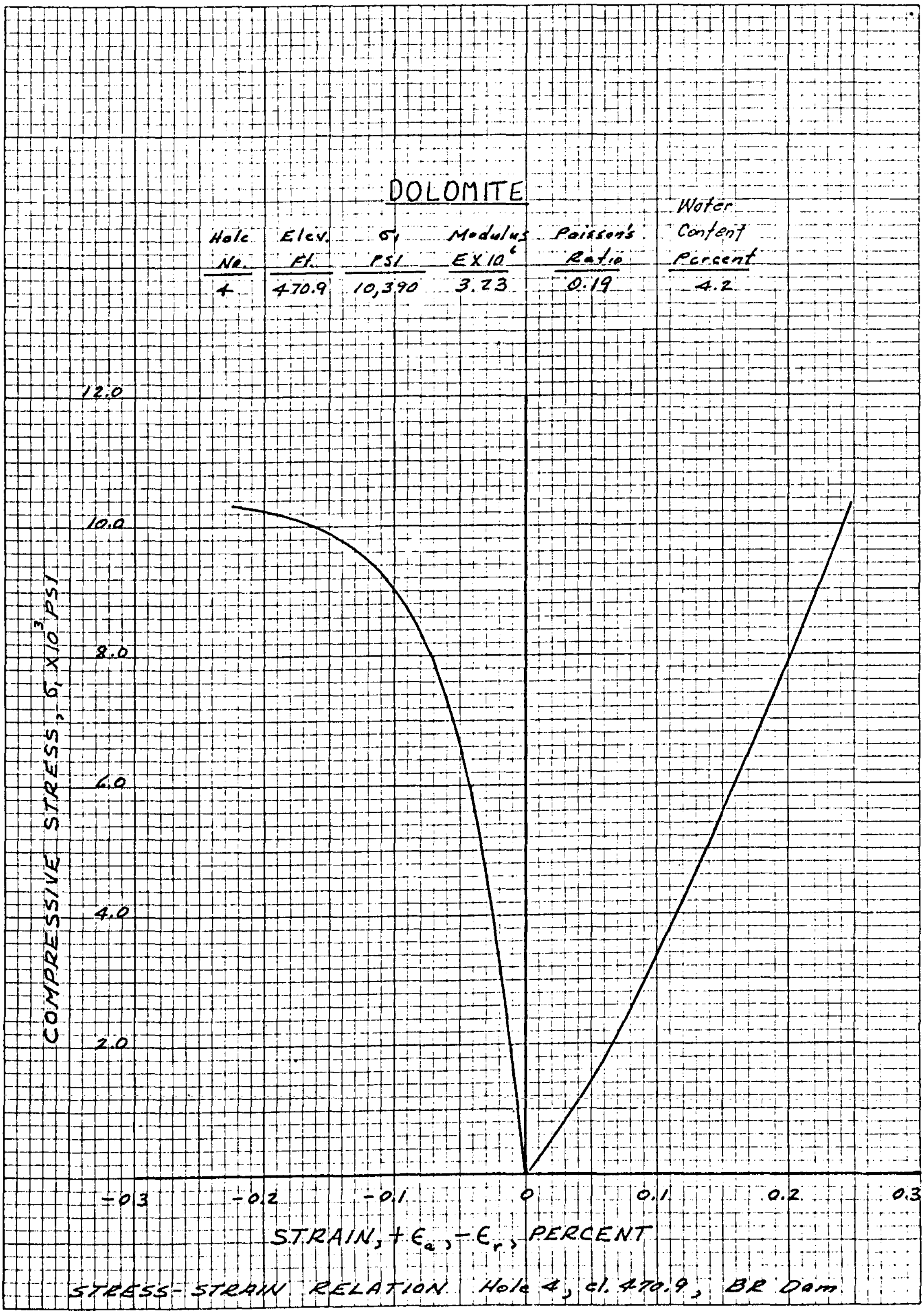
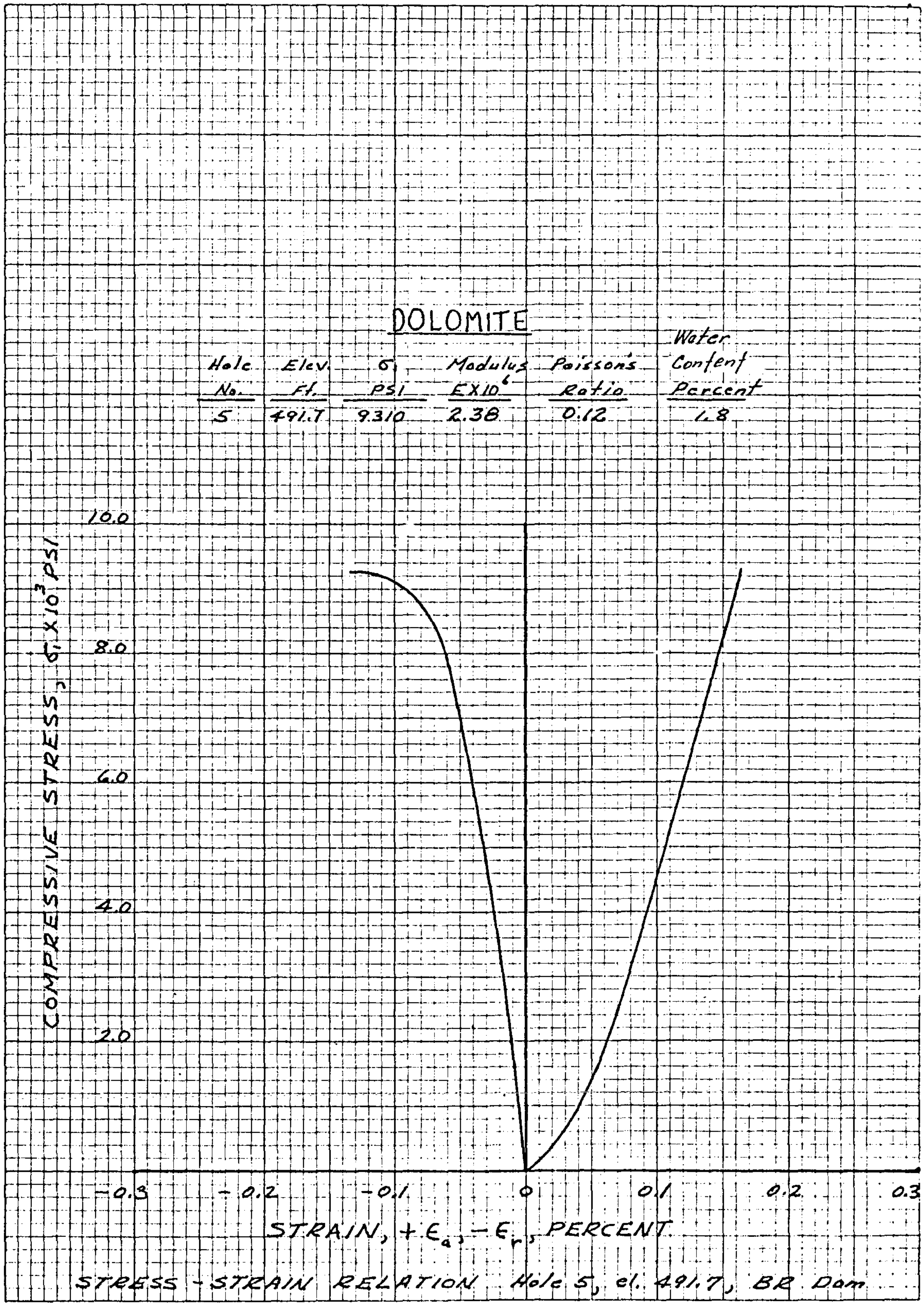

DOLOMITE
Hole No. | Elev. Ft. | σ_1 PSI | Modulus $E \times 10^6$ | Poisson's Ratio | Water Content Percent
4 | 470.9 | 10,390 | 3.23 | 0.19 | 4.2
12.0
10.0
8.0
6.0
4.0
2.0
COMPRESSIVE STRESS, $\sigma_1 \times 10^3$ PSI
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, $+\epsilon_a$, $-\epsilon_r$, PERCENT
STRESS-STRAIN RELATION Hole 4, el. 470.9, BR Dam

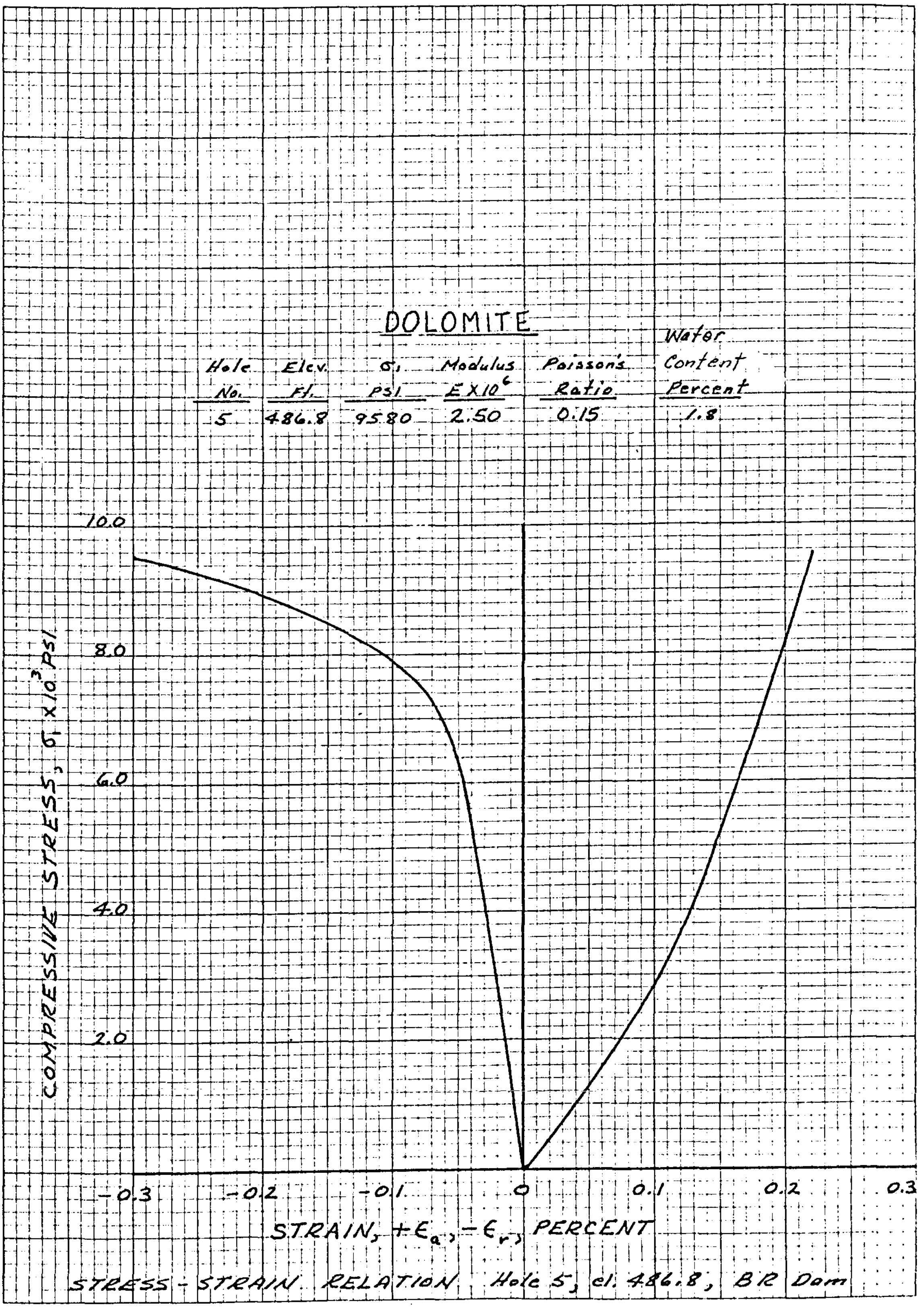
PLATE 23



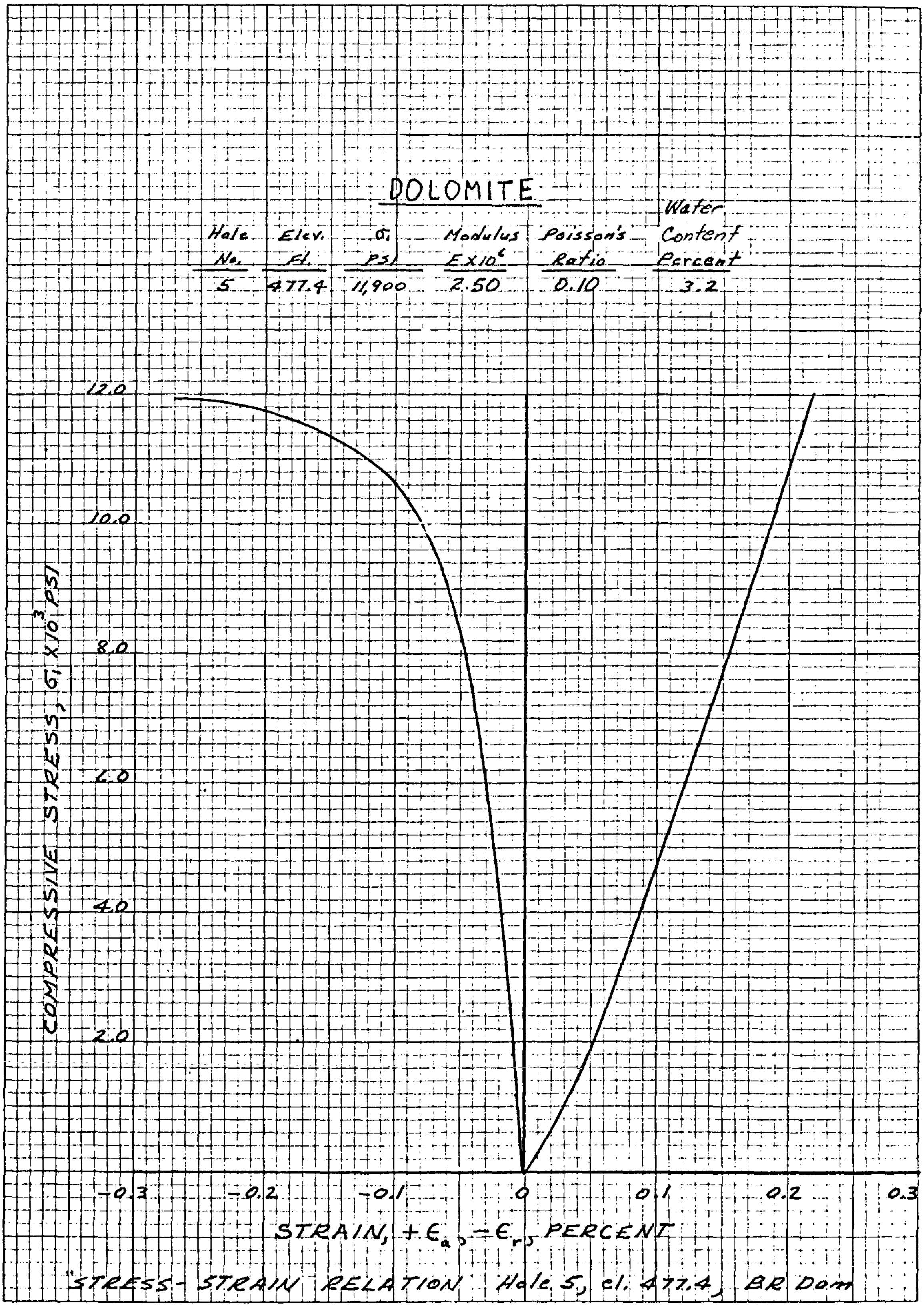
DOLOMITE

Hole No.	Elev. Ft.	σ_1 PSI	Modulus $E \times 10^6$	Poisson's Ratio	Water Content Percent
5	491.7	9.310	2.38	0.12	1.8

COMPRESSIVE STRESS, $\sigma_1 \times 10^3$ PSI
10.0
8.0
6.0
4.0
2.0
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, $+\epsilon_a$, $-\epsilon_r$, PERCENT
STRESS - STRAIN RELATION Hole 5, el. 491.7, BR Dam

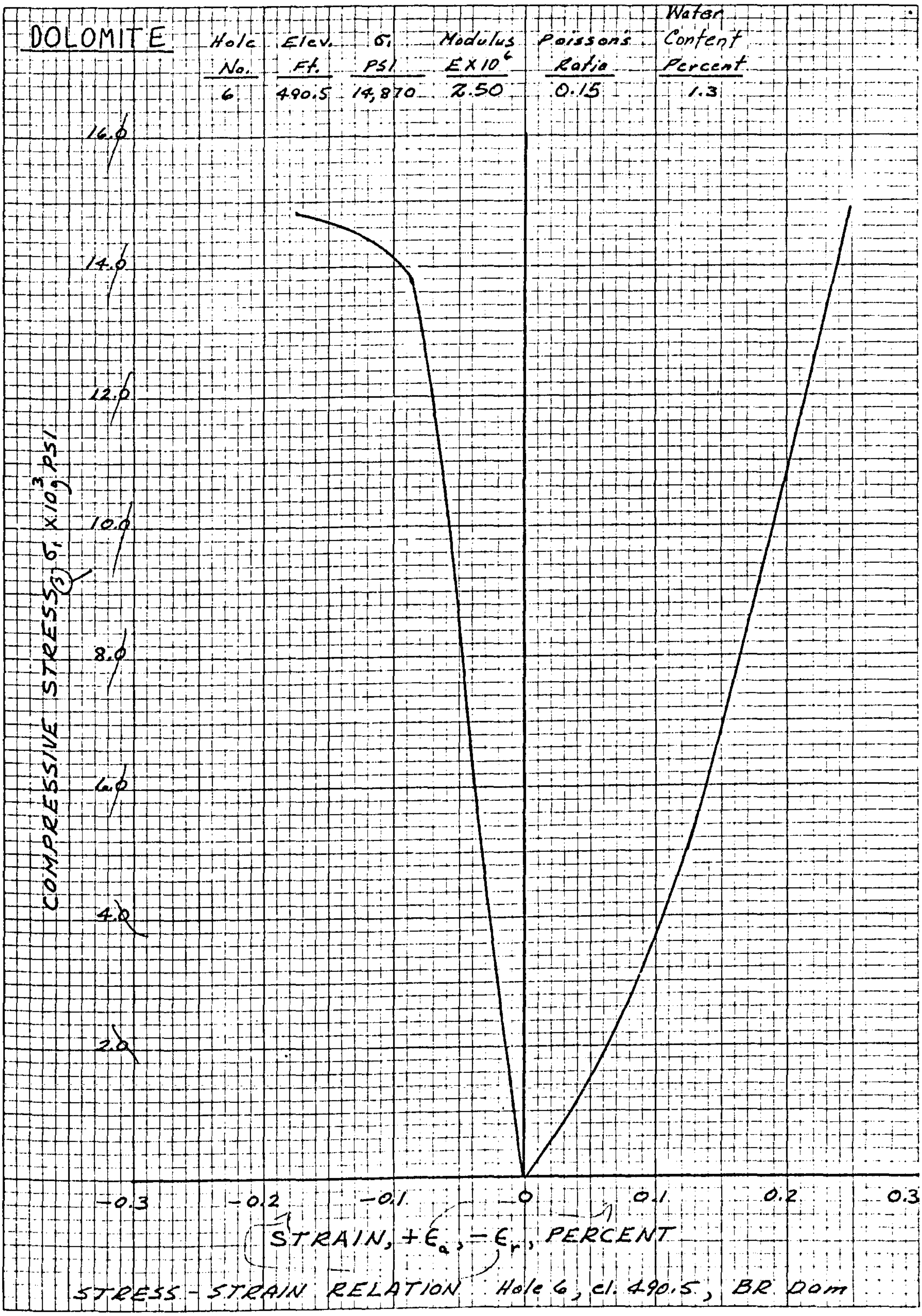


DOLOMITE
Hole No.
Elev. Ft.
σ1 PSI
Modulus E X10^6
Poisson's Ratio
Water Content Percent
5
486.8
9580
2.50
0.15
1.8
10.0
8.0
6.0
4.0
2.0
COMPRESSIVE STRESS, σ1 X 10^3 PSI
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, +ϵa, -ϵr, PERCENT
STRESS - STRAIN RELATION Hole 5, el. 486.8, BR Dam



DOLOMITE
Hole No.
Elev. Ft.
σ_1 PSI
Modulus E X10^6
Poisson's Ratio
Water Content Percent
5
477.4
11,900
2.50
0.10
3.2
12.0
10.0
8.0
6.0
4.0
2.0
COMPRESSIVE STRESS, σ_1 X10^3 PSI
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, $+\epsilon_a$, $-\epsilon_r$, PERCENT
STRESS-STRAIN RELATION Hole 5, el. 477.4, BR Dam

PLATE 26



DOLOMITE
Hole No. 6
Elev. Ft. 490.5
σ_1 PSI 14,870
Modulus $E \times 10^6$ 2.50
Poisson's Ratio 0.15
Water Content Percent 1.3
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
COMPRESSIVE STRESS, $\sigma_1 \times 10^3$, PSI
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, $+\epsilon_a$, $-\epsilon_r$, PERCENT
STRESS - STRAIN RELATION Hole 6, el. 490.5, BR Dom

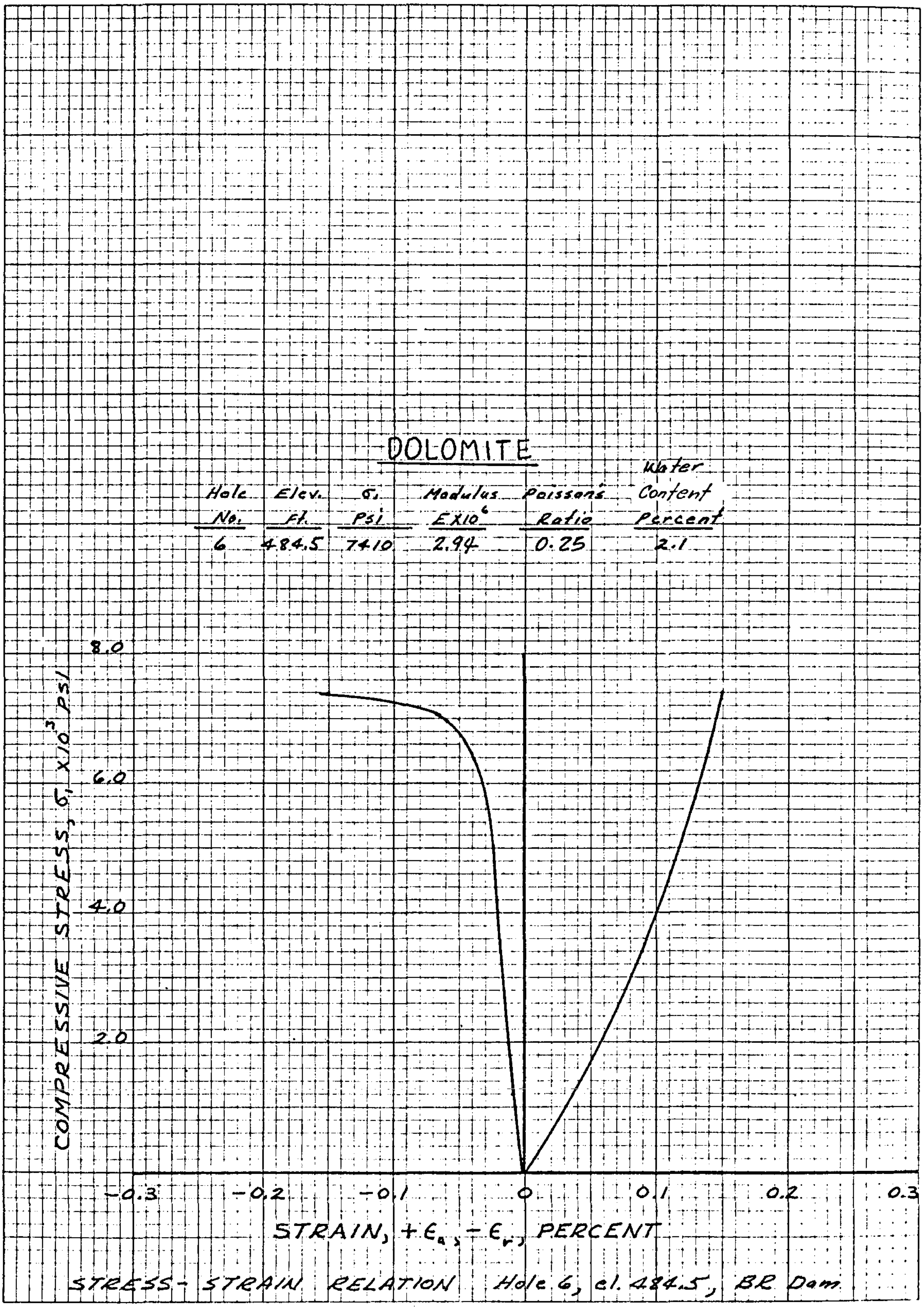
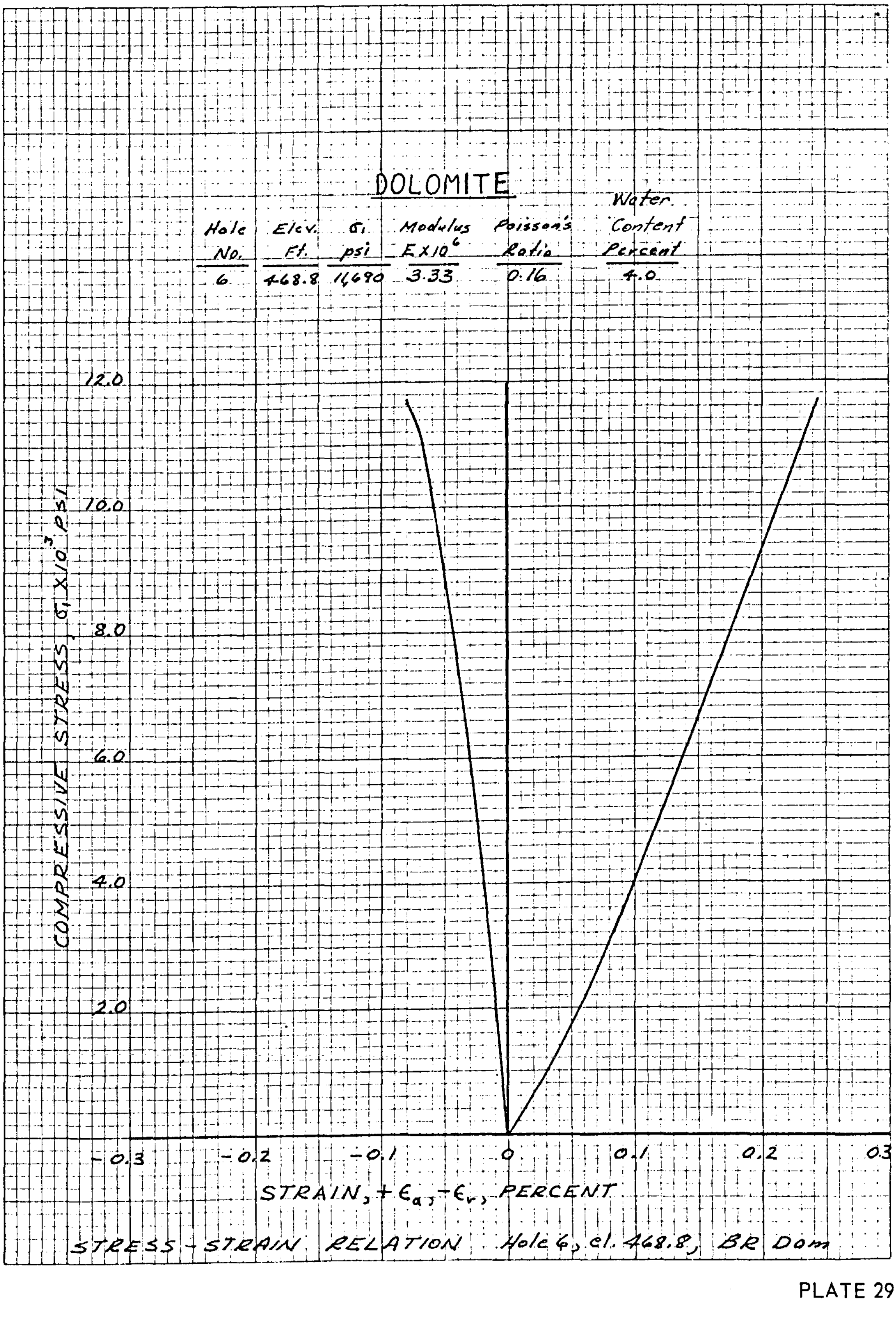
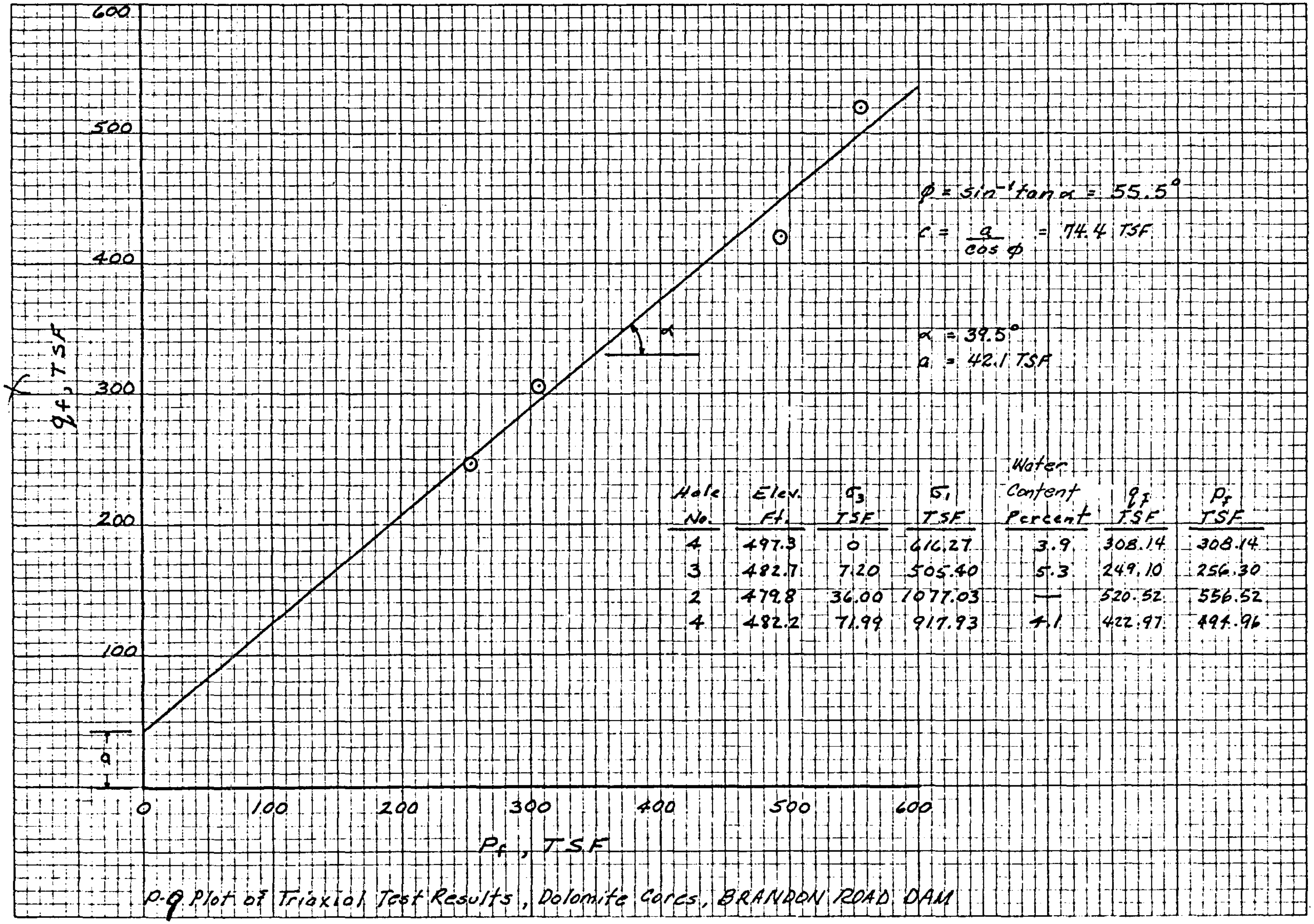

DOLOMITE
Hole No.
Elev. Ft.
σ₁ psi
Modulus E X10⁶
Poisson's Ratio
Water Content Percent
6
484.5
7410
2.94
0.25
2.1
8.0
6.0
4.0
2.0
COMPRESSIVE STRESS, σ₁ X10³ PSI
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, +ε_a, -ε_r, PERCENT
STRESS-STRAIN RELATION Hole 6, el. 484.5, BR Dam

PLATE 28


DOLOMITE
Hole No. | Elev. Ft. | σ1 psi | Modulus E X 10^6 | Poisson's Ratio | Water Content Percent
6 | 468.8 | 11,690 | 3.33 | 0.16 | 4.0
12.0
10.0
8.0
6.0
4.0
2.0
COMPRESSIVE STRESS, σ1 X 10^3 PSI
-0.3
-0.2
-0.1
0
0.1
0.2
0.3
STRAIN, +εa, -εr, PERCENT
STRESS - STRAIN RELATION Hole 6, el. 468.8, BR Dom



$\phi = \sin^{-1} \tan \alpha = 55.5°$

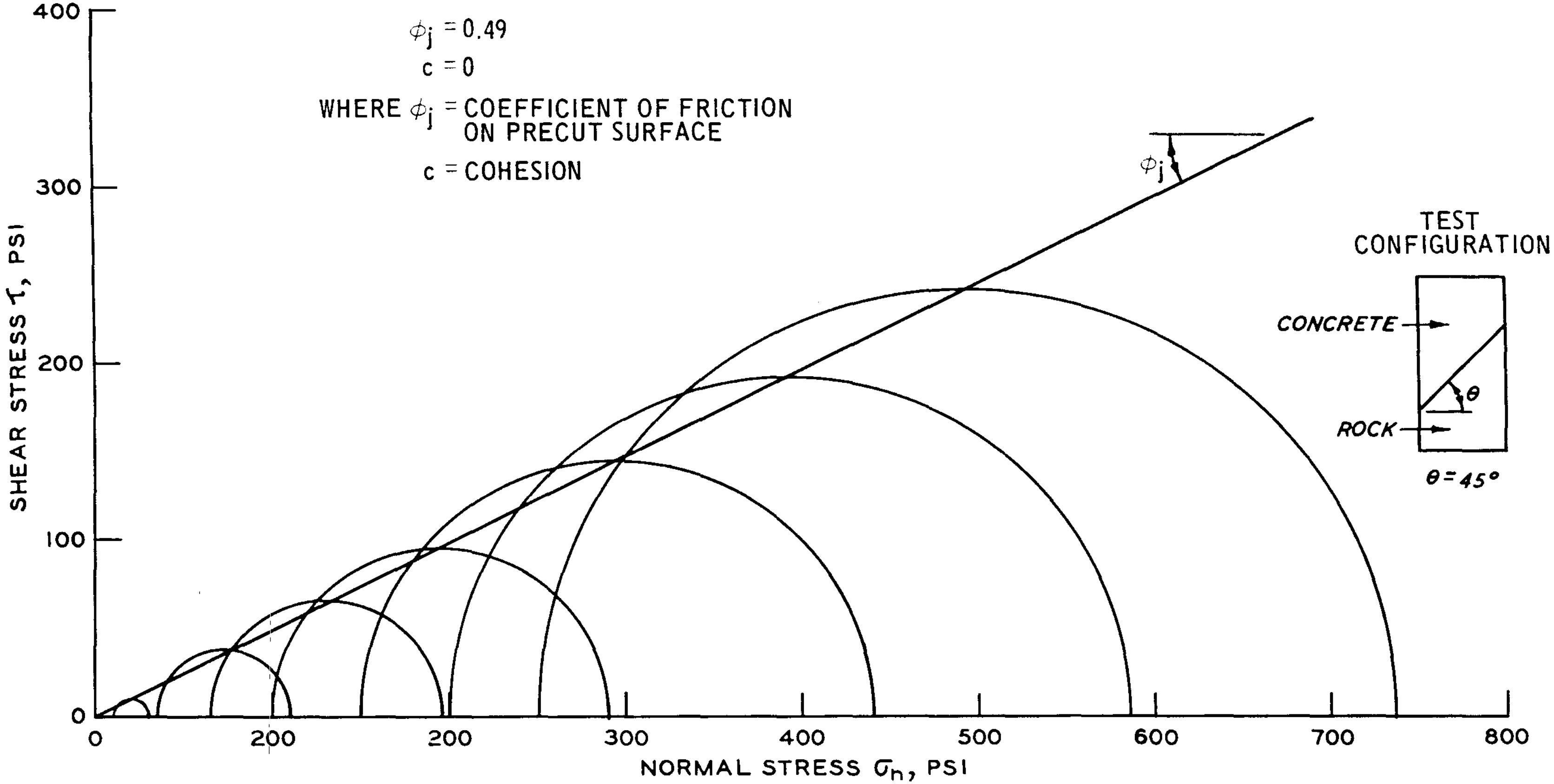
$c = \dfrac{a}{\cos \phi} = 74.4$ TSF

$\alpha = 39.5°$

$a = 42.1$ TSF

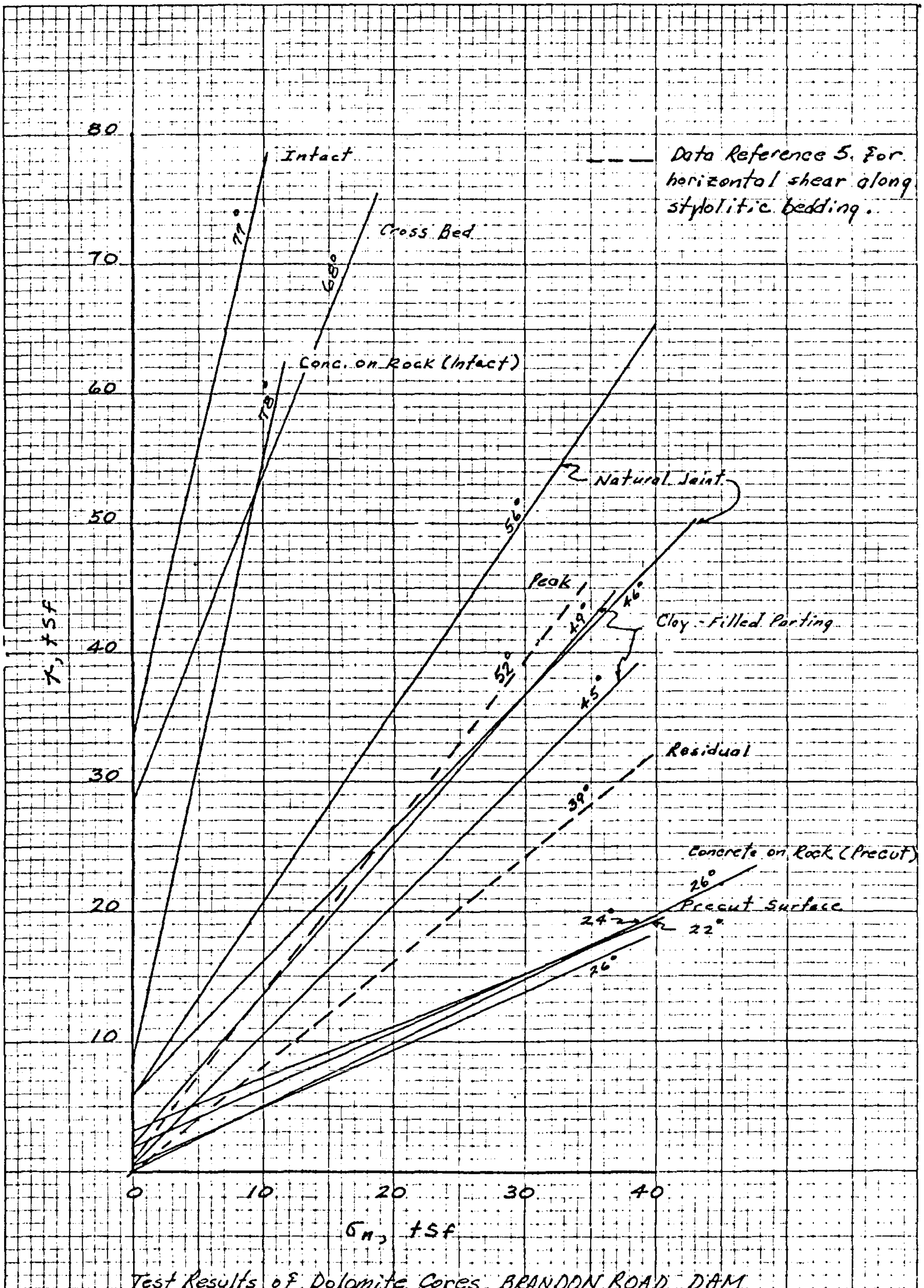
Hole No.	Elev. Ft.	σ_3 TSF	σ_1 TSF	Water Content Percent	q_f TSF	p_f TSF
4	497.3	0	616.27	3.9	308.14	308.14
3	482.7	7.20	505.40	5.3	249.10	256.30
2	479.8	36.00	1077.03	—	520.52	556.52
4	482.2	71.99	917.93	4.1	422.97	494.96

p-q Plot of Triaxial Test Results, Dolomite Cores, BRANDON ROAD DAM

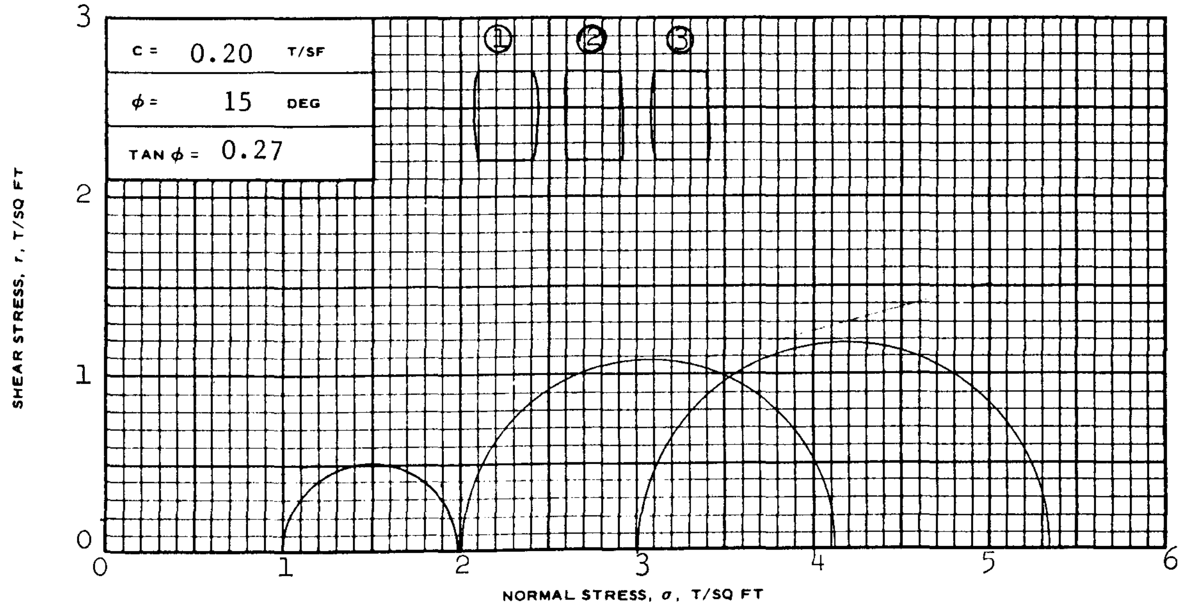


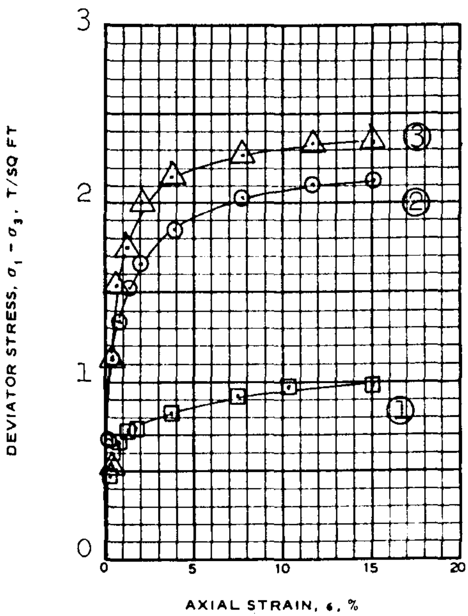
BRANDON ROAD DAM

MOHR'S DIAGRAM
CONCRETE ON DOLOMITE
PRECUT SURFACE



Test Results of Dolomite Cores, BRANDON ROAD DAM





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	103.9	109.3	114.1
	DRY DENSITY LB/CU FT	γd_o	40.8	39.6	38.4
	SATURATION, %	S_o	93.4	93.8	94.0
	VOID RATIO	e_o	2.70	2.83	2.95
BEFORE SHEAR	WATER CONTENT, %	w_c	61.7	51.3	52.9
	DRY DENSITY LB/CU FT	γd_c	58.9	66.1	65.5
	SATURATION, %	S_c	95.1	96.2	97.8
	VOID RATIO	e_c	1.58	1.30	1.32
	FINAL BACK PRESSURE, T/SQ FT	u_o	2.88	2.88	2.88
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.99	2.12	2.34
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	652	625	492
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.38	1.37	1.39
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY (CH), black;oily odor

LL 96	PL 39	PI 57	Gs 2.43	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST R

REMARKS: See attached sheet for effective values.

PROJECT BRANDON ROAD DAM REHABILITATION

BORING NO. | SAMPLE NO. 1

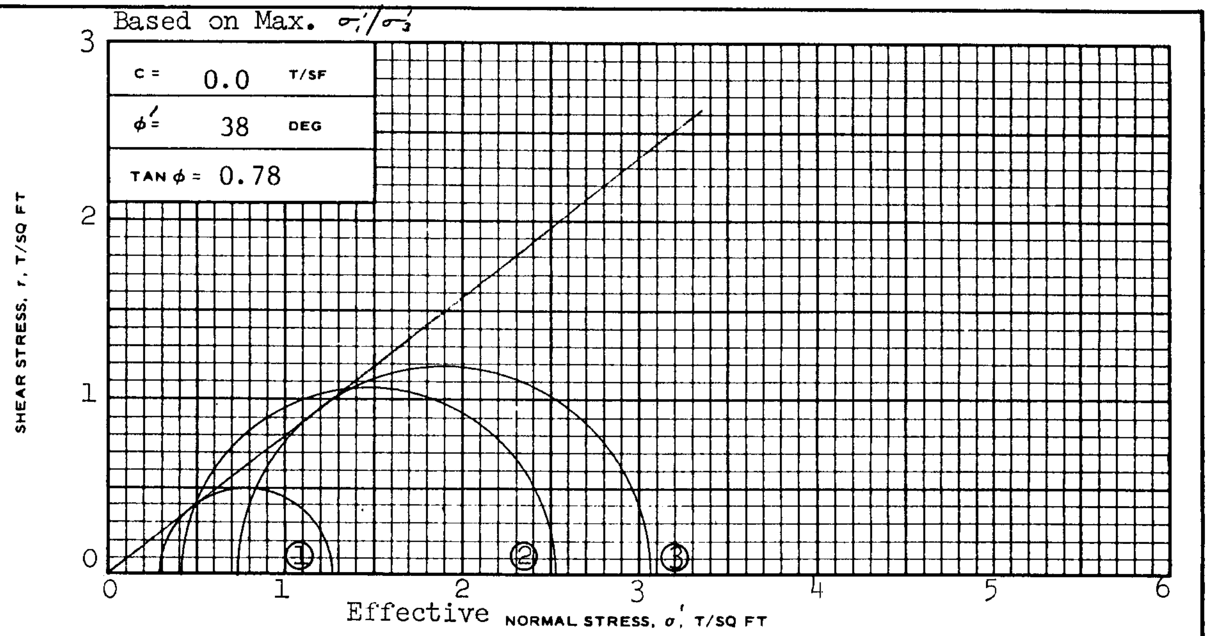
DEPTH/ELEV 519.0-516.5

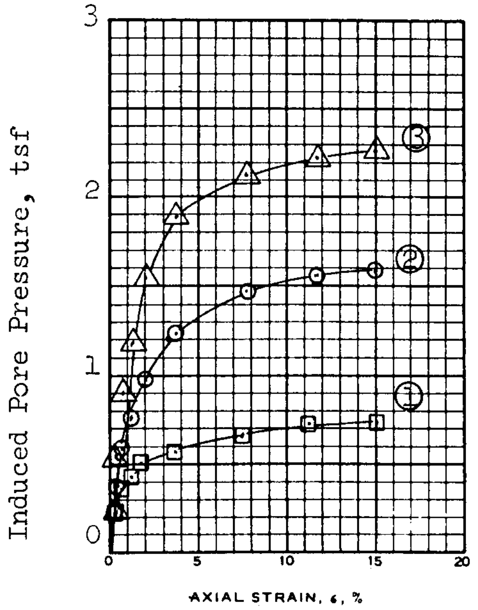
LABORATORY USAEWES | DATE 4 Feb. 1977

Sheet 1 of 2.

JMS **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.						
INITIAL	WATER CONTENT, %	w_o				
	DRY DENSITY LB/ CU FT	γd_o				
	SATURATION, %	S_o				
	VOID RATIO	e_o				
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3				
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$				
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f				
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o				
INITIAL HEIGHT, IN.		H_o				

CONTROLLED- TEST

DESCRIPTION OF SPECIMENS

LL	PL	PI	Gs	TYPE OF SPECIMEN	TYPE OF TEST

REMARKS:

PROJECT BRANDON ROAD DAM REHABILITATION

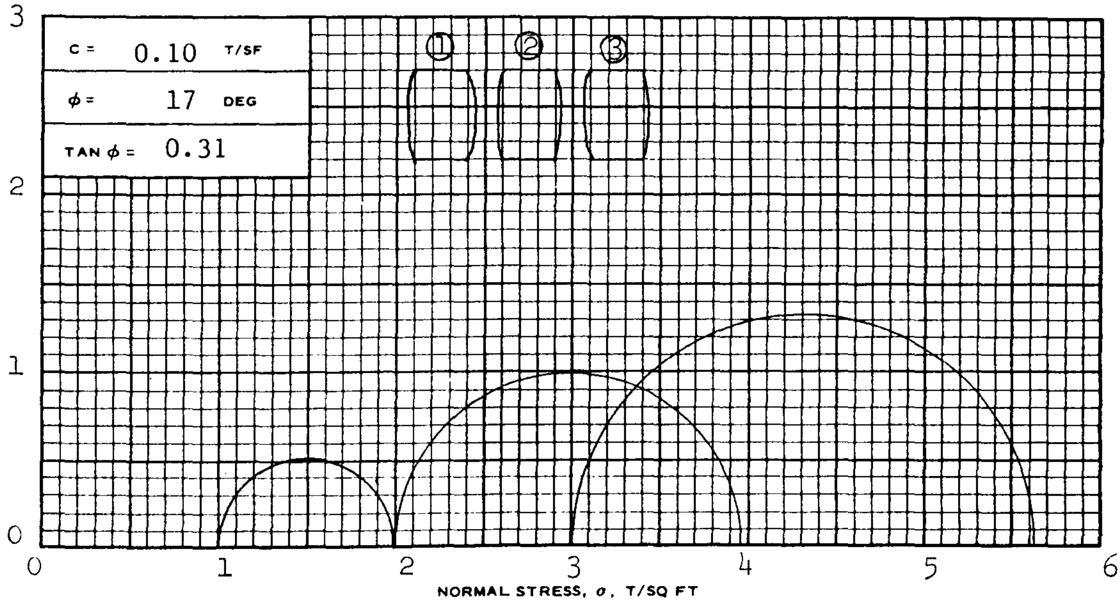
BORING NO. SAMPLE NO. 1

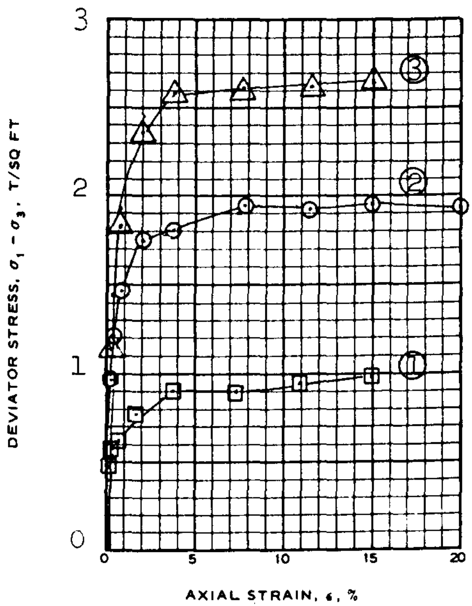
DEPTH/ELEV 519.0-516.5

LABORATORY USAEWES DATE 4 Feb. 1977

Sheet 2 of 2.

JMS **TRIAXIAL COMPRESSION TEST REPORT**





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	108.0	112.1	111.0
	DRY DENSITY LB/CU FT	γd_o	39.4	39.2	39.8
	SATURATION, %	S_o	90.6	93.3	94.3
	VOID RATIO	e_o	3.04	3.06	3.00
BEFORE SHEAR	WATER CONTENT, %	w_c	66.6	55.7	52.6
	DRY DENSITY LB/CU FT	γd_c	53.0	65.0	66.9
	SATURATION, %	S_c	84.9	98.2	97.2
	VOID RATIO	e_c	2.00	1.45	1.38
	FINAL BACK PRESSURE, T/SQ FT	u_o	2.88	2.88	2.88
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.99	1.95	2.64
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	682	325	625
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.39	1.40	1.39
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- strain TEST

DESCRIPTION OF SPECIMENS ORGANIC SILT(OH), gray;pockets of decayed fibrous organic matter

LL 112 | PL 46 | PI 66 | Gs 2.55

~~TYPE OF SPECIMEN~~ UNDISTURBED ~~TYPE OF TEST~~ R

REMARKS: See attached sheet for effective values.

PROJECT BRANDON ROAD DAM REHABILITATION

BORING NO. | SAMPLE NO. 2

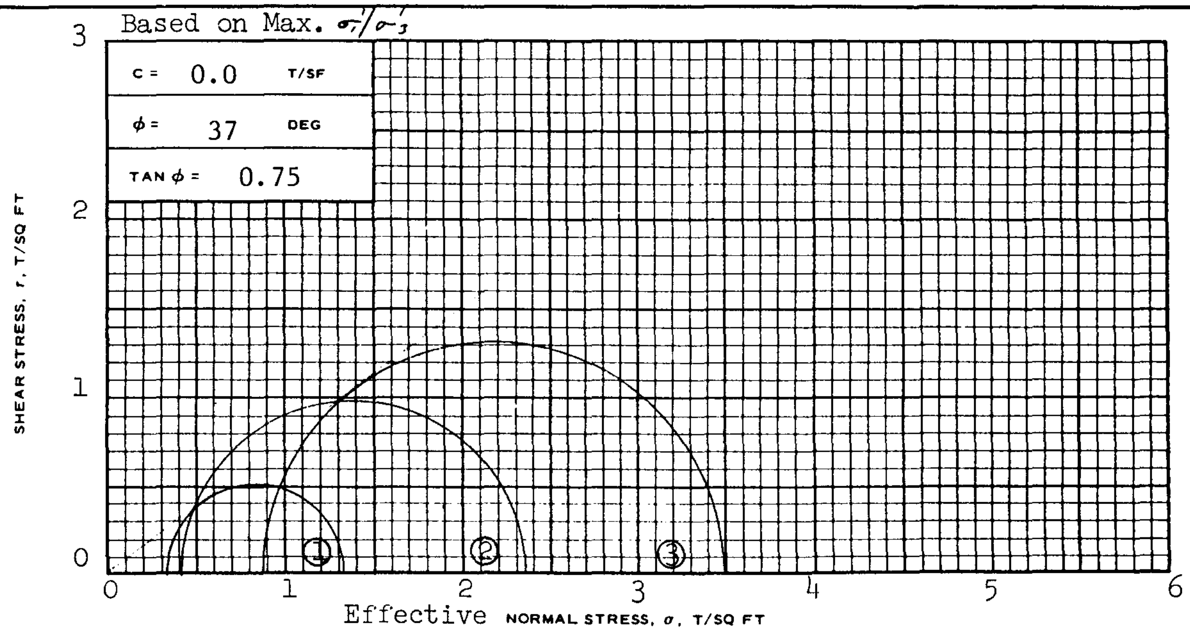
DEPTH/ELEV 516.0-513.5

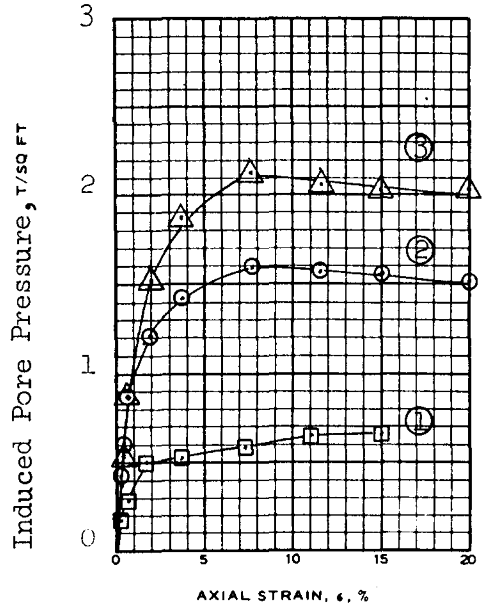
LABORATORY USAEWES | DATE 8 Feb. 1977

Sheet 1 of 2.

JMS **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110–2–1906)*





SPECIMEN NO.							
INITIAL	WATER CONTENT, %	w_o					
	DRY DENSITY LB/ CU FT	γd_o					
	SATURATION, %	S_o					
	VOID RATIO	e_o					
BEFORE SHEAR	WATER CONTENT, %	w_c					
	DRY DENSITY LB/CU FT	γd_c					
	SATURATION, %	S_c					
	VOID RATIO	e_c					
	FINAL BACK PRESSURE, T/SQ FT	u_o					
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3					
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$					
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f					
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$					
INITIAL DIAMETER, IN.		D_o					
INITIAL HEIGHT, IN.		H_o					

CONTROLLED- TEST

DESCRIPTION OF SPECIMENS

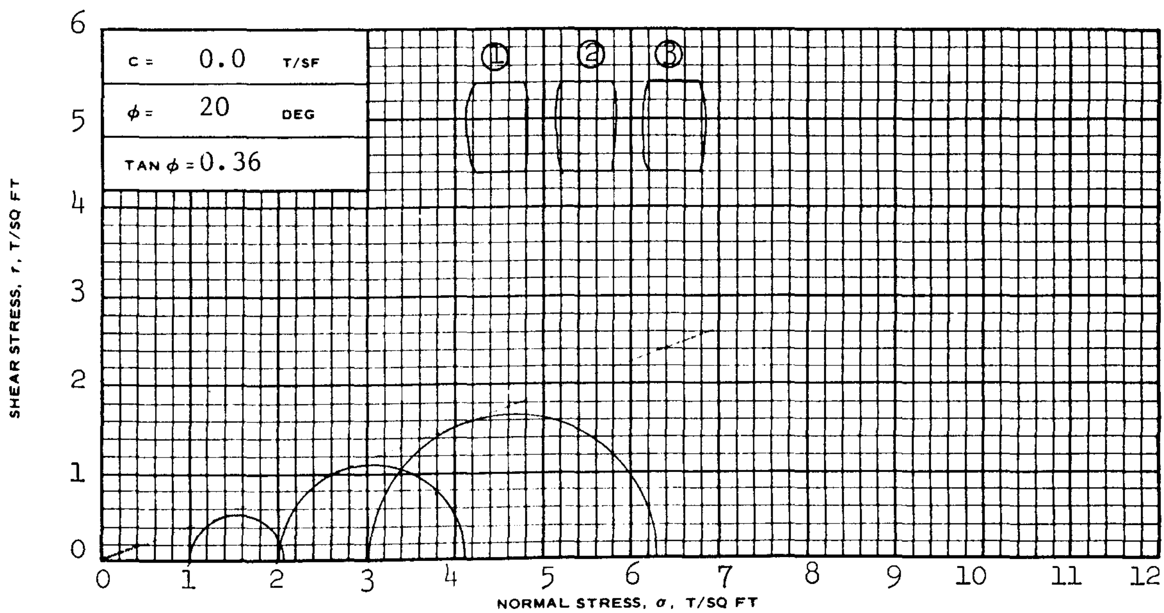
LL	PL	PI	Gs	TYPE OF SPECIMEN	TYPE OF TEST

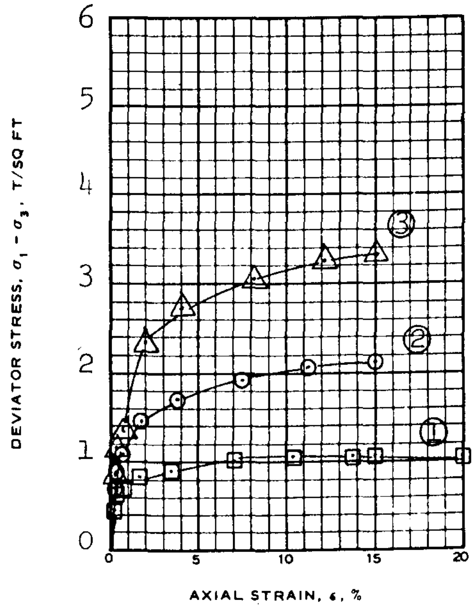
REMARKS:

PROJECT	BRANDON ROAD DAM REHABILITATION
BORING NO.	SAMPLE NO. 2
DEPTH/ELEV	516.0-513.5
LABORATORY USAEWES	DATE 8 Feb. 1977
JMS	TRIAXIAL COMPRESSION TEST REPORT

Sheet 2 of 2.

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110–2–1906)*





SPECIMEN NO.			1	2	3
INITIAL	WATER CONTENT, %	w_o	82.1	95.7	107.5
	DRY DENSITY LB/CU FT	γ_{d_o}	49.1	43.2	39.1
	SATURATION, %	S_o	97.8	94.5	92.3
	VOID RATIO	e_o	1.94	2.34	2.69
BEFORE SHEAR	WATER CONTENT, %	w_c	69.2	65.7	59.3
	DRY DENSITY LB/CU FT	γ_{d_c}	56.0	62.7	72.3
	SATURATION, %	S_c	100+	100+	100+
	VOID RATIO	e_c	1.57	1.30	0.995
	FINAL BACK PRESSURE, T/SQ FT	u_o	2.88	2.88	2.88
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.06	2.13	3.30
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	495	652	600
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$			
INITIAL DIAMETER, IN.		D_o	1.40	1.42	1.41
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00

CONTROLLED- strain TEST

DESCRIPTION OF SPECIMENS ORGANIC SILT(OH), dark gray; fibrous organic matter

LL 127	PL 58	PI 69	Gs 2.31	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST R

REMARKS: See attached sheet for effective values.

PROJECT BRANDON ROAD DAM REHABILITATION

BORING NO. | SAMPLE NO. 4

DEPTH/ELEV 510.0-507.9

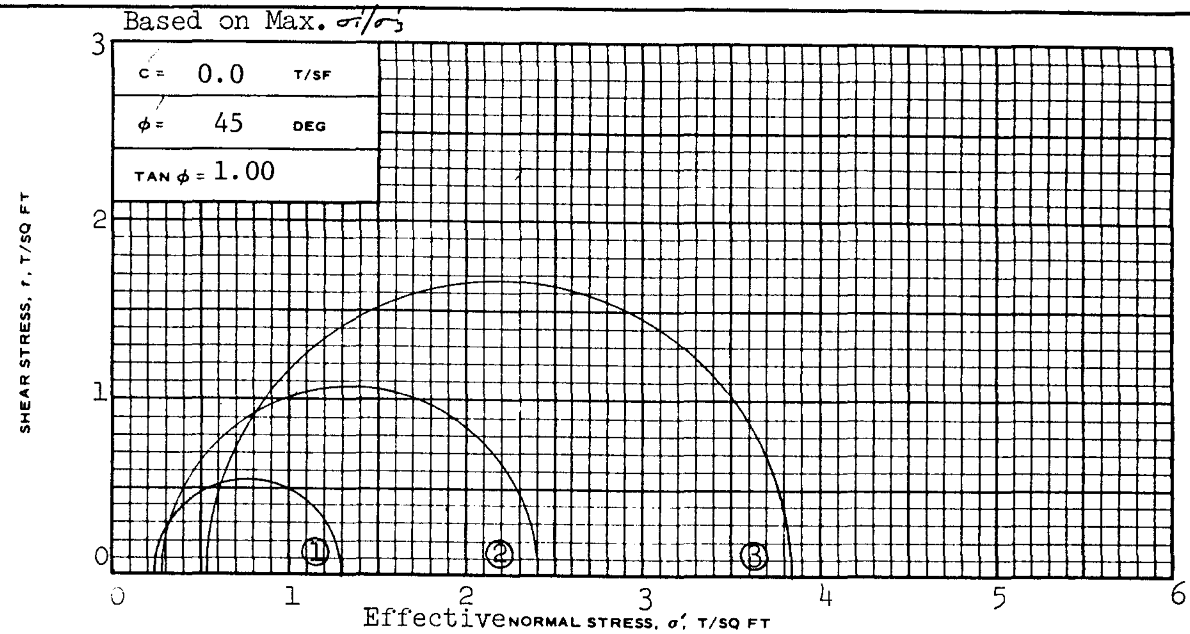
LABORATORY USAEWES | DATE 1 Feb. 1977

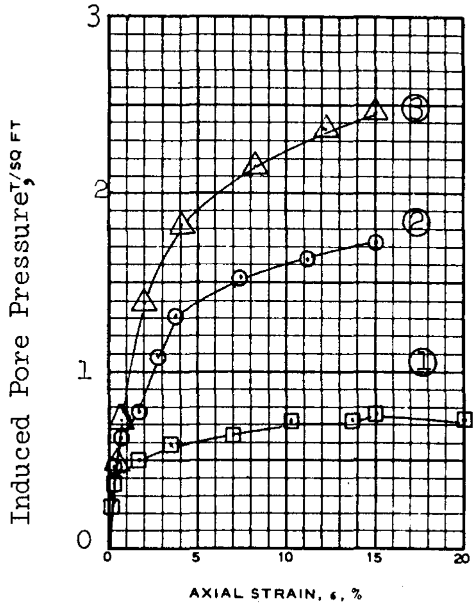
Sheet 1 of 2.

JMS

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.						
INITIAL	WATER CONTENT, %	w_o				
	DRY DENSITY LB/CU FT	γd_o				
	SATURATION, %	S_o				
	VOID RATIO	e_o				
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3				
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$				
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f				
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o				
INITIAL HEIGHT, IN.		H_o				

CONTROLLED- TEST

DESCRIPTION OF SPECIMENS

LL	PL	PI	Gs	TYPE OF SPECIMEN	TYPE OF TEST

REMARKS:

PROJECT BRANDON ROAD DAM REHABILITATION

BORING NO. SAMPLE NO. 4

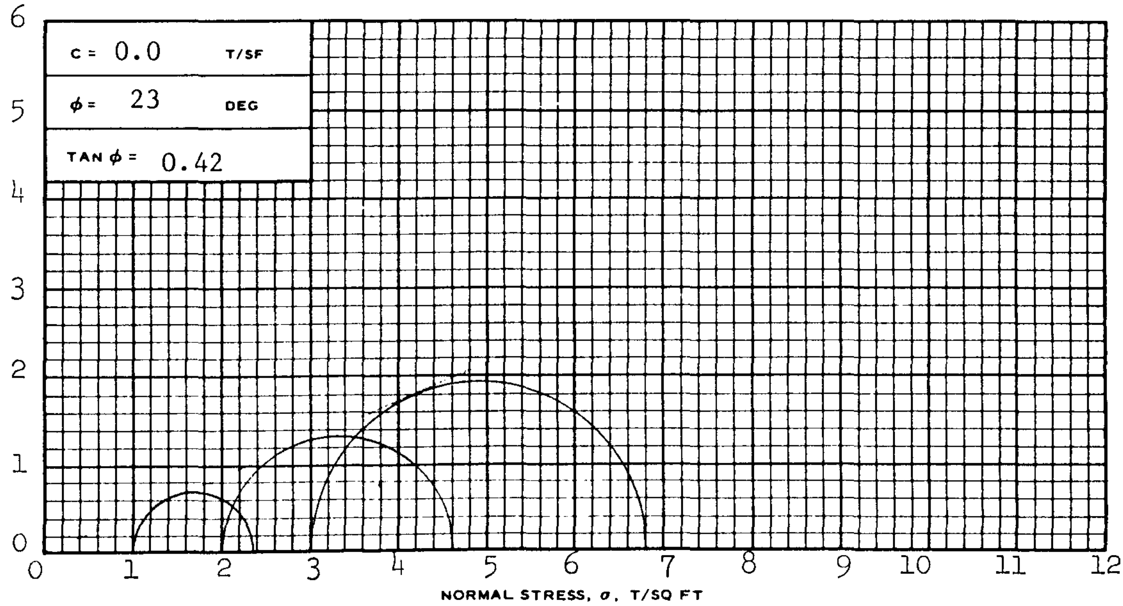
DEPTH/ELEV 510.0-507.9

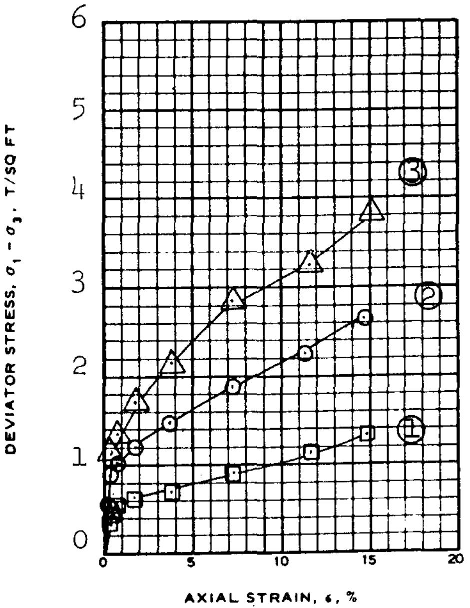
LABORATORY USAEWES DATE 1 Feb 1977

Sheet 2 of 2.

JMS **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.			1	2	3	
Before	WATER CONTENT, %	w_o	96.0	69.2	63.0	
	DRY DENSITY LB/CU FT	γd_o	45.5	56.6	60.1	
	SATURATION, %	S_o	100	100	100	
	VOID RATIO	e_o	2.33	1.68	1.52	
After Shear	WATER CONTENT, %	w_c	81.1	55.4	52.7	
	DRY DENSITY LB/CU FT	γd_c	51.0	64.7	66.5	
	SATURATION, %	S_c	100	100	100	
	VOID RATIO	e_c	1.97	1.34	1.28	
	FINAL BACK PRESSURE, T/SQ FT	u_o	2.88	2.88	2.88	
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.34	2.62	3.81	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$ MIN		t_f	634	632	634	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.42	1.39	1.39	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

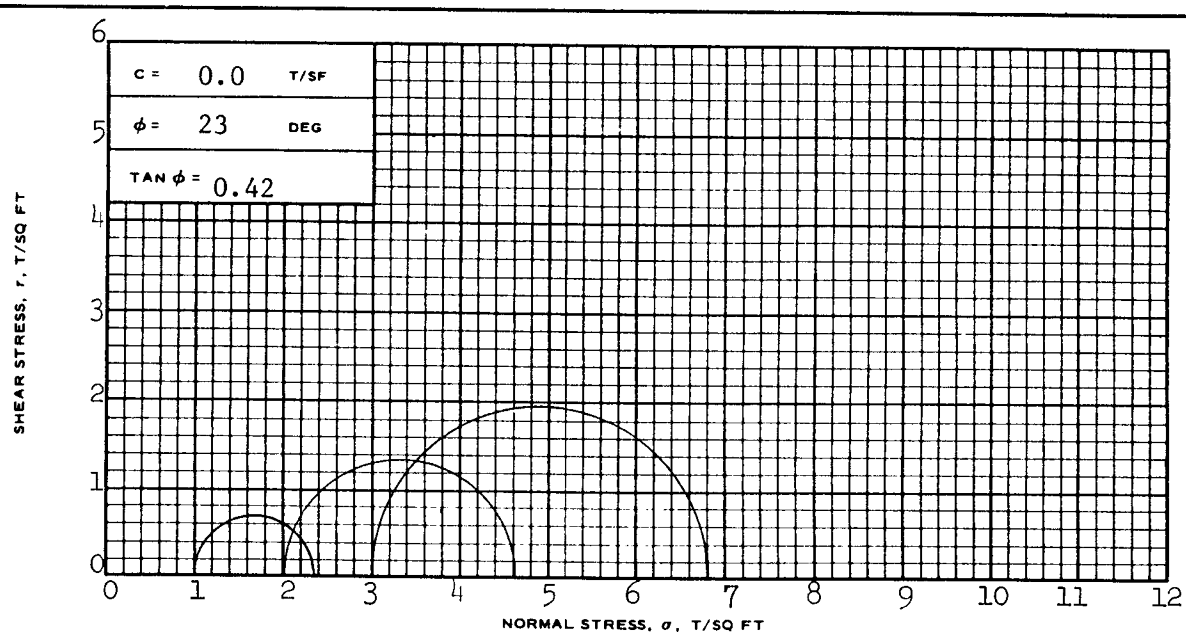
CONTROLLED- strain TEST

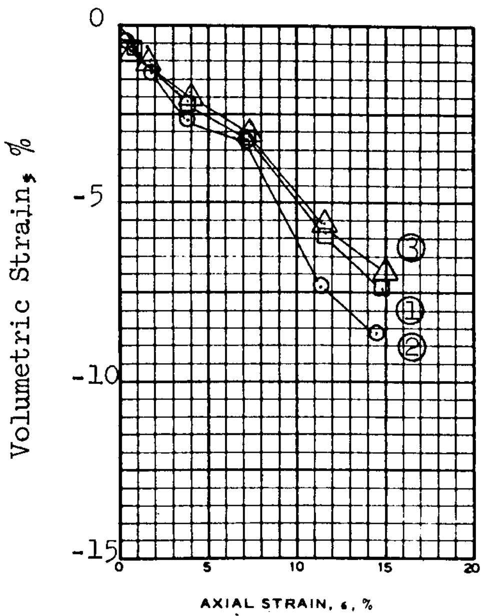
DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH), black;oily odor

LL 96	PL 39	PI 57	Gs 2.43	TYPE OF SPECIMEN UNDIST.	TYPE OF TEST S triaxial

REMARKS:	PROJECT BRANDON ROAD DAM REHABILITATION
	BORING NO. — SAMPLE NO. 1
	DEPTH/ELEV 519.0-516.5
	LABORATORY USAEWES — DATE 11 Feb. 1977
Sheet 1 fo 2.	JMS **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.						
INITIAL	WATER CONTENT, %	w_o				
	DRY DENSITY LB/CU FT	γd_o				
	SATURATION, %	S_o				
	VOID RATIO	e_o				
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3				
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$				
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f				
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o				
INITIAL HEIGHT, IN.		H_o				

CONTROLLED- TEST

DESCRIPTION OF SPECIMENS

LL	PL	PI	Gs	TYPE OF SPECIMEN	TYPE OF TEST

REMARKS:

PROJECT BRANDON ROAD DAM REHABILITATION

BORING NO. SAMPLE NO. 1

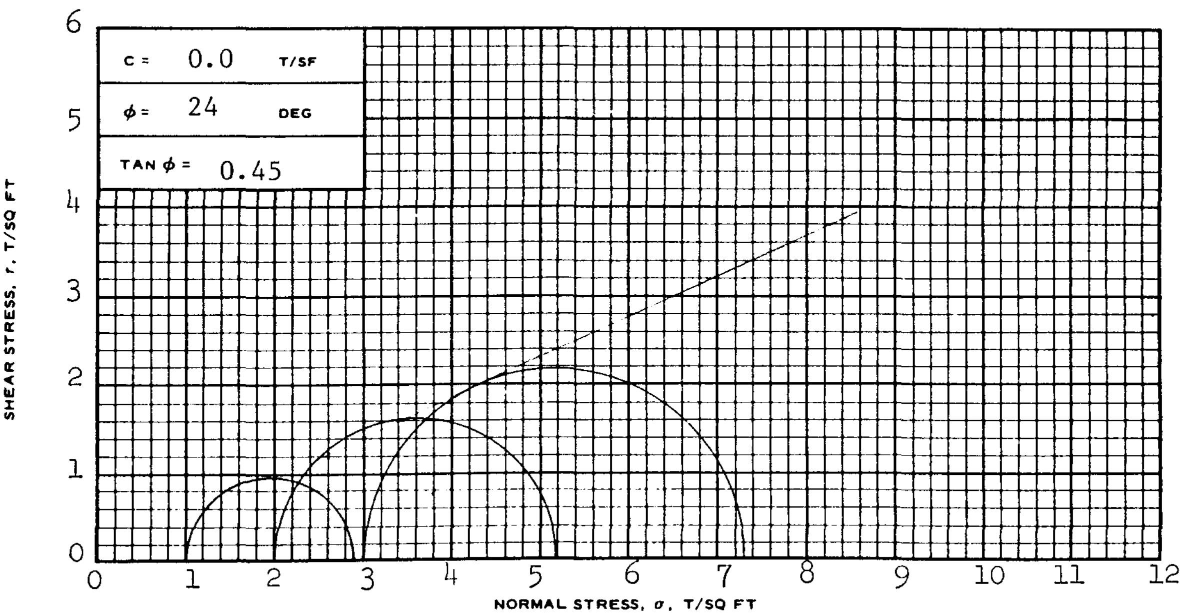
DEPTH/ELEV 519.0-516.5

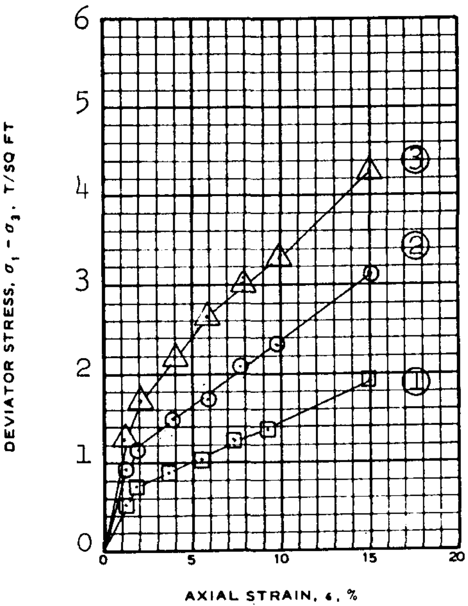
LABORATORY USAEWES DATE 11 Feb. 1977

Sheet 2 of 2.

JMS **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110–2–1906)*





SPECIMEN NO.			1	2	3	
Before	WATER CONTENT, %	w_o	73.9	69.2	65.8	
	DRY DENSITY LB/CU FT	γd_o	55.1	57.6	59.4	
	SATURATION, %	S_o	100	100	100	
	VOID RATIO	e_o	1.89	1.76	1.68	
After Shear	WATER CONTENT, %	w_c	59.7	57.0	56.4	
	DRY DENSITY LB/CU FT	γd_c	63.1	64.9	65.2	
	SATURATION, %	S_c	100	100	100	
	VOID RATIO	e_c	1.52	1.45	1.44	
	FINAL BACK PRESSURE, T/SQ FT	u_o	2.88	2.88	2.88	
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	1.89	3.18	4.32	
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	675	636	622	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o	1.40	1.40	1.40	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED- strain TEST

DESCRIPTION OF SPECIMENS ORGANIC SILT(OH), gray;pockets of decayed fibrous organic matter

LL 112	PL 46	PI 66	Gs 2.55	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST S Triaxial

REMARKS:

PROJECT BRANDON ROAD DAM REHABILITATION

BORING NO. | SAMPLE NO. 2

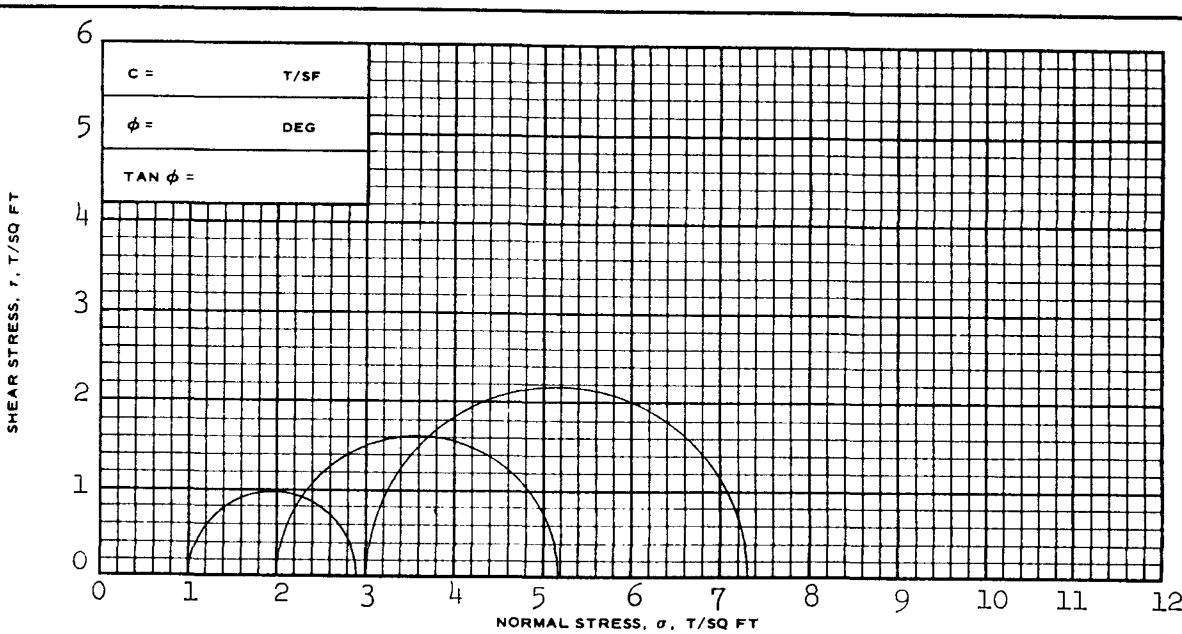
DEPTH/ELEV 516.0-513.5

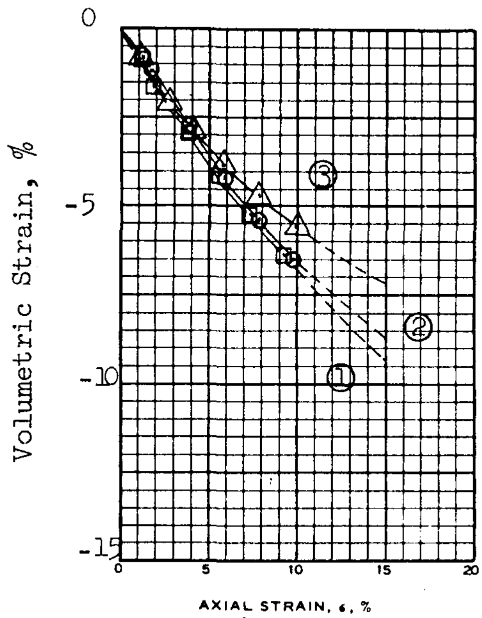
LABORATORY USAEWES | DATE 15FEB77

Sheet 1 of 2.

JMS **TRIAXIAL COMPRESSION TEST REPORT**

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*





SPECIMEN NO.						
INITIAL	WATER CONTENT, %	w_o				
	DRY DENSITY LB/ CU FT	γd_o				
	SATURATION, %	S_o				
	VOID RATIO	e_o				
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γd_c				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3				
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$				
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f				
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o				
INITIAL HEIGHT, IN.		H_o				

CONTROLLED- TEST

DESCRIPTION OF SPECIMENS

LL	PL	PI	Gs	TYPE OF SPECIMEN	TYPE OF TEST
REMARKS:				PROJECT BRANDON ROAD DAM REHABILITATION	
				BORING NO.	SAMPLE NO. 2
				DEPTH/ELEV 516.0-513.5	
				LABORATORY USAEWES	DATE 15FEB77
Sheet 2 of 2.				JMS **TRIAXIAL COMPRESSION TEST REPORT**	

ENG FORM NO. 2089
REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*

APPENDIX A: ABBREVIATIONS

Dol - Dolomite
Sh - Shale
Ch - Chert
Cl - Clay
Chy - Cherty
Sty - Stylolitic Bed
Interb - Interbedded
Sf - Soft
Inc - Inclusion
Lyr - Layer
Nod - Nodule
W/ - With
V - Very
Vert - Vertical
Slg - Slightly
Mod - Moderately
Fi - Fine
Bl - Blue
Br - Brown
Gry - Gray
Grn - Green
Drk - Dark
Fr - Fracture
Ptg - Parting
Jt - Joint
SB - Structural Break
BP - Bedding Plane
Prob MZ - Probably Missing Zone
FA - Fine Aggregate
CA - Coarse Aggregate
Nat - Natural
Conc - Concrete
Pc - Piece
Const - Construction
Lt - Light
Gr - Grain

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Stowe, Richard L
Concrete and rock tests, rehabilitation work, Brandon Road Dam, Illinois Waterway, Chicago District / by Richard L. Stowe. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1978.
46, [53] p., 42 leaves of plates : ill. ; 27 cm. (Miscellaneous paper - U. S. Army Engineer Waterways Experiment Station ; C-78-4)
Prepared for U. S. Army Engineer District, Chicago, Chicago, Ill.
References: p. 46.

1. Brandon Road Dam. 2. Concrete tests. 3. Dam foundations. 4. Dam stability. 5. Field tests. 6. Grouting. 7. Illinois Waterway. 8. Rock tests (Laboratory). I. United States. Army. Corps of Engineers. Chicago District. II. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper ; C-78-4.
TA7.W34m no.C-78-4