No. FHWA-RD-78-140

DESIGN AND CONSTRUCTION OF COMPACTED SHALE EMBANKMENTS

Vol. 4 Field and Laboratory Investigations, Phase III

October 1978

Interim Report

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Prepared for

FEDERAL HIGHWAY ADMINISTRATION
Offices of Research & Development
Washington, D.C. 20590

FOREWORD

This report presents the results of the field and laboratory investigation of shales at six highway embankment sites in five eastern States. The report will be of interest to other researchers in the field and to engineers from the States where the selected embankments are located.

The technical guidance resulting from this report and three previous volumes is contained in the fifth and final volume of this series. This five-volume set presents the results of FCP Project 4D study, "Development of Methodology for Design and Construction of Compacted Shale Embankments." The program was conducted by the U. S. Army Engineer Waterways Experiment Station for FHWA under Purchase Order No. 4-1-0196 during the period July 1, 1974, to January 30, 1979.

Acknowledgement is given to the following advisory group members who provided consultation to the program: Mr. J. Armstrong (Montana), Mr. R. Bashore (Ohio), Mr. H. Mathis (Kentucky), Mr. G. Meaders (Virginia), Mr. R. Prysock (California), Mr. D. Royster (Tennessee), Mr. W. Sisiliano (Indiana), Mr. B. Thompson (West Virginia), Dr. J. K. Mitchell (University of California), and Dr. L. E. Wood (Purdue University).

In contrast to the other volumes of this series, this report is receiving limited distribution to the advisory group members and other shale researchers.

Charles F. Scheffey

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

Technical Report Documentation Page

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15. Supplementary Notes

FHWA Contract Manager--Mr. A. F. DiMillio (HRS-21)

16. Abstract This fourth report covers Phase III field sampling and pressuremeter tests at six shale embankments and laboratory tests on undisturbed embankment samples and on unweathered samples of parent shales used in the embankments. The six embankments, in Tennessee, West Virginia, Kentucky, Ohio (two), and Indiana were selected to cover a range in age and performance for shales placed as rockfill at three sites and as soilfill at three sites. Special K_oCU triaxial compression and density tests on 5-in.-diameter embankment samples provided data on in situ density, modulus, and shear strength. Pressuremeter tests gave supplemental modulus and strength data.

Embankment samples showed shale deterioration to be minor, except at the Indiana embankment, compared with softening of random mixtures of soil, shale, and rock placed during construction at the other sites. Predicted settlement from modulus data was smaller than actual settlements. Undrained strength of saturated laboratory compacted shales correlated with in situ strength.

Unweathered parent shales were used for jar soaking and slake-durability index tests, point load tests, compaction tests, and soaked compression tests. Point load index appears promising for field identification of rocklike shales if correlations are made during design studies. Compaction tests on modeled gradations can be used for coarse graded shales. Soaked compression of minus 3/4-in. compacted shale correlated with slake-durability index and can be used to estimate long-term settlement.

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PREFACE

The study of the methodology for design and construction of compacted shale embankments is a 4-year investigation funded by the U. S. Department of Transportation, Federal Highway Administration, under Intra-Goverment Purchase Order No. 4-1-0196, Work Unit No. FCP 34D5-012.

The work was initiated during June 1974 by the Geotechnical Laboratory (GL) of the U. S. Army Engineer Waterways Experiment Station (WES). Mr. William E. Strohm, Jr., Research Group, Engineering Geology and Rock Mechanics Division (EGRMD), GL, was the principal investigator during the period of this report. The work reported herein was performed under the supervision of Mr. Strohm and by Mr. George H. Bragg, Jr., formerly of EGRMD, Messrs. Mark A. Vispi, George L. Regan, Henry E. McGee, and James L. Drake, Explorations Branch, EGRMD, and Messrs. Robert T. Donaghe, Richard W. Peterson, Willie J. Hughs, and Ms. Catherine M. Ryan, Soil Mechanics Division, GL. Fieldwork was accomplished with the assistance of State Highway District personnel in Tennessee, West Virginia, Kentucky, Ohio, and Indiana. The report was prepared by Mr. Strohm. The investigation was accomplished under the general supervision of Dr. Don C. Banks, Chief, EGRMD, and Mr. James P. Sale, Chief, GL.

Directors of WES during the conduct of this portion of the study and preparation of the report were COL G. H. Hilt, CE, and COL J. L. Cannon, CE. Technical Director was Mr. F. R. Brown.

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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	25.4	metres
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
gallons (U. S. liquid)	3.785412	cubic decimetres
pounds (mass)	0.4535924	kilograms
tons (2000 lb, mass)	907.1847	kilograms
pounds (mass) per cubic foot	16. 01846	kilograms per cubic metre
pounds (force) per square inch	6894.757	pascals
pounds (force) per square foot	47.88026	pascals
tons (force) per square foot	95.76052	kilopascals
degrees (angle)	0.01745329	radians

DESIGN AND CONSTRUCTION OF COMPACTED SHALE EMBANKMENTS; VOL. 4, FIELD AND LABORATORY INVESTIGATIONS, PHASE III

PART I: INTRODUCTION

Background

1. Construction of the modern interstate highway system in much of the United States has required large, high embankments using economically available shale* from adjacent cuts or borrow areas. Settlements of 1 to 3 ft** in many shale embankments have required frequent overlaying to maintain grade. Raising of bridge abutments founded on approach embankments of shale has also been required. In some shale embankments continuing settlements are followed by slope failure and slides, while in others the settlement stops and no further distress occurs. The more severe settlements and slope failures have occurred in the East Central States where the climate is humid. Repair of failures is expensive, amounting to nearly $2 million in one case for three slides where reconstruction was required over a period of 18 months.

2. The underlying cause of excessive settlement and slope failures in highway shale embankments appears to be deterioration or softening of certain shales with time after construction. Some shales are rocklike when excavated, but when placed as rockfill, deteriorate or soften into weak soil. Other shales, often interbedded with limestone or sandstone, break down when excavated; but large-size durable rocks often prevent adequate compaction. The difficulties encountered in using shale in highway embankments are complicated by variations in geology and physical properties of sedimentary rocks, depth of weathering, climate and groundwater conditions, and the weather and construction methods. The main difficulty is determining which shales can be placed as rockfill

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* Shale is used as a general term for weak sedimentary rocks such as claystone, siltstone, mudstone, etc.

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

in thick lifts (2 to 3 ft) and which shales must be placed as soil and compacted in thin lifts (8 to 12 in.).

Scope of Study

3. The need for comprehensive guidance on the use of shales in highway embankments and on procedures for evaluation and treatment of existimg shale embankments led to a 4-year study started in July 1974 and accomplished in three phases. Phase I addressed problem areas, factors causing shale deterioration in embankments, and current practices, while Phase II developed needed information on evaluation and remedial treatment of existing shale embankments. Phases I and II were completed in the first year, and the results were published in two Federal Highway Administration (FHWA) reports:

- a. "Design and Construction of Compacted Shale Embankments, Vol 1, Survey of Problem Areas and Current Practices," Interim Report, August 1975, Report No. FHWA-RD-75-61.
- b. "Design and Construction of Compacted Shale Embankments, Vol 2, Evaluation and Remedial Treatment of Shale Embankments," Interim Report, August 1975, Report No. FHWA-RD-75-62.

4. Phase III covered the development of sampling and testing procedures for shales, evaluation of potential problems, design criteria, and construction control techniques, with detailed guidance provided in a technical manual. An important part of Phase III was the development of preliminary criteria for anticipating performance of shales in embankments from simple slaking indexes. Lift thickness and service performance information for 92 embankments provided by 15 State highway agencies was correlated with index tests on associated shale samples collected during the first 18 months; and the results, offered for further study and trial at the State level, are contained in "Design and Construction of Compacted Shale Embankments, Vol 3, Slaking Indexes for Design," Interim Report, February 1977, Report No. FHWA-RD-77-1.

5. The scope of Phase III was modified to include sampling and testing of in situ and parent shales at existing embankments of various

ages and construction methods to better define the role of shale deterioration considering embankment distress and the durability of the unweathered parent shales used in the embankments. This information was needed in developing criteria to assess potential problems with shale durability, considering normal design and construction practices and recommendations for special design features, construction methods and compaction procedures, and construction control techniques to ensure adequate long-term performance.

Objectives of Phase III Investigations

6. Undisturbed samples from selected shale embankments provided information on the character of embankment materials, type, degree of shale deterioration, in situ density, water content, compressibility, and shear strength. Unweathered parent shale samples were used to determine durability indexes for comparison with in situ conditions and service performance. The parent shales were also used to develop compaction test procedures for gradations having more than 35 percent of plus 3/4-in. sizes and a simple test on compacted samples to assess expected compressibility of saturated shales.

7. Information for the embankments on excavation and placement methods, placement sequence, mixing of shales with soil and rock, lift thickness, and type and amount of compaction was used to evaluate the influence of these variables on service performance. Additional information was obtained during the fieldwork and from current Joint Highway Research Project (JHRP) studies on shale sampling requirements, sample preparation and testing procedures, shale durability classification criteria, compaction equipment and procedures, and applicability of test strips for shales.

8. The resulting guidance is contained in "Design and Construction of Compacted Shale Embankments, Vol 5, Technical Guidelines," Report No. FHWA-RD-78-141, December 1978.

Scope of Report

9. This report presents the results of the Phase III field and

laboratory investigation of shales at six embankment sites selected from 14 to cover a range of ages, various degrees of distress, and different types of construction. Fieldwork and results obtained are described first and followed by laboratory tests on embankment shale samples, a summary of embankment properties, and laboratory tests on parent shales.

PART II: SAMPLING AND TESTING OF SELECTED SHALE EMBANKMENTS

Embankment Selection

10. Selection of the shale embankments for field sampling and testing was based on the availability of information that could be used to determine the following:

- *a*. Specific location and availability of the parent shale used in the embankment (as from adjacent cut).
- *b*. Location within the embankment of the different shales still exposed in the original source (adjacent cut).
- *c*. Construction specifications and/or special provisions and dates of construction.
- *d*. Methods and sequence of excavation and placement.
- *e*. Lift thickness, type of compaction equipment, and amount of compaction.
- *f*. Results of any compaction control tests.
- *g*. Field observations and performance records after construction.

11. A total of 14 embankments in nine States (Virginia, West Virginia, Tennessee, Kentucky, Ohio, Indiana, South Dakota, Montana, and Utah) were identified from State highway agency reports and information letters, which included embankments constructed with shale as rockfill in thick lifts or shale as soil in thin lifts, with ages ranging from 2 to 18 years and with various degrees of distress. The appropriate State highway agencies and project offices were visited. Construction records were reviewed, and available project engineers and inspectors were interviewed. The accumulated information was summarized in separate memorandum reports for each location (available from the U. S. Army Engineer Waterways Experiment Station (WES) study files). The final selection of the six embankments listed in Table 1 was made in consultation with the FHWA Contract Manager. These embankments represent the more severe problem areas with shale formation characteristics that cause the greatest design and construction problems.

Table 1. Summary of Information on Embankments Selected for Field Sampling and Testing

State	Embankment Location	Date of Construction	1976 Age yr	Max. Height at ℄ ft	Original Outer Slopes	Construction Records	Shale Placement and Compaction Procedures	Embankment Shale Source	Correlation w/location in Embankment	Formation and Type of Shale	Type and Magnitude of Problem	Contributing Factors	Previous Field and Lab Investigation	Remedial Treatment and Effect
Tennessee	I-75, Campbell County, N of Cove Lake, sta. 950 to 966	1965 and 1966	11	100	1-1/2:1	Plan and sections	Placed as rock-fill; sheepsfoot, 18-in. lifts	Three cuts, sta. 820 to 950	Deeper in cuts for sta. 950 to 960	Hance - weak shale, gray shale, siltstone, sandstone, under clay and coal	Settlement and cracks first 5 yr; cont. settlement (~2-ft total)	Surface infiltration	Detailed inves by Law Engr. 1972	Overlying to maintain grade
West Virginia	U. S. 460, Mercer County, E of Princeton, via Kellysville, sta. 551+00 to 555+50	1970	6	90	2:1	Excavate and fill sta., equip, w.c., in-place densities, % comp.	Placed as soil and rockfill; vib. and pad foot rollers; 8 in. for soft shale, 24 in. for hard shale	Two adjacent cuts	sta. versus date and height of fill	Hinton - near vert. shale, siltstone, occ. sandstone and limestone	Settlement to 18 in. from end const. to mid-1975 then stopped	Surface infiltration and high groundwater	Detailed by WV incl well logging of in-place density and w.c.	Overlying to maintain grade
Kentucky	I-75, Boone County, MP 178.8, Florence, sta. 670+50 to 682+00	1959 and 1960	16	45	2:1	Excavate and fill sta.	Placed as soil and rockfill; sheepsfoot; thin lifts for soft shale, thick for hard shale and rock	Borrow east of sta. 674 to 680	Uniform deposits	Bull Fork interbedded shale and limestone (50%)	Slope failure in shoulder; 1968 repair by pile wall, failed 1973; slopes flattened 1974	Surface infiltration	Detailed by Ky 1974	Slopes flattened to 4:1 in 1974, stable to date
Ohio	I-74, Hamilton County, east bridge approach emb. at SR 128; Miamitown, sta. 424+12 to 440+00	1961	15	55	3:1	Details from Ohio DOT ltrs	Placed as soil; 4 coverages with tamping roller plus 2 coverages with 50-ton roller on 8-in. lifts	Two adjacent cuts west, sta. 386-401 and sta. 403-419	Uniform deposits	Fairview and Kope, shale with thin-bedded limestone (20%)	None; only 3-in. settlement. Example of good performance	N/A	Detailed settlement vs depth	N/A
Ohio	I-74, Hamilton County, west bridge approach emb. at Wesselman Road, east of Miamitown, sta. 480+00 to 492+50	1962	14	55	2:1	Details from Ohio DOT ltrs	Placed as rock-fill in approximately 3-ft lifts	Adjacent cut, sta. 468 to 480	Uniform deposit	Kope - shale with thin-bedded limestone (20%)	Settlement 12 in. by 1964; 20 in. by 1968; grouted 1968, no further problems	Possible found soil settlement	Borings and test pit after grouting	Cement grouting apparently effective
Indiana	I-74, Harrison County, Mile 165.2, east of Saint Leon, sta. 426+00 to 440+50	1962	14	115	2:1	General info from published reports	Placed as rock-fill up to 4-ft thick lifts	Adjacent cuts to the west	Uniform deposit	Dillsboro - thin-bedded shale (65%) and limestone (35%)	Settlement max. of 2 ft by 1976; sloughing area on north slope	Surface infiltration	Detailed inves by D'Appolonia Engr for ISHC	Recommended gravel toe berm north slope

Field Sampling and Testing

12. During March and April 1976, at each of the six embankments, undisturbed samples were obtained from one boring; and pressuremeter tests were made in a companion boring. Work was accomplished by the WES. Chunk samples of unweathered shales used in the embankments were obtained from adjacent cuts with the aid of the highway districts. Traffic safety control for borings on the shoulders of embankments was provided by highway districts in Tennessee and West Virginia. Borings at the Kentucky, Ohio, and Indiana sites were made in the median. The procedures used and pertinent information on each embankment are briefly described in this section.

Undisturbed sampling

13. Five-inch undisturbed samples were obtained using a special 5- by 6-1/4-in.-double tube core barrel sampler for soil and rock with adjustable cutting shoes and a carboloy bit (Vol 2). The inner barrel was a standard 5-in. ID thin-wall steel sample tube. Compressed air was used to remove cuttings. Sampling was continuous except for clean-out when a lost sample could not be recovered. Maximum sample length was 30 in. All samples were pushed from the sample tube (inner barrel), photographed, and logged. Selected lengths of samples (generally 12 in. long) were preserved in wax in a cardboard tube (with metal base). All borings were backfilled after all sampling and testing was completed.

14. At the last location, after completion of work at three locations, accumulated undisturbed samples were packed in sawdust in a stakebody truck, transported to the WES laboratory, and stored in a humid room.

Pressuremeter tests

15. The pressuremeter boring, located approximately 20 ft from the undisturbed sample boring, was advanced to one-half the total depth using a 2-15/16-in.-diameter tricon-roller bit and compressed air. Pressuremeter tests, using the Menard apparatus shown in Figure 1, were made at 5-ft-depth intervals downhole and then at intermediate

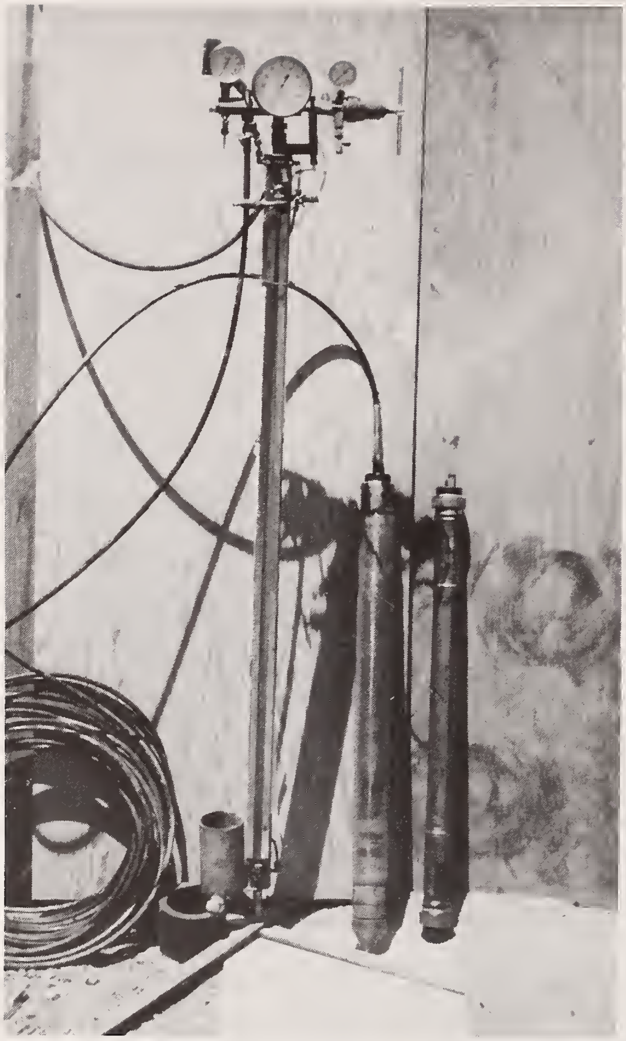


Figure 1. Menard pressuremeter apparatus
(NX size probe with urethane sheath and
bare probe on right; compressed nitrogen
gas source not shown)

intervals uphole. The boring was completed to the base of the embankment, and the pressuremeter tests were completed using the same pattern as in the upper half of the boring. Test procedures and examples of test results are given in Appendix A and are summarized in Part III.

Parent shale samples

16. Chunk samples of the unweathered, parent shale used in the embankment were obtained near the base of the source cut with the assistance of highway district personnel using a backhoe. Selected unweathered chunks were placed and sealed in 55-gal drums. A minimum of one drum of each major shale type was obtained. The drums were stored in highway district yards and transported by truck to the WES.

Description of Selected Embankments and Results of Field Investigations

I-75 Tennessee

17. The selected I-75 TN embankment, located four miles north of Cove Lake, was constructed in 1965. It is 100 ft high with side slopes of 1.5:1 (2 vertical on 3 horizontal) extending from stations 950 to 985 (north to south) across a relatively wide valley. The embankment crosses the valley on an undulating saddle between the flanks of Pine Mountain to the north (station 950) and Little Cumberland Mountain to the south (station 985). A general view looking toward Pine Mountain is shown in Figure 2. The shale section between stations 950 and 966 was chosen for sampling and testing. This portion of the embankment was constructed with weathered to hard shale from cuts in the Hance formation between stations 820 and 950. Shale ripped from the cuts was placed in 18-in. loose lifts and compacted with a sheepsfoot roller pulled by a dozer. The remaining portion of the embankment was constructed with sandstone blasted from cuts in the Lee formation ahead of station 985. The sandstone was placed in 3-ft lifts and compacted by hauling and spreading equipment.

18. Performance. During construction, cracks occurred in the shale fill and were attributed to settlement. Shortly after being

Figure 2. Tennessee I-75 embankment looking north, sta. 985 to 950

opened to traffic, longitudinal cracking developed in both the northbound and southbound lanes of the shale section of the embankment. Nonuniform settlements also occurred and required 6 in. of overlay on the northbound lane and 8 in. of overlay on the southbound lane. No further significant distress occurred. However, settlements continued, requiring periodic overlaying. The total settlement is believed to be about 2 ft. The sandstone section of the embankment showed no signs of distress.

19. Previous investigation. The I-75 Tennessee embankment was one of several exhibiting distress that were investigated in 1972 by Law Engineering for the Tennessee Department of Transportation. The shale fill portion, stations 950 to 966, showed cracking in both traffic lanes. Minor amounts of seepage, attributed to surface infiltration, were observed in the lower third of the north slope. Samples from embankment borings indicated gray weathered shale, siltstone of all sizes, and fragments of shale embedded in tan to gray silty clay. Water levels were at or below the original ground surface at the base of the embankment. Dry densities ranged from 125 to 130 pcf, water contents ranged from 8 to 15 percent, and the calculated degree of saturation ranged from 60 to 80 percent. Pressuremeter tests indicated undrained strengths of 700 to 2000 psf in the upper third and 1700 to 3100 psf in the lower two-thirds of the fill. Unconsolidated-undrained triaxial shear tests indicated that the strengths at in situ water contents ranged from 1100 to 8500 psf and a saturated shear strength of 300 psf. Stability analyses indicated factors of safety of 0.9 to 1.2. The irregular settlements were attributed to nonuniform compression of the fill which occurred as shale chunks were broken down by infiltrating water.

20. WES field sampling and testing. The undistributed sample boring was located at station 960+82 on the northbound lane shoulder (Figure 2) within 18 ft of a 1972 boring by Law Engineering at station 961. The location was chosen near the lowest grade elevation for this section where maximum infiltration was possible. Undisturbed samples, Figures 3 and 4, were loose to firm and contained shale and

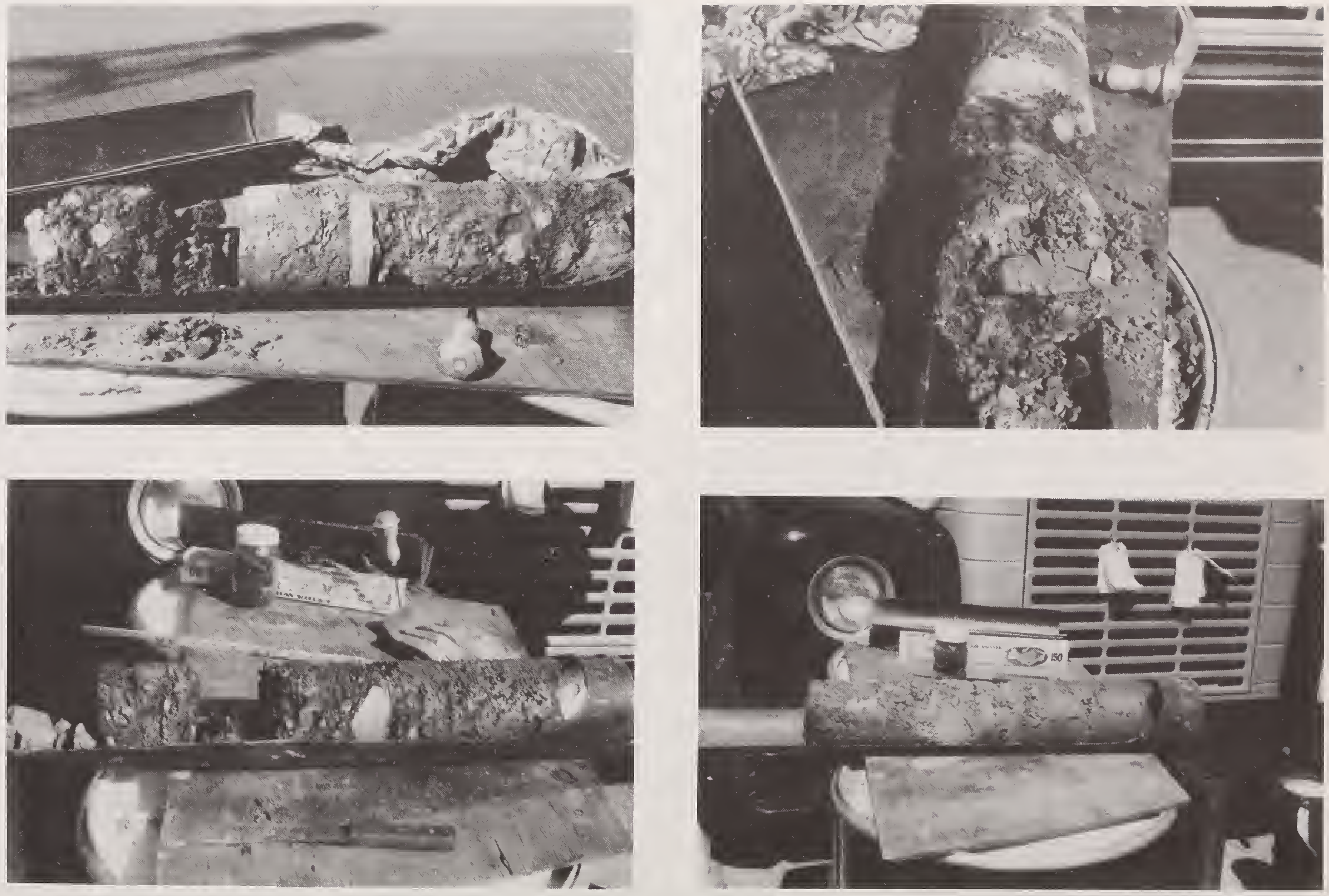


Figure 3. Character of undisturbed samples saved from Tennessee I-75 shale embankment boring

Figure 4. Character of undisturbed wasted samples from Tennessee I-75 shale embankment boring

siltstone chunks, shale fragments imbedded in gray to tan silty clay, some large pieces of sandstone (Figure 4), and occasional layers of tan to orange gravelly, silty clay. Water seepage into the borehole was encountered at 28 ft and at 78 ft after drilling through 2 ft of sandstone chunks. The character of shale pieces ranged from hard to soft and friable with iron oxide stains and with little evidence of severe deterioration. Five loose samples fell apart when pushed from the sample tube or during placement in cardboard cylinders prior to encasing in wax. Where weathered shale foundation material was encountered, a total of 34 samples were obtained to a depth of 90 ft. Twenty-nine samples were preserved for laboratory examination and testing. The overall character of the samples indicated a heterogeneous mixture of soil from weathered overburden, shale, and occasional sandstone boulders. It appeared that considerable soil and finer grained shale pieces were mixed with harder shale and siltstone chunks and with sandstone boulders during construction. It was apparent that the loosely compacted material settled and increased in water content from infiltrating surface water.

21. Two pressuremeter test borings were located at stations 960+33 and 960+59, about 20 ft from the undisturbed boring. Because of difficulties encountered in using the equipment for the first time and squeezing-in of hard chunks, only five valid tests were completed to a depth of 60 ft. Modulus values ranged from 600 to 6000 psi, and in situ undrained shear strengths ranged from about 1000 to 6000 psf. Test results are discussed in Part III.

22. Parent shale samples were obtained from two cuts south of the embankment between stations 820 and 950. Highway department personnel indicated that overburden soil, weathered shale, and siltstone from the upper portions of the cuts had been placed in low areas to build up the grade northward toward the large shale embankment. Shale and siltstone from the lower sections of these cuts were then hauled to the shale embankment section. Unweathered chunks of the shale in the cut near the ditch were broken out using a backhoe operated by State highway district personnel (Figure 5). This shale was dark gray and hard, and corresponded to hard chunks of shale found in the embankment samples. Two

Figure 5. Excavating unweathered parent shale chunks, Tennessee I-75

barrels of shale chunks were obtained from each cut. The barrel lids were sealed with a rubber gasket and clamping ring.

U. S. 460, West Virginia

23. The selected U. S. 460 embankment shown in Figure 6 is one of six along a downhill grade about 12 miles east of Princeton, West Virginia. These embankments experienced 12 to 18 in. of settlement after construction in 1971. The embankment between stations 551 to 555+50 was chosen because fill materials could be related to locations in the adjacent cuts from information on station, offset, and date in construction records. Fill for the other embankments came from several different cuts in the near vertical beds of shales, siltstones, and some sandstone within the Hinton formation. Because of the complex sequence of excavation and placement, the source of different shales within any of the other embankments could not be adequately defined. The selected embankment is 90 ft high and symmetrical with 2:1 (1 vertical on 2 horizontal) side slopes.

24. Select rockfill foundation drainage blankets at least 4 ft thick were specified on the downhill side of through embankments and on benches cut into unweathered rock on steeper slopes (specified by stations) for sidehill embankments. During construction, required quantities of sandstone and limestone for select rockfill were inadequate, and hard shale was used at the higher foundation elevations beneath the embankments. Embankment construction specifications required hard shale to be compacted in 2-ft lifts and soft shale, random material, and soil to be compacted in 8-in. lifts. Lifts were spread using D-8 and D-9 dozers. Two types of compaction rollers were used, a vibratory steel drum and a static "pad foot." Field density tests were used for compaction control when the amount of plus 3/4-in. material was less than 35 percent of the total sample.

25. Performance. Shortly after concrete paving was completed in early 1971, settlement depressions were evident in four of the embankments including the one from stations 551 to 555+50. The concrete was replaced with asphalt pavement, and the section was opened in the fall of 1971. Pavement settlement at the center of the

Figure 6. U. S. 460 West Virginia embankment

embankments required overlays in 1972 and 1973. Settlements continued at a rate of about 6 in. per year until mid-1975. Total settlement for the highest embankment (200 ft) was approximately 25 in. and was attributed to the breaking down of shale and the consolidation of soil layers caused by subsurface seepage from uphill terrain and infiltration of surface water.

26. Investigations. A field investigation by the West Virginia Department of Highways in 1972 included 11 split spoon borings in four of the embankments with slope indicator casing installed in six borings and steel casing installed in five borings. The boring log at station 553+50 indicated brown soft shale, with boulders; blow counts ranged from 29 to 66; water contents ranged from 8 to 18 percent; and liquid limits ranged from 25 to 30, with a plasticity index of 18. The recorded water table was at a depth of 7 ft. Slope indicator data showed small displacements over the height of the embankment, indicating minor lateral bulging. The steel-cased borings were used for moisture-density logging by a commercial well-logging firm. Four sets of moisture-density logs were obtained from June 1973 to May 1974 using a directional probe, which gave inconsistent readings because of spiraling of the cable. A fifth set of readings (June 1974) was made with an omnidirectional (360 deg coverage) probe. The data for station 553+50 is shown in Figure 7. Dry densities ranged from about 103 to 130 pcf and water contents from about 4 to 10 percent. The purpose of the logging was to measure water content and density changes with depth that might be associated with shale deterioration. The results indicated an overall gradual increase in density with time.

27. WES field sampling and testing. The undisturbed sample boring was located on the shoulder (Figure 6) at station 553+75 within 25 ft of the State highway boring at station 553+50. Undisturbed samples (Figure 8) were loose to dense mixtures of clay, gravel-size pieces of gray to black shale and brown siltstone, and occasional large pieces of hard shale, siltstone, sandstone, and limestone. Brown silty clay and siltstone predominated. Shale deterioration (Figure 8, 66.5 to 67.5 ft) was minor compared to the mixture of materials which appeared to have

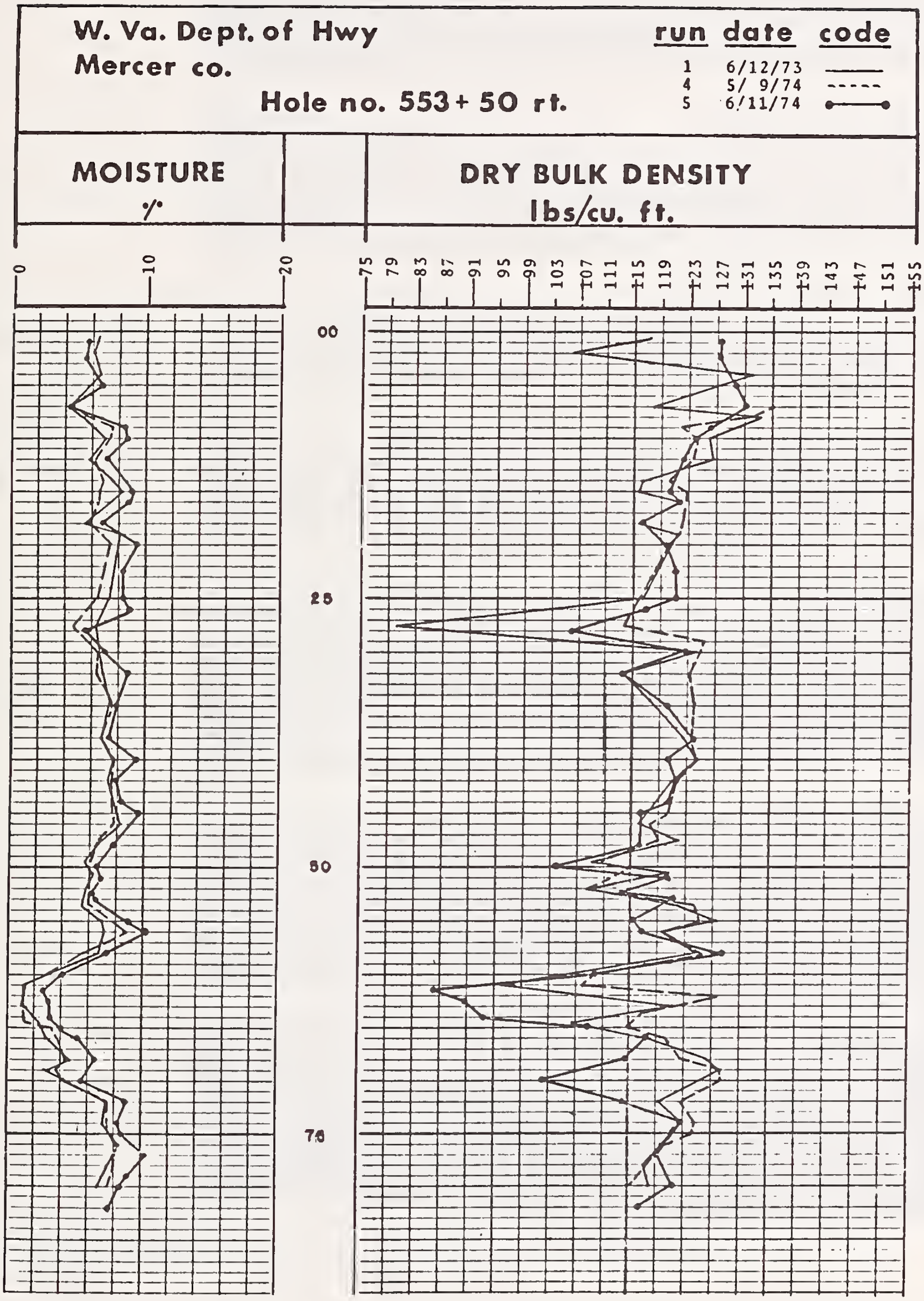
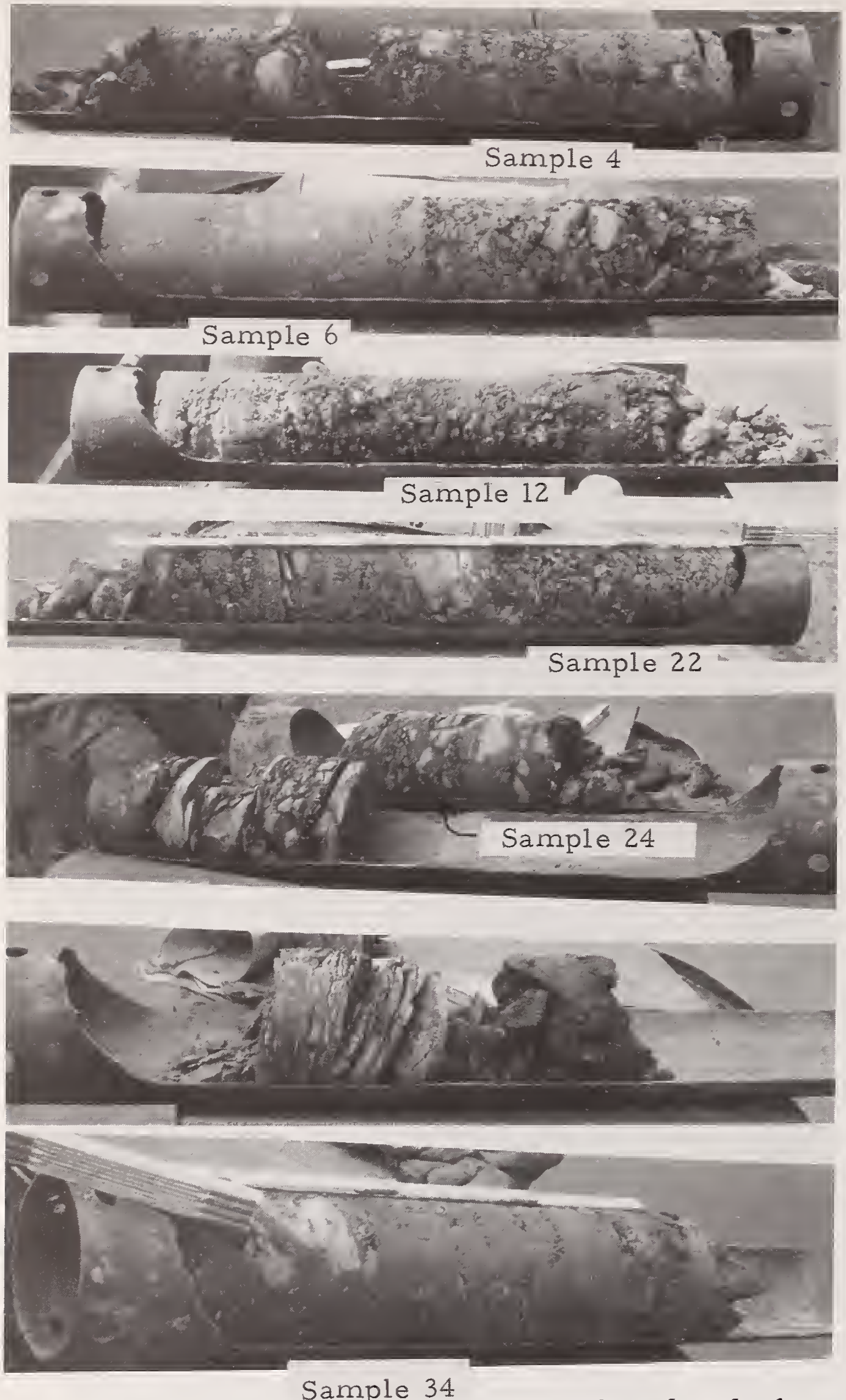


Figure 7. Results of well-logging tests, sta. 553+50, West Virginia, U. S. 460 embankment



Depth, ft
12.5 - 14.8

17.5 - 20.0

32.5 - 35.0

53.5 -56.0

59.6 - 61.6

66.5 - 67.5

81.7 - 83.6

Note: Closed end of sample holder is at bottom of sample.

Figure 8. Character of undisturbed samples, West Virginia, U. S. 460 shale embankment

been placed during construction. Water was encountered in the boring at a depth of 80 ft and bedrock at 84 ft. A total of 36 samples were obtained, and 28 of these were preserved for laboratory examination and testing (see Part III).

28. Pressuremeter tests were performed in a boring at station 553+25 to a depth of 75 ft. A total of 14 tests were considered valid. Other tests did not produce valid data because the diameter of the borehole at the test depth was too large. Modulus values ranged from 1500 to 5000 psi. In situ undrained shear strengths ranged from 1000 to 5000 psf. Test results are summarized in Part III.

29. Parent shale samples were excavated along the ditch line in adjacent cuts. Five barrels of unweathered chunks of shale and siltstone of the same type as observed in undisturbed samples were obtained.

Barrel No.	Location	Material Type
1	557+50 rt	Brown siltstone
2	559+00 rt	Dark brown shale
3	557+00 lt	Dark gray shale
4	556+00 lt	Brown siltstone
5	543+50 lt	Black shale

I-75, Kentucky

30. Several small embankments along I-75 south of Covington experienced shallow slope failures in 2:1 slopes about 6 to 8 years after construction in 1959-1960. The slides were investigated by the Kentucky Bureau of Highways and repaired by reconstruction to 4:1 side slopes. A repaired embankment about 40 ft high at Mile Post 179.9 to 179.7 (stations 670 to 682) shown in Figure 9 was selected for undisturbed sampling and pressuremeter tests. The embankment was constructed using interbedded shale and limestone from an adjacent borrow area and cut in the Bull Fork formation (Ordovician age). This formation contains soft, medium-gray fissile shale (moderately fossiliferous) in layers 1 to 10 in. thick and is interbedded with limestone layers, usually less than 1 ft thick.

31. Fill materials for the embankment at stations 670 to 682 were

Figure 9. General view of I-75, Kentucky shale embankment after reconstruction MP 179.7 to 179.9

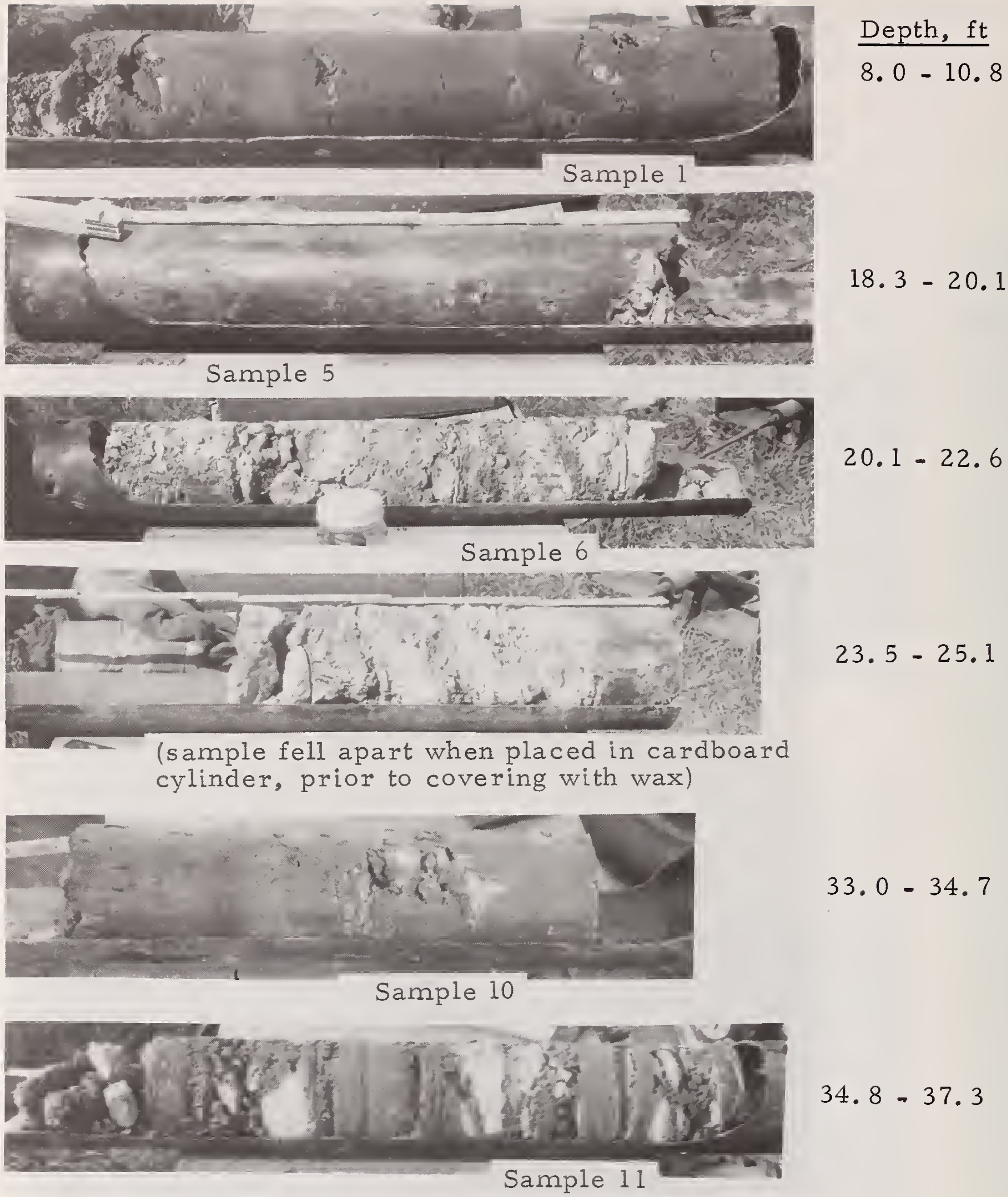
taken from a borrow pit area east of stations 674 to 680. Materials were placed in fairly thin lifts as soil with some rock lifts. Three scrapers, one dozer, and one roller were listed as equipment in the construction diary.

32. Performance. Shallow shoulder slides were repaired in 1968 using wood piling and old guardrail. Intermittent pavement overlaying was required to maintain grade; and in 1973, the guardrail retaining wall slumped away from the shoulder. Quarry stone was used to rebuild the shoulder; and permanent reconstruction to 4:1 slopes was accomplished in 1974.

33. Investigation. The shoulder slides were investigated in early 1974 by the Bureau of Highways. Slope inclinometer data indicated creep movements in the left slope at station 637+50. Triaxial tests on embankment samples from station 674+50 indicated a drained shear strength of 31.5 deg for $\bar{\phi}$ and zero cohesion. Stability analyses of the existing 2:1 slope indicated a factor of safety of 1.37 for a shear surface extending into the foundation ($\bar{\phi}$ = 21.5 deg , $\bar{c}$ = 389 psf). For the right slope, a shear plane was defined at station 677+50, and a back analysis for the 2:1 slope indicated $\bar{\phi}$ = 22 deg with zero cohesion. For a 4:1 slope, the factor of safety was 1.63.

34. WES field sampling and testing. The undisturbed sample boring and pressuremeter boring were located in the median at stations 678+55 and 677+50, respectively. Undisturbed samples (Figure 10) indicated dense, brown clay with limestone fragments in the upper 20 ft and loose to dense mixtures of limestone and shale chunks in the lower 20 ft. Some shale chunks showed deterioration in the form of separations along bedding and softening into friable, clayey shale. A total of 14 undisturbed samples were obtained, and 12 were preserved for laboratory examination and testing. Seven valid pressuremeter tests indicated a modulus of 600 to 3600 psi and in situ undrained shear strengths ranging from 1000 to 5000 psf. Test results are discussed in Part III.

35. Four barrels of weathered, light-brown to gray clay shale interbedded with thin limestone were obtained from the base of a 12-ft



Note: Bottom of sample is at closed end of sample holder.

Figure 10. Character of undisturbed samples, I-75, Kentucky shale embankment at MP 179.7 to 179.9

cut slope at the south end of the embankment. Unweathered shale was not obtained because the borrow area had been depleted during construction.

I-74, Ohio

36. The two shale embankments in Ohio were selected to include one which was well compacted with very small settlements and one in which the same type of shale was placed as rockfill with subsequent large settlements. The two are bridge approach embankments on I-74 northwest of Cincinnati near Miamitown. This location cuts through shale interbedded with limestone in the Fairview and Kope formations (Ordovician age).

37. East embankment, SR 128. The first site is at State Route 128 (Figure 11) where shale with about 25 to 30 percent limestone was compacted in 8-in. lifts by four coverages of a heavy tamping roller and two coverages of a 50-ton, 4-wheel, pneumatic-tired roller. Construction was completed in 1961. Measured settlements of the 55-ft high embankment amounted to only 3 in. The sample from 17.5 to 20 ft (Figure 12) contained an 8-in. gray shale and limestone layer between light-brown, silty clay. The undisturbed samples (Figure 12) of the east approach embankment indicate that the fill was well compacted in thin lifts. The samples were moist and included unweathered chunks of Kope shale, limestone, and light-brown, silty clay layers. A total of 18 undisturbed samples were obtained to a depth of 50 ft. Brown, loamy clay, indicative of the natural soil, was encountered at a depth of 53 ft. Fourteen samples were preserved for laboratory examination and testing. Sixteen pressuremeter tests were conducted in the east approach embankment to a depth of 44 ft and three in the west embankment from 44 to 50 ft. The results indicated modulus values of 1500 to 6000 psi and in situ undrained shear strengths as high as 12,300 psf. Two barrels each of the unweathered chunks of the gray shale were obtained from the Fairview and Kope formations in the two major source cuts just west of the embankment.

38. West embankment, Wesselman Road. The second site is at Wesselman Road and Taylor Creek (Figure 13) (1.3 miles east of State Route 128) where 50-ft-high bridge approach embankments were constructed in 1962 using lifts as thick as 3 ft. Approximately 22 in. of settlement

a. East approach embankment

b. West approach embankment

Figure 11. General view of I-74, Ohio embankments at SR 128

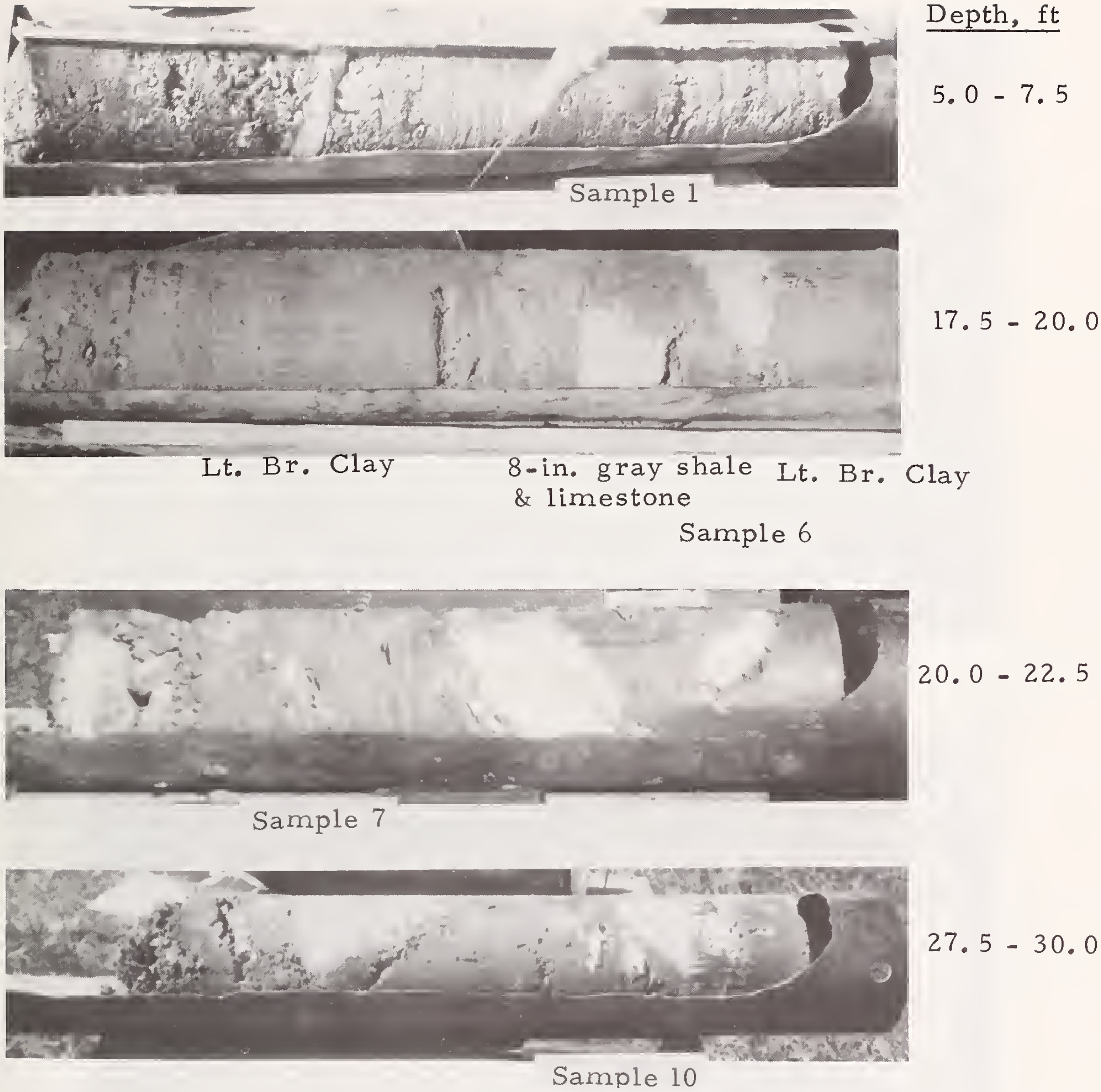


Figure 12. Undisturbed samples I-74, Ohio at SR 128, sta. 424+90

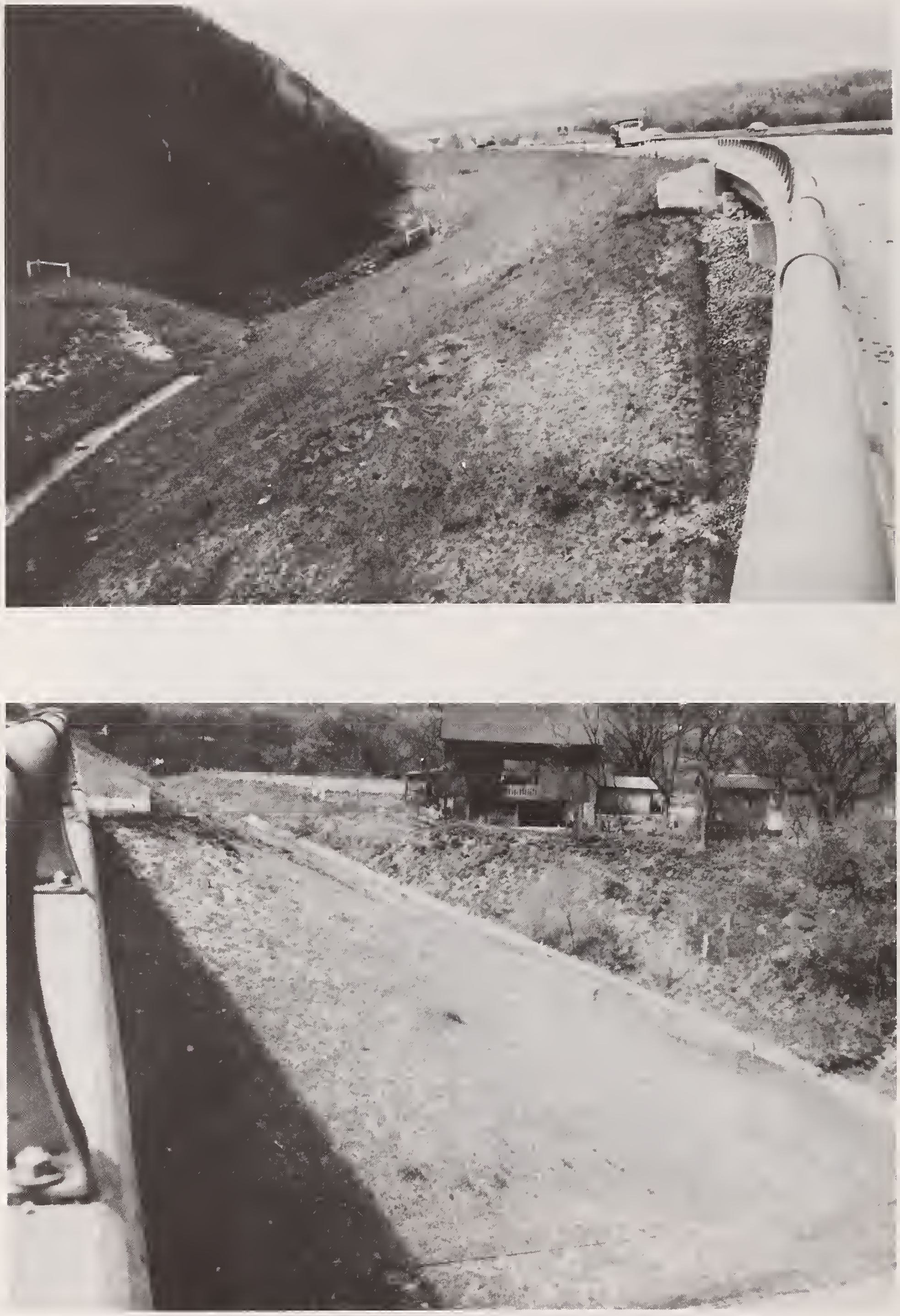


Figure 13. West approach embankment I-74, Ohio at Wesselman Road

at the bridge abutments occurred from 1962 to 1968. Settlement of the soil foundation (80 ft to bedrock) may have contributed to the problem, since 15 ft of soft foundation material was removed at the start of construction. A test pit made in 1968, after cement grouting of the embankment (very low grout take), indicated no large voids in the fill. The undisturbed sample boring was located in the median of the west approach embankment at station 492+00, 75 ft west of the abutment. The samples included loose to firm, dark-gray to brown shale chunks and gravel-size pieces, with clay and limestone chunks and layers of tan to brown, silty clay. Pictures of the sampler were not made. The condition of shale pieces ranged from hard to soft and friable. Twenty-one samples were obtained to a depth of 55 ft, where stratified, dark-brown, clay foundation material was encountered. Sixteen samples were preserved for laboratory examination and testing. Sixteen valid pressuremeter tests were performed to a depth of 35 ft at station 492+15. Modulus values ranged from 1000 to 4000 psi, and undrained shear strengths ranged from 2000 to 8000 psf. Test results are discussed in Part III. Two barrels of unweathered, gray, shale chunks were taken from the source cut in the Kope formation at the west end of the embankment.

I-74, Indiana

39. The selected embankment, stations 426+00 to 440+40 (west to east), is the longest of several embankments experiencing large settlements and slope failure along a five-mile section of I-74 from U. S. 52 near the Ohio-Indiana State line to Indiana State Route 1 near Saint Leon. The embankments were constructed in 1964 and 1965. Interbedded shale and limestone (60 to 70 percent shale and 30 to 40 percent limestone) of the Dillsboro formation (Ordovician age) was used in lifts as thick as 4 ft for most of the embankments. A comprehensive investigation of the existing stability of embankments along this section was made by E. D'Appolonia Consulting Engineers, Inc.* (EDCE) in 1975

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* E. D'Appolonia Consulting Engineers, Inc. (EDCE), "Summary of Investigation and Recommendations, Evaluation of Embankment Stability (ISHC Project No. I-74-4 (73) 163)," Final Report in four Volumes, revised 1977, Indiana State Highway Commission, Indianapolis, Indiana, Nov 1977.

and 1976 for the Indiana State Highway Commission (ISHC). The embankment from stations 426+00 to 440+40 (Figure 14) was selected because the fill material was all shale and limestone and it had experienced large settlements. It was also extensively investigated in the EDCE work. This embankment with 2:1 slopes crosses two skewed ravines separated by a sidehill slope. The height of the embankment along the center line ranges from approximately 60 to 100 ft. The traffic lanes are separated by a 52-ft-wide median, and the grade slopes three percent.

40. Cause of settlement. Settlement data summarized by EDCE from ISHC records and measurements during 1975 and 1976 indicated a maximum settlement of 2.2 ft in the westbound lane at station 434+00. This location is over the sidehill foundation area where the embankment height is 60 ft. Lateral displacement measurements from March 1976 to May 1975 along the westbound shoulder showed 1.3 in. localized movement between stations 433 and 435. Observations of surface conditions indicated settlement around catch basins in the grass median, ponding of water in the median, and an open crack along the westbound pavement at the shoulder.

Figure 14. Indiana I-74 embankment, sta. 426+00 to 440+40 (north slope, looking west)

These subsurface conditions and the possible breakage of subsurface drain pipe aggravated infiltration of surface water. Test pits by EDCE in the median to a maximum depth of 18 ft showed soft, gray, clayey shale (~70 percent) and hard, gray limestone (~30 percent), with shale sizes ranging from crumbly fragments to 3-ft chunks and limestone sizes ranging from gravel to 4-ft slabs. Void sizes were 2 to 12 in., and the material appeared poorly compacted. At one location, a 2-ft layer of soft, brown, silty clay was found which is similar to residual overburden soil on natural slopes. On the north slope of the embankment, a large seepage area with shallow sloughing was evident. Borings encountered seepage water at a depth of about 30 ft, but piezometers in the embankment and the residual soil foundation layers indicated that piezometric levels remained near the base of the embankment. Thus, infiltrating surface water, following diverse paths through the loose embankment fill had softened the shale in the 10-year period since construction.

41. Evaluation of the embankment stability by EDCE included extensive laboratory shear strength tests on undisturbed samples of the embankment clay shale and undisturbed samples of the residual, silty, clay soil and weathered, shale foundation layers. Drained shear strength parameters are summarized in Table 2. The stability evaluation indicated critical shear surfaces through the weathered, shale foundation layer with factors of safety for the north slope at station 436+00 of 1.14 for existing shear strengths being 0.98 assuming loss of cohesion in the embankment and weathered shale and 0.88 assuming progressive failure. A sand and gravel berm to be constructed in 50-ft-wide increments was recommended, in addition to reconstructing the shoulders and median to include an impervious layer, repairing buried drainage pipes, paving the median ditch with asphalt, and reconstructing deteriorated embankment toe ditches with gravel lining.

42. <u>WES field sampling and testing.</u> The undisturbed sample boring was located in the median at station 435+83 near EDCE boring 118 at station 436+00. Samples (Figures 15 and 16) indicated loose to firm, gray clay and shale fragments with gravel-size and larger pieces of limestone. Seepage water and wet zones were encountered at depths of 18,

Table 2. Shear Strength Parameters, Embankment and Foundation Materials, Sta. 426+00 to 440+40 (from EDCE Investigation Report)

Description	Type of Sample	Water Content %	Dry Density pcf	Consolidated Undrained with Pore Pressure Measurement		Consolidated Drained Repeated Direct Shear, Peak and Residual ( )	
				$\bar{c}$ tsf	$\bar{\phi}$ deg	$\bar{c}$ tsf	$\bar{\phi}$ deg
Embankment material:							
Stiff gray and brown silty clay, some rock fragments; sta. 436, westbound shoulder, 64-ft depth	Shelby tube	13.5	125	0.18	24.4		
Stiff yellowish-brown clay to silty clay, some rock fragments; sta. 434, north slope, 27- to 33-ft depth	Shelby tube	18 to 19	129 to 135	0.08	25.7		
Residual soil foundation:							
Brown and gray clay trace of rock fragments, north slope	Shelby tube	17 to 22	124 to 134	0.06	25.5		
Weathered shale foundation:							
Very stiff, gray clay, trace of rock fragments; sta. 440+25, test pit, 14-ft depth	Block sample undisturbed	17 to 19	112 to 116	0.21 0.56 0.47	30.1 28.2 28.9		
	Block sample remolded	30 32 34	92 92 88	0 0 0.14	26.6 25.5 23.1		
Gray, clayey shale, sta. 440+25, test pit, 13- to 14-ft depth	Block sample undisturbed	11 to 17	114 to 125			0.71 (0.19)	23.8 (10.8)
Brown clay, sta. 440+25, test pit, 13-ft depth	Block sample undisturbed	44	76			0.18 (0.18)	14.5 (5.5)

Figure 15. Character of shale samples, Indiana I-74, sta. 435+83, 5.0 to 28.9 ft

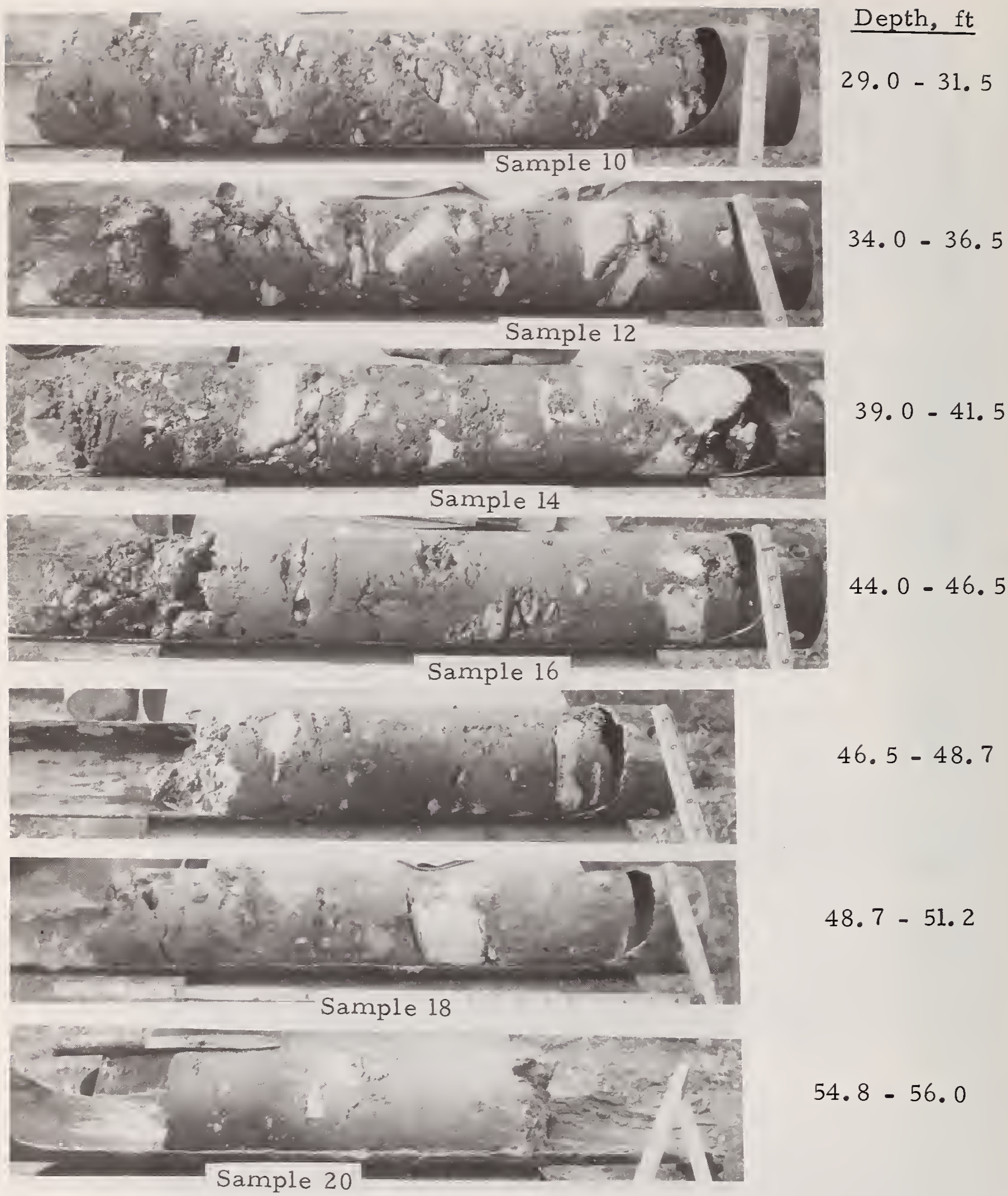

Figure 16. Character of shale samples, Indiana I-74, sta. 435+83, 29.0 to 56.0 ft

34, 40, and 43 ft. Layers of light-brown clay with limestone gravel (indicative of residual soil) were evident in samples from depths of 28, 38, 47, and 53 ft. Light-brown clay with limestone fragments (residual soil) was encountered at a depth of 55 ft and weathered shale at 63 ft. The boring was terminated at a depth of 66 ft. The water surface in the boring was at a depth of 52 ft. From Figure 16 it appears that material below about 42 ft was denser than that at shallower depths. Shale fragments were generally soft and could be remolded with fingers. Of the 23 samples obtained, 21 were preserved. Six were left with the Seymore District for use by the ISHC, and 15 were saved for laboratory examination and testing (Part III).

43. Pressuremeter tests were performed at 5-ft-depth intervals in a boring at station 435+20. Seven of 11 tests were valid. The modulus values ranged from 1000 psi to 2500 psi, and undrained shear strengths ranged from 1000 to 5000 psf and showed an increase with depth.

44. Barrel samples of unweathered shale were excavated from a cut at the west end of the embankment. The inplace hardness of the shale layers (12 in. thick) and limestone layers (1 to 2 in. thick) prevented excavation of large shale chunks with a backhoe. Consequently, only one barrel of shale pieces, 1 to 6 in. in size, was obtained.

Summary of Shale Conditions

45. Visual examination of embankment samples during the field investigation indicated that deterioration of shale chunks into silt and clay was less than expected, except for the Indiana embankment. Many of the samples contained a mixture of overburden (residual) soil, small shale and rock particles, and large rock which apparently occurred during construction. Such mixtures would be difficult to compact when larger rock or shale chunks exceeded about 40 percent of the total material. The resulting densification of smaller particles would be minimal. This condition was evident from loose material in a number of samples. In addition, the random variation with depth in types of materials and loose zones indicated that infiltrating water probably followed erratic paths, causing softening and compression in an irregular fashion.

PART III: TESTS ON SHALE EMBANKMENT SAMPLES

General

46. Selected samples from each of the six embankments were examined and tested in the laboratory to determine the following:

- a. Type and degree of deterioration of shale pieces and/or softening of soil, shale, and rock mixtures.
- b. Range of particle sizes and material types that could indicate mixing of soil, shale fragments, large shale pieces, and rock during construction.
- c. In situ density, water content, compressibility, and shear strength.

A major objective of the laboratory tests was to obtain information about the character of embankment materials, in situ shale conditions, and material properties needed to assess the influence of construction practices and shale durability on performance. The information was also needed to develop some type of accelerated weathering index test that could be used to predict long-term performance of compacted shales.

Tests Performed

47. Special triaxial compression tests to determine compressibility properties and shear strength were performed on 22 samples. Ten of the triaxial samples were used for special gradation tests and also for specific gravity and Atterberg limits tests on the minus No. 40 sieve fraction. The density and water content of the 22 triaxial samples and 30 other samples were measured. Direct shear tests were performed on shale layers of five samples containing large limestone fragments. All samples tested were examined, and a visual description with emphasis on condition of shale pieces was recorded.

Selection of Triaxial Test Samples

48. The 5-in.-diameter samples for triaxial compression tests were

selected first to utilize as many samples as possible with a minimum of large (greater than 2 in.) shale, siltstone, or limestone pieces. Radiographs (X-ray pictures) were made of samples which appeared to have pieces no larger than 2-3 in. The radiograph negatives (true scale) showed intact pieces of shale, siltstone, limestone, etc., within the sample as dark areas which could be measured directly on the negative. An example of radiograph pictures is shown in Figure 17. Since testing two samples from each embankment was desirable, it was necessary to use samples with 3-in.-size pieces. However, it was usually possible to select the 10-in.-long test sample (from the 12-in. sample length) with the larger pieces at one end.

Test Procedures

Density and water content

49. The wet density of triaxial samples was measured after trimming and patching the rough sides and/or ends. The patching caused wet densities to be approximated 5 pcf too high and reported densities have been adjusted. The water content was determined after triaxial testing. The 30 samples used only for density and water content tests were subjected to the following procedure: The cardboard tube was peeled off, and the sample ends were cut off on a motorized steel cutting saw to expose sampled material. The height and total diameter of the sample (including wax jacket) were measured to determine volume of sample and wax. The wax was then removed and weighed and the volume of wax calculated from its known specific gravity (0.9). The sample, assumed to have retained its 5-in. diameter (as sampled diameter), was weighed and then oven-dried to determine the dry weight. The total volume of the sample (with wax jacket) minus the volume of wax was compared with the sample volume (calculated from the measured length and assumed 5-in. diameter) as a check for gross errors. The sample volume and measured wet and oven-dry weights were used to calculate wet density, water content, and dry density.

Special gradation tests

50. An important observation during Phase I studies was that

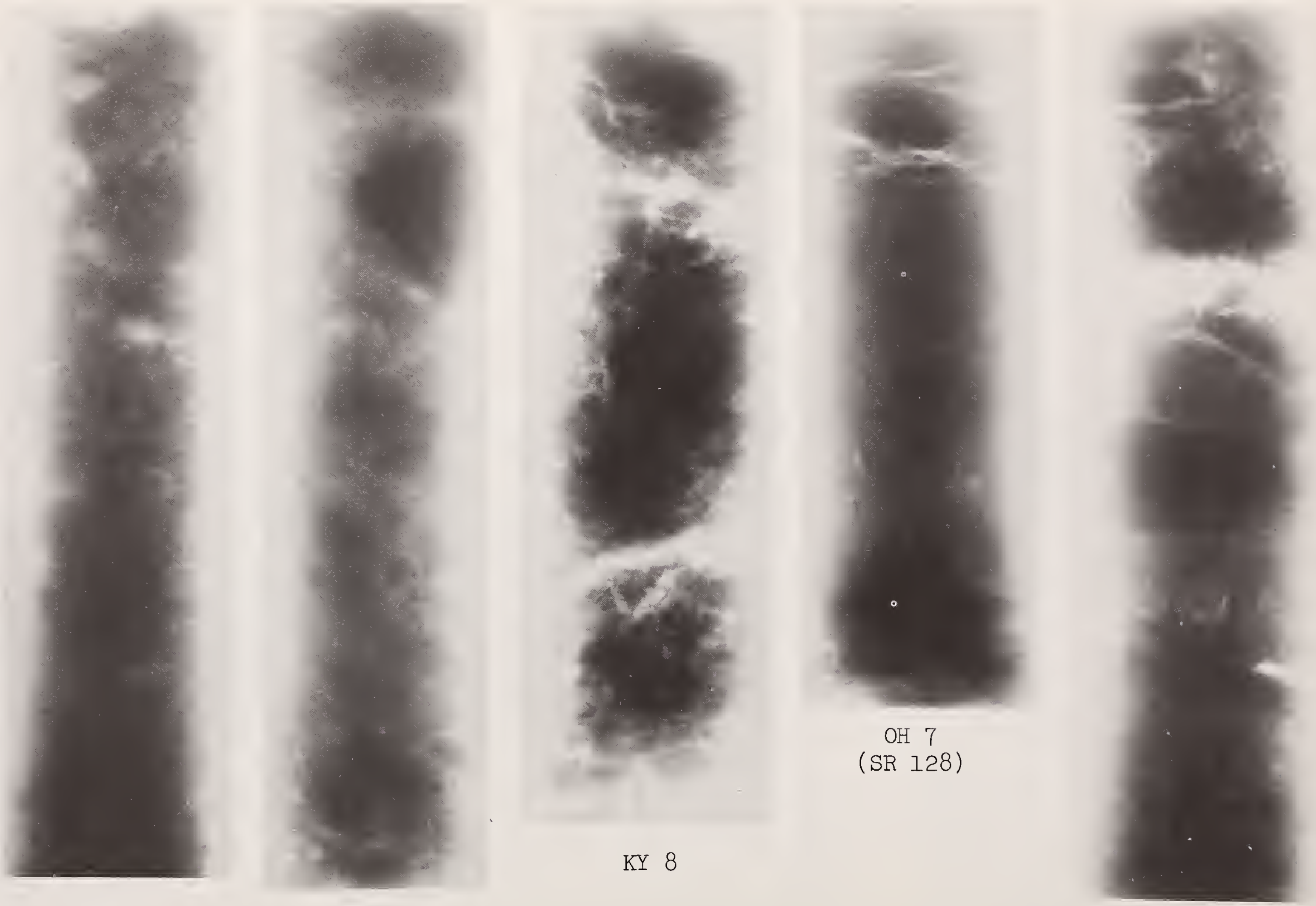

Figure 17. Example of radiograph pictures of shale embankment samples.

shales, when allowed to dry even for a short time, tended to slake when emersed in water. Conversely, when shales at their natural water content were emersed in water, very little slaking occurred. The procedure involved gently breaking the sample apart by hand in the humid room to segregate large particles, then soaking the sample to disaggregate smaller particles, followed by wet sieving (sieve sizes used were 2-in., No. 4, No. 10, No. 40, and No. 200). Although slaking was minimized, it was not possible to break the sample apart without breaking softer shale and siltstone pieces. Although the true gradation was not measured, the test results did provide a relative indication between samples, since the same procedure was used for all tests.

Atterberg limits and specific gravity tests

51. Atterberg limits were performed on a minus No. 40 sieve fraction using a blenderized procedure outlined in Corps of Engineers Manual EM 1110-2-1906. Specific gravity tests on a minus No. 40 sieve fraction also used procedures outlined in EM 1110-2-1906.

Triaxial compression tests

52. Sample preparation. Samples were prepared for testing by peeling off the cardboard tube and carefully cutting away the wax along two sides with a band saw. The top section of wax was lifted from the sample and a half-round section of 5-in.-diameter sample tube was placed over the exposed sample length. The sample and tube were then turned over so that the sample rested in the sample tube and the wax section remaining on top was removed. The samples had been covered with Saran wrap in the field before being covered with wax. The Saran wrap made it easier to remove the wax in the laboratory humid room. After removing the wax, the 10-in. sample length was chosen, using photo prints of the X-radiographs as a guide. The excess portions at the end of the sample were carefully cut or gently pried away in small segments. The ends and sides of rough samples were patched with fines from removed sample material. The dimensions of the sample were measured, and the sample was installed in the triaxial chamber with base cap and membrane. A 5-in.-diameter by 0.25-in.-thick porous stainless-steel disk was used for a

bearing surface at the top and bottom of the test sample.

53. Test procedures. To obtain as much information as possible from one 5-in.-diameter by 10-in.-high test sample, a special consolidated undrained triaxial test was used. The test sample was fitted with two girth gages to monitor lateral strains at the one-third points of sample height. The girth gage incorporated an electronic sensor (potentiometer) which enabled changes in diameter as small as 0.00005 in. or 0.001 percent in diametric strain (i.e., lateral strain) to be measured. The test sample was consolidated in increments to the in situ overburden stress by increasing the axial load and simultaneously increasing the chamber pressure to maintain zero lateral strain. Thus K_o consolidation was achieved since K_o is the ratio of lateral stress to the vertical stress for zero lateral strain or

$$K_o = \frac{\sigma_{3c}}{\sigma_{1c}}$$

where

σ_{3c} = lateral consolidation stress

σ_{1c} = vertical consolidation stress

The test sample was then unloaded to a principal stress ratio approaching 1.0 (axial load and chamber pressure decreased) and reloaded to the initial in situ overburden stress to relieve stress caused by removing the sample from the ground. At this point, the test sample was at the in situ stress conditions, and the measured K_o value was equivalent to the in situ value. Drainage lines were then closed, and the axial load increased (with the chamber pressure constant) until the deviator stress started to peak. A second and third consolidation stage to higher lateral stresses holding K_o constant, with undrained loading after each stage, were used to determine the shear strength.

Direct shear tests

54. Consolidated drained direct shear tests were performed on clay shale layers from two samples from the Kentucky I-75 embankment and three samples from the Indiana I-74 embankment. Specimens were carefully trimmed into a 3-in.-square cutting frame, 0.62 in. deep,

and tested submerged, as outlined in EM 1110-2-1906. Three specimens for each test were consolidated to pressures of 0.5, 1.0, and 2.0 tsf and sheared at a displacement rate of 0.00018 in./min to a total displacement of 0.5 in. (total shear time of 46 hours).

Character of Shale Embankment Materials

55. Examination of selected embankment samples confirmed observations during field sampling about apparent mixing of fine-grained materials with rock during construction and the heterogeneous nature of embankment materials. Descriptors of the embankment samples used for laboratory tests are given in Table 3. The condition of shale and siltstone pieces ranged from soft to friable to hard. Softening of some shale and siltstone pieces was the major form of deterioration, except for the Indiana I-74 embankment where shale had softened into stiff to soft clay shale. In the Ohio I-74 embankment at State Route 128 shale pieces were similar to the unweathered parent shale.

Water content and density

56. Water contents and densities (Table 4) were determined for the total sample and included the effects of large intact pieces of shale, siltstone, sandstone, or limestone. Water contents greater than 17 percent were measured in one sample (No. 10) from the West Virginia embankment and in three samples (Nos. 1, 4, 5) and one direct shear specimen (from sample 11) from the Kentucky embankment. The highest dry density of 130 pcf was measured for sample 23 from the Tennessee embankment. This sample had a 4-in.-long core of black shale and two shale pieces, 2 in. by 3 in. (Table 3). Since the natural dry density of intact shale, siltstone, sandstone, or limestone can range from 130 to 150 pcf, it can be reasoned that samples with an overall low dry density and numerous intact large pieces of dense shale, siltstone or limestone could have a low density for the soillike matrix.

57. A comparison in Table 5 of the range and average for water contents and dry densities indicates that the Ohio State Route 128 and the Indiana I-74 embankments had significantly higher average dry densities.

Table 3. Description of Selected Shale Embankment Samples

Sample No.	Description
	Tennessee I-75 Embankment, Sta. 960+82
4	Top 1/3: Gray, gravelly clay with 1-in. sandstone pieces near top and few black shale pieces, 1- to 2-in. size; friable Bottom 2/3: Light brown, silty clay with few black shale fragments
5	Dark gray to black shale with intact shale pieces, 1-1/2 to 3 in., friable; mixed with clayey soil and gravel; large shale cobble on one side
6	Top half: Gray shale with two 2- to 3-in. sandy shale pieces and smaller silty shale pieces Bottom half: Dark brown, silty clay with 1/4- to 1/2-in. dark brown to black shale pieces
7	Brown to orange, gravelly clay mixture with few sandstone pieces to 2 in.
11	Brown, gravelly, silty clay mixture with friable gray and brown shale pieces 1 to 1-1/2 in. (one 1-in. by 2-in. shale piece near top)
12	Dark brown, gravelly, silty clay mixture with brown to black shale pieces, 1 to 1-1/2 in.; few 1/4- to 1/2-in. sandstone fragments
14	Light black to dark gray clay and shale mixture; shale pieces 1 to 2 in., friable and soft; few hard pieces to 2 in., loose
17	Dark brown, silty clay and gray shale (25 percent) mixture, shale pieces friable; few sandstone pieces 2 to 3 in. and some orange, soft, sandy clay
23	Black shale, hard, gravel sizes; two shale pieces 3 in. by 3 in. and 4-in.-long core of black shale at bottom
26	Brown, gravelly, silty clay mixture with medium-hard, gray to brown sandstone pieces 1 to 2 in. near top on one side; one 2-in. piece of dark gray, hard, sandy shale at bottom on other side
31	Dark gray to brown, gravelly, silty clay and medium-hard shale pieces 1/4 to 1 in. (50 percent); few shale pieces 2 to 3 in.
33	Black shale pieces to 3 in. and few sandstone pieces to 2 in., fairly dense
	West Virginia U. S. 460 Embankment, Sta. 553+75
1	Brown, gravelly, siltstone pieces to 1 in. and silty clay mixture; one 4-in.-long core section of brown, silty shale; siltstone pieces medium hard
3	Light brown, weathered siltstone with 50 to 60 percent siltstone fragments to 1-in. size, cubical shape with soft clay coating; no evidence of deterioration
6	Brown, silty clay and hard siltstone pieces to 3-in. size with about 10 percent sandstone pieces (Fig. 8)
10	Mixture of light brown to orange clay shale pieces to 4 in., medium soft and silty clay; few hard, brown, siltstone pieces and shale pieces about 3/4 in.; some tough, wet, orange plastic clay
12	Mottled dark gray, brown, and tan weathered shale with about 10 percent friable shale fragments to 1-1/2 in. (Fig. 8)

Table 3. Description of Selected Shale Embankment Samples (Continued)

Sample No.	Description
	West Virginia U. S. 460 Embankment, Sta. 553+75 (Continued)
14	Brown, gravelly, silty clay with 4-in. core section of olive brown shale (short sample)
16	Tan and yellow, clayey, sandy gravel with siltstone pieces to 4-in. and 1/2-in. layer yellow-tan clay
19	Medium hard, black clay shale pieces to 1/2 in. with 2-in. by 3-in. piece of light brown to dark gray siltstone and gray shale gravel-size pieces; 3 in. of light brown, gravelly, silty clay with small pieces of brown siltstone at bottom
21	Mixture of brown, gravelly, silty clay with siltstone pieces to 3 in. and smaller pieces of gray shale; one 4-in. core section of brown shale
29	Mixture of dark gray, hard shale sand sizes to 1-1/2 in. and clay
30	Mixture of light brown, hard, siltstone pieces to 4-in. size and smaller pieces of black claystone (limey) with wet, orange clay infilling and brown, gravelly, silty clay
33	Light brown, weathered shale with few hard shale pieces to 3 in.
	Kentucky I-75 Embankment, Sta. 678+55
1	Brown, silty clay with few 1-in. limestone pieces (Fig. 10)
3	Brown, silty clay with fragments of white clay and white clay streaks
4	Brown, gravelly shale and clay shale pieces to 1/2 in.; moist, dense with mottled brown to dark brown clay chunk on one end
5	Dense, mottled, light and dark brown clay with few limestone pieces 1/2 in. to 1 in.
6	Gray clay shale with 1/2-in. concretions and limestone cobbles (Fig. 10)
8	Mixture small, gray shale pieces with limestone pieces 1/2 in. to 1 in. by 3 in.
10	Dense, reddish-brown (mottled light and dark) clay with several limestone chunks 3 to 4 in. in center half (Fig. 10)
11	Layers of light brown clay shale and limestone cobbles (Fig. 10)
13	Olive brown to gray clay shale, inclined bedding, plastic, friable; small root
	Ohio I-74 Embankment at SR 128, Sta. 424+90
1	Gray clay shale pieces to 3 in., medium hard with gray clay (Fig. 12)
2	Dense mixture of gray, clayey shale and gray shale pieces to 3 in.
4	Dense, gray, gravel-size shale with several medium-hard, brown, stained shale pieces to 3 in. and few small limestone pieces
7	Dense, brown clay with few small (1/2 in. to 1 in.) pieces of gray to mostly brown clay shale, friable, medium-hard; one 2-in. piece of brown shale on one side (Fig. 12)

Table 3. Description of Selected Shale Embankment Samples (Concluded)

Sample No.	Description
	Ohio I-74 Embankment at SR 128, Sta. 424+90 (Continued)
10	Compact gray shale to 3-in. size with few limestone pieces 2 in. to 3 in. (Fig. 12)
12	Dense, gray clay shale and many gray shale pieces to 3-in. size; one 1-in. by 3-in. limestone piece at top and one 3-in. by 4-in. gray shale piece at bottom on one side
15	Dense, gray clay shale with many medium hard gray shale pieces to 3-in. size; trace brown, silty clay
18	Dense, gray clay shale and medium-hard, gray shale pieces to 4 in.; one large piece of sandstone
	Ohio I-74 Embankment at Wesselman Road, Sta. 492+00
1	Gray clay shale, numerous pieces of gray shale to 3 in.; one piece 1/2-in. by 3-in. limestone inclined about 30 deg near base and smaller piece of limestone near top
3	Mixture of light brown and gray clay and hard gray shale pieces to 2 in.
5	Gray clay shale and numerous gray shale pieces to 3 in., loose
7	Gray clay shale gravel sizes and shale pieces to 3 in., medium hard to soft
11	Gray, friable clay shale to 1/4 in. with numerous medium-hard, unweathered shale pieces to 2 in.
14	Mixture of gray clay and shale pieces to 2 in. with limestone pieces to 3 in.
17	Mixture gravelly, gray clay with gray, medium-hard shale pieces to 4 in. and several limestone pieces to 2 in.
18	Mixture gravelly clay shale and gray shale pieces to 3 in.
	Indiana I-74 Embankment, Sta. 435+83
6	Gray, stiff clay shale and limestone pieces to 4 in. (Fig. 15)
7	Gray clay shale (wet) and limestone pieces to 3 in. (about 60 percent limestone); clay shale remoldable with fingers (Fig. 15)
10	Gray clay shale with shale and limestone pieces to 3 in., shale pieces more cubical in shape, crumbly to hard (Fig. 16)
14	Mixture gray, clayey shale to 1/2 in. and limestone pieces to 3 in. (40 to 50 percent limestone) (Fig. 16)
16	Gray clay with concretions to 1/2 in. and limestone pieces to 3 in.
18	Gray shale and limestone pieces 2 in. to 3 in.; brown clay and shale fragments at one end (Fig. 16)
20	Light brown clay and limestone cobbles
22	Tan, sandy clay, hard with one 2-1/2 in. limestone piece

Table 4. Laboratory Test Results for Shale Embankment Samples

Embankment	Sample No.	Sample Depth, ft	Sample* Water Content %	Sample* Wet Density pcf	Sample* Dry Density pcf	Specific Gravity Solids G_s	Atterberg Limits LL %	Atterberg Limits PL %	Atterberg Limits PI %	Maximum Size in.	Gradation Percent Passing: 3-in.	No. 4	No. 10	No. 40	No. 200	K_o Consolidated Undrained Triaxial Tests, Effective Stress K_o	ν'	Total Stress K	ν	E psi	c kg/cm^2	ϕ deg	Consolidated Drained Direct Shear Tests c' kg/cm^2	ϕ' deg
Tennessee	4	13.8-15.0	17	125	107	----	--	--	--	2	---	---	---	---	---	0.53	0.35	0.75	0.43	4,270	0.4	15	---	---
I-75,	5	16.4-17.7	14	131	115	2.78	42	24	18	3	100	59	50	37	31	0.50	0.33	0.75	0.42	1,180	0.6	16	---	---
sta. 960+82	6	18.9-20.2	13	130	115	----	--	--	--	3	---	---	---	---	---	0.45	0.31	0.72	0.41	5,400	0.3	25	---	---
	7	21.5-22.7	12	121	108	----	--	--	--	2	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	11	29.8-31.1	15	126	109	----	--	--	--	2	---	---	---	---	---	0.41	0.29	0.70	0.41	5,570	0.7	22	---	---
	12	32.5-33.8	15	125	108	2.70	36	22	14	1 1/2	100	83	76	67	58	0.53	0.35	0.75	0.43	3,120	0.3	21	---	---
	14	37.3-38.5	12	113	101	----	--	--	--	2	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	17	44.7-46.0	10	134	122	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	23	58.4-59.7	4	135	130	----	--	--	--	4	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	26	67.4-68.7	14	128	112	----	--	--	--	2	---	---	---	---	---	0.50	0.33	0.75	0.42	5,450	0.6	20	---	---
	31	81.8-83.0	13	128	113	----	--	--	--	3	---	---	---	---	---	0.50	0.33	0.75	0.42	6,410	1.4	15	---	---
	33	85.8-86.8	9	115	106	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
West Virginia	1	6.1-7.4	11	127	114	----	--	--	--	4	---	---	---	---	---	----	----	----	----	---	---	---	---	---
U.S. 460,	3	11.1-12.2	11	131	118	2.74	37	22	15	3	100	42	30	20	16	0.30	0.23	0.62	0.38	4,580	0.3	27	---	---
sta. 553+75	6	18.6-19.9	11	128	115	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	10	28.6-29.9	25	113	90	----	--	--	--	4	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	12	33.0-34.0	16	120	103	2.67	42	23	19	1 1/2	100	69	57	42	35	0.42	0.30	0.71	0.42	8,250	0.2	26	---	---
	14	39.5-40.2	10	123	111	----	--	--	--	4	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	16	42.8-43.9	16	129	111	2.63	38	19	19	4	99	61	53	41	36	0.50	0.33	0.73	0.42	7,390	---	---	---	---
	19	50.4-51.7	14	124	109	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	21	54.6-55.9	10	126	114	----	--	--	--	4	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	29	70.0-71.0	8	134	124	2.74	29	16	13	2	100	60	43	28	21	0.35	0.26	0.64	0.39	4,560	---	---	---	---
	30	72.2-73.3	12	130	116	----	--	--	--	4	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	33	79.5-80.8	12	135	120	----	--	--	--	3	---	---	---	---	---	0.53	0.35	0.74	0.42	44,600	---	---	---	---
Kentucky	1	9.4-10.7	20	128	102	----	--	--	--	1	---	---	---	---	---	0.50	0.33	0.74	0.42	5,360	1.0	12	---	---
I-75,	3	14.4-15.7	17	128	104	----	--	--	--	1/4	---	---	---	---	---	0.50	0.33	0.74	0.42	10,040	1.7	7	---	---
sta. 678+55	4	16.3-17.6	21	124	102	----	--	--	--	1/2	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	5	19.0-20.0	24	114	93	----	--	--	--	1	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	6	20.4-21.7	10 to 12	116 to 118	104 to 108	----	--	--	--	4	---	---	---	---	---	----	----	----	----	---	---	---	0	30
	8	26.3-27.5	5	106	101	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	10	33.4-34.7	17	129	110	----	--	--	--	4	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	11	35.9-37.2	18 to 22	127 to 132	107 to 112	----	--	--	--	4	---	---	---	---	---	----	----	----	----	---	---	---	0.15	25
	13	39.0-39.8	17	111	95	----	--	--	--	---	---	---	---	---	---	----	----	----	----	---	---	---	---	---
Ohio I-74 at	1	6.1-7.4	10	131	119	----	--	--	--	3	---	---	---	---	---	0.50	0.33	0.73	0.42	8,900	0.9	28	---	---
SR 128,	2	8.6-9.9	10	130	118	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
sta. 424+90	4	13.6-14.9	9	133	122	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	7	20.2-21.2	17	125	104	----	--	--	--	2	---	---	---	---	---	0.50	0.33	0.74	0.42	2,540	1.3	12	---	---
	10	28.6-29.9	12	138	123	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	12	33.6-34.9	11	136	123	2.82	36	19	17	3	100	71	66	57	52	0.42	0.30	0.67	0.40	20,500	2.1	10	---	---
	15	41.1-42.4	11	135	122	----	--	--	--	3	---	---	---	---	---	0.45	0.31	0.69	0.41	6,340	2.4	10	---	---
	18	47.9-49.2	10	138	126	----	--	--	--	4	---	---	---	---	---	----	----	----	----	---	---	---	---	---
Ohio I-74 at	1	6.2-7.4	12	132	118	----	--	--	--	3	---	---	---	---	---	0.40	0.29	0.67	0.40	7,100	0.9	23	---	---
Wesselman Road,	3	10.7-11.7	13	116	103	----	--	--	--	2	---	---	---	---	---	----	----	----	----	---	---	---	---	---
sta. 492+00	5	16.0-17.2	8	120	111	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	7	21.4-22.4	11	112	100	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	11	30.0-31.3	11	124	112	2.79	43	22	21	2	100	61	52	39	33	0.34	0.25	0.64	0.39	5,900	0.9	25	---	---
	14	36.3-37.4	7	125	117	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	17	43.6-44.9	10	124	113	----	--	--	--	4	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	18	46.1-47.4	12	132	118	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
Indiana	6	20.1-21.4	10 to 13	131 to 135	117 to 120	----	--	--	--	4	---	---	---	---	---	----	----	----	----	---	---	---	0.2	24
I-74,	7	22.7-23.7	9	121	111	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
sta. 435+83	10	30.3-31.4	8	137	127	2.76	35	17	18	3	100	51	43	32	28	0.47	0.32	0.73	0.41	5,750	0.9	24	---	---
	14	40.0-41.4	10	124	113	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	16	45.0-46.0	8 to 12	136 to 137	122 to 126	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	0.3	24
	18	49.8-50.8	7	133	124	----	--	--	--	3	---	---	---	---	---	----	----	----	----	---	---	---	---	---
	20	54.8-56.0	12 to 14	135 to 137	119 to 122	----	--	--	--	4	---	---	---	---	---	----	----	----	----	---	---	---	0.2	23
	22	60.0-61.0	10	131	119	2.77	37	18	19	2 1/2	100	81	78	68	56	0.40	0.29	0.68	0.40	13,400	---	---	---	---

*Total sample otherwise for triaxial sample or direct shear specimen where these tests are indicated.

For the State Route 128 embankment, the higher average dry density of 120 pcf reflects the greater compaction effort used (paragraph 37), while for the Indiana embankment the high average dry density of 120 pcf probably reflects the large percentage of dense limestone pieces (Figures 15 and 16) or consolidated clay and clay shale layers at significant depths. The low average dry density of 103 pcf for the Kentucky embankment probably reflects plastic clay and/or loose granular materials (Figure 10).

Classification

58. Based on the Atterberg limits and gradation, the materials tested classify according to the Unified Soil Classification System (USCS) and AASHTO System as indicated below:

Embankment	Sample No.	Classification AASHTO	USCS
Tennessee I-75	5	A-2-7	SC
	12	A-6	CL
West Virginia U. S. 460	3	A-2-6	GC
	12	A-2-7	SC
	16	A-6	SC
	29	A-2-6	SC
Ohio I-74, SR 128	10	A-6	CL
Ohio I-74, Wesselman Road	11	A-2-7	SC
Indiana I-74	10	A-2-6	SC
	22	A-6	CL

Compressibility

59. The initial consolidation and rebound stage of the K_o triaxial compression tests provided an estimate of the in situ stress conditions characterized by K_o (coefficient of earth pressure at rest) and compressibility characterized by Poisson's ratio, $\nu(\nu = K_o/(1 + K_o))$, and initial tangent modulus, E. Test results for individual tests are contained in Appendix B. Effective stress K_o values (Table 4) ranged from 0.40 to 0.53, which corresponds to the range for normally consolidated soils, except for three samples with values as low as 0.30 (West Virginia samples 3 and 29, Ohio at Wesselman Road sample 11). The majority of the calculated values for Poisson's ratio

Table 5. Summary of Water Content and Density Data for Shale Embankment Samples

Embankment	Water Content, %		Dry Density, pcf	
	Range	Average	Range	Average
Tennessee I-75	4 - 17	12	101 - 122	110
West Virginia U. S. 460	8 - 25	14	103 - 124	114
Kentucky I-75	5 - 24	17	93 - 112	103
Ohio I-74 at SR 128	9 - 17	11	105 - 126	120
Ohio I-74 at Wesselman Road	8 - 13	10	100 - 118	112
Indiana I-74	7 - 14	10	111 - 127	120

ranged from 0.29 to 0.35. Initial tangent moduli from undrained loading varied from 1,200 to 13,000 psi (84 to 914 kg/cm^2), except for two samples for which modulus values were much higher. The total stress values for K and ν shown in Table 4 are used in evaluating pressuremeter data in the next section.

60. The range of initial tangent moduli versus lateral confining pressure, σ_{3C}, for the shale embankment samples is compared in Figure 18 with other data. Initial tangent modulus values usually increase with increase in confining pressure (Kulhawy, Duncan, and Seed, 1969),* and this trend is suggested for the shale embankment samples by arbitrary trend lines (grouping of different samples was precluded by their heterogeneous character). However, the comparison in Figure 18 indicates that modulus values for the shale embankment samples are generally higher than the range of values from the isotropically consolidated undrained (ICU) triaxial compression tests on laboratory compacted shales tested for Corps of Engineers (CE) projects (Table 6 extracted from Table 14, Vol 1) and by Abeyesekera (1977).**

Shear strength

61. For triaxial tests in which a second and third stage of consolidation and axial loading were achieved (i.e., brittle failure did not occur during the first or second loading stage), the undrained shear strength was defined (see Appendix B for individual test results). Undrained strength parameters c, ϕ, listed in Table 4, indicate considerable variation. As shown in Figure 19, values of cohesion, c, range from 0.2 to 2.4 kg/cm^2, with the higher c values associated with smaller ϕ values for samples containing either numerous large pieces of intact shale, siltstone, and limestone (e.g., Ohio State

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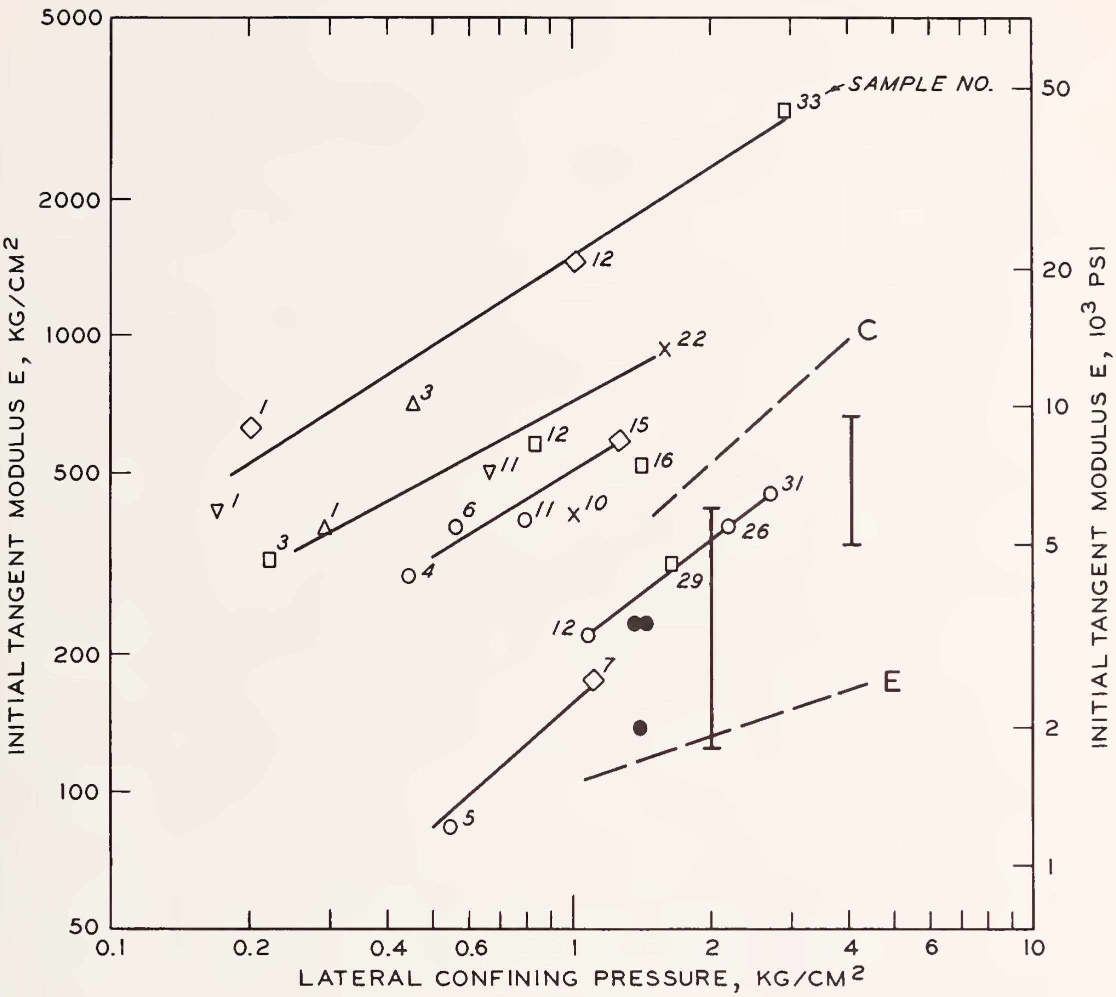
* Kulhawy, F. H., J. M. Duncan, and H. Bolton Seed, "Finite Element Analyses of Stresses and Movements in Embankments during Construction," Contract Report S-69-8, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Nov 1969.

** Abeyesekera, R. Aubrey, "Stress Deformation on Strength Characteristics of a Compacted Shale," Interim Report JHRP-TI-24, HRP Project C-36-51 for Indiana State Highway Commission, Joint Highway Research Project, Purdue University, West Lafayette, Indiana, Dec 1977.



LEGEND

K_0CU TRIAXIAL TESTS ON SHALE EMB. SAMPLES:

○	TN, I-75	◇	OH, I-74, SR 128
□	WV, US 460	▽	OH, I-74, W.R.
△	KY, I-75	×	IN, I-74

I EST. FROM CE PROJECT TESTS, PROJECTS 4, 8, 9, 10, 12 (TABLE 6)

● EST. FROM 3 CU TESTS, ABEYESEKERA, 1977

C, E CU TRIAXIAL TESTS (UNDRAINED) ON OROVILLE DAM CLAYEY SANDY GRAVEL AND DRAMMEN CLAY (FIGURE 11, KULHAWY, DUNCAN, & SEED, 1969)

Figure 18. Range of modulus, E , for tests on shale embankment samples compared with other data

Table 6. CU and CD Shear Strength Parameters, CE Projects
(Extracted from Table 14, Vol 1)

Project	Material Description	As Compacted Water Content %	As Compacted Dry Density pcf	As Tested Water Content %	Shear Strength R(CU) c tsf	Shear Strength R(CU) φ deg	Shear Strength R̄ or S (CD) c' tsf	Shear Strength R̄ or S (CD) φ' deg
4. East Fork Lake, OH, 4 miles SW of Batavia	Crushed, gray, shaley limestone (50%) and calcareous shale (50%)	1.9	130	2	1.9	36	--	--
		2.3	129	3	1.6	36	--	--
		1.6	130	Sat.	1.5	12	0.7	38
5. Carr Fork Lake, KY, 2 miles from Vicco	Crushed shale and sandstone	5.8	123	Sat.	1.2	16	--	--
6. Fishtrap Lake, KY, 7 miles SE of Pikesville	Crushed gray shale	3	128	3	1.0	18	--	--
		3	128	Sat.	--	--	0.5	36
7. Paintsville Lake, KY, Paintsville	Crushed gray, silty, clayey shale	7	129	7	1.0	18	--	--
		7	129	Sat.	--	--	0.0	37
		7	129	Sat.	--	--	0.0	33
8. Red River Lake, KY, 45 miles E of Lexington	Crushed, gray and green-gray limestone and shale	5	120	Sat.	0.4	16	0.6	36
	Crushed, light-gray, clayey shale	5	124	Sat.	0.25	18	0.0	40
9. Taylorsville Lake, KY, 20 miles SE of Taylorsville	Crushed brown to green-gray, weathered, soft shale (68%) and gray, interbedded limestone (32%)	12	115	12	1.2	20	--	--
		12	115	12	1.1	21	--	--
		11	116	Sat.	0.0	14	0.3	35
		11	116	Sat.	0.3	11	0.0	40
	Crushed, gray, interbedded mod.-hard to mod.-soft, calcareous shale and hard fossiliferous limestone	2.4	127	Sat.	0.25	16.5	0.25	37
		2.5	127	Sat.	0.25	15.5	0.0	40
10. Yatesville Lake, KY, 4 miles south of Louisa	Crushed, red to gray, clay shale, indurated clay; and siltstone	4	128	Sat.	0.25	20	0.0	34
11. Beech Fork Lake, WV, 6.5 miles S of Huntington	Crushed, indurated clay	9	123	Sat.	0.17	17	--	--
		11	123	Sat.	0.21	17	--	--
12. R. D. Bailey Lake, WV, near Justice	Crushed, brown clay shale	2.5	113	Sat.	0.0	16	0.0	32 & 35
	Crushed, gray clay shale	1.1	115	Sat.	0.0	16	0.0	31 & 39
	Brown, weathered shale & claystone from test fill	8.5	125	Sat.	0.5	18	0.0	35
		8.0	126	9	--	--	0.0	32
13. Beltsville Dam, PA, 4 miles E of Lehighton	Crushed, weathered shale	17	114	Sat.	0.0	24	0.0	35
	Broken, brown, weathered shale from test fill	14	120	Sat.	0.0	22	0.0	37
		14.7	116	Sat.	0.7	20	0.0	36
		4.2	119	Sat.	0.6	25	0.0	31
		12.2	118	Sat.	2.0	20	0.0	36
	Gray, silty, weathered shale from test fill	8.3	121	Sat.	1.5	10	--	--
		7.2	126	Sat.	1.5	13	--	--
		8.8	130	Sat.	2.3	20	--	--
		7.9	131	Sat.	0.0	26	0.0	38
		5.8	132	Sat.	0.4	28	0.0	38
	Partly weathered shale from test fill	7.1	126	Sat.	1.5	13	0.0	40
		9.1	126	Sat.	--	--	0.0	35
		8.8	130	Sat.	3.3	14	0.0	43
		8.4	121	Sat.	1.5	10	0.0	42
14. Raystown Dam, PA, 3 miles SE of Huntington	Mixture of crushed siltstone, shale, and sandstone	3.0	124	Sat.	--	--	0.0	37
		3.1	116	Sat.	--	--	0.0	34
	Mixture of shale, siltstone, and sandstone from test fills	7	128 to 142	Sat.	--	--	0.0	33 to 39
15. Trexler Lake, PA, 12 miles NE of Allentown	Brown-gray, broken shale from test pit	2.8	114	Sat.	1.15	16	0.65	43.5
	Brown-gray, broken, clayey shale from test pit	2.1	115	Sat.	0.65	19	0.65	45
	Brown-gray, broken, silty, clayey shale from test pit	2.5	122	Sat.	0.8	20	0.60	45

Route 128, samples 12 and 15) or development of saturation pore pressure (e.g., Kentucky samples 1 and 3 with high initial water contents). The limits of strength parameters shown in Figure 19 by dashed lines are compared in Figure 20 with strength parameters for the CE projects listed in Table 6 (extracted from Table 14, Vol 1 report). The CE data in Figure 20 indicates that significant strength reductions can occur upon saturation of compacted shales. The range of c and ϕ values for the embankment samples includes the majority of values for saturated shales, with maximum particle sizes from 1 to 3 in. and dry densities from 113 to 130 pcf. Comparison of Figures 19 and 20 also indicates that the drained strength ϕ' values for the Kentucky and Indiana shale specimens are significantly lower than for any of the shales used in the CE project tests. However, the drained strength c' and ϕ' values for the Kentucky and Indiana samples are within the range for the EDCE test results shown in Table 2. Also, the Kentucky specimen ϕ' values compare with ϕ' values from recent tests on compacted shale by Abeyesekera (1977).

Summary of Embankment Properties

62. Properties for each of the six shale embankments investigated are summarized in Figures 21-26. Each figure shows an abbreviated log and plot of the following data versus depth:

- a. In situ water content and dry density.
- b. Estimated horizontal earth pressure from the K_o triaxial compression tests and pressuremeter tests.
- c. Estimated modulus values from the triaxial compression tests and pressuremeter tests.
- d. Estimated in situ shear strengths from triaxial compression tests and pressuremeter tests.
- e. Shear strength parameters, c and ϕ from laboratory tests and pressuremeter test results for which an interpretable stress-path plot was developed.

Available data from previous investigations by others are also shown on the plots in Figures 21, 22, and 26, for the Tennessee, West Virginia, and Indiana embankments.

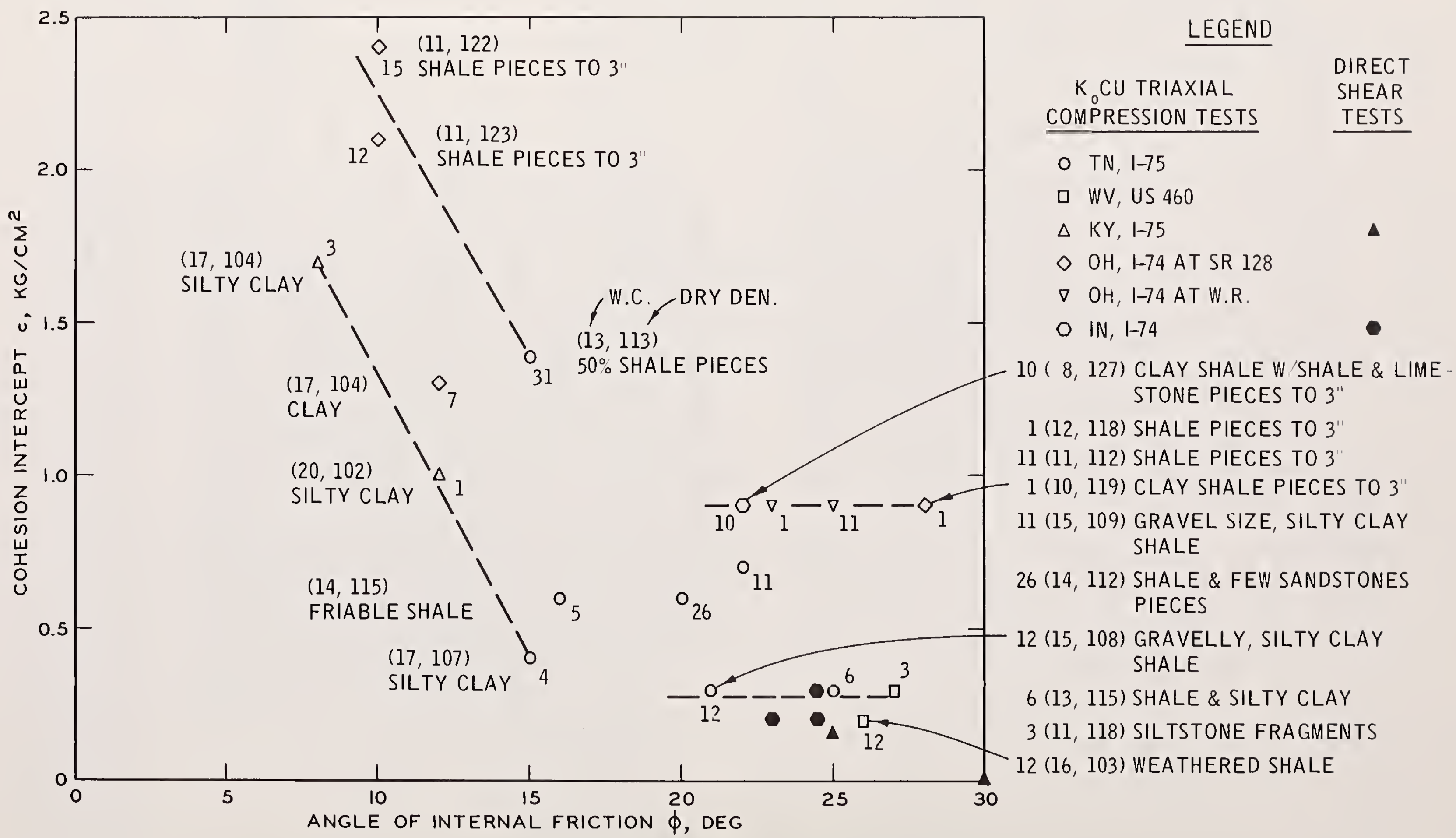


Figure 19. Range of strength parameters (c,ϕ) for embankment samples

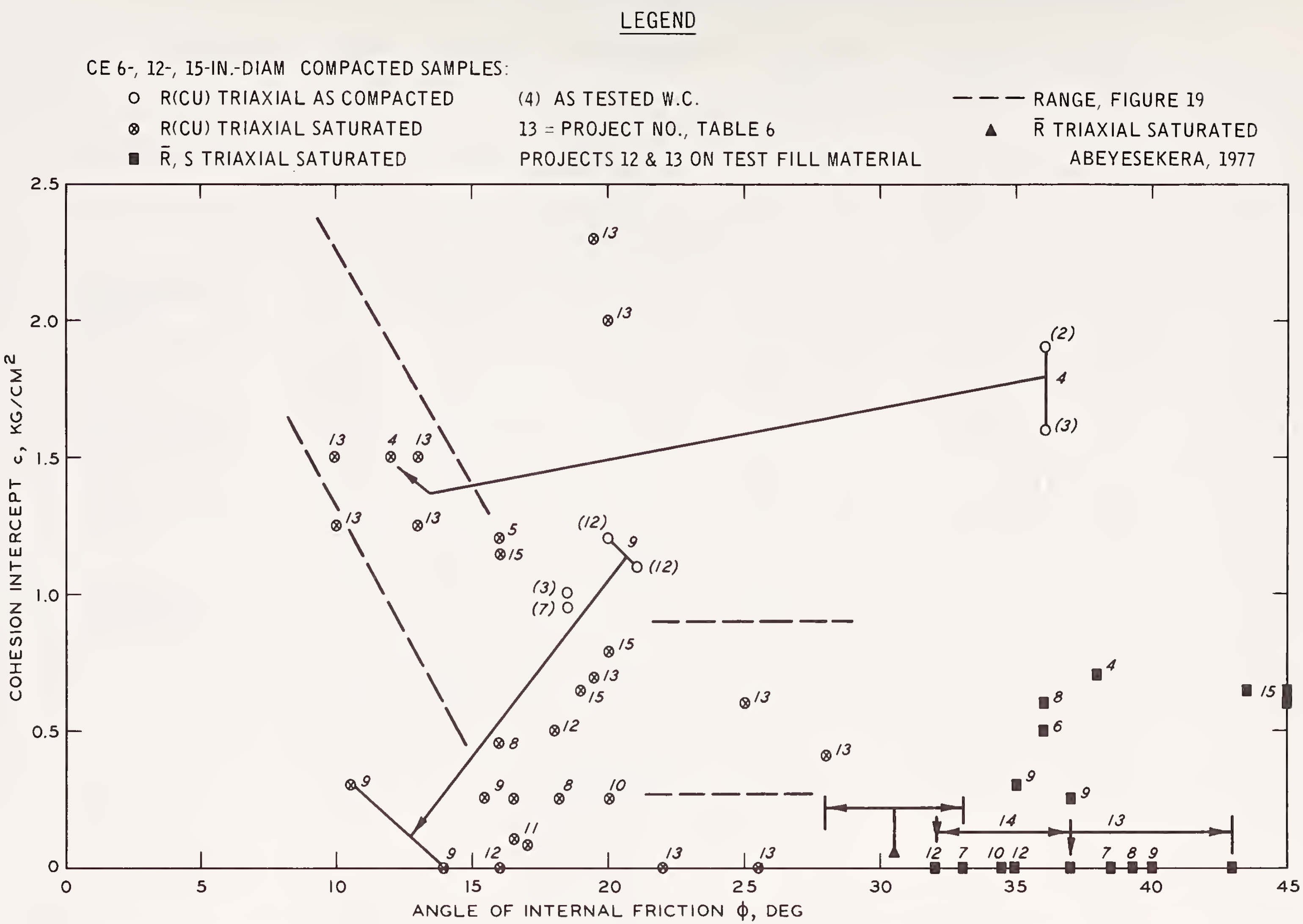


Figure 20. Comparison of strength parameters (c,ϕ) for saturated, compacted shales

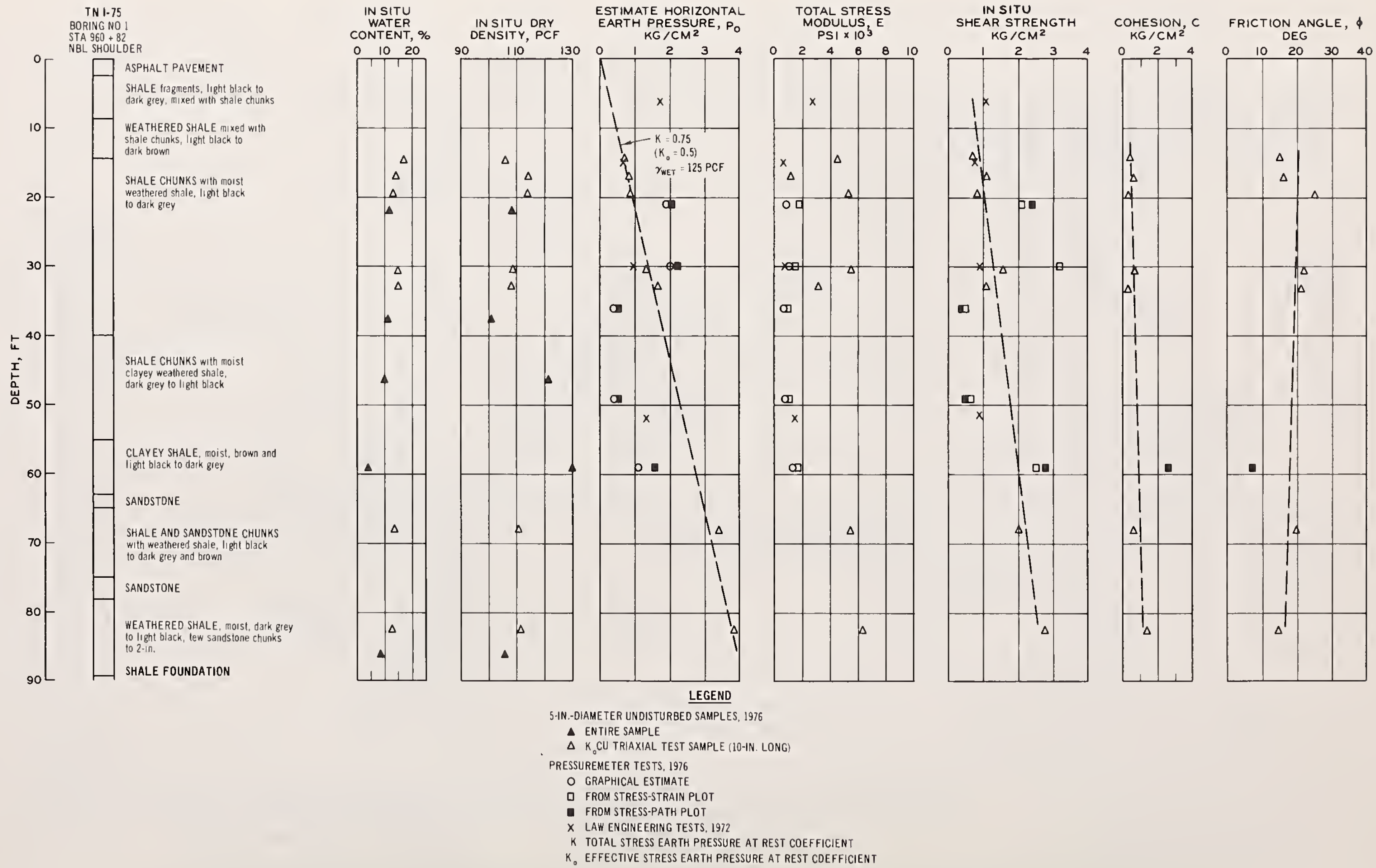


Figure 21. Summary of embankment properties, Tennessee I-75, sta. 960+82

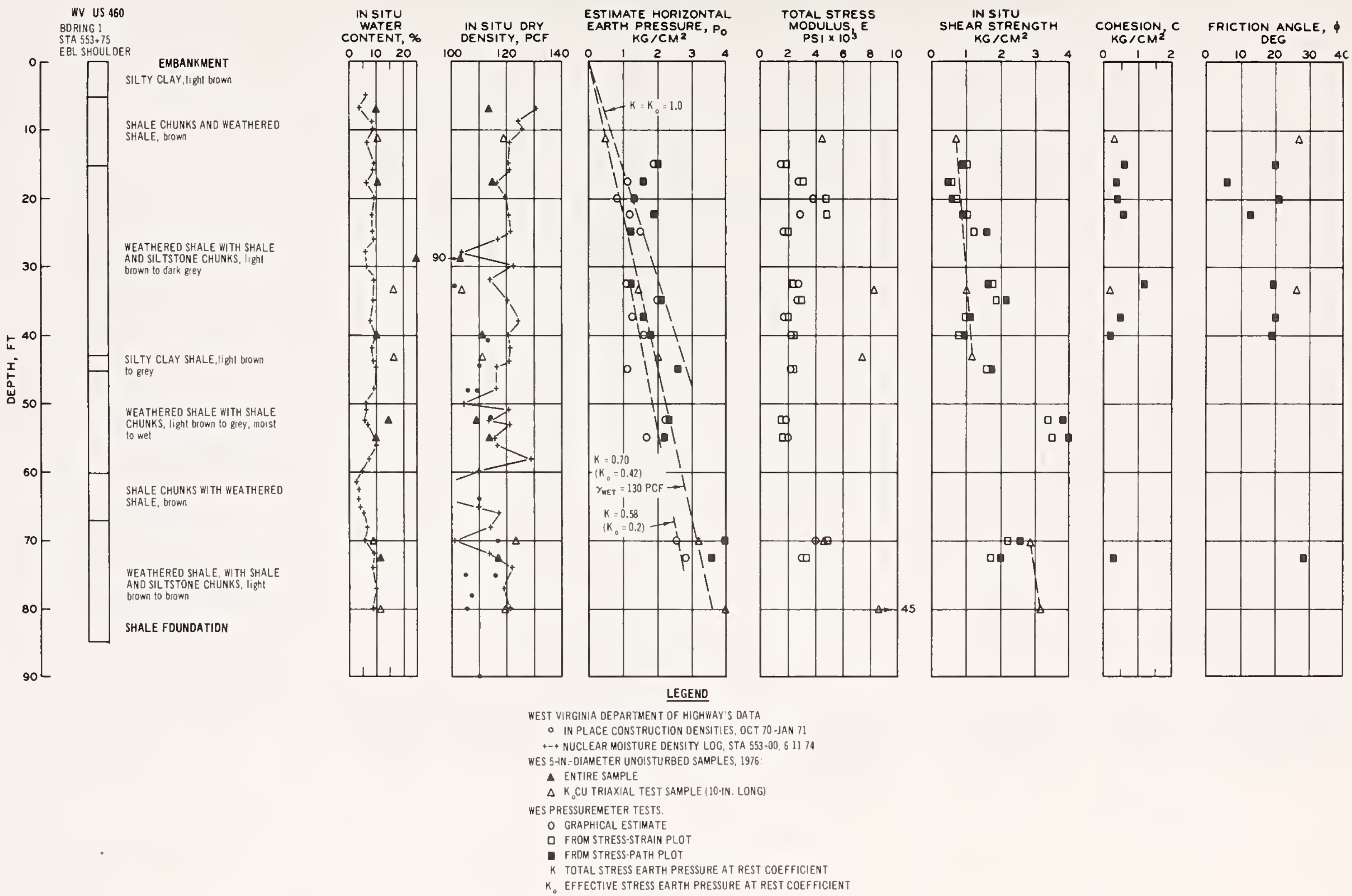


Figure 22. Summary of embankment properties, West Virginia, U. S. 460, sta. 553+75

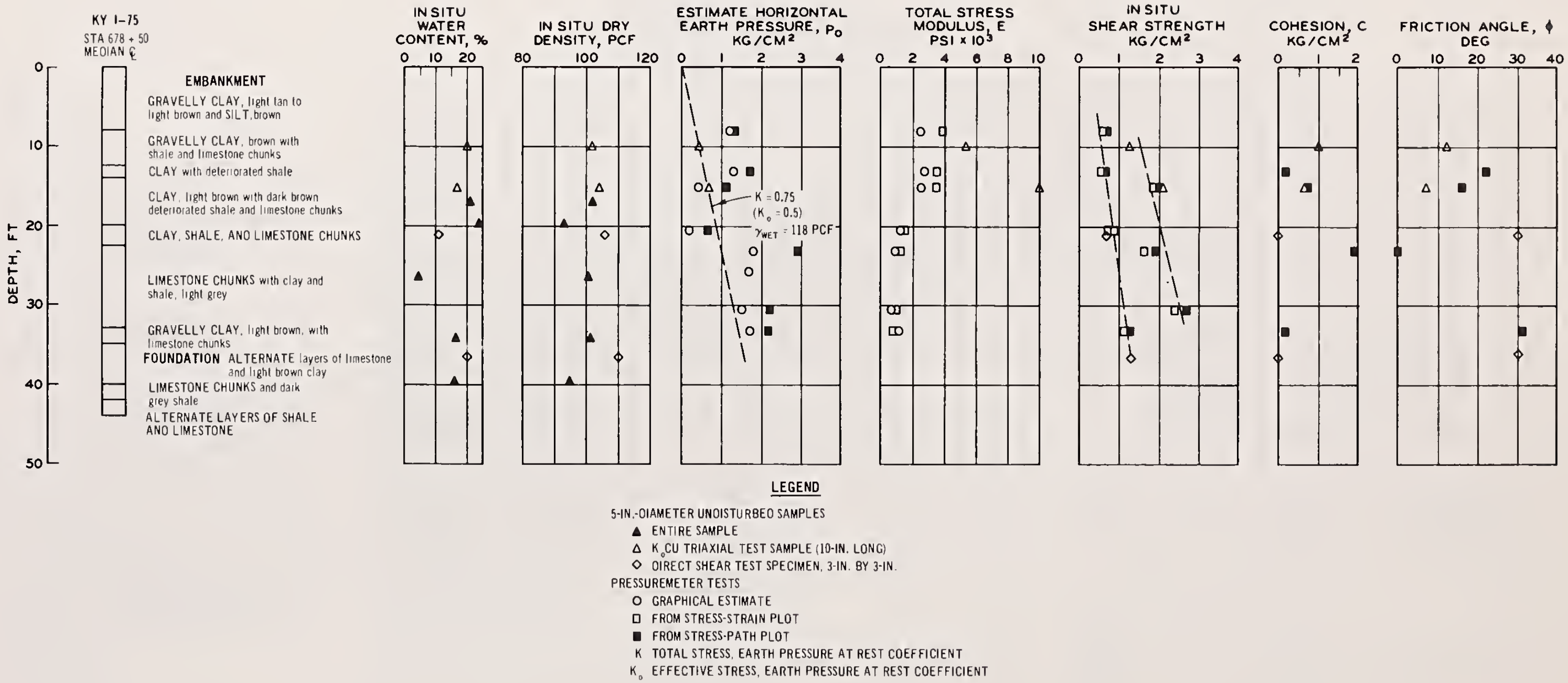


Figure 23. Summary of embankment properties, Kentucky I-75, sta. 678+55

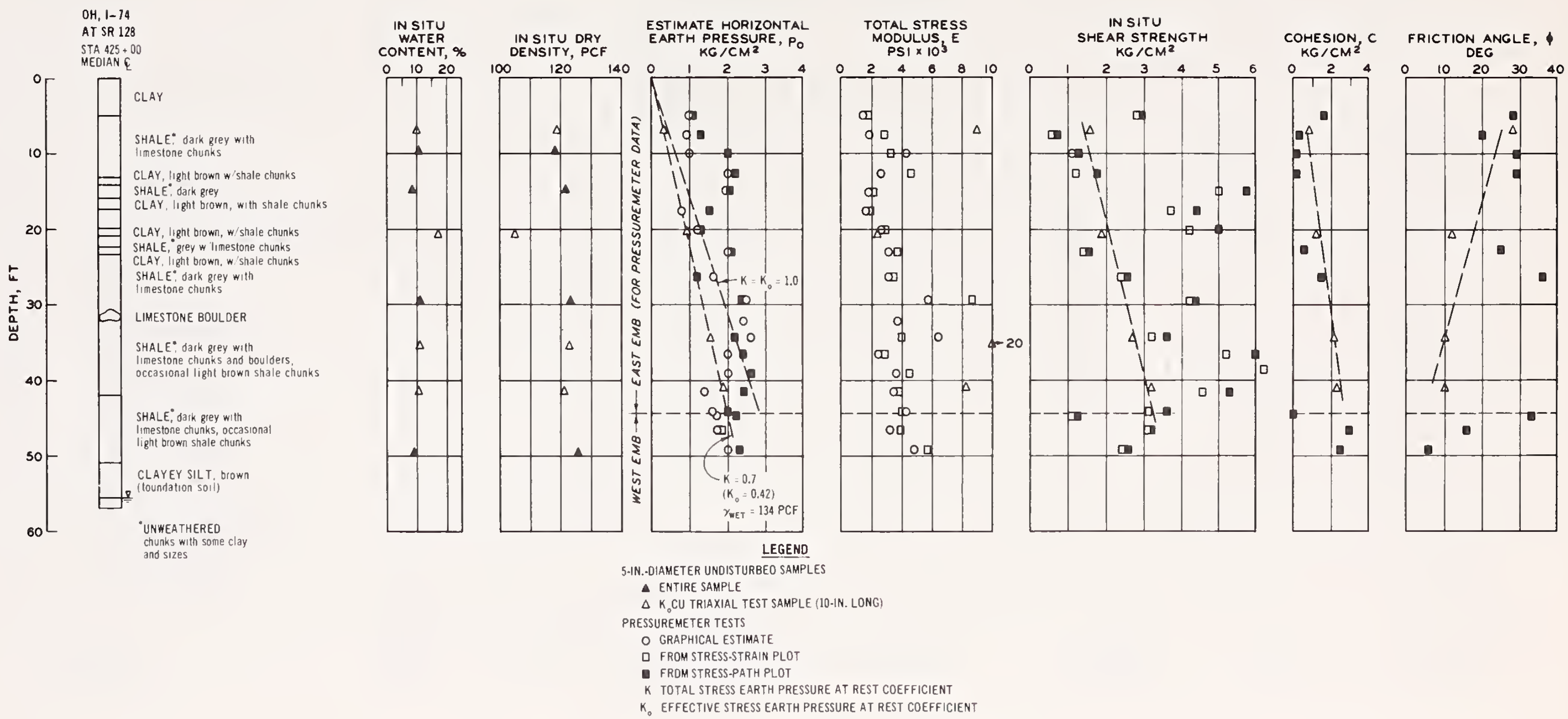


Figure 24. Summary of embankment properties, Ohio I-74 at SR 128, sta. 424+90

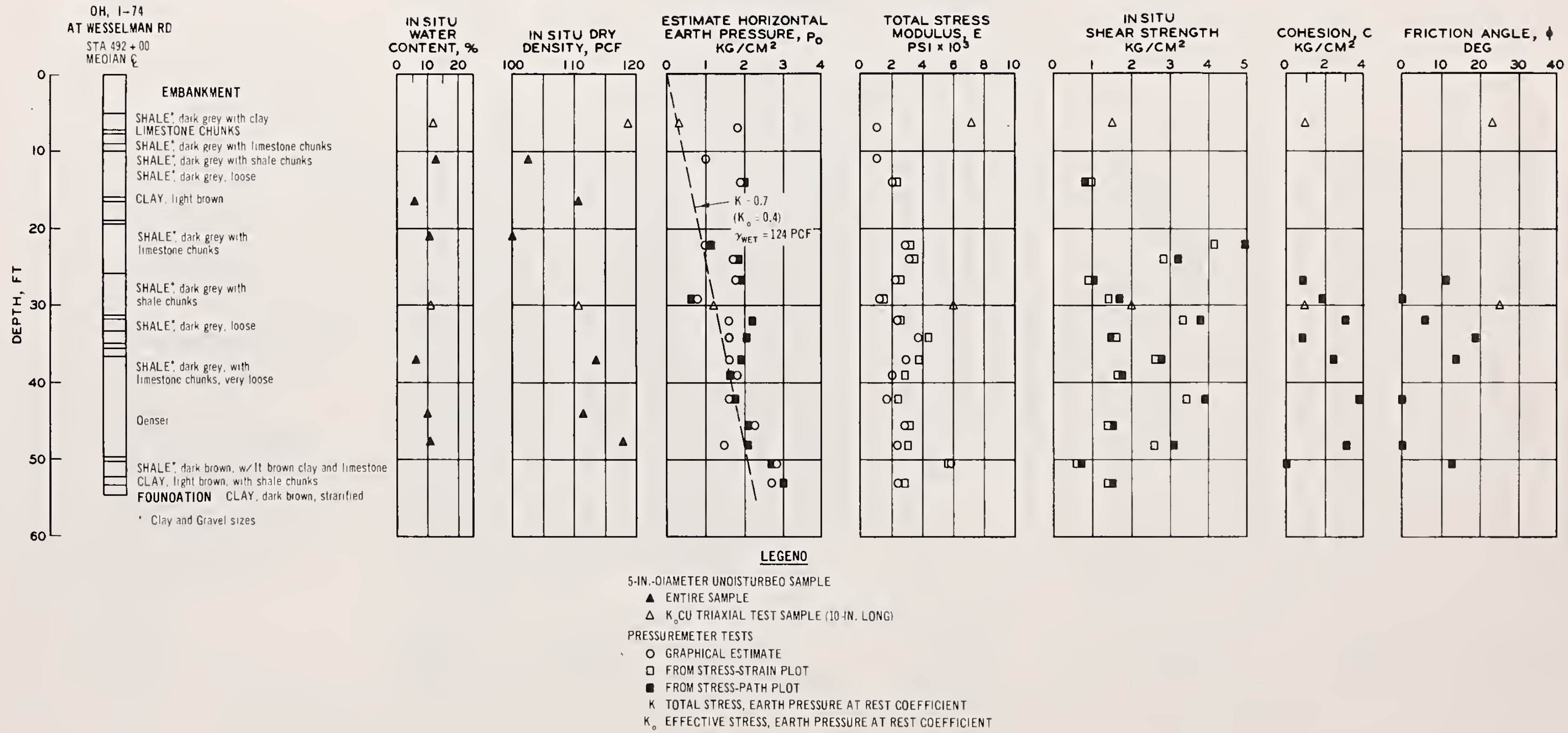


Figure 25. Summary of embankment properties, Ohio I-74 at Wesselman Road, sta. 492+00

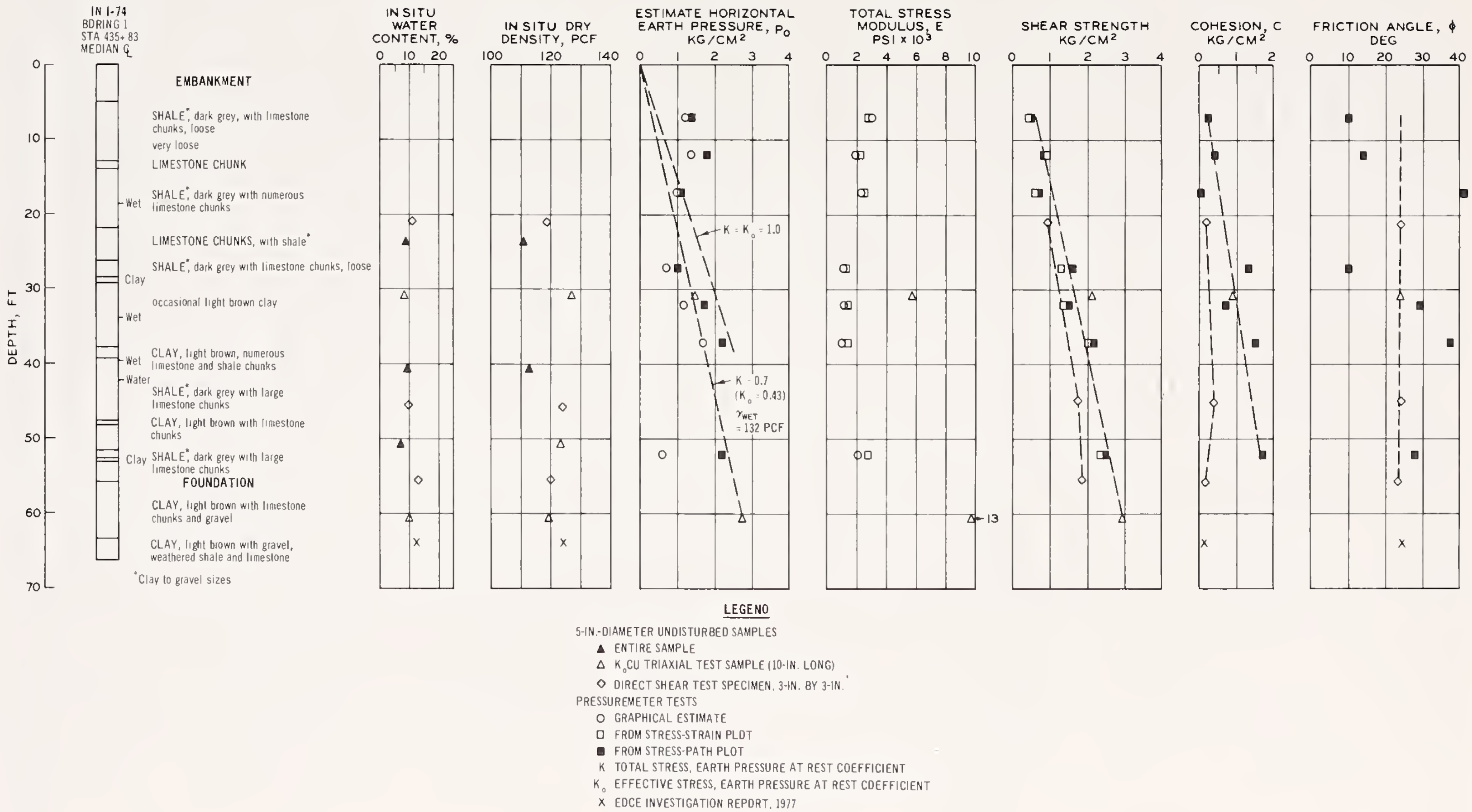


Figure 26. Summary of embankment properties, Indiana I-74, sta. 435+83

Tennessee I-75 embankment

63. Water content and density. The in situ water content and dry density values in Figure 21 are based on the total samples and reflect the combined effect of dense, intact pieces of shale, etc., and a loose to dense, soillike matrix. The sample with a density of 130 pcf included a 4-in.-long section of intact shale (paragraph 56). The general range of dry densities from 110 to 115 pcf about ten years after construction (Table 1) is still relatively low, considering that AASHTO-T99 compaction of coarse-grained shale (Part IV) produced dry densities of 115 pcf or higher.

64. Horizontal earth pressure. The total stress horizontal earth pressure was estimated from K_o triaxial test data using measured values of K_o and the following relationship:

$$K = K_o - (1 - K_o) \frac{\gamma water}{\gamma wet}$$

where

K = total stress horizontal earth pressure coefficient

K_o = effective stress horizontal earth pressure coefficient

γwater = unit weight of water

γwet = unit wet weight of embankment material.

Although the above relationship is valid for saturated materials, it is used as an approximation in the following. The average increase in total horizontal earth pressure, p_o (depth · γwet · K) versus depth, is shown by the dashed line (K-line) in the p_o plot in Figure 21. The total stress horizontal pressure (K-line) was used to evaluate the validity of p_o estimated from pressuremeter tests (total stress type test). Thus values of p_o from pressuremeter tests that are significantly higher than p_o indicated by the K-line are unrealistic, probably because the probe was pushing against a large intact cobble or boulder of shale, siltstone, etc. The pressuremeter tests were used to provide supplemental information on in situ properties of the embankments. It was also desirable to evaluate the pressuremeter tests as a low-cost means of evaluating existing shale embankments exhibiting distress. Valid shear strengths from pressuremeter tests require a reasonably good estimate of p_o. Consequently, it was important to establish a means, such as the K-line, for evaluating estimated values of p_o from the pressuremeter test. For the Tennessee embankment, estimated p_o values from pressuremeter tests were either considerably higher (perhaps because of large shale pieces) or considerably lower (perhaps because of soft zone) than the values indicated by the K-line. Two of the p_o values from the 1972 Law Engineering pressuremeter tests in an adjacent boring were close to values indicated by the K-line.

65. Modulus values. The total stress tangent modulus (E) values from K_o triaxial compression tests ranged from about 1000 to 6000 psi and suggested an increase in modulus with depth. Modulus values from pressuremeter tests, which cause a major stress increase in the horizontal direction, are usually about one-third those from axially loaded laboratory samples. In Figure 21, the pressuremeter modulus values were all close to 1000 psi, similar to values reported in the 1972 Law Engineering investigation report, and approximately one-fifth the trend of values indicated by K_o triaxial compression tests.

66. Shear strength. The in situ undrained shear strength from K_o triaxial tests was based on the first stage axial loading after reconsolidation to the estimated in situ overburden pressure (see figures in Appendix B). The in situ shear strength plot in Figure 21 shows an increase in shear strength with depth. Estimated strengths from four pressuremeter tests (which produced reasonable stress-strain and stress-path plots (Appendix A)) show considerable scatter. The shear strength parameters (c, ϕ) in the last plot in Figure 21 indicate an increase in cohesion with depth and a slight decrease in ϕ. The one pressuremeter test that defined an interpretable stress-path plot for c, ϕ gave what appears to be too high a value for cohesion and too low a value for ϕ.

West Virginia U. S. 460

67. Water content and density. The plots shown in Figure 22 for the West Virginia U. S. 460 embankment indicate water contents of 10 to 16 percent, which are higher than water content ranges indicated by the

nuclear moisture log (from Figure 7). Dry densities generally of 110 to 120 pcf are equal to or less than those indicated by the nuclear density log, except at a depth of 70 ft. The embankment sample densities are similar to in-place construction densities (94 to 102 percent of maximum dry density for the minus 3/4-in. fraction) for the depth interval of 32 to 55 ft and generally higher from 70 to 80 ft. No conclusive increase in density since construction is apparent because of the variability inherent in the heterogeneous nature of embankment materials. However, the nuclear moisture density measurements could be more representative since they include a larger volume than the individual samples and have indicated an overall increase in density with time (paragraph 26).

68. Lateral earth pressure. Values of horizontal earth pressure (Figure 22) from pressurementer tests are reasonably close to the K-line, with graphical estimates considerably lower than estimates from a stress-path plot. High values of p_o from pressuremeter tests probably reflect the influence of large shale or siltstone chunks. The lower average value of $k_o = 0.42$ indicates a stiffer material compared with the Tennessee I-75 embankment.

69. Modulus. The modulus values from triaxial tests, indicating a range of 4000 to 8000 psi, are about two to four times the pressuremeter modulus values, except at a depth of 70 ft.

70. Shear strength. Estimated in situ shear strengths from triaxial tests show a slight increase with depth and correspond with lower values from pressuremeter tests to a depth of 45 ft. Estimated cohesion values from pressuremeter tests are slightly higher than those from two triaxial tests in which more than one stage of loading was possible. However, ϕ values estimated from pressuremeter tests were considerably lower than those from the two triaxial tests.

Kentucky I-75

71. Water content and density. Water contents (Figure 23) in the upper 20 ft were around 20 percent, while those below 20 ft tended to be lower. Densities generally between 100 pcf and 110 pcf do not show any significant difference in the upper 20 ft compared to the lower 20 ft, which contain predominantly shale and limestone chunks (paragraph 34).

72. Earth pressure and modulus. Values of p_o from pressuremeter tests are higher than the K-line values and indicate a higher range in the lower 20 ft. The relatively high K_o value (0.5) indicates a softer soil structure. Modulus values estimated from pressuremeter tests are higher in the upper 20 ft and lower than values estimated from the two triaxial tests.

73. Shear strengths. Estimated in situ shear strengths (Figure 23) appear to group into separate ranges for undrained and drained conditions. The lower range defined by the two direct shear tests might also represent in situ strengths from pressuremeter tests on free-draining material. The higher range defined by strengths from the two triaxial tests might represent undrained strengths (high c and low ϕ values) for which the shear strength at lower normal stresses is higher than drained strength (zero or small c and high ϕ values). The estimated strength parameter (c,ϕ) from the triaxial tests and from pressuremeter tests at depths of 15 and 23 ft have relatively high c values and low ϕ values compared to the other values from direct shear tests and from pressuremeter tests at depths of 13 and 33 ft.

Ohio I-74

74. Data for the two bridge approach embankments, Ohio I-74 at State Route 128 (Figure 24) and Ohio I-74 at Wesselman Road (Figure 25), indicate some significant differences. For the well-compacted embankment at State Route 128, densities, modulus values, and shear strengths are higher than at Wesselman Road. The highest strength values from pressuremeter tests were probably caused by large shale chunks.

Indiana I-74

75. The data for the Indiana embankment shown in Figure 26 indicate relatively high densities which include the influence of considerable limestone. However, relatively low modulus values and the consistent trend for in situ strengths indicate the dominant influence of the softer clay and clay shale matrix. Strengths from the two direct shear tests define the lower boundary for in situ strengths, with c' and ϕ' values similar to those determined in a comparable EDCE test (Table 2).

Discussion

76. Properties of the six embankments vary widely, as might be expected from the heterogeneous mixture of materials. However, trends are evident for densities, modulus, and in situ shear strengths of the different embankments. Pressuremeter results provide useful supplemental information, provided sufficient tests are made to establish trends. The two major properties of interest are compressibility and shear strength.

77. Compressibility. The three embankments placed as rockfill in thick lifts (Tennessee I-75, Ohio I-74 at Wesselman Road, and Indiana I-74) exhibited total settlements of about 2 ft. A simple analysis was made to determine the range of modulus values required for settlements as large as 2 ft. The analysis used influence charts developed by Poulos (Poulos, Booker, and Ring, 1972)* and reproduced in Elastic Solutions for Soil and Rock Mechanics (Poulos and Davis, 1974).** These influence charts are based on construction sequence finite element solutions for three embankment slopes (20, 30, and 40 deg) and two values of Poisson's ratio (0.30 and 0.48). As shown by the curves in Figure 27, modulus values of 1000 psi or less are required to predict settlements of 2 ft. As shown by the dashed line in Figure 27, minimum values of modulus from pressuremeter tests correlate directly with measured settlements perhaps fortuitously. Since the higher laboratory measured modulus values reflect existing conditions, it could be argued that the curves in Figure 27 are applicable to future settlements that might amount to 1 to 4 in. for the Ohio, Indiana, Tennessee, and West Virginia embankments. On this basis, the modulus values at the time of construction for the Indiana, Tennessee, and West Virginia embankments would have been extremely low (less than 1000 psi), which seems unlikely. Thus, while the use of simplified charts for predicting settlement is

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* Poulos, H. G., J. R. Booker, and G. J. Ring, "Simplified Calculations of Embankment Deformations," Research Report R192, The University of Sidney, May 1972 (NTIS PB-218-376).

** Poulos, H. G., and E. H. Davis, Elastic Solutions for Soil and Rock Mechanics, John Wiley and Sons, Inc., 1974.

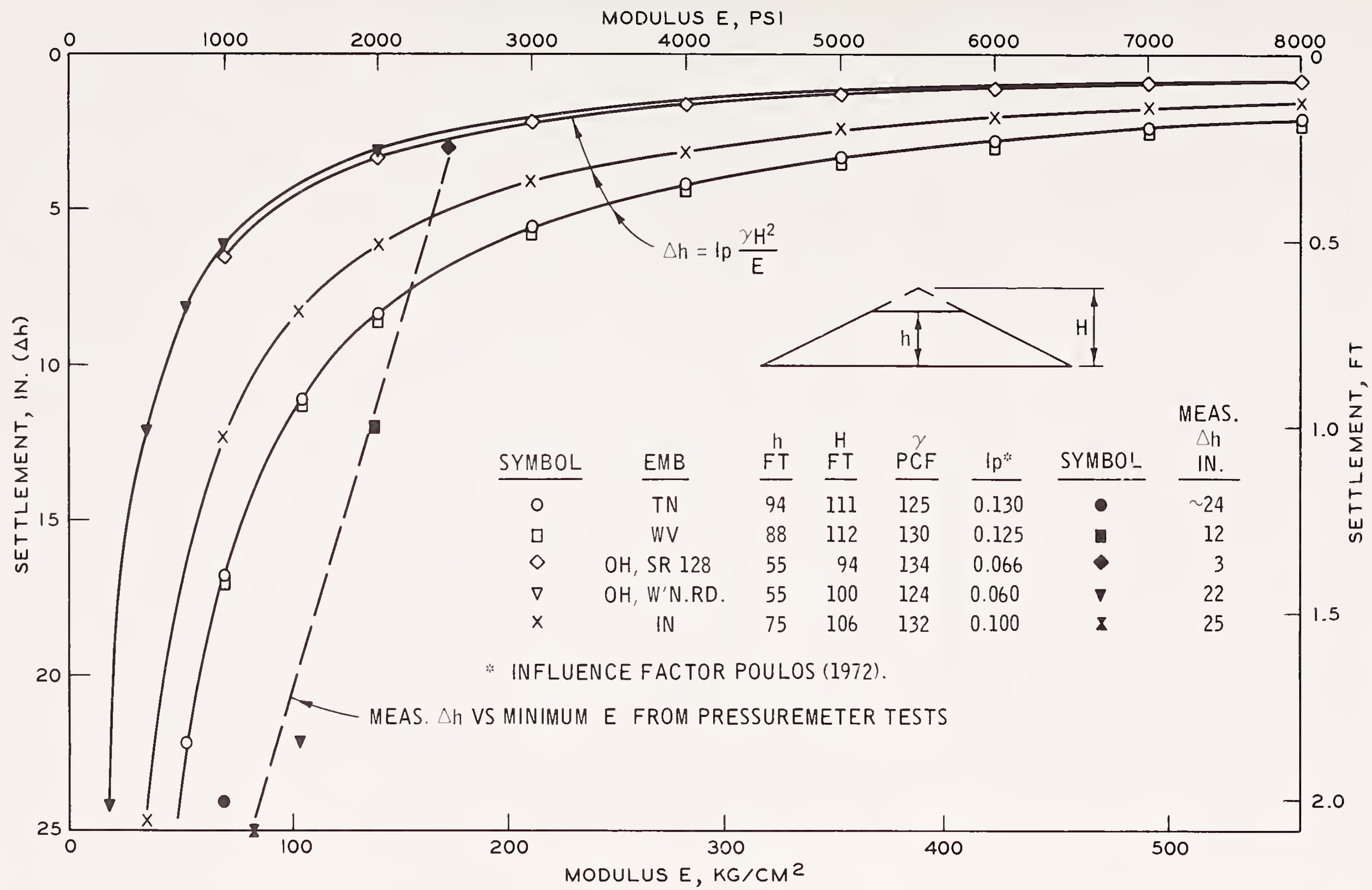

Figure 27. Settlement versus modulus

attractive, the results do not appear to be applicable for shale embankments.

78. Shear strength. The existing shear strength of the shale embankments studied correlates with the undrained shear strength from laboratory tests on saturated, compacted shales. Thus the long-term shear strength of compacted shale embankments can be estimated from undrained laboratory tests on compacted, saturated samples. Consolidated drained direct shear tests on laboratory compacted and saturated samples for CE projects produce shear strengths that are too high for long-term shear strength predictions.

Improvements in Pressuremeter Testing

79. Many of the pressuremeter tests were not valid because the hole was too large. Wobble in the drill rod string caused oversize sections in some borings. A large volume increase was required to push the expanding probe membrane against the borehole wall. The capacity of the volumeter and expansion of the membrane was reached before the test could be carried to completion. A better test procedure would be to drill the 2-15/16-in.-diameter hole in 7-ft increments and perform two tests 3 ft apart. This procedure would also minimize squeezing-in of the hole before testing. If drilling mud was required to prevent caving and to facilitate testing below depths of 100 ft (see Appendix A), hole advancement and testing by increments would minimize softening of shales by the drilling mud.

PART IV: UNWEATHERED PARENT SHALE TESTS

80. This part presents the results of tests on the unweathered parent shale samples from the embankment sites. The shales were used to determine jar slake indexes and slake-durability indexes and to develop the following:

- a. Compaction test procedures for shales containing large shale pieces.
- b. An index compression test on compacted, soaked shale samples for use in predicting long-term settlement.

Point load index tests were also performed on intact shale pieces to study the usefulness of this test for field identification of shales of different durability.

Jar Slake and Slake-Durability Index Tests

81. The jar slake index and slake-durability index for unweathered shales from the six embankment sites was determined using the two-cycle procedures described in Volumes 1 and 3 and the commercial apparatus shown in Figure 28. The results reported in Volume 3 are shown in Table 7 for convenience.

82. The parent shale from two cuts east of the Tennessee I-75 embankment was hard and considered to be a rocklike shale on the basis of the index tests. The shales and siltstone from the West Virginia U. S. 460 embankment site varied from hard to soft, and material from barrels 1 and 4 (WV 4 and WV 7) and barrels 2 and 3 (WV 5 and WV 6) were combined (on the basis of similar slake-durability index values) for the compaction tests.

83. The available material from the cut adjacent to the Kentucky I-75 embankment was predominantly weathered clay shale with only a few pieces of unweathered shale. Consequently, only a normal compaction test (AASHTO T99) was performed on this material.

84. The parent shales from the two Ohio I-74 embankments were very similar and had a low slake-durability index indicating soillike

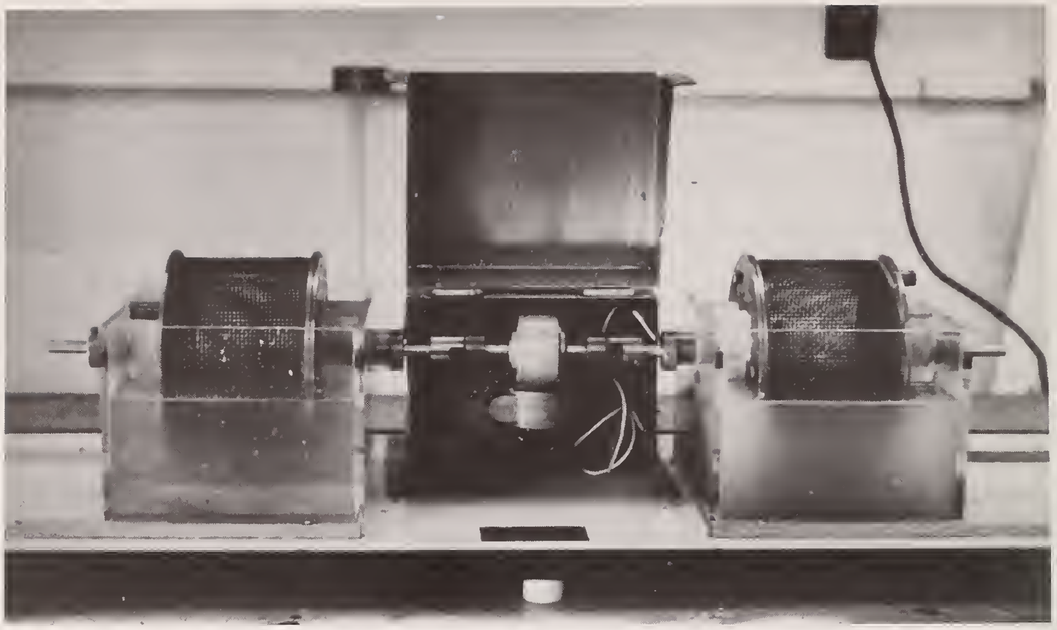

Figure 28. Slake-durability test apparatus

material. The Indiana I-74 parent shale had the lowest slake-durability index, although the in situ material was hard and difficult to excavate because of closely spaced limestone strata, 1 to 2 in. thick (paragraph 44).

Point Load Tests

General

85. A limited series of point load tests were performed on sawed cubes of hard parent shale from the Tennessee embankment site and irregular pieces of a medium-soft shale and sawed cubes of a harder siltstone from the West Virginia embankment site. The purpose was to see if an expedient field test such as the point load test could be used as an index to distinguish shales of different durability. The point load test apparatus used is shown in Figure 29 and is available from Terrametrics, Golden, Colorado. It was necessary to obtain lower range pressure gages and to develop a calibration curve of pressure (psi) versus load (pounds force) for each pressure gage with a load cell in the point load tester.

Table 7. Description, Jar Slake, and Slake-Durability Tests, Parent Shale Samples

A. Shale Sample Descriptions (from Vol 3)

Test Sample Number	Color	Structure	Fragment Shape	Texture (other than shaley)	Formation	Geologic Age
TN 20	Dark	Bedded	Tabular	Sandy	Slatestone (Hance)	Pennsylvanian
TN 21	Gray	Bedded	Irregular	Silty	Slatestone (Hance)	Pennsylvanian
WV 4	Light olive gray	Indistinct	Tabular	---	Hinton	Mississippian
WV 5	Olive gray	Indistinct	Tabular	---	Hinton	Mississippian
WV 6	Brownish-black	Laminated	Irregular	---	Hinton	Mississippian
WV 7	Pale brown	Laminated	Irregular	Sandstone	Hinton	Mississippian
WV 8	Medium dark gray	Indistinct	Irregular	Silty	Hinton	Mississippian
KY 31	Dark gray	Indistinct	Irregular	---	Breathitt	Pennsylvanian
KY 32	Olive black	Fissile	Platy	---	Breathitt	Pennsylvanian
OH 8	Dark greenish-gray	Indistinct	Tabular	---	Kope	Ordovician
OH 9	Greenish-gray	Indistinct	Tabular	---	Kope	Ordovician
IN 12	Greenish-gray	Faintly bedded	Irregular	Sandy	Dillsboro	Ordovician

B. Jar Slake and Slake-Durability Tests

Test Sample Number	Water Content Before Test, percent				Jar Slake Index, I_J, dry	Slake-Durability Index, I_D, percent				pH of Water after I_D Test		
	1	2	3	Avg.		1	2	3	Avg.	Cycle 1	Cycle 2	Avg.
TN 20	3.0	3.1	---	3.1	6	96	96	---	96	---	---	---
TN 21	2.0	1.9	---	2.0	---	98	98	---	98	8.0	8.0	8.0
WV 4	7.8	8.1	---	8.0	2	74	71	---	73	7.9	---	7.9
WV 5	8.1	7.6	---	7.8	2	52	59	---	56	7.9	8.0	8.0
WV 6	6.3	5.3	---	5.8	4	29	47	---	38	7.5	7.4	7.5
WV 7	7.9	7.7	---	7.8	6	86	88	---	87	7.5	7.6	7.5
WV 8	2.2	2.2	---	2.2	6	97	94	---	96	7.8	8.0	7.9
KY 31	9.4	7.4	8.8	8.5	3	40	59	49	49	7.4	7.6	7.5
KY 32	6.2	5.9	---	6.1	6	92	91	---	91	---	---	---
OH 8	7.4	7.6	7.7	7.6	2	40	21	13	24	7.0	7.9	7.5
OH 9	9.4	9.4	---	9.4	3	31	33	---	32	---	---	---
IN 12	7.3	6.2	---	6.8	2	10	6	---	8	---	---	---

Figure 29. Hand-held point load tester

Procedure

86. Brook (1977)* showed that a straight line was obtained on a log-log plot of point load versus fracture area for cores, disks, and irregular pieces of the same rock. He proposed defining the point load strength index, T_{500} , as the tensile stress at an area of 500 mm^2 (i.e., T_{500} = point load at area of 500 mm^2 divided by 500 mm^2). This is in contrast to the International Society of Rock Mechanics standard for point load index based on a 50-mm-diameter core size (1963 mm^2).

87. On the basis of Brook's procedure, pieces of the shales were tested at various water contents from 3 percent (by soaking the hard shale) to about 6 percent (for the softer shale) to simulate field water contents. The shale pieces were loaded perpendicular to bedding, and the fracture area was traced on paper and measured with a digital planimeter. An example of the data sheet for one test is shown in Figure 30.

Results

88. The results of the point load tests are summarized in Figure 31. This plot of point load versus fracture area indicates a relatively good correlation for the hard Tennessee shale (TN 20) and the softer West Virginia shale (WV 6). However, the harder West Virginia siltstone (WV 8) showed considerable scatter and ranged between the hard Tennessee shale and the softer West Virginia shale. Values of I_p show considerable scatter for individual tests; and the average values for the TN 18 shale specimens and the WV 8 siltstone specimens, both with slake-durability index, I_D = 96 percent, are significantly different. For a point load index I_p** (based on an area of 1963 mm^2), the two shales (TN 20 and WV 6) with different I_D values do not show a significant difference. Using a point load index based on an area of 500 mm^2, as Brook suggests, would require specimens with an equivalent

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* Brook, N., "The Use of Irregular Specimens for Rock Strength Tests," International Journal of Rock Mechanics, Mining Science, and Geomechanics, Vol 14, 1977.

** Donaghe, R. T., and F. C. Townsend, "Scalping and Replacement Effects on the Compaction Characteristics of Earth Rock Mixtures," Special Technical Publication No. 599, ASTM, June 1976.

SUBJECT	COMPUTED BY:	DATE:	FILE NO
POINT LOAD TEST	CHECKED BY:	DATE:	SHEET NO.

SAMPLE SOURCE: WV 460 B. 2&3 SPECIMEN NO.: WV 5-3 (I_D = 37)

DESCRIPTION: Drk Br Shale laminated; saturated

TYPE SPECIMEN: CORE( )
IRREGULAR(x)

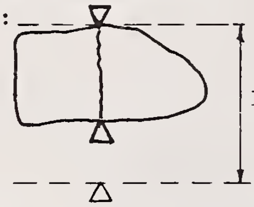
DIMENSIONS: 30.5 (D) 50 (L) 60 (W) mm

POINT LOAD, P = 65 PSI x C = 1210* NEWTONS

PRESSURE GAUGE USG 300psi
CALIB. FACTOR A= ______
C = A/0.2248 = ______

LOAD APPLIED: PERPIN(x) PARALLEL ( ) TO BEDDING
NO APPARENT BEDDING ( )

SKETCH IN TESTER:



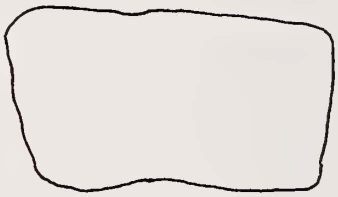
D = 30.5 mm
(Before Failure)
D^2 = 930 mm^2

POINT LOAD INDEX, $I_P = P/D^2$ = 1.3 MPa
$I_s(50)$ = ______ MPa

*CALIBRATION CHART:
P= 272 LBF/0.2248= 1210 N

FAILURE DESCRIPTION: Irregular vertical crack thru specimen

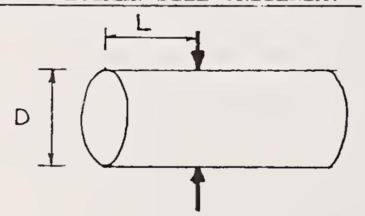
TRACED OUTLING OF FAILED SECTION:
AREA = 1400 mm^2



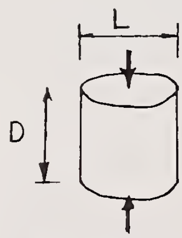
SPECIMEN WATER CONTENT:
TARE NO. 26

TARE + WET SPECIMEN: 159.7 GR
TARE + DRY SPECIMEN: 148.4 GR
WEIGHT OF WATER: 11.3 GR
TARE WEIGHT: 5.1 GR
DRY WEIGHT: 143.3 GR
WATER CONTENT: 7.9 %

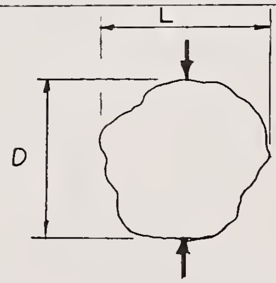
SPECIMEN SIZE CRITERIA:



L>0.7D
Diametrial Test
cores: D ≅ 50 mm preferable



$\frac{D}{L} = 1.1 \pm 0.5$
Axial Test



D ≅ 50 mm (2-in.)
$\frac{D}{L}$ = 1.0 to 1.4
Irregular Lump Test

Figure 30. Data sheet for point load test

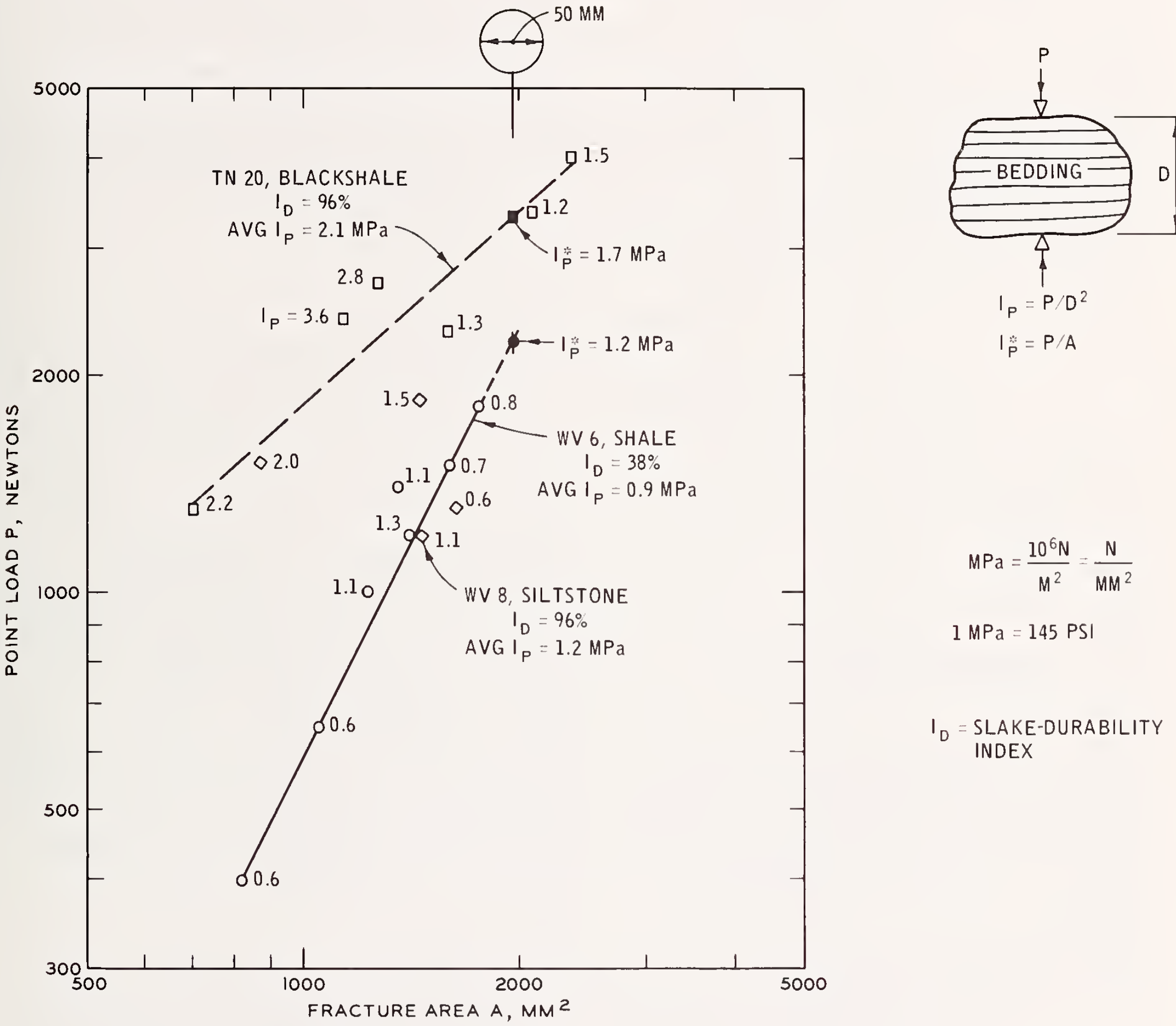


Figure 31. Results of point load tests

square size of less than 1 in. This small size is not considered applicable for shales because of the frequent spalling which occurred during the testing of larger specimens.

89. Based on the limited tests, it appears that a unique relationship between point load and fracture area (or average I_p value) would be required for each different shale (or siltstone, etc.) on a particular project. If a unique point load relationship could be established for a particular shale, then the slake-durability index for that shale could also be identified and used as a basis for determining whether the shale should be treated as soil or could be used as rockfill.

Compaction Tests

90. Unweathered shales from the West Viginia, Ohio, and Indiana embankment sites were broken down to 3-in. maximum size using the 25-lb drop hammer in a large, mechanical, impact compaction device described by Donaghe and Townsend (1976).* Gradations with 70 to 80 percent gravel size were prepared, and standard effort (AASHTO T99) compaction tests were performed in a 12-in.-diam mold. Various gradations of the minus 3/4-in. fraction of the harder West Virginia shale (barrels 1 and 4) were used for compaction tests in a 6-in.-diameter CBR mold to determine the gradations producing dry densities similar to those obtained in the 12-in.-diam mold. Test results are shown in Figures 32 and 33.

West Virginia U. S. 460

91. The gradations for the compaction tests on the harder parent shales (barrels 1 and 4) from the West Virginia embankment are shown in Figure 32a. Compaction test results (Figure 32b) indicated that scalping and replacement produced densities significantly lower than densities for the minus 3-in. material compacted in the 12-in.-diameter mold. The proportional gradation of minus 3/4-in. material was selected

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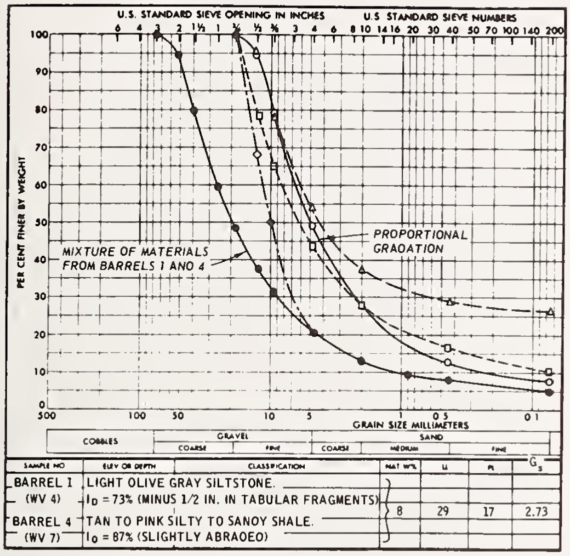
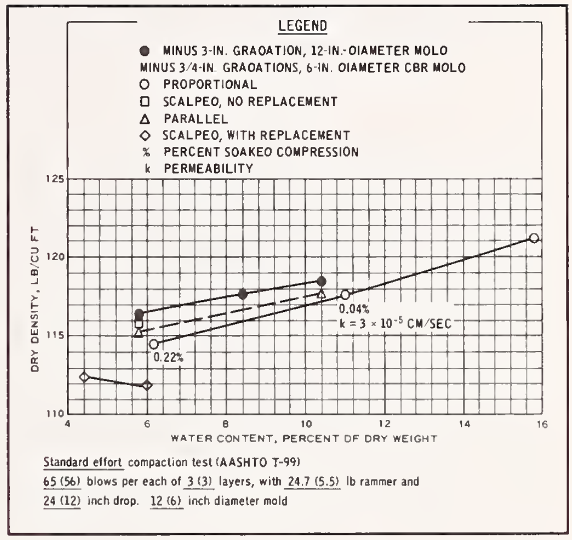
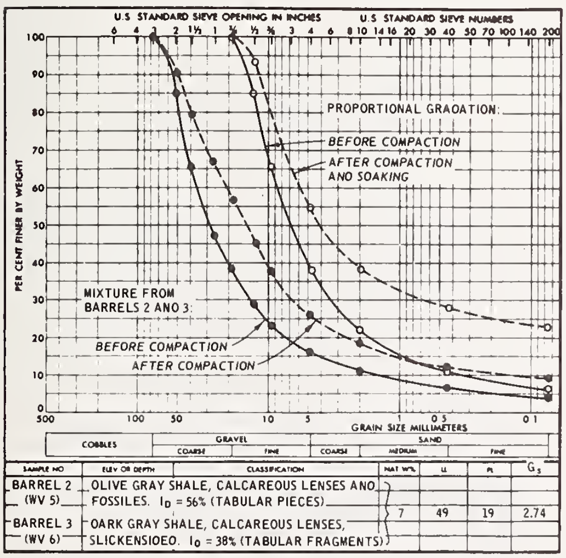
* Donaghe, R. T., and Townsend, F. C., "Scalping and Replacement Effects on the Compaction Characteristics of Earth Rock Mixtures," Special Technical Publication No. 599, ASTM, Jun 1976.



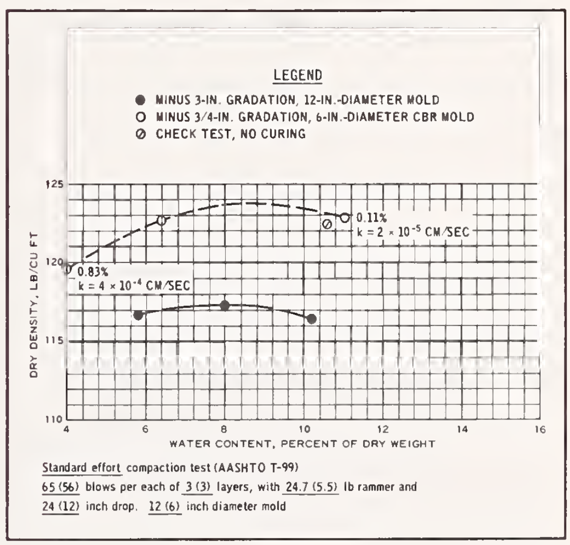
a. Gradation curves, barrels 1 & 4



b. Compaction results, barrels 1 & 4

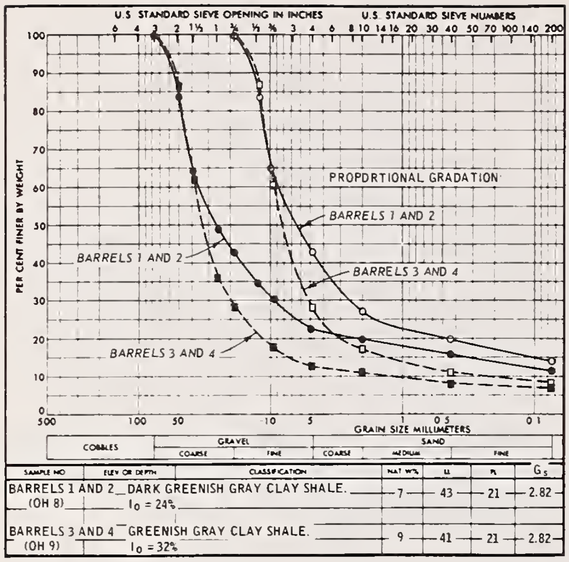


c. Gradation curves, barrels 2 & 3

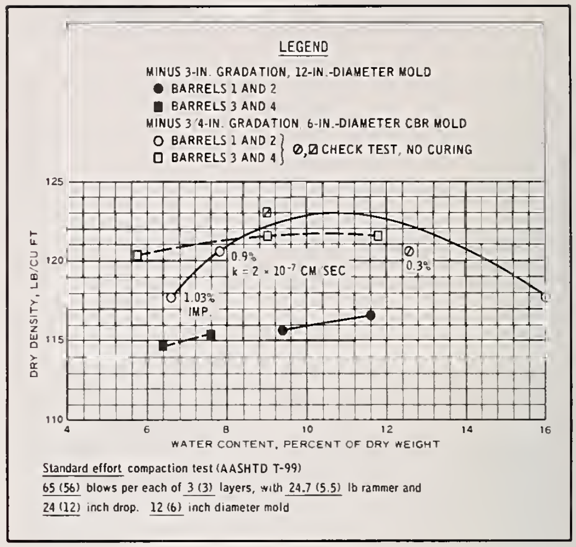


d. Compaction results, barrels 2 & 3

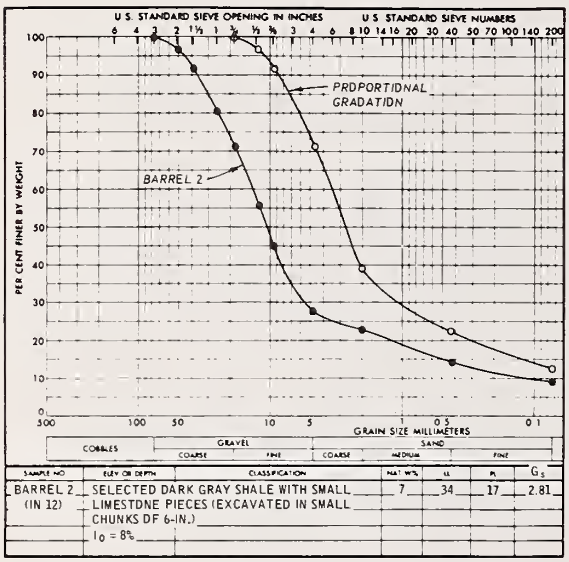
Figure 32. Gradations and compaction test results, parent shales, West Virginia, U. S. 460



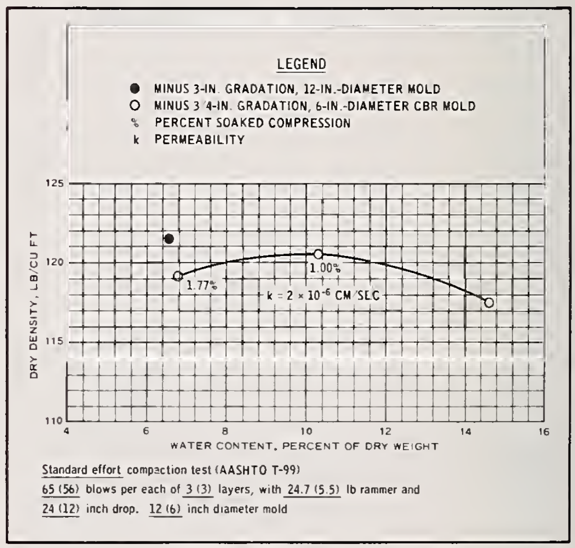
a. Gradation curves, Ohio I-74



b. Compaction results, Ohio I-74



c. Gradation curves, Indiana I-74



d. Compaction results, Indiana I-74

Figure 33. Gradations and compaction test results, parent shales, Ohio and Indiana I-74 sites

as standard for the other shales for the following reasons.

a. Densities were similar to those from tests in the 12-in.-diam mold.

b. The proportional gradation had the smallest percentage of minus No. 10 material and the highest coefficient of uniformity, Cu (ratio of D_{60} to D_{10}), which would better reproduce the seepage characteristics of the coarser material.

c. Proportional gradation for 1.4-in.-diam test specimens (minus No. 4 sieve material) has been shown to produce stress-strain, strength, and Poisson's ratio parameters similar to those from 36-in.-diam specimens of a minus 6-in. granular material (Nobari and Duncan, 1972).*

Also shown in Figure 32b are permeabilities and percent compression after soaking under a 1000-lb surcharge load (see paragraph 98).

92. Compaction test results for the softer parent shales (barrels 2 and 3) from the West Virginia embankment site are shown in Figure 32c and 32d (gradations after compaction indicate a significant reduction in particle sizes, especially for the proportional gradation). Densities of the minus 3/4-in. gradation were significantly higher than for the minus 3-in. gradation. A check test with no curing before compaction did not show any difference. Compression was higher for the softer shale than for the harder shale, but permeability was similar.

Ohio and Indiana, I-74 sites

93. Ohio I-74. Results of the compaction tests on the Kope shale from the Ohio embankment sites (Figure 33a and 33b) show similar results. The minus 3/4-in. proportional gradation produced higher densities than the minus 3-in. gradation. Soaked compression of the minus 3/4-in. compacted sample was higher, and the soaked sample was impervious.

94. Indiana I-74. Only one barrel of parent shale was obtained from the Indiana embankment site (paragraph 44). Consequently, only one 12-in.-diameter sample was compacted. For this soft Dillsboro

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* Nobari, E. S., and Duncan, J. M., "Effects of Reservoir Filling on Stresses and Movements in Earth and Rockfill Dams," Contract Report 5-72-2, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Jan 1972.

shale, the proportional gradation (Figure 33c) produced slightly lower densities (Figure 33d), the highest soaked compression, and a lower permeability than the West Virginia shales and siltstone.

Discussion

95. Results of the compaction tests using coarse-graded, minus 3-in. material as a simulated prototype indicate that proportional gradation for the minus 3/4-in. fraction produces similar densities. A scalped gradation with no replacement for the minus 3/4-in. fraction could also be used, since the resulting gradation is very close to the proportional gradation (Figure 32a), and the resulting density is very close to that for the minus 3-in. gradation.

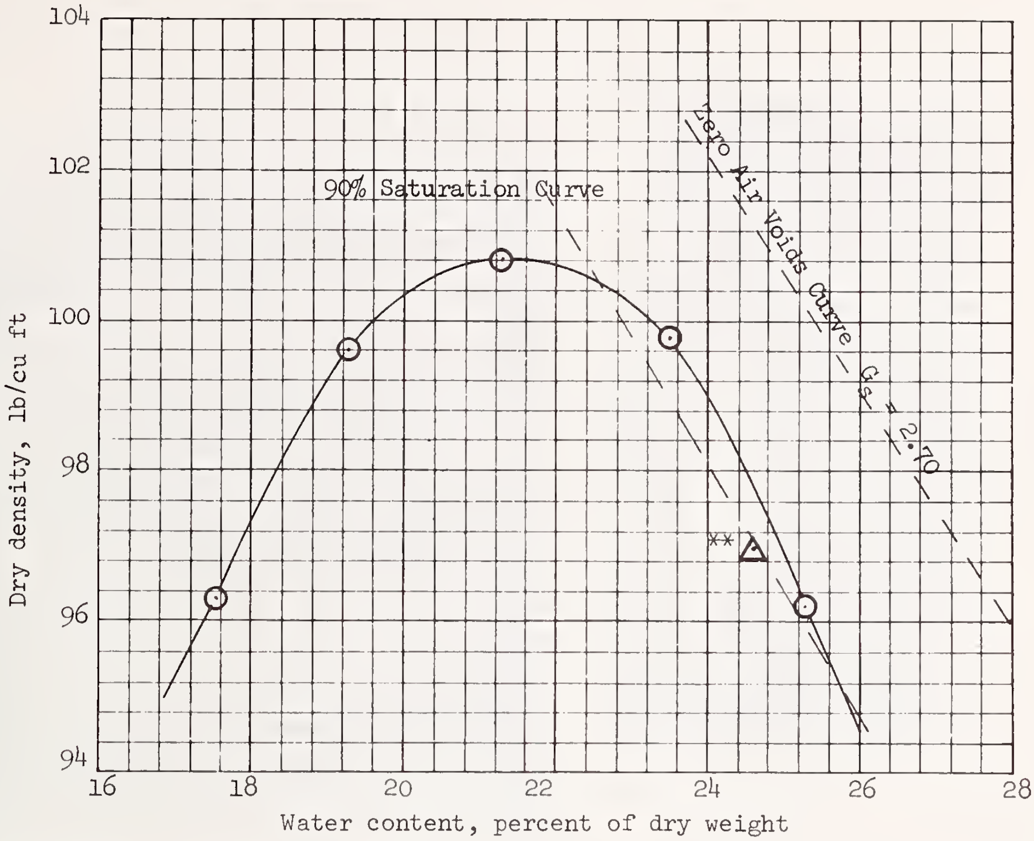
Compaction Test on Weathered Clay Shale, Kentucky I-75

96. Because unweathered shale used in the Kentucky I-75 embankment was not available, a compaction test was performed on the weathered clay shale from an adjacent cut. This material was used in the upper 20 ft of the embankment.

97. Results of the standard effort compaction test (AASHTO T99) are shown in Figure 34. The natural water content of the cut material was 25 percent and corresponded to a compacted dry density of 97 pcf. The optimum water content was 21 percent, and the maximum dry density was 101 pcf. One compaction test point, using standard effort and a 6-in.-diameter mold with the material at the natural water content, also gave a dry density of 97 pcf. These laboratory values of dry density are generally lower than the densities of 114 and 118 pcf listed in Table 4 for samples containing some limestone.

Soaked Compression Tests

98. The compacted samples of minus 3/4-in. material (compacted in a mold lubricated with silicon grease) were used to develop an index type compression test for use in estimating long-term settlement. The



Standard compaction test

25 blows per each of 3 layers, with 5.5 lb rammer and

12 inch drop. 4 inch diameter mold

Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
		LEAN CLAY(CL), light brown	2.70	45	22		
				*49	22		

Sample No.				
Natural water content, percent				
Optimum water content, percent	21.3			
Max dry density, lb/cu ft	100.8			

Remarks *Blenderized.	Project COMPACTED SHALE EMBANKMENT STUDY
** Natural w, 6" diam	Weathered shale from cut slope
mold, 5.5# rammer,	Area Kentucky I-75, station 670+00
standard effort.	Boring No. Date 7 May 1976

Figure 34. Compaction test results, Kentucky I-75 weathered parent clay shale

test consisted of cyclic soaking and draining under an arbitrarily chosen load of 1000 lb (equivalent to 40 ft of embankment height with a wet density of 127 pcf). The test apparatus is shown in Figure 35.

Equipment

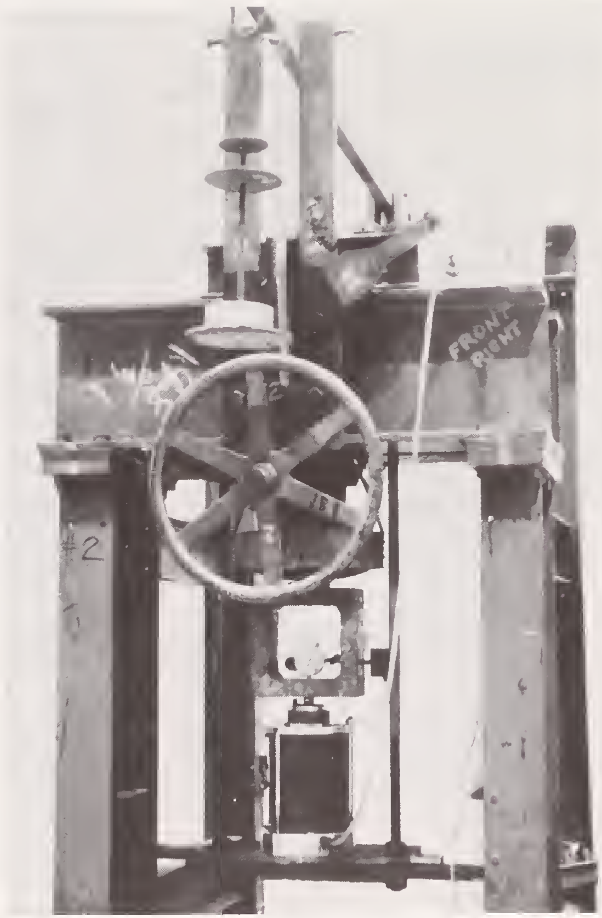
99. The CBR mold containing the 4.5-in.-high sample was fitted with a specially made base containing a 5-in.-diameter porous stone and outlet and a recessed collar (Figure 35b). A thin rubber ring, 1.2-in. wide (inside diameter of 5 in. and outside diameter of 6.2 in.) was used between the stone, sample bottom, and CBR mold base as a seal and to prevent seepage around the sample during soaking under a 4-ft head. Two lever type (100 to 1 ratio) loading machines (Figure 35a), readily available in the soils laboratory, were used.

100. Since the top of the sample was 3.75 in. below the mold collar, the interval between the top of sample and reaction frame of the loading machine was filled with a porous stone (5.9-in. diameter), a metal disk with a depression, and a steel ball (3/4-in. diameter). A dial gauge resting on the steel ball (through a hole in the reaction frame) was used to monitor compression. The test results were corrected for compression of metal pieces and porous stones.

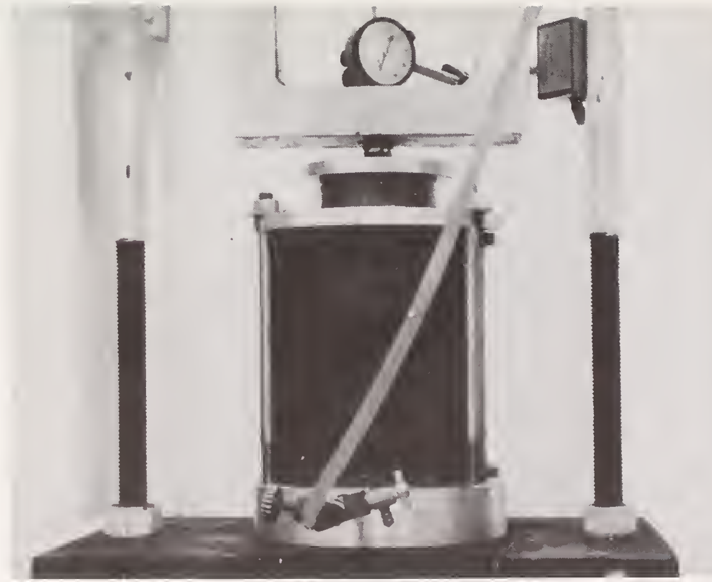
Procedure

101. For each compression test, the 1000-lb load was applied to the compacted sample, and compression readings were recorded until equilibrium occurred. Water, under a 4-ft head, was admitted at the base of the sample, and compression readings were again recorded until water filled the space at the top of the CBR mold (Figure 35c). Readings of increase in water level in the mold and head difference between the mold and the reservoir on top of the loading frame (plastic bucket) versus time were recorded. A falling head type permeability test was thus performed. The water was then drained from the sample. Several cycles of soaking and drainage did not produce any significant increase in compression.

102. After the test, the force required to push the sample 0.1 in. out of the mold was measured to determine side friction. The samples were examined after the test to determine the amount of softening of



a. Loading frame



b. CBR mold, base, and collar

c. Top of CBR mold, scale, and water surface

Figure 35. Soaked compression apparatus

shale particles. Compacted samples after testing are shown in Figure 36.

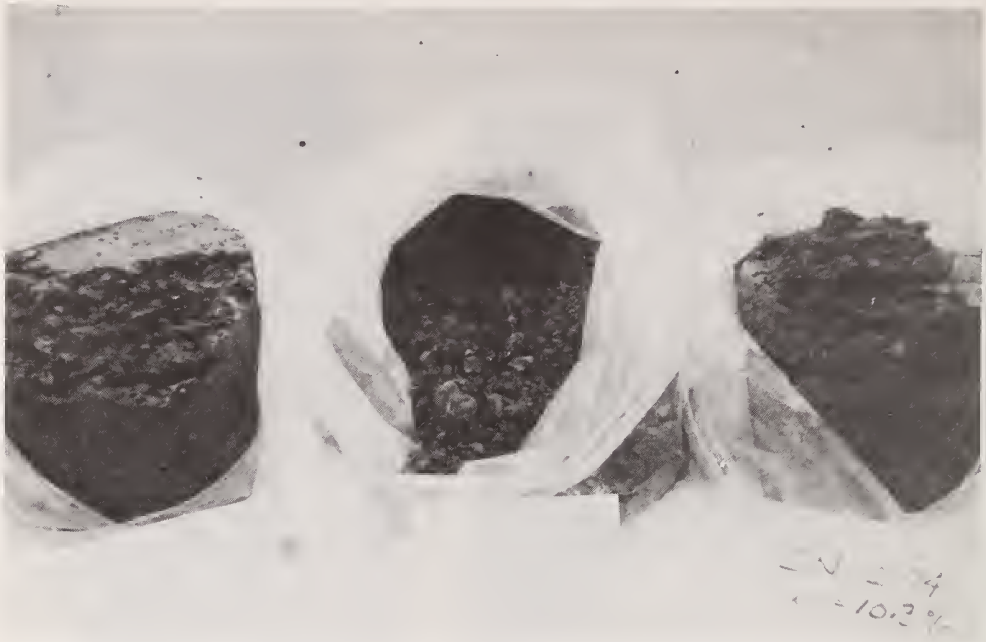


Figure 36. Compacted minus 3/4-in. parent shale after soaked compression tests

Test results

103. A typical compression time plot is shown in Figure 37. Only a small amount of compression (0.2 percent) occurred before soaking. However, after soaking over a 10-day period, the additional compression amounted to more than one percent (termed percent soaked compression). The results of the compression tests on compacted samples of parent shales from the West Virginia, Ohio, and Indiana embankments related to slake-durability are summarized in Figure 38. A trend of increasing soaked compression with decreasing slake-durability index is apparent. In addition, a 2- to 3-pcf increase in compacted dry density caused a significant reduction in soaked compression. A rough idea of the average settlement for an 80-ft-high embankment is shown at the bottom of the plot. Permeability data has been shown in Figures 32 and 33.

104. Examination of the samples after the test indicated various degrees of softening of shale pieces. Consequently, the test is considered a useful index for estimating long-term settlement of samples compacted under the following conditions:

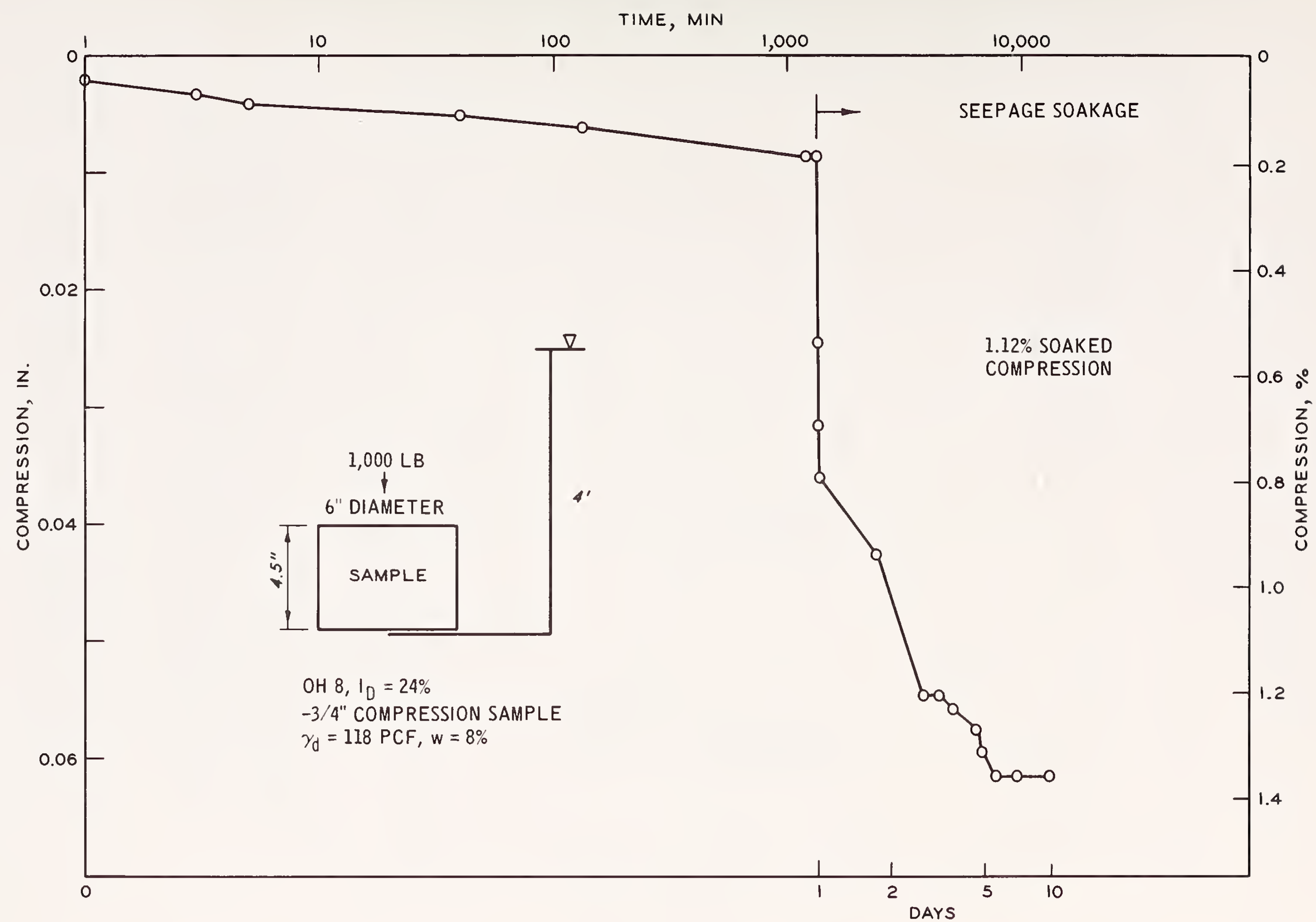


Figure 37. Compression time plot for compacted shale sample

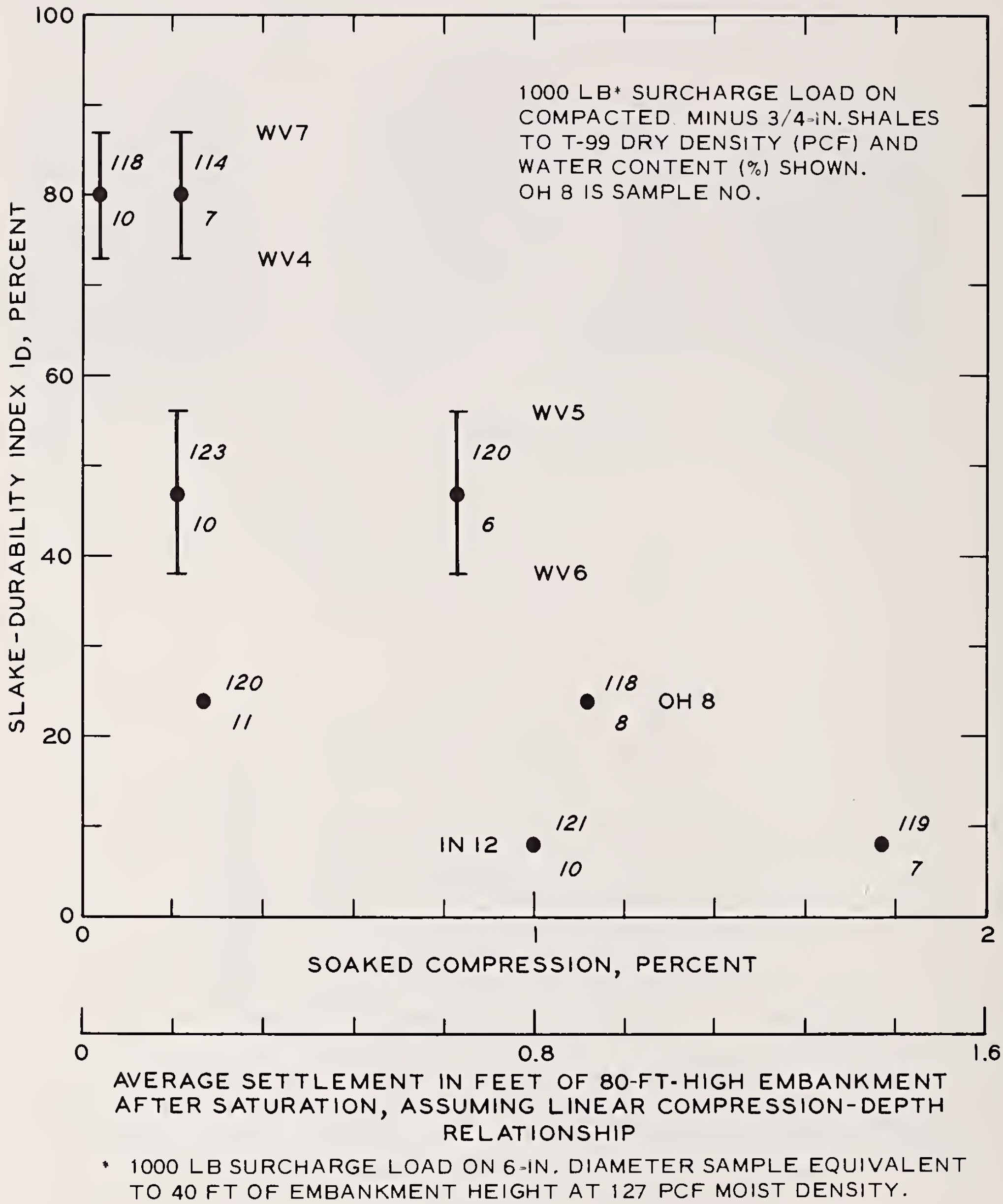


Figure 38. Soaked compression of minus 3/4-in. compacted shale samples related to slake-durability index

a. Using minus 3/4-in. material with a gradation modeling the gradation expected during construction.

b. Compacting samples to the expected field dry density and at the expected field water content.

PART V: CONCLUSIONS AND RECOMMENDED USAGE

Shale Embankments Investigated

105. The six shale embankments investigated were intended to cover different ages and include shales used as soil and as rockfill. This objective was achieved with ages of 6 years (West Virginia U. S. 460), 11 years (Tennessee I-75), and 14 to 16 years (Kentucky I-75, Ohio I-74 two, Indiana I-74) and included three embankments with shale placed as rockfill in thick lifts (Tennessee, Ohio, Indiana). However, the effect of age on shale deterioration was not fully apparent, except at the Indiana I-74 site, where shale pieces had softened into stiff clay. Generally, considerable mixing of overburden (residual) soils, small shale and rock particles, and large rock during construction was apparent from examination of undisturbed samples. Random variations in types of materials and loose zones with depth indicated that infiltrating water probably followed erratic paths, causing softening, compression, and settlement in an irregular fashion.

Shale properties

106. Properties of the six embankments varied widely because of the random mixing of different materials. However, trends were indicated for densities which were higher in the well-compacted Ohio I-74 embankment at State Route 128 and lower in the Tennessee I-75 embankment. Total sample densities included intact dense shale and limestone chunks. Thus, densities of clayey soil and shale fragment components were lower than indicated by the total sample density. Trends were also evident for modulus and shear strength.

Settlement

107. Modulus values estimated from pressuremeter tests, as in other investigations, were much lower than from laboratory K_o CU triaxial compression tests. The high modulus values from the triaxial tests indicated small additional settlements of 1 to 4 in., using published influence values derived from finite element analyses. Although low modulus values from pressuremeter tests correlated roughly with

measured settlements, such low modulus values would seem unlikely at the end of construction. Thus, pseudo-elastic analyses of these shale embankments does not appear appropriate and reinforces the concept of the softening of the soil and shale components which leads to excessive settlements.

Shear strength

108. The existing shear strength of the shale embankments investigated correlated with the undrained shear strengths of laboratory compacted, saturated shales. This correlation applies to long-term shear strength, since four embankments were 14 to 16 years old. High strengths estimated from pressuremeter tests reflected the influence of large pieces of shale and limestone. Testing at frequent intervals (every 3 to 5 ft) is required to establish lower limits of strengths, and considerable experience is required to perform the tests and evaluate the results. Where feasible, pressuremeter tests should be made by advancing the hole and testing in short increments, such as 5 ft. This procedure would prevent an oversize hole caused by drill rod wobble and allow testing before squeezing-in of the hole when deep holes are drilled before tests are started.

Point Load Tests

109. The point load test appears promising in providing a quick field index test to distinguish hard, rocklike shales from softer shales. However, no absolute relation was apparent between point load index and slake-durability. Separate testing programs would be required for each project to determine specific application for the shales involved.

Modeled Gradations for Compaction of Oversize Shales

110. The use of proportional gradations or scalping with no replacement appears to be the best means of estimating compaction properties of shales with more than 35 percent oversize pieces. AASHTO T-99 compaction tests on minus 3/4-in. proportional or scalped gradations

can be used to develop compaction properties during design and to perform check tests during construction. This procedure would mainly apply to shales classified as soillike and compacted in thin lifts.

Soaked Compression Index for Long-Term Settlement

111. Soaked compression of compacted shale samples can provide an index for estimating long-term settlement potential. This test can also be used to measure permeability and evaluate relative porosity to infiltration of surface water. The importance of reducing settlement potential by extra compaction to increase densities 2 to 4 pcf can be evaluated during design and correlated with a slake-durability index. The applied load during soaked compression can be tailored to actual project embankment heights. This index test also applies mainly to soillike shales compacted in thin lifts.

APPENDIX A: PRESSUREMETER TESTS

1. This appendix describes the Menard pressuremeter, field test procedures, and methods used to evaluate test results to obtain an estimate of modulus, shear strength, and a stress-strain curve.

Pressuremeter and Operation

Pressuremeter

2. A schematic diagram of the pressuremeter (Figure 1) used for the field tests is shown in Figure 39. The volume increase of the probe measuring cell induced by the applied pressure (controlled by the regulator and gage 2) is measured on the graduated site tube on the front of the volumeter. The pressure reducer is set to keep the pressure in the outer (air) cell slightly lower (about 1 kg/cm^2) than in the measuring cell. The pressure difference allows the rubber membrane of the measuring cell to push against the outer sheath without bulging at the ends (i.e., maintain a cylindrical shape).

3. An NX-size probe was used with a measuring cell length of 18.3 in. (465 mm) and a filled volume (V_f) of 1688 cm^3 (when pressurized to just touch the outer sheath). A urethane outer sheath (2.75-in. outside diameter) was used to prevent puncture during the tests. Compressed nitrogen was used as the high-pressure source. The volumeter with a capacity of 1500 cm^3 (maximum volume increase of probe without bursting is rated at 1700 cm^3) was filled with water which was colored green by adding food coloring.

Filling probe

4. After filling the volumeter, water was forced through the coaxial tubing under a pressure of 1 kg/cm^2 to remove all air. The probe, leaned at an angle of 30 deg from the vertical, was filled by inserting and removing the end fitting of the coaxial tubing (while water was flowing out) into the top of the probe several times until no air bubbles could be shaken out of the probe. The end of the coaxial tubing was then fastened tightly to the probe.

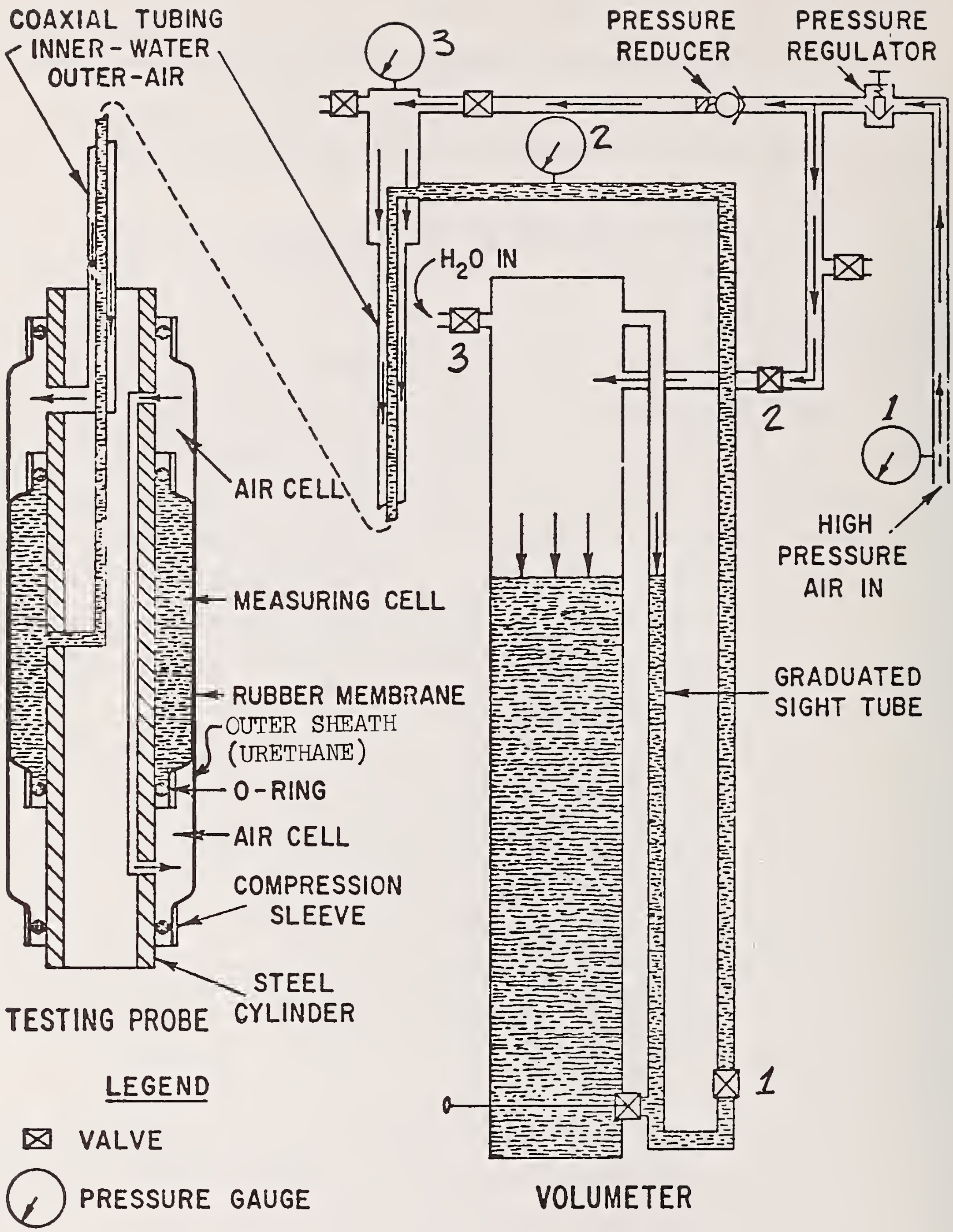


Figure 39. Schematic diagram of Menard pressuremeter device (from "Embankment Testing with the Menard Pressuremeter," California Department of Transportation, Highway Research Report No. M&R 632509-2, May 1968)

Calibration

5. A pressure test was made with the probe held vertical to detect any leaks and to obtain a pressure-volume calibration for expansion of the outer urethane sheath (inertia test) before lowering into the boring. This inertia calibration test was required at the start of testing each day and before and after testing in a boring. It was found that the stiffness of the urethane sheath changed with use, and a family of inertia calibration curves used to correct the test results is shown in Figure 40. Calibration of the volumeter and coaxial tubing for volume expansion under the pressures used in the tests was not required, since the correction was minute compared to volume increase of the probe during a test.

Test Procedures

6. The probe was attached to small diameter aluminum rods (5 and 10 ft long) by a special fitting which screwed to the top of the probe. The valve on the waterline to the measuring cell (valve 1, Figure 39) was closed to prevent expansion of the probe during lowering to the test depth. The probe was lowered down the 3-in.-diameter boring with the coaxial tubing taped to the rods at about 3-ft intervals to prevent the tubing from jamming the probe against the borehole wall during recovery. When the probe was at the test depth, valve 1 (Figure 39) was opened and volume readings recorded until equilibrium occurred.

7. The test was then conducted by increasing the fluid pressure in 0.5- or 1.0-kg/cm^2 increments depending on the limiting pressure from previous tests. The pressure increments and volume readings at 15, 30, and 60 sec are shown on an example data sheet in Figure 41. The maximum volume increase was about 1600 cm^3 without danger of rupturing the rubber membrane. After completion of the test, the pressure was released; and the fluid was pushed out of the measuring cell and back up into the volumeter by increasing the outer cell pressure (Figure 39: valve 2 closed, valve 3 opened, and pressure monitored by gage 3 increased). The probe could then be pulled uphole to the next test depth.

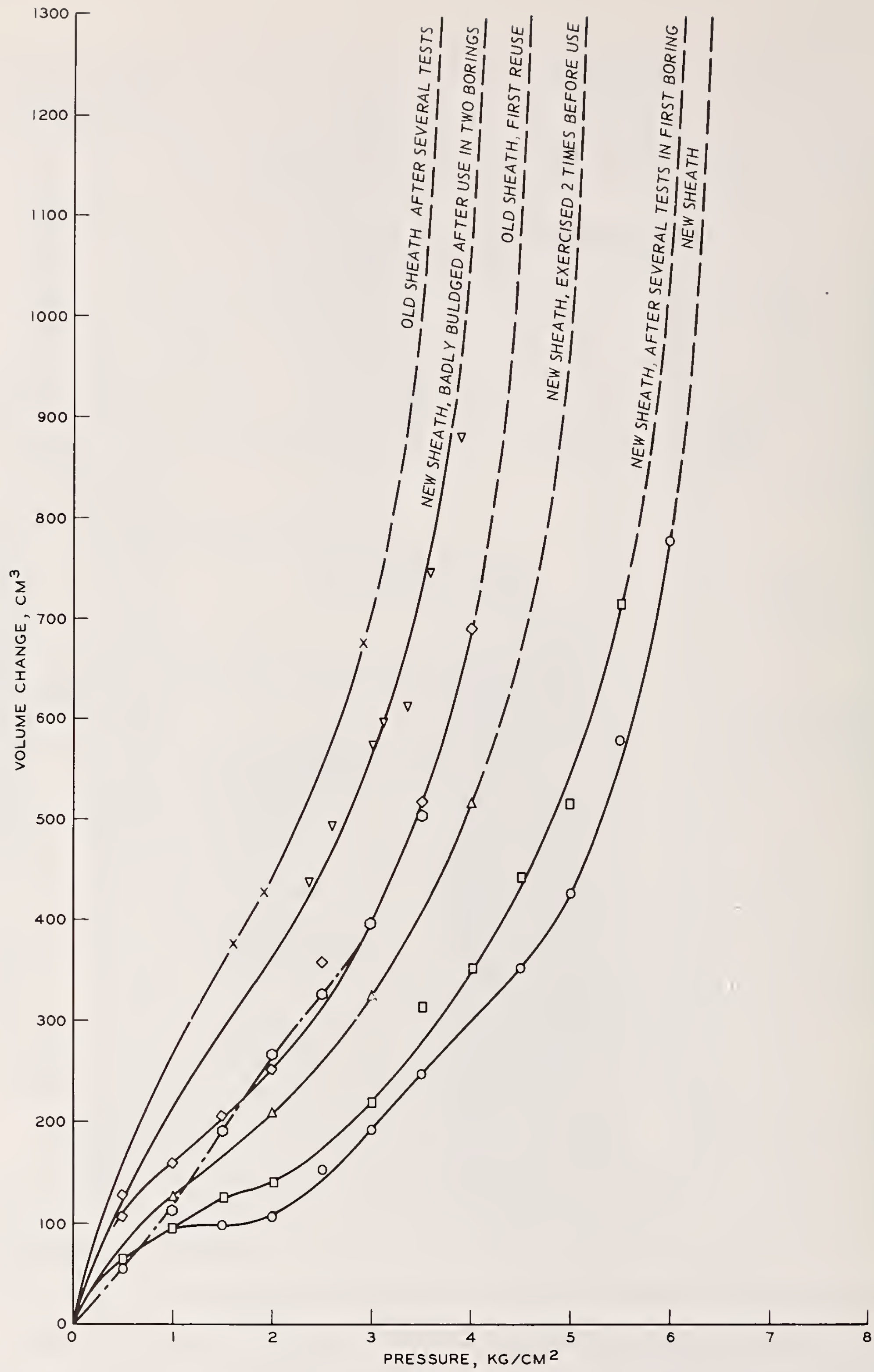


Figure 40. Inertia curves for NX-size urethane sheaths

MENARD PRESSUREMETER TEST DATA

SITE	DATE	BORING NO.	DEPTH, FT	OPERATOR	PAGE
WV US 460	3-31-76	PM-1	72.5	W.E.S. & H.M.	1 OF 1

PROBE SIZE & TYPE: NX LONG URETHANE SHEATH

REMARKS: Volumeter reading start of testing: $V_{P=0} \doteq 90 cm^3$
Differential pressure setting: 0 Kg/cm²
(0.5 Kg/cm² bleeds back to 0)

TIME	Applied LOAD KG/CM² (Pressure)	VOLUME - CM³ 15 SEC	30 SEC	60 SEC	CREEP (30-60)	DIFF (60 SEC)	REMARKS
1414	0	500					76.5-Ft head of water = 2.3 Kg/cm²
1417	0	508					Add to Applied load.
1419	0	508					
1420	1	570	595	635	40		
1421	2	712	728	738	10	103	
1423	3	780	790	804	14	166	
1424	4	830	838	853	15	49	
1425	5	871	878	890	12	37	
1426	6	908	915	926	11	36	
1427	7	943	950	960	10	34	
1428	8	985	993	1005	13	45	
1430	9	1033	1040	1052	12	47	
1431	10	1105	1128	1147	19	95	
1433	11	1175	1192	1222	30	75	

Figure 41. Pressuremeter test data sheet

Tests were made at 5-ft depth intervals, with additional tests at intermediate intervals (2.5 ft) in some borings.

8. After each test the uncorrected volume for 60-sec readings was plotted versus pressure to determine if the test was valid or if the hole diameter was too large. Too large a diameter produced a plot similar to the curves in Figure 40 with an "S" shape when the probe was expanded into the borehole wall after a large volume increase.

Correction for water head

9. Since the pressuremeter tests were performed in a dry hole, a correction of 0.03 kg/cm^2 was required for each foot of depth below the water level in the volumeter. The correction amounted to 1 kg/cm^2 at a depth of about 30 ft below ground surface. At depths less than 30 ft, the pressure reducer was used to produce a total differential pressure of 1.0 kg/cm^2 and left open below this depth. At a depth of 100 ft, the hydrostatic head of 3 kg/cm^2 can exceed the stiffness of a well-used urethane sheath by 2 kg/cm^2. Thus, the 100-ft depth is a limiting depth, since the measuring cell is pushed into the borehole wall with an effective pressure of 2 kg/cm^2 under the hydrostatic pressure.

Data Reduction

Graphical interpretation

10. The applied pressures, corrected for water head at the test depth, were plotted against measured volume. The plotted field-data curve was laid over the family of inertia-correction curves (Figure 40) and an appropriate correction curve selected to best fit the initial portion of the field-data curve. The results for two tests are shown in Figure 42.

11. The corrected curve was used to obtain a graphical estimate of pressuremeter modulus, E_p , using the following relationship:

$$\text{Shear modulus,} \quad G = \left[\left(V_{m_o} - V_i\right) + V_f\right] \cdot \frac{\Delta P}{\Delta V}$$

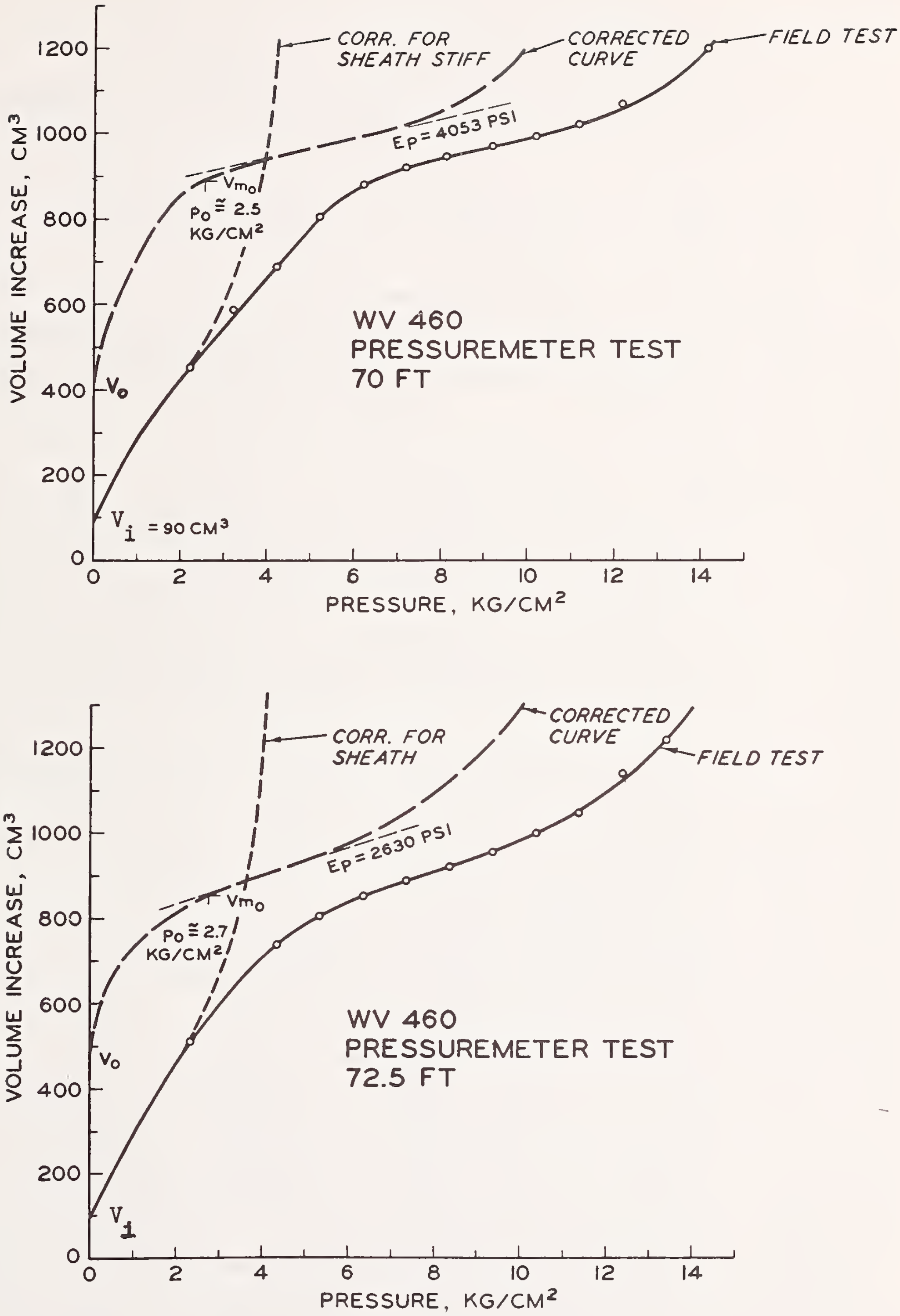


Figure 42. Correction of pressure volume data for sheath stiffness

where

V_{m_o} = measured volume at p_o

V_i = volumeter reading before insertion of probe in borehole

V_f = filled volume of probe (1688 cm^3)

$\frac{\Delta P}{\Delta V}$ = slope of linear segment of pressuremeter curve

and

$E_p = 2G(1 - \nu)$; $\nu = 0.3$ to 0.5 .

The value of p_o (the theoretical in situ lateral earth pressure) was estimated from the beginning of the linear portion of the pressuremeter curve.

Stress-path plot

12. The Menard theory used to obtain a stress-path plot and an estimate of in situ shear strength and shear strength parameters c and ϕ is illustrated in Figure 43. The Menard theory for calculating R values ($R = q$ = radius of Mohr Circle) is described by Parsons (1961)* and was developed into an interactive computer graphics program by E. D'Appolonia (1977).**

13. The baseline (dashed line, Figure 43a) at the volume corresponding to p_o is used to calculate R values. The volume at p_o is the total true volume of the measuring cell, A_o . The value of $A_o = (V_{mo} - V_i) + V_f$ is defined above.

14. If the correct value of p_o is used to establish the baseline, the resulting stress-path curve shown in Figure 43b will have its vertical linear segment starting at $q = 0$ (point A, Figure 43b). At point A, the pressuremeter has theoretically pushed the deformed borehole wall back to its original position, corresponding to zero shear stress at the true in situ lateral earth pressure. For other assumed values of p_o greater or less than the true value, point A will be

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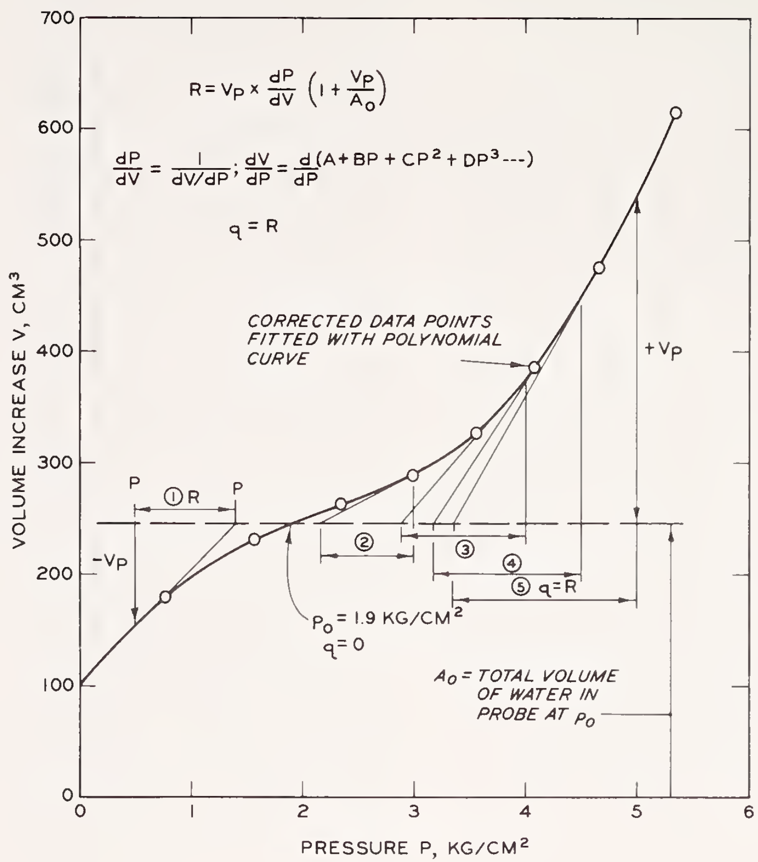
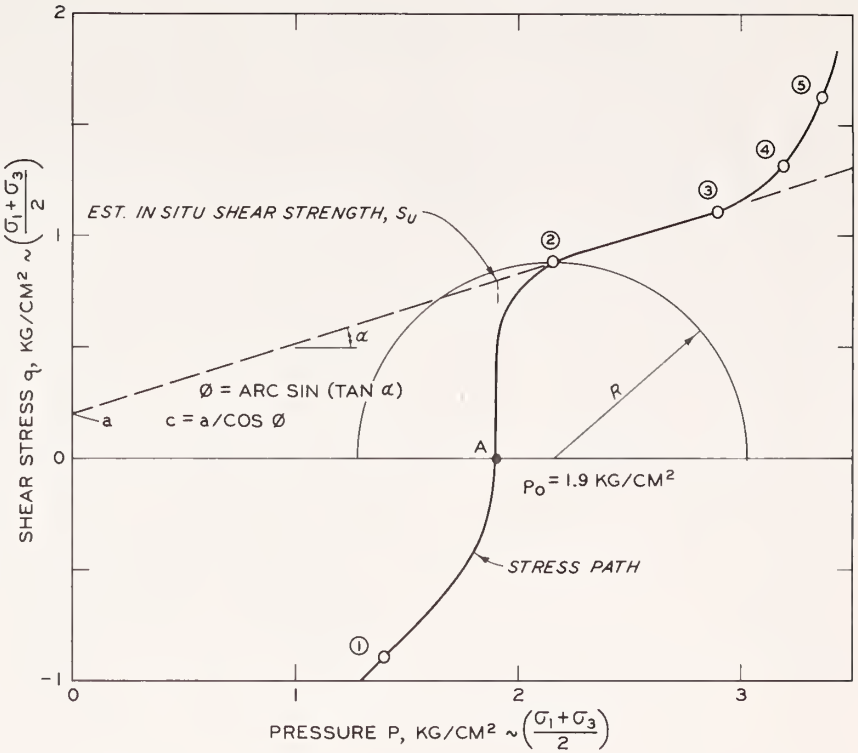
* Parsons, Brinckerhoff, Quade, and Douglas, "The Menard Pressuremeter, Technical Digest," prepared for the American Soil Testing Company, Sep 1961 (reproduced in E. D'Appolonia, 1977, Vol 3).

** E. D'Appolonia Consulting Engineers, Inc. (EDCE), "Summary of Investigations and Recommendations, Evaluation of Embankment Stability (ISHC Project No. I-74-4 (73) 163)," Final Project Report - Vol 4 (Pressuremeter), Indiana State Highway Commission, Indiana, Nov 1977.



a. Procedure for determining R, q, and p from pressure volume curve



b. Stress-path plot

Figure 43. Illustration of Menard theory for obtaining stress-path plot and estimating shear strength

above or below the $q = 0$ line. Because of the excessive time required for a graphical interpretation and because the selected p_o value may not be correct, a computer assisted graphical analysis program was used. The program generates a family of stress-path curves from a smooth, continuous polynomial curve fitted to the corrected pressure-volume data points. The program also includes generation of a stress-strain plot using the selected p_o value.

Stress-strain plot

15. Theory and procedures developed by Landanyi (1972)* and based on expansion of a cavity were used to obtain a stress-strain curve for use in estimating modulus E and shear strength S_u. The procedure is illustrated in Figure 44. The segment of the corrected pressure, injected-volume curve above p_o (Figure 44a) is used, and the change in volume, ΔV divided by the total volume, is plotted against change in pressure above p_o (Figure 44b). The formulas for stress and strain shown above Figure 44b have been transformed from plain strain to axial symmetry. The corresponding numbered points in Figures 44a and 44b are shown on the plot of stress versus strain in Figure 45. The procedure for estimating S_u in Figure 45 was arbitrarily chosen. The graphical interpretation or hand calculation using the formulas shown in Figure 44 to obtain a stress-strain plot requires considerable time, and a computer assisted graphic analysis is much faster. However, the graphic interpretations shown in Figures 43 and 44 provide an insight into the effect of the shape and curvature of the corrected pressuremeter curve on the shape of the stress-path plot and stress-strain curve.

Example plots

16. Example plots for three tests using the interactive graphics computer program are shown in Figures 46 and 47 (pages 143-146). Figure 46 shows results for the test at a depth of 40 ft in the West Virginia (U. S. 460) embankment. In Figure 46(A) the corrected pressure versus volume data points were fitted with a fourth-degree polynomial.

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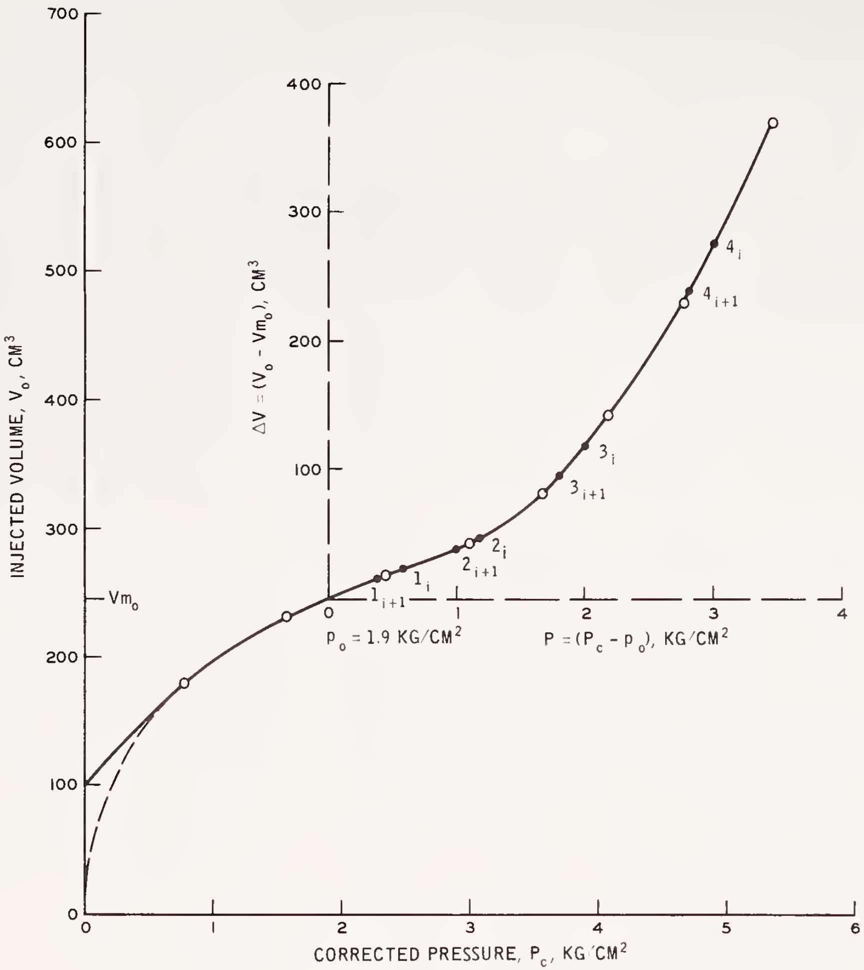
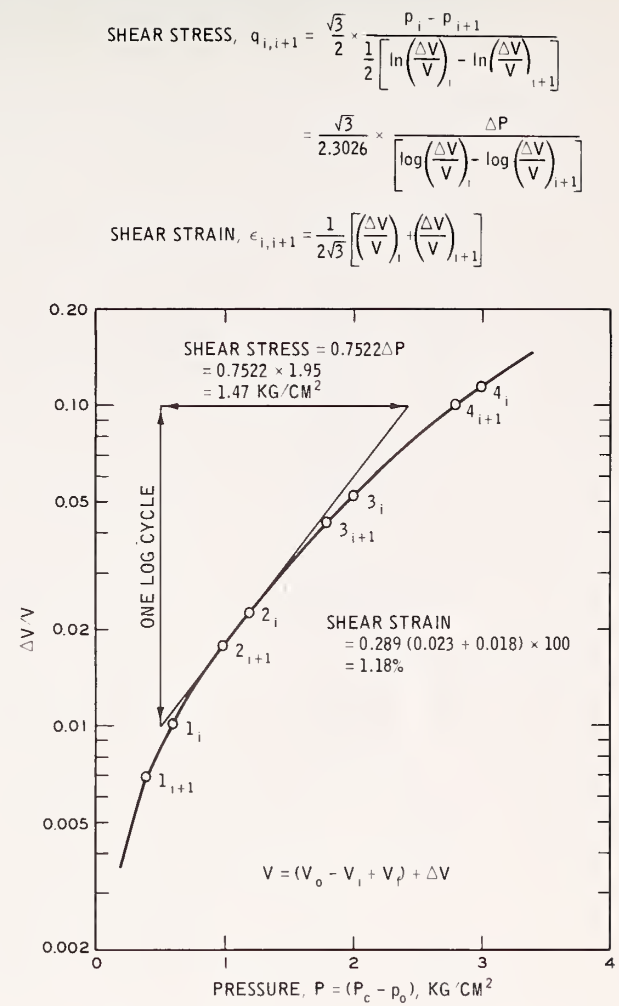
* Landanyi, B., "In-situ Determination of Undrained Stress-Strain Behavior of Sensitive Clays with a Pressuremeter," Canadian Geotechnical Journal, Vol 9, 1972.



a. Interpretation of pressure-volume curve



b. Graphical determination of stress and strain

Figure 44. Illustration of Ladanyi's method for estimating stress and strain

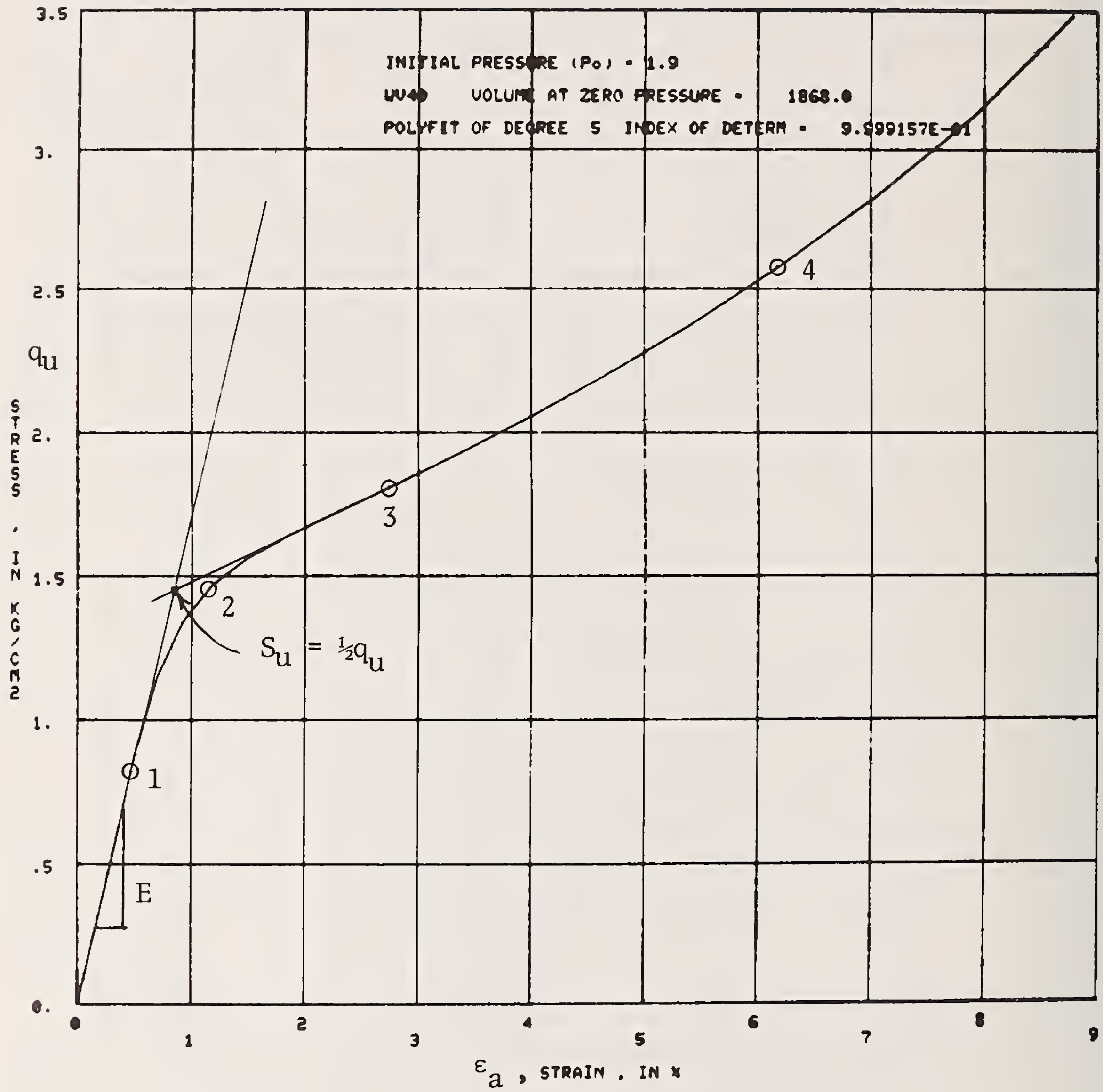


Figure 45. Stress-strain plot

The polynominal fitting is required to produce a smooth continuous curve for generating the stress-path plot and stress-strain plot. The stress-path plot is a family of 10 curves, and the curve with its linear segment just starting at $q = 0$ is selected as the correct curve. This curve defines p_o and is used to graphically estimate $\alpha(\sin \phi = \tan \alpha)$, $a(c = a/\cos \phi)$, and S_u as shown. The stress-strain curve for the selected p_o value is used to estimate E and S_u. In Figure 46(B) (page 143-144) the same corrected pressure volume data points were fitted with a fifth-degree polynomial. Although the latter curve fits the data points only slightly better, the resulting p_o value is higher; and the shear strength and modulus values are slightly lower. Consequently, good judgment is required in choosing the degree of polynomial. If the original test is not carefully performed, the data properly corrected, and the results evaluated for validity, then the graphical analysis becomes a game of curve fitting. The results could be entirely misleading.

17. Figure 47 (page 145-146) shows the results of two tests at depths of 70 and 72.5 ft. These two tests show significantly different results. Figure 47(A) for the test at 70 ft has a pressure volume curve which continued to curve upward at the high pressure and produced stress-path curves which bent over into a horizontal direction. Thus, the c, ϕ parameters could not be defined. In this case, S_u was taken as the maximum value for the stress-path curve. An alternate conservative procedure of drawing a line from $p = q = 0$, tangent to the stress-path curve, and defining S_u at p_o could be used. The resulting stress-strain curve indicates strain softening. In Figure 47(B), the selected stress-path curve can be used to estimate c and ϕ. The stress-strain curve indicates a strain-hardening material.

<u>Computer graphics program</u>

18. The graphics program used for analyses of the corrected pressuremeter data was coded for the GE 635 computer graphics system and requires a cathode ray tube (CRT) graphics terminal and hard copy machine. An example of the interactive time share session (questions and responses) used to obtain the plots shown in Figure 47 is summarized in Figure 48. A program listing is shown in Table 8.

```
LIST WU40

010 2,3
020 .78,180
030 1.58,232
040 2.34,263
050 3.0,290
060 3.58,328
070 4.09,388
080 4.67,475
090 5.35,615

ready

*OLD BRAGG
READY
*GCS2D
device-TK4

        06/13/78    11.005

DO YOU WISH USER INSTRUCTIONS, TYPE YES OR NO
=NO
DO YOU WANT TO USE DATA FILES, TYPE YES OR NO
=YES
NAME OF DATA FILE
=WU40
DO YOU WANT DATA SHIFTED TO ZERO MEAN, YES OR NO
=NO
INPUT VOLUME AT P=0 FROM CORRECTED CURVE.
=1868.0


L E A S T   S Q U A R E S   P O L Y N O M I A L S

     NUMBER OF POINTS = 8
     MEAN VALUE OF X  =  3.173750E 00
     MEAN VALUE OF Y  =  3.463750E 02
     STD ERROR OF  Y  =  1.422603E 02

NOTE:  CODES FOR 'WHAT NEXT?' ARE:

     0 = INPUT THE NEXT SET OF DATA
     1 = COEFFICIENTS ONLY
     2 = ENTIRE SUMMARY
     3 = FIT NEXT HIGHER DEGREE
     4 = PLOT DATA POINTS AND CURVE
     5 = PLOT P-Q CURVES FROM MENARD THEORY
     6 = PLOT STRESS-STRAIN CURVE USING LADANYI'S METHOD
     7 = STOP

POLYFIT OF DEGREE  3  INDEX OF DETERM =   9.994698E-01
WHAT NEXT
=3
```

```
POLYFIT OF DEGREE  4  INDEX OF DETERM =   9.998604E-01
WHAT NEXT
=4
COPY THEN PRESS ANY CHARACTER AND RETURN
```

Volume-pressure (V-P) plot drawn on CRT screen

```
WHAT NEXT
=5
INPUT TRIAL PRESSURE
=1.4
DO YOU WANT SMALLER SCALE? YES=1,NO=2
=2
```

Stress-path (P-Q) plot drawn

```
WHAT NEXT
=6
NAME OF FILE OR TITLE?
=WU40
ENTER INITIAL PRESSURE
=1.7
DO YOU WANT TABLE OF STRESS AND STRAIN? YES=1, NO=2
=2
```

Stress-strain plot drawn

```
WHAT NEXT
=3

POLYFIT OF DEGREE  5  INDEX OF DETERM =   9.999157E-01
WHAT NEXT
=4
COPY THEN PRESS ANY CHARACTER AND RETURN
```

Volume-pressure plot drawn

```
WHAT NEXT
=5
INPUT TRIAL PRESSURE
=1.5
DO YOU WANT SMALLER SCALE? YES=1,NO=2
=2
```

Stress path plot drawn

```
WHAT NEXT
=6
NAME OF FILE OR TITLE?
=WU40
ENTER INITIAL PRESSURE
=1.9
DO YOU WANT TABLE OF STRESS AND STRAIN? YES=1, NO=2
=2
```

Stress-strain plot drawn

Figure 48. Example of time-share interactive graphics session to draw pressuremeter plots

Table 8. Listing of Interactive Graphics Program

```
BRAGG                       16:12:12  06/20/78

10* MENARD DEC 17,1976
20C
30C
40C  MODIFIED TO INCLUDE THE FOLLOWING:
50C          1. PLOTTING OF DATA POINTS LOTTING OF STRESS-STRAIN
80C             CURVE BY LADANYI'S METHOD
90C  FOR PRESSUREMETER DATA REDUCTION
100C
110C
120 COMMON X(100)
130 COMMON N,M,U(15),VAP
140 DIMENSION A(15),B(15),S(15),G(15),
150& P(100),Y(100),C(100),Q(100),
160& RRR(100),PEE(100),VA(52),STRESS(51),
170& STRAIN(51),DVOV(51),DELV(51),VOL(51)
180 CHARACTER FNAM*4(4)
190 CHARACTER YLABEL*40
200 INTEGER FNAME,OKST/O400000000000/,OKST2/O403700000000/
210 DATA FNAM(1),FNAM(4)/" /    ",  ;    "/
220   DATA JYES,JNO/"YES","NO"/
230C
240   PRINT:"DO YOU WISH USER INSTRUCTIONS, TYPE YES OR NO"
250   READ 10,IYES
260 FNAME=10
270  10 FORMAT(A3)
280   IF(IYES-JYES) 30,580,30
290  30 PRINT:"DO YOU WANT TO USE DATA FILES, TYPE YES OR NO"
300   READ 10,IYES
310   IF(IYES-JYES) 35,590,35
320  35 PRINT:"ENTER NUMBER OF POINTS, INITIAL DEGREE OF POLYNOMIAL"
330   READ:M,N
340 X01=0.0
350  36 Z=0
360   O=1
370   K=12
380   N=N+1
390   IF(N.GT.12) GO TO 576
400   IF(M.LT.N) GO TO 616
410   IF(M.GT.100) GO TO 570
420 T7=Z;T8=Z;W7=Z
430   IF(IYES-JNO) 302,301,302
440 301 PRINT:"ENTER DATA POINTS IN X(1),Y(1), ... ,X(N),Y(N) ORDER"
450   READ:(X(I),Y(I),I=1,M)
460 302 PRINT:"DO YOU WANT DATA SHIFTED TO ZERO MEAN, YES OR NO"
470   READ 10,IYES
480   IF(IYES-JYES) 305,303,305
490 303 M=M+1
500   DO 304 I=1,M
510   IM=M-I+2
520   X(IM)=X(IM-1)
530 304 Y(IM)=Y(IM-1)
540  X(1)=0.0;Y(1)=0.0
```

Table 8. Listing of Interactive Graphics Program (Continued)

```
BRAGG     CONT            16:12:12  06/20/78

 305 DO 310 I=1,M
       W7=W7+X(I)
       T7=T7+Y(I)
 310 T8=T8+Y(I)*Y(I)
       T9=(M*T8-T7*T7)/(M*M-M)
       W77=W7/M
       T77=T7/M
       T99=SQRT(T9)
     PRINT:"INPUT VOLUME AT P=0 FROM CORRECTED CURVE."
     READ:VAP
       PRINT 314,M,W77,T77,T99
 314 FORMAT(///" L E A S T   S Q U A R E S   P O L Y N O M I A L S"/,
&  //7X"NUMBER OF POINTS =",I2/7X"MEAN VALUE OF X  =",1PE14.6/
&  7X"MEAN VALUE OF Y  =",1PE14.6/7X"STD ERROR OF  Y  =",1PE14.6
&  ///" NOTE:  CODES FOR "WHAT NEXT?" ARE:"//7X"0 = INPUT THE ",
&  "NEXT SET OF DATA"/7X"1 = COEFFICIENTS ONLY"/7X"2 = ENTIRE ",
&  "SUMMARY"/7X"3 = FIT NEXT HIGHER DEGREE"/7X"4 = PLOT ",
&  "DATA POINTS AND CURVE"/7X"5 = PLOT P-Q CURVES",
&  "  FROM MENARD THEORY",
&  /7X"6 = PLOT STRESS-STRAIN CURVE USING LADANYI'S METHOD",
&  /7X"7 = STOP"//)
C
 DO 1150 I=1,15
 U(I)=0.0
 1150 CONTINUE
 DO 352 I=1,M
       P(I)=Z
 352 Q(I)=0
       DO 362 I=1,11
 362 A(I)=Z;B(I)=Z;S(I)=Z
      E1=Z;F1=Z
       W1=M
       N4=K
       I=1
       K1=2
       IF(N.NE.0) K1=N4
 380 W=Z
       DO 386 L=1,M
 386 W=W+Y(L)*Q(L)
       S(I)=W/W1
       IF(I-N4.GE.0.OR.I-M.GE.0) GO TO 428
       E1=Z
       DO 398 L=1,M
 398 E1=E1+X(L)*Q(L)*Q(L)
       E1=E1/W1
       A(I+1)=E1
       W=Z
       DO 416 L=1,M
       V=(X(L)-E1)*Q(L)-F1*P(L)
       P(L)=Q(L)
       Q(L)=V
 416 W=W+V*V
```

Table 8. Listing of Interactive Graphics Program (Continued)

```
BRAGG     CONT              16:12:12  06/20/78

     F1=W/W1
     B(I+2)=F1
     W1=W
     I=I+1
     GO TO 380
428 DO 432 L=1,12,1
432 G(L)=Z
     G(1)=0
     DO 464 J=1,N
     S1=Z
     DO 448 L=1,N
  IF(L.NE.1)G(L)=G(L)-A(L)*G(L-1)
IF(L.GT.2)G(L)=G(L)-B(L)*G(L-2)
448 S1=S1+S(L)*G(L)

     U(J)=S1
     L=N
     DO 460 I2=2,N
     G(L)=G(L-1)
     L=L-1
460 CONTINUE
     G(1)=Z
464 CONTINUE
     T=Z
     DO 488 L=1,M
     C(L)=Z
     J=N
     DO 482 I2=1,N
     C(L)=C(L)*X(L)+U(J)
     J=J-1
482 CONTINUE
     T3=Y(L)-C(L)
     T=T+T3*T3
488 CONTINUE
     IF(M.NE.N) GO TO 496
     T5=0
     GO TO 498
496 T5=T/(M-N)
498 Q7=1-T/(T9*(M-1))
     PRINT 500,N-1,Q7
500 FORMAT(/" POLYFIT OF DEGREE ",I2,"  INDEX OF DETERM = ",1PE14.6)
     PRINT:"WHAT NEXT"
     READ:IR
  512 IF(IR.EQ.0) GO TO 513
     IF(IR.EQ.3) GO TO 564
IF(IR.EQ.4) GO TO 700
IF(IR.EQ.5) GO TO 800
IF(IR.EQ.6) GO TO 801
IF(IR.EQ.7) GO TO 802
  GO TO 515
  513 CALL DETACH (10,ISTAT, )
  IF (ISTAT.EQ.0.OR.ISTAT.EQ.1.OR.ISTAT.EQ.OKST) GO TO 30
```

Table 8. Listing of Interactive Graphics Program (Continued)

```
BRAGG     CONT              16:12:12  06/20/78

 IF (ISTAT.EQ.OKST2) GO TO 30
 PRINT 5555,ISTAT
 5555 FORMAT (" ERROR DEACCESSING FILE - STATUS = " 012)
 STOP
 515 PRINT  516
516 FORMAT(//" TERM",8X,"COEFFICIENT"/)
  DO 526 J=1,N
  I2=J-1
526 PRINT 527,I2,U(J)
527 FORMAT(I4,7X,1PE14.7)
  IF(IR.EQ.1) GO TO 558
  PRINT 530
530 FORMAT(//" X-ACTUAL",12X,"Y-ACTUAL",3X,"Y-CALC",8X,"DIFF",
& 9X,"PCT-DIFF"/)
  DO 550 L=1,M
  Q8=Y(L)-C(L)
  IF(C(L)-0.0) 540,548,540
540 Q88=100.0*Q8/C(L)
  PRINT 551,X(L),Y(L),C(L),Q8,Q88
  GO TO 550
548 PRINT 552,X(L),Y(L),C(L),Q8
550 CONTINUE
551 FORMAT(1P5E14.6)
552 FORMAT(1P4E14.6,3X,"INFINITE")
  T55=SQRT(T5)
  PRINT 553,T55
553 FORMAT(/10X"STD ERROR OF ESTIMATE FOR Y = ",1PE14.6)
  IF(K.EQ.N) GO TO 576
558 CONTINUE
CALL URESET
CALL UBELL
CALL UPAUSE
PRINT:"WHAT NEXT"
  READ:IR
  GO TO 512
564 N=N+1
  IF(M.LT.N) GO TO 616
  GO TO 428
570 PRINT:"PROGRAM SIZE LIMIT IS 100 DATA POINTS."
 GO TO 576
576 PRINT:"ELEVENTH DEGREE IS THE LIMIT."
 GO TO 558
580 PRINT 585
585 FORMAT(///" THIS PROGRAM FITS LEAST-SQUARES POLYNOMIALS TO ",
& "PRESSUREMETER",/,"DATA, USING AN ORTHOGONAL POLYNOMIAL METHOD.",
& "   LIMITS ARE"/"  11-TH DEGREE FIT AND A MAX OF 100 DATA ",
& "POINTS. THE PROGRAM"/"ALLOWS THE USER TO SPECIFY",
& " THE ",
& "LOWEST DEGREE POLYNOMIAL TO BE"/" FIT, AND THEN FITS THE ",
& "POLYNOMIALS IN ORDER OF ASCENDING"/" DEGREE.  AT EACH ",
& "STAGE, THE INDEX OF DETERMINATION IS"/" PRINTED, AND THE ",
& "USER HAS THE CHOICE OF GOING TO THE NEXT"/" HIGHEST DEGREE ",
```

Table 8. Listing of Interactive Graphics Program (Continued)

```
BRAGG     CONT            16:12:12  Ø6/2Ø/78

& "FIT, SEEING EITHER OF TWO SUMMARIES OF FIT,"/" PLOTTING THE",
& " DATA POINTS AND THE FITTED CURVE,CALCULATING"/,
& "AND PLOTTING P-Q STRESS PATHS USING MENARD'S EQUATION,"/,
& "CALCULATING ",
& "STRESS-STRAIN CURVES BY LADANYI'S METHOD AND TABULATING AND"/,
& "PLOTTING THE RESULTS OR ENTERING NEW READ DATA.")
 PRINT 586
 586 FORMAT ("THE USER SHOULD PROVIDETHE FOLLOWING INFORMATION"/,
& "WHEN IT IS REQUESTED:"//7X,
& "1.  THE NUMBER OF DATA POINTS AND THE INITIAL (LOWEST)"/11X,
& "DEGREE POLYNOMIAL TO BE FIT."/7X"2.  THE DATA POINTS IN ",
& "X(1),Y(1),X(2),Y(2), ... ,X(N), "/11X"Y(N) ORDER."//" NOTE: ",
& "EACH VALUE ENTERED MUST BE FOLLOWED BY A COMMA (,)."/,
& "IF DATA FILES ARE USED THE FORMAT IS:"/,
& "       1ST       LINE  NUMBER OF POINTS (M), STARTING DEGREE OF FI
& "  2ND THRU (M+1) LINE  X(M),Y(M)"
& ////)
C
   GO TO 3Ø
590 PRINT:"NAME OF DATA FILE"
READ 591,FNAM(2),FNAM(3)
591 FORMAT(2A4)
CALL ATTACH(1Ø,FNAM,1,Ø,ISTAT,)
IF(ISTAT.EQ.Ø.OR.ISTAT.EQ.1.OR.ISTAT.EQ.OKST) GO TO 4545
IF(ISTAT.EQ.OKST2) GO TO 4545
PRINT 3535,ISTAT
3535 FORMAT("ERROR ACCESSING FILE - STATUS = "O12)
STOP
4545 CONTINUE
592 FORMAT(V)
  READ(FNAME,592)KK,M,N
DO 6ØØ I=1,M
6ØØ READ(FNAME,592)(KK,X(I),Y(I))
  GO TO 36
616 PRINT 617,N-1
617 FORMAT(/" TOO FEW POINTS FOR FITTING DEGREE",I4)
  GO TO 558
C
C
C  *************** PLOT POINTS AND CURVE ***************
C
C
 7ØØ PRINT:"COPY THEN PRESS ANY CHARACTER AND RETURN"
CALL UBELL
CALL UPAUSE
 N=N+1
CALL USTART
CALL USET ("INCHES")
IF(X(M)-1Ø)7Ø1,7Ø1,7Ø2
 7Ø1 XSCALE=1Ø.Ø
 GO TO 7Ø3
7Ø2 XSCALE=2Ø.Ø
```

Table 8. Listing of Interactive Graphics Program (Continued)

```
BRAGG     CONT           16:12:12  06/20/78

 703 CONTINUE
 IF (Y(M)-1000)704,704,705
704 YSCALE=1000.0
 GO TO 706
705 YSCALE=2000.0
 706 CONTINUE
CALL USET ("LINE")
CALL USET ("GRIDAXES")
CALL USET ("XBOTHLABELS")
CALL UPSET ("XLABEL","      PRESSURE , IN KG/CM2\")
CALL USET ("YBOTHLABELS")
CALL UPSET ("YLABEL","      VOLUME , IN CM3\")
CALL UAXIS (0.0,XSCALE,0.0,YSCALE)
DO 2 L=1,M
X0=X(L)
Y0=Y(L)
CALL USET ("N+")
CALL UPEN (X0,Y0)
CALL USET ("LINE")
2 CONTINUE
CALL UMOVE (0.0,0.0)
X0=0.0
X01=0.0
Y01=0.0
DO 5 I=1,100
Y0=0.0
DO 4 J=1,N
IF(J.EQ.1) GO TO 900
Y0=(U(J)*(X0**(J-1)))+Y0
GO TO 4
900 Y0=U(J)
4 CONTINUE
X01=X0
Y01=Y0
CALL USET ("L")
CALL UPEN (X01,Y01)
IF(X01.GT.X(M)) GO TO 6
X0=X0+.2
5 CONTINUE
6 N=N-1
CALL URESET
CALL UEND
1199 PRINT 1200,FNAM(2),FNAM(3),VAP
 1200 FORMAT (/,2A4," VOLUME AT ZERO PRESSURE =",F10.1,/)
PRINT 500,N-1,Q7
PRINT 516
DO 998 J=1,N
I2=J-1
998 PRINT 527,I2,U(J)
GO TO 558
800 CONTINUE
C
```

Table 8. Listing of Interactive Graphics Program (Continued)

```
BRAGG     CONT          16:12:12  06/20/78

C
C
C
CALL MENARD
C
1500 PRINT 1200, FNAM(2),FNAM(3),VAP
C
PRINT 500,N-1,Q7
GO TO 550
801 CONTINUE
C
C
C
C
CALL LADANYI
C
C
C
C
GO TO 1500
802 CONTINUE
STOP;END
C  ******************************************************
C
C
C
SUBROUTINE MENARD
C
C
C
C  ******************************************************
COMMON X(100)
COMMON N,M,U(15),VAP
DIMENSION RRR(100),PEE(100)
 DATA JYES, JNO /"YES","NO"/
MMTRY=1
 810 PRINT: "INPUT TRIAL PRESSURE"
READ: PRES
815 MTRY=1
I=1
X0=PRES
820 Y0=0.0
DO 840 J=1,N+1
IF(X0)899,841,824
824 CONTINUE
825 IF(J.EQ.1) GO TO 830
Y0=(U(J)*(X0**(J-1)))+Y0
GO TO 840
830 Y0=U(J)
840 CONTINUE
841 CONTINUE
IF(MTRY.GT.1)GO TO 855
```

```
BRAGG     CONT          16:12:12  06/20/78

VOL0=Y0
X0=0.0
MTRY=2
GO TO 820
855 CONTINUE
IF(X0)859,870,859
859 CONTINUE
DO 860 J=2,N+1
IF(J.EQ.2) GO TO 850
DY0=((J-1)*U(J)*(X0**(J-2)))+DY0
GO TO 860
850 DY0=U(J)
860 CONTINUE
GO TO 880
870 DY0=U(2)
Y0=U(1)
880 DPDV=1.0/DY0
VOLN=Y0-VOL0
A0=VAP+VOL0
RRR(I)=(VOLN*DPDV*(1.0+(VOLN/A0)))
PEE(I)=X0-RRR(I)
I=I+1
IF(X0-X(M))897,898,898
897 CONTINUE
IF(X(M)-10.0)1400,1400,1300
1300 X0=X0+.2
GO TO 820
1400 X0=X0+.1
GO TO 820
898 CONTINUE
 DO 70 IM=1,I
LM = IM
IF (RRR(IM)+0.001) 60,60,70
70 CONTINUE
60 KM = I-2
IF(MMTRY.GT.1)GO TO 701
CALL USTART
CALL USET ("LINE")
CALL USET ("GRIDAXES")
CALL USET ("XBOTHLABELS")
CALL UPSET ("XLABEL","        P,IN KG/CM2\")
CALL USET ("YBOTHLABELS")
CALL UPSET ("YLABEL","        Q, IN KG/CM2\")
PRINT: "DO YOU WANT SMALLER SCALE? YES=1,NO=2"
READ: IRR
IF(IRR.EQ.1)GO TO 900
CALL UERASE
CALL UAXIS(0.0,6.0,-1.0,4.5)
GO TO 701
900 CALL UERASE
CALL UAXIS(0.0,10.0,-1.0,9.0)
 701 CALL USET ("MOVE")
```

Table 8. Listing of Interactive Graphics Program (Continued)

```
BRAGG     CONT          16:12:12  06/20/78

 CALL UMOVE (PEE(LM),RRR(LM))
CALL USET ("LINE")
 DO 10 I = LM,KM
CALL UPEN (PEE(I),RRR(I))
10 CONTINUE
 CALL USET ("MOVE")
CALL UMOVE (0.0,0.0)
MMTRY=MMTRY + 1
PRES=PRES +.1
IF(MMTRY-11)815,815,705
705 CONTINUE
  YB=MMTRY
 YA=4.5-(YB-12)
  CALL UMOVE(0.0,YA)
 CALL UALPHA
 PRINT: "ANOTHER PRESSURE? YES=1, NO=2"
 READ: IANS
 IF (IANS.EQ.1) GO TO 810
899 CONTINUE
CALL UEND
RETURN
END
C  ******************************************************
C
C
C
C
 SUBROUTINE LADANYI
C
C
C
C  ******************************************************
COMMON X(100)
COMMON N,M,U(15),VAP
DIMENSION VA(52),STRESS(52),STRAIN(52),DVOV(52),VOL(52)
CHARACTER FNAM*4(4)
PRINT: "NAME OF FILE OR TITLE?"
READ 70,FNAM(2),FNAM(3)
70 FORMAT (2A4)
 PRINT:"ENTER INITIAL PRESSURE"
 READ,XO
PO=XO
 DO 1 I=1,52
 YO=0.0
 DO 2 J=1,N+1
IF(XO)10,4,3
 3 CONTINUE
 IF(J.EQ.1)GO TO 4
 YO=(U(J)*(XO**(J-1)))+YO
GO TO 2
 4 YO=U(1)
 2 CONTINUE
```

```
BRAGG     CONT              16:12:12  06/20/78

VA(I)=Y0
VOL(I)=VAP+VA(I)
DVOV(I)=(VA(I)-VA(1))/VOL(I)
IF(X0.GT.X(M)) GO TO 45
X0=X0+.2
Y0=0.0
1 CONTINUE
45 KK=I-1
PRINT: "DO YOU WANT TABLE OF STRESS AND STRAIN? YES=1, NO=2"
READ: IRRR
IF(IRRR.EQ.2) GO TO 30
PRINT:"CLEAR SCREEN FOR TABLE THEN PRESS ANY CHARACTER AND RETURN"
CALL UPAUSE
PRINT 50,FNAM(2),FNAM(3),P0
50 FORMAT (2A4,"INITIAL PRESSURE = ",1F4.1,///)
PRINT:"     ---------------------------------------------"
PRINT 40
PRINT:"     ---------------------------------------------"
30 DO 8 I=2,KK
X02=I
STRESS(I)=(.2*SQRT(3.0))/(DLOG(DVOV(I+1))-DLOG(DVOV(I)))
STRAIN(I)=(100.0/SQRT(3.0))*(DVOV(I+1)+DVOV(I))
 STRAIN(I)=0.5*STRAIN(I)
 P=(X02-1)*.2
 DV=VA(I)-VA(1)
IF (IRRR.EQ.2) GO TO 80
 PRINT 100,P,DV,DVOV(I)
 PRINT 200,STRESS(I),STRAIN(I)
 100 FORMAT (F10.1,F10.2,2X,F10.6)
 200 FORMAT (32X,2F10.2)
80 IF(I.NE.26) GO TO 8
IF(I.EQ.KK) GO TO 10
IF (IRRR.EQ.2) GO TO 66
PRINT 88
88 FORMAT("       ---------------------------------------------")
PRINT 89
89 FORMAT (//////////)
PRINT 88
PRINT 40
40 FORMAT ("       DP       DV       DV/V0       STRESS   STRAIN")
PRINT:"     ---------------------------------------------"
66 CONTINUE
8 CONTINUE
10 CONTINUE
IF (IRRR.EQ.2) GO TO 85
PRINT:"     ---------------------------------------------"
PRINT:"     COPY THEN PRESS ANY CHARACTER AND RETURN!"
CALL UPAUSE
85 DO 60 I=2,KK
IF (STRESS(I+1).LT.STRESS(I)) GO TO 65
60 CONTINUE
65 KKM=I
```

Table 8. Listing of Interactive Graphics Program (Concluded)

```
BRAGG     CONT           16:12:12  06/20/78

CALL USTART
CALL USET ("LINE")
CALL USET ("GRIDAXES")
CALL USET ("XBOTHLABELS")
CALL UPSET ("XLABEL","      STRAIN , IN %\")
CALL USET ("YBOTHLABELS")
CALL UPSET ("YLABEL","      STRESS , IN KG/CM2\")
CALL UAXIS(0.0,STRAIN(KK),0.0,STRESS(KKM))
CALL UMOVE (0.0,0.0)
DO 20 I =2,KK
CALL UPEN (STRAIN(I),STRESS(I))
20 CONTINUE
CALL UEND
PRINT 25,PO
25 FORMAT (///,"INITIAL PRESSURE (PO) =",1F4.1,/)
CALL USET ("LINE")
RETURN
END
```

APPENDIX B: K_o CU TRIAXIAL COMPRESSION AND DIRECT SHEAR TEST DATA

This appendix contains K_o CU triaxial compression and direct shear test data for embankment samples as listed below.

Embankment	Sample No.	Depth, ft	Figure No.
	K_o CU Triaxial Compression Tests		
Tennessee I-75, sta. 960+82	4	13.75-15.0	49
	5	16.4 -17.7	50
	6	18.9 -20	51
	11	29.8 -31.1	52
	12	32.5 -33.8	53
	26	67.4 -68.7	54
	31	81.8 -83.0	55
West Virginia U. S. 460, sta. 553+75	3	11.1 -12.2	56
	12	33 -34	57
	16	43 -44	58
	29	70 -71	59
	33	80	60
Kentucky I-75, sta. 678+55	1	9.4 -10.7	61
	3	14.4 -15.7	62
Ohio I-74 at SR 128, sta. 424+90	1	6.1 - 7.4	63
	7	20.2 -21.2	64
	12	33.6 -34.9	65
	15	41.4 -42.4	66
Ohio I-74 at Wesselman Road, sta. 492+00	1	6.2 - 7.4	67
	11	30.0 -31.3	68
Indiana I-74, sta.435+83	10	30.3 -31.4	69
	22	60.0 -61.0	70
	Direct Shear Tests		
Kentucky I-75, sta. 678+55	6	21.0	71
	11	36.5	72
Indiana I-74, sta. 435+83	6	21.0	73
	16	45.5	74
	20	55.0	75

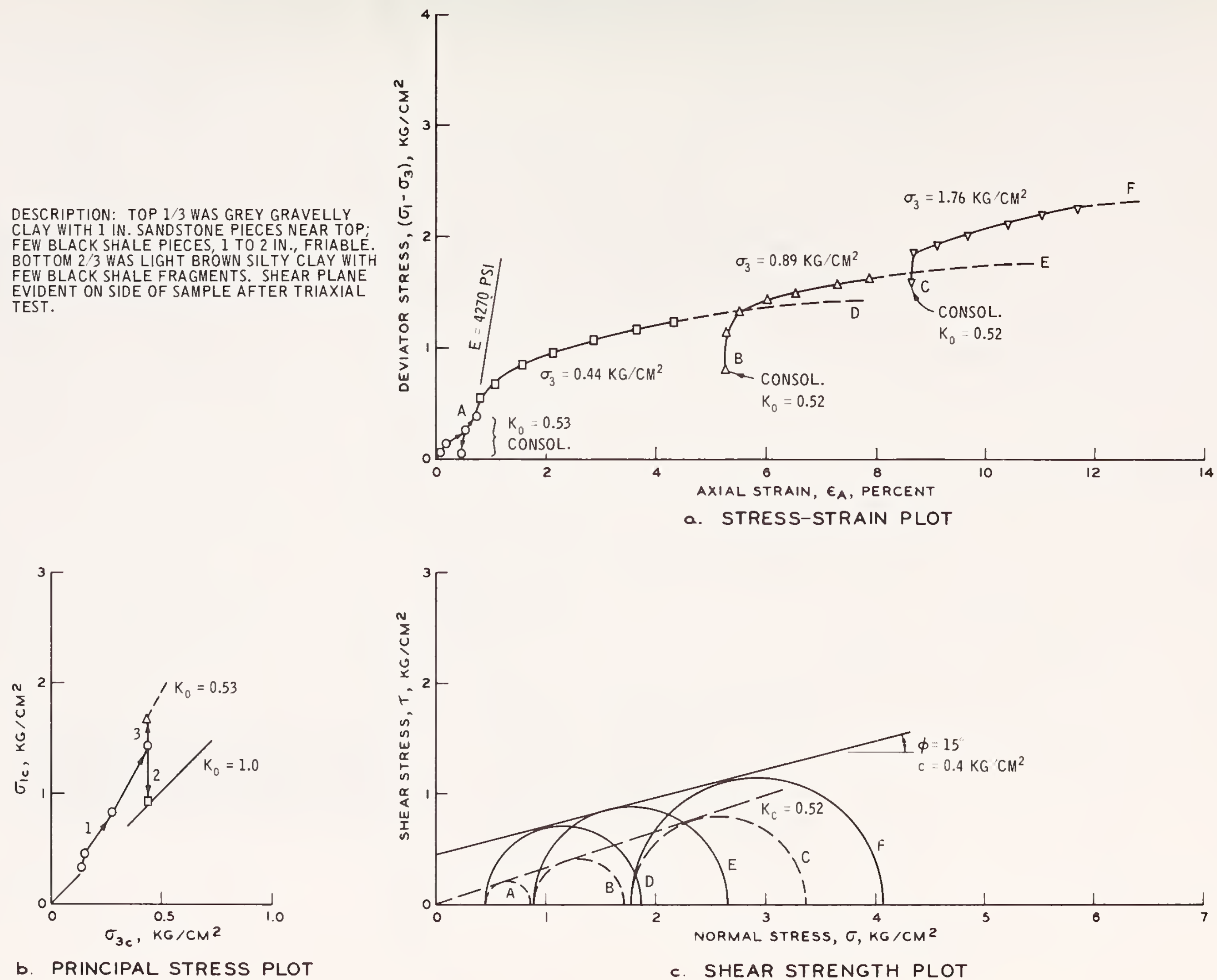


Figure 49. TN I-75, sample No. 4, depth: 13.75-15 ft

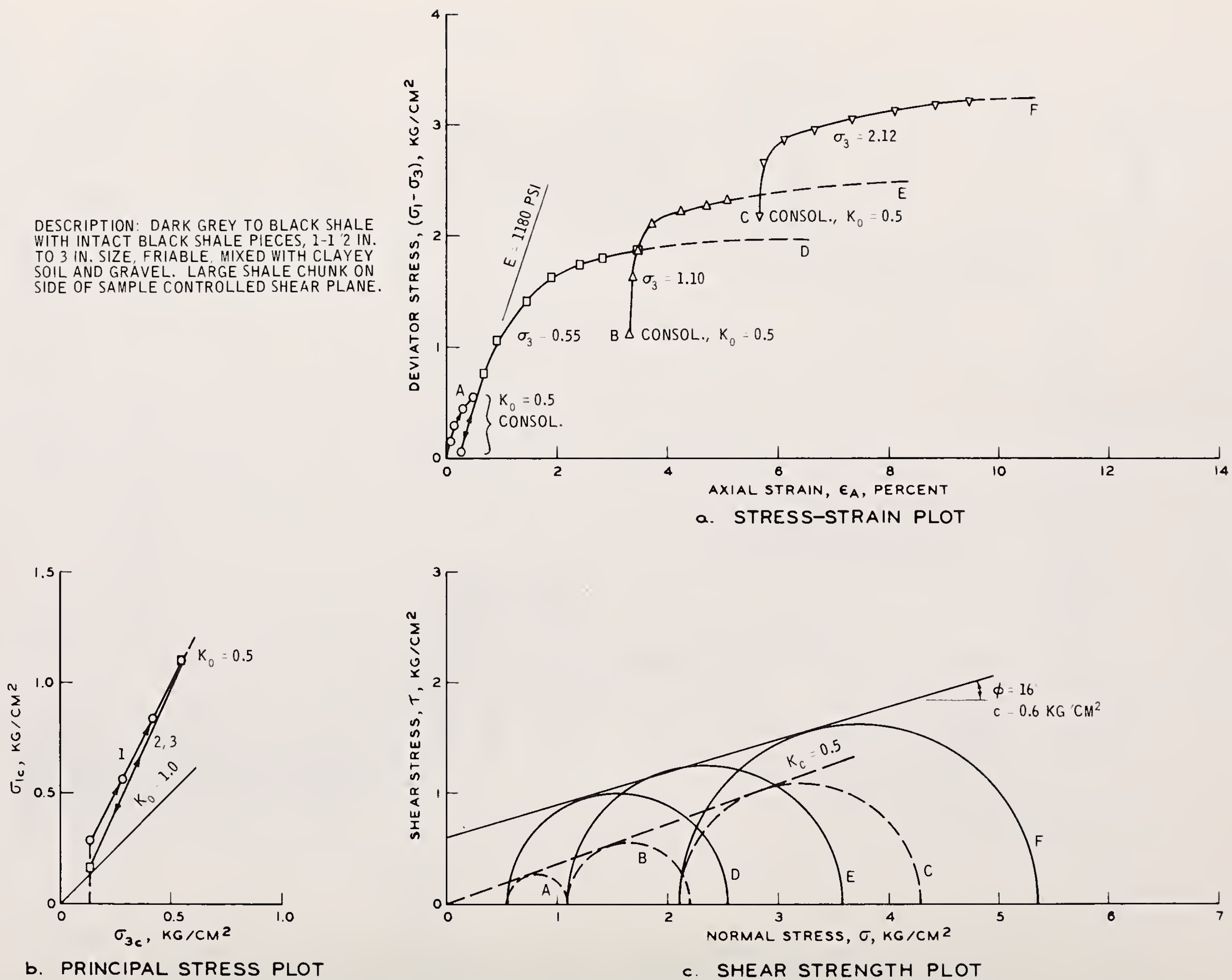


Figure 50. TN I-75, sample No. 5, depth: 16.4-17.7 ft

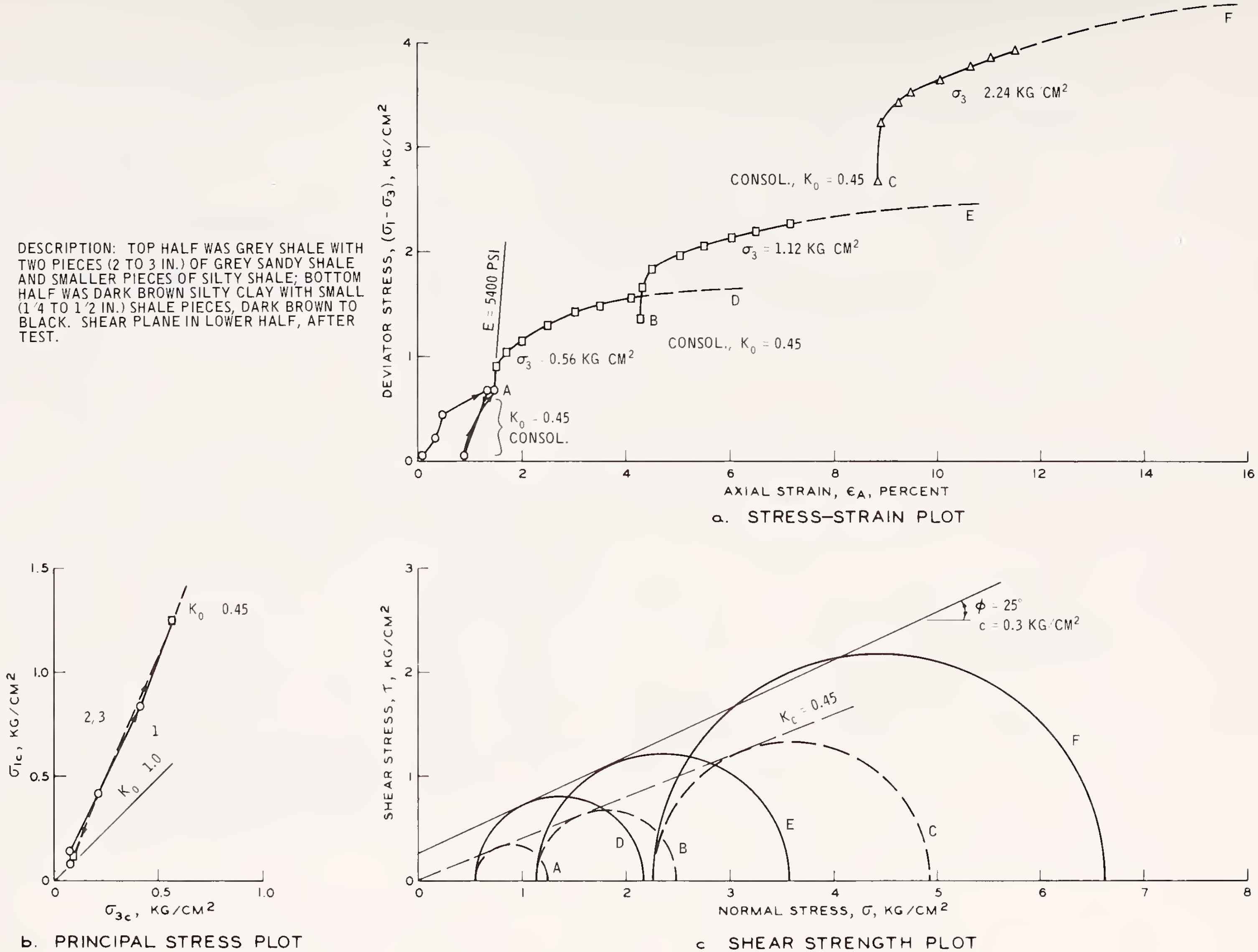
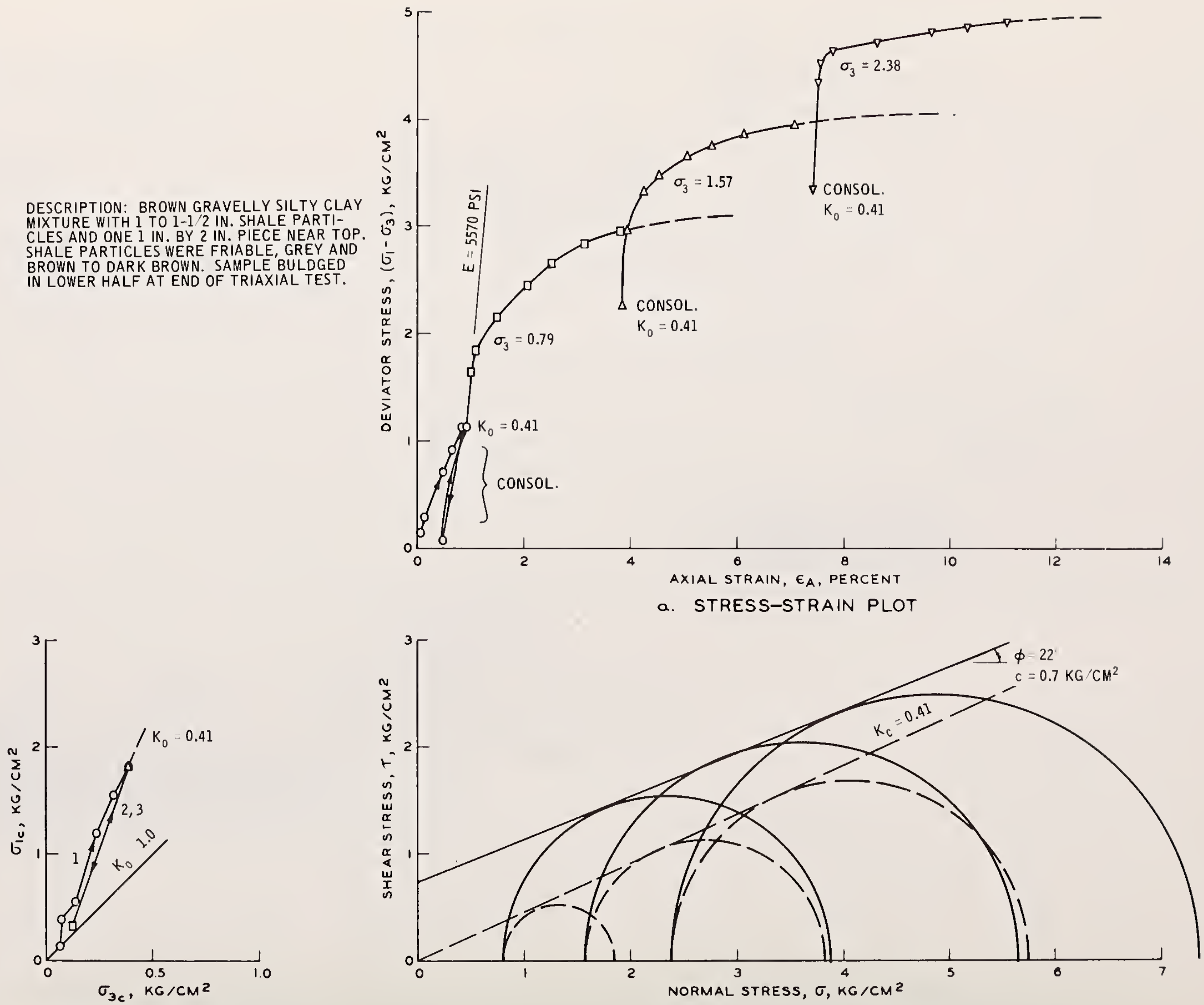


Figure 51. TN I-75, sample No. 6, depth: 18.9-20 ft



a. STRESS–STRAIN PLOT

b. PRINCIPAL STRESS PLOT

c. SHEAR STRENGTH PLOT

Figure 52. TN I-75, sample No. 11, depth: 29.8-31.1 ft

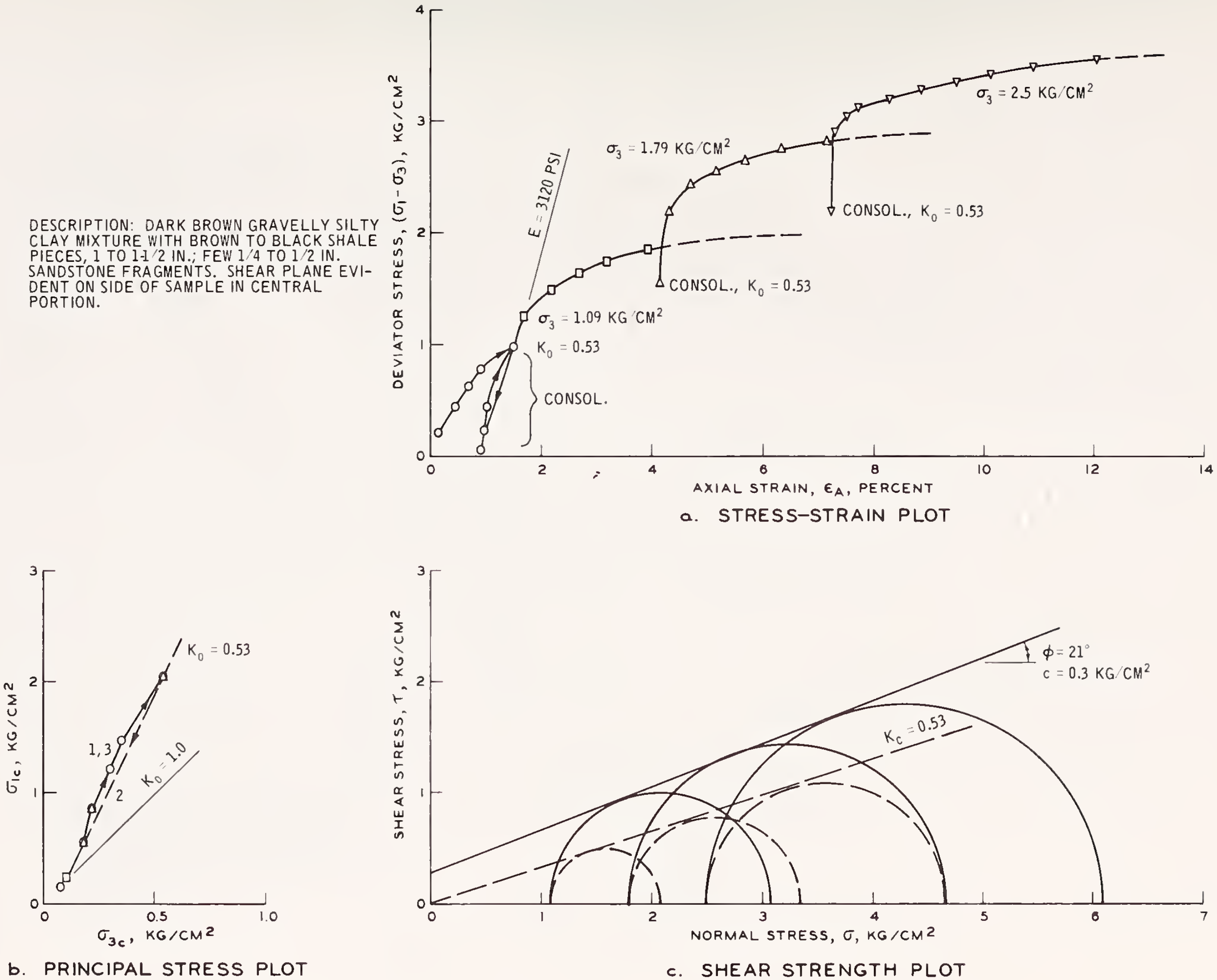


Figure 53. TN I-75, sample No. 12, depth: 32.5-33.8 ft

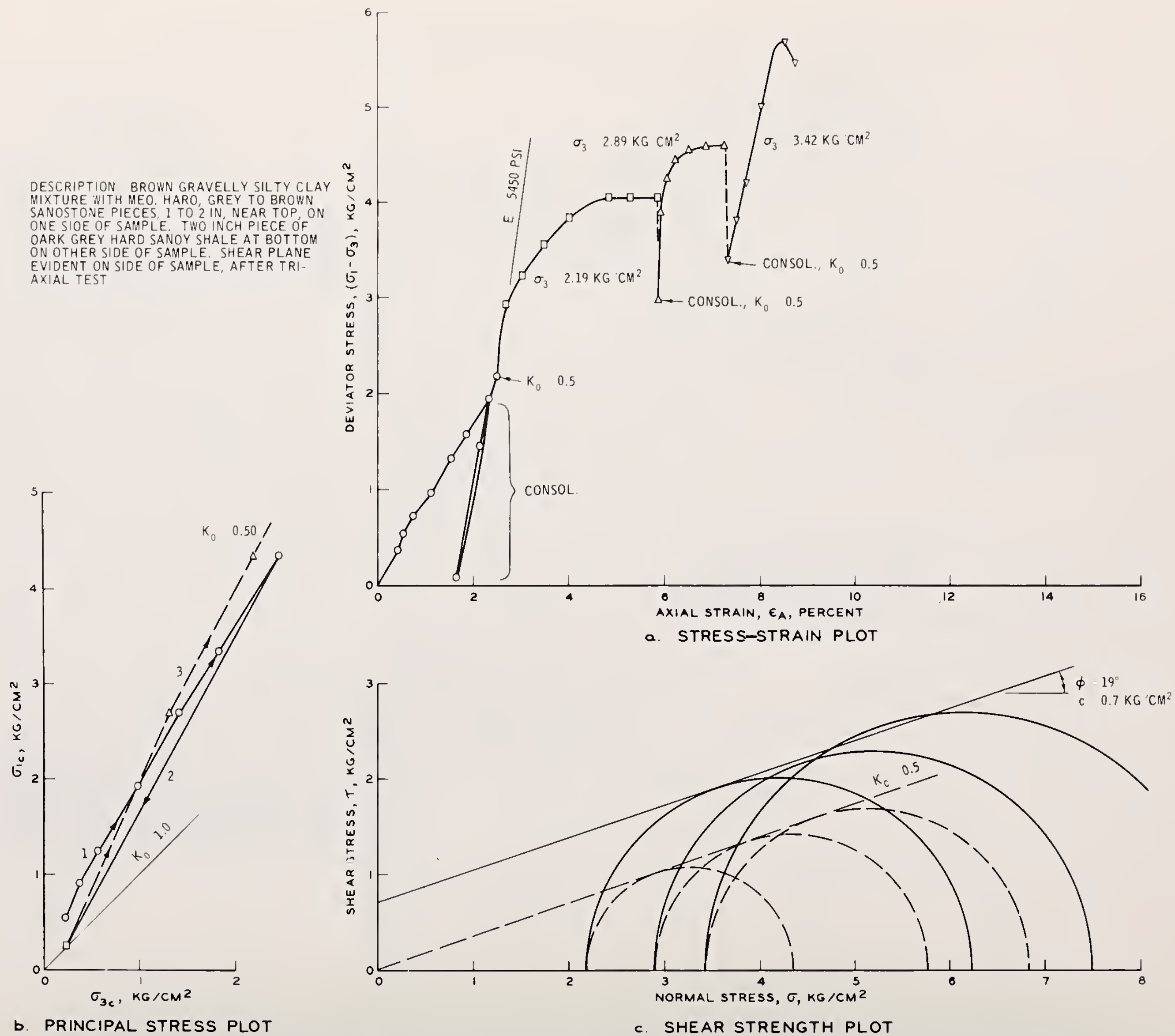


Figure 54. TN I-75, sample No. 26, depth: 67.4–68.7 ft

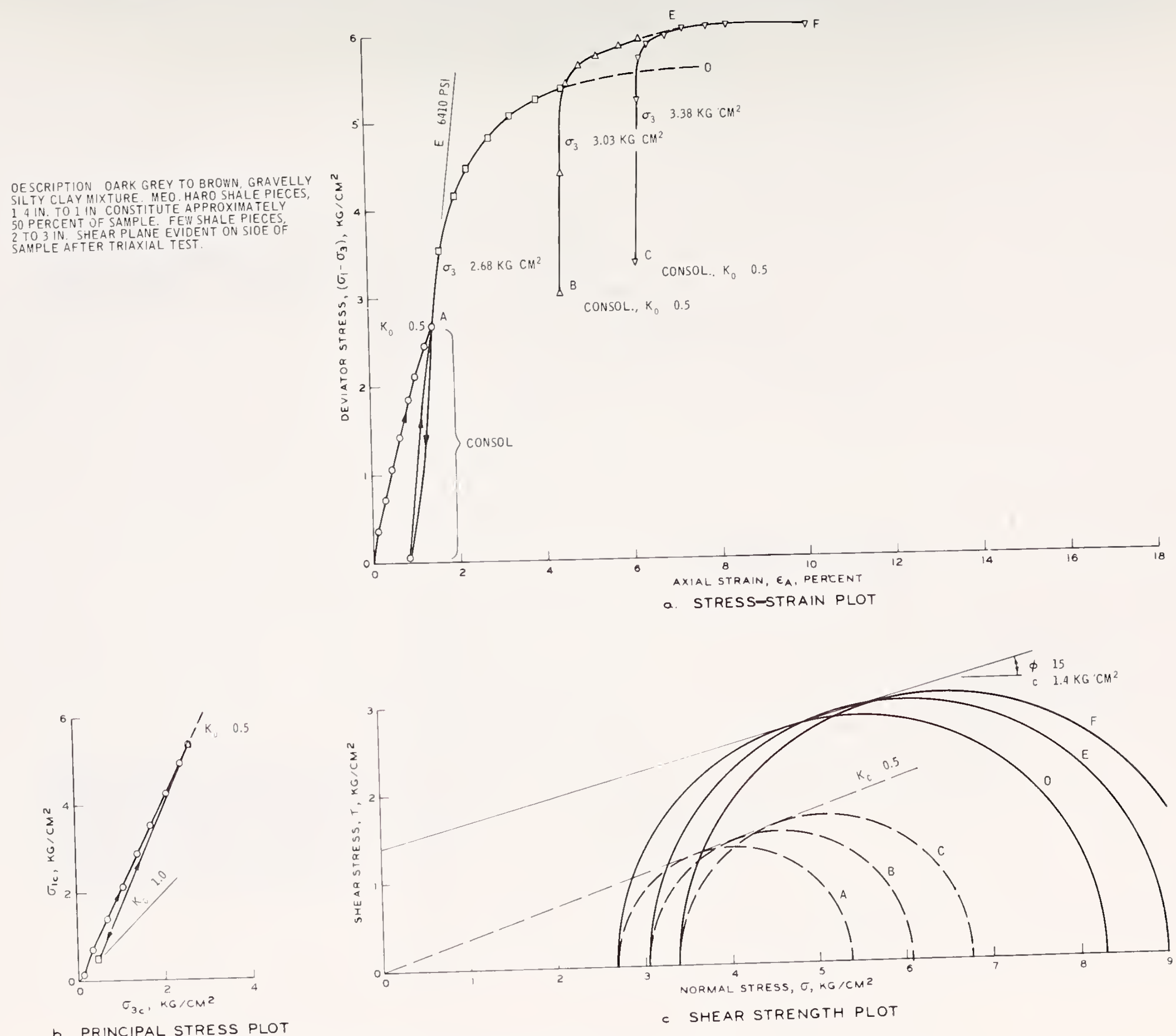
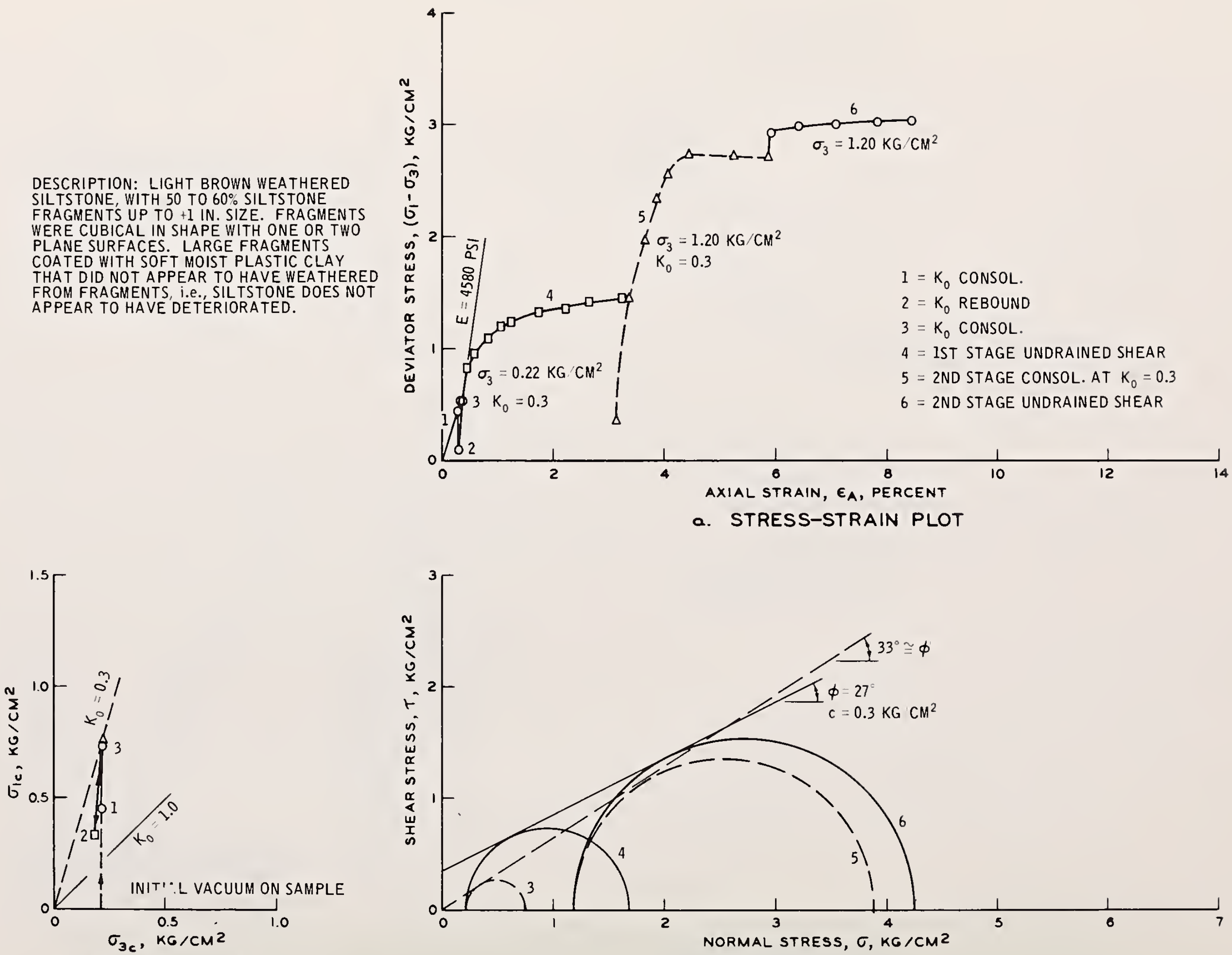


Figure 55. TN I-75, sample No. 31, depth: 81.8-83.0 ft



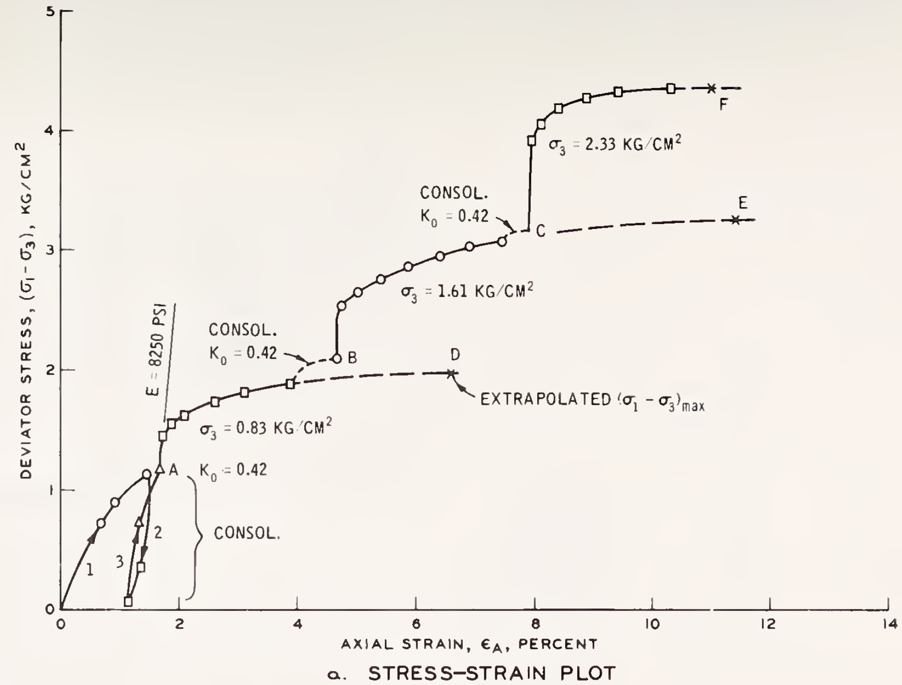
a. STRESS–STRAIN PLOT

b. PRINCIPAL STRESS PLOT

c. SHEAR STRENGTH PLOT

Figure 56. WV US 460, sample No. 3, depth: 11.1-12.2 ft

DESCRIPTION: MOTTLED DARK GREY, BROWN AND TAN WEATHERED SHALE. CONTAINED FRIABLE SHALE FRAGMENTS FROM SAND SIZE TO APPROXIMATELY 1-1/2 IN. MAXIMUM (ABOUT 10 PERCENT OF SAMPLE).



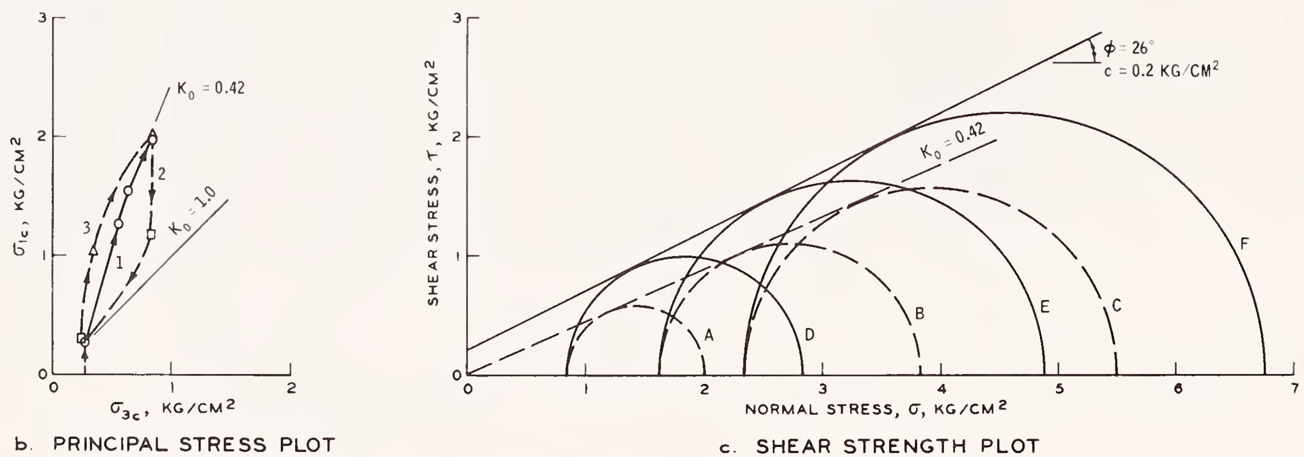
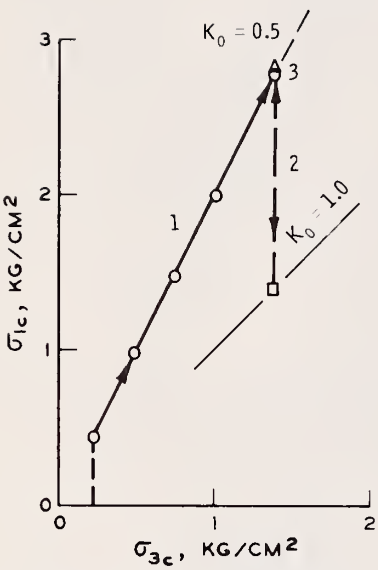
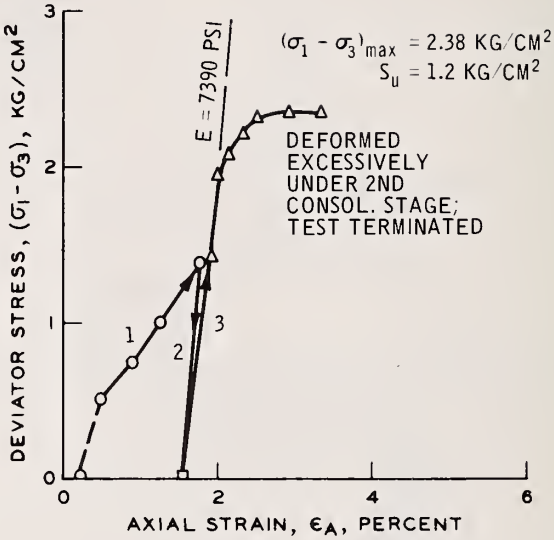


Figure 57. WV US 460, sample No. 12, depth: 33-34 ft

DESCRIPTION: TAN AND YELLOWISH TAN CLAYEY SANDY GRAVEL (GC). CONTAINED SILTSTONE PARTICLES UP TO 4 IN. MAXIMUM SIZE AND 1/2 IN. THICK LAYER OF YELLOWISH TAN CLAY (CH) 5 IN. FROM TOP OF SPECIMEN; HORIZONTAL CRACK APPROXIMATELY 3-1/2 IN. FROM TOP.



b. PRINCIPAL STRESS PLOT



a. STRESS-STRAIN PLOT

Figure 58. WV US 460, sample No. 16, depth: 43-44 ft

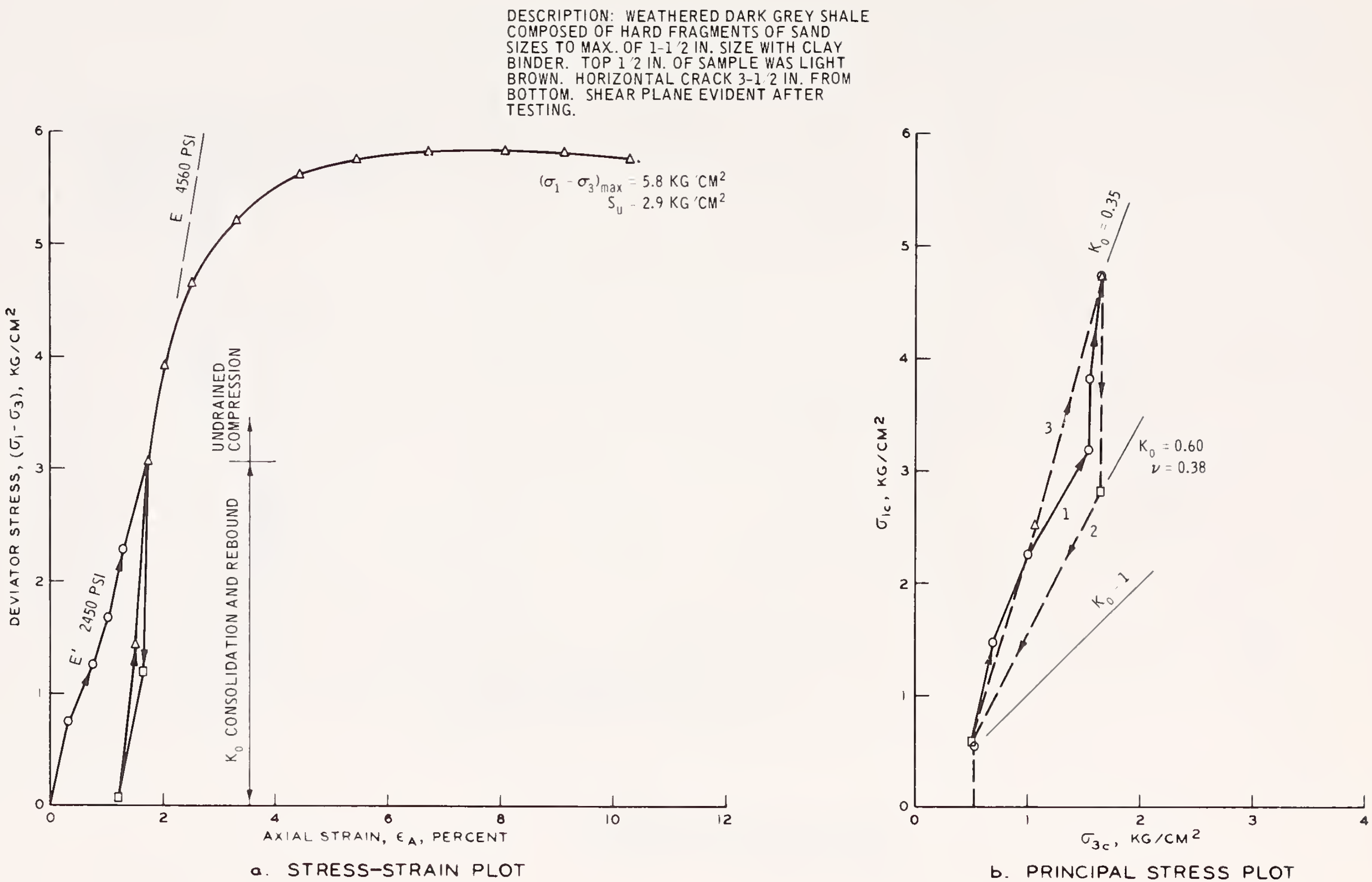


Figure 59. WV US 460, sample No. 29, depth: 70-71 ft

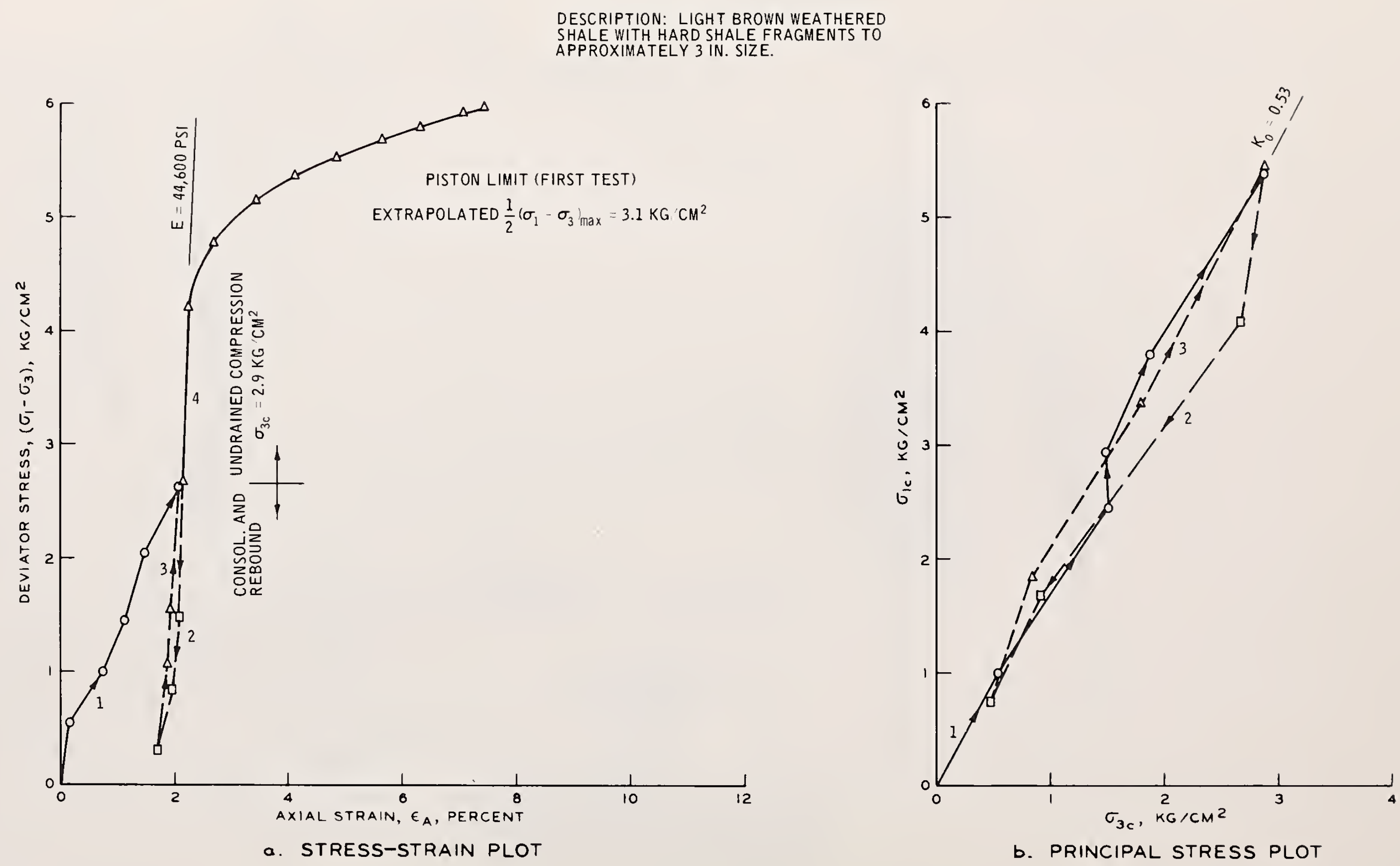
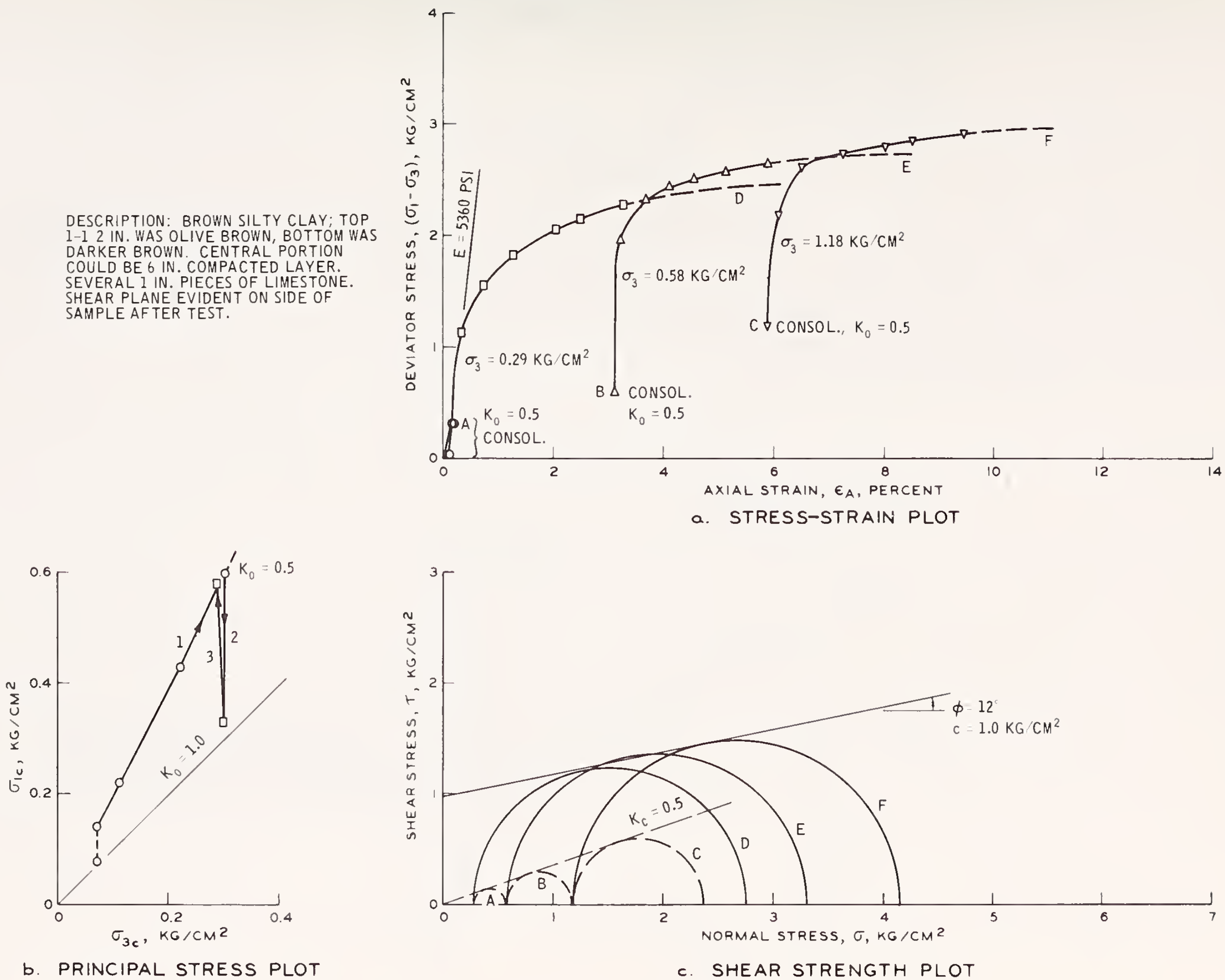


Figure 60. WV US 460, sample No. 33, depth: 80 ft



a. STRESS-STRAIN PLOT

b. PRINCIPAL STRESS PLOT

c. SHEAR STRENGTH PLOT

Figure 61. KY I-75, sample No. 1, depth: 9.4-10.7 ft

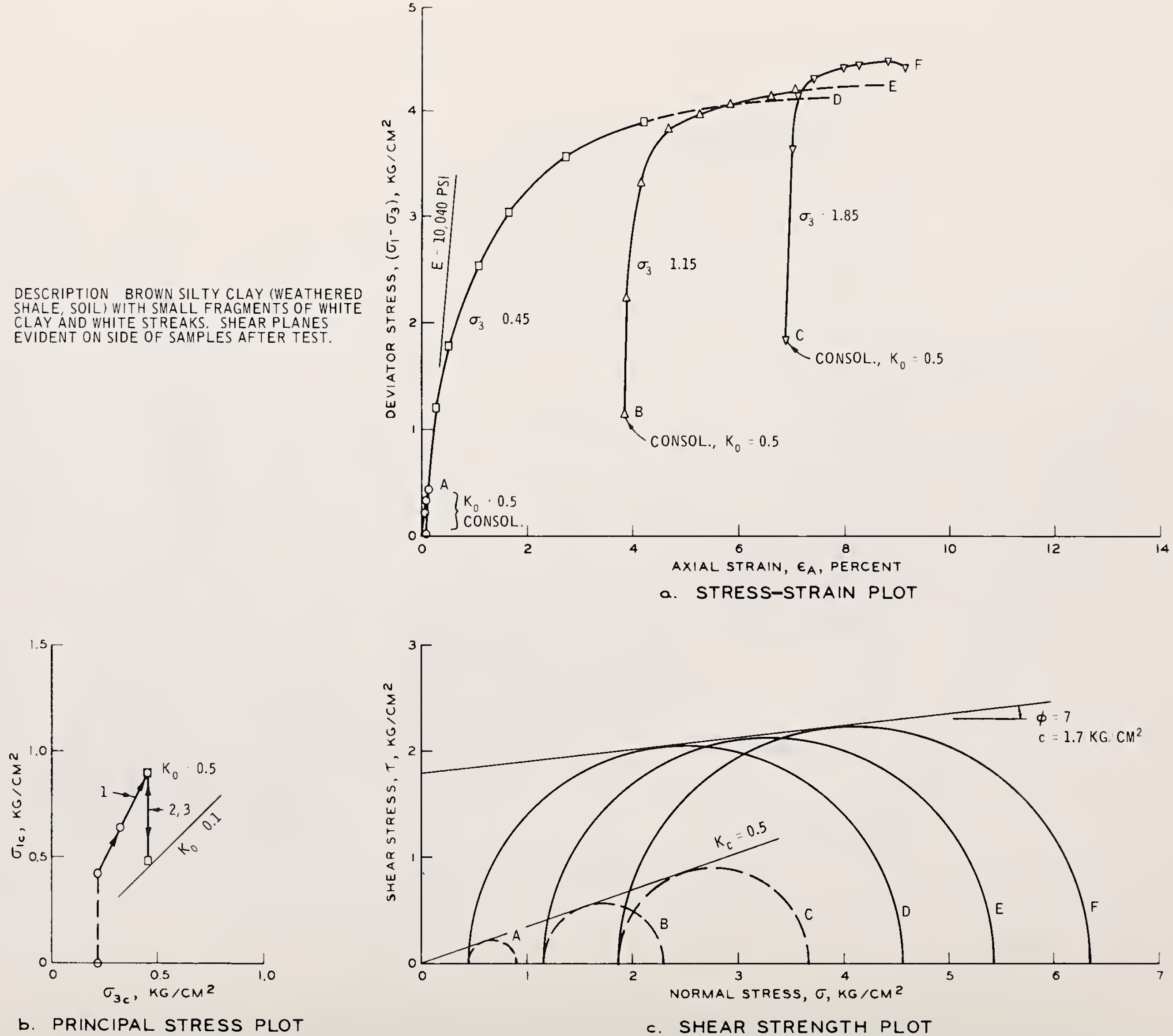


Figure 62. KY I-75, sample No. 3, depth: 14.4-15.7 ft

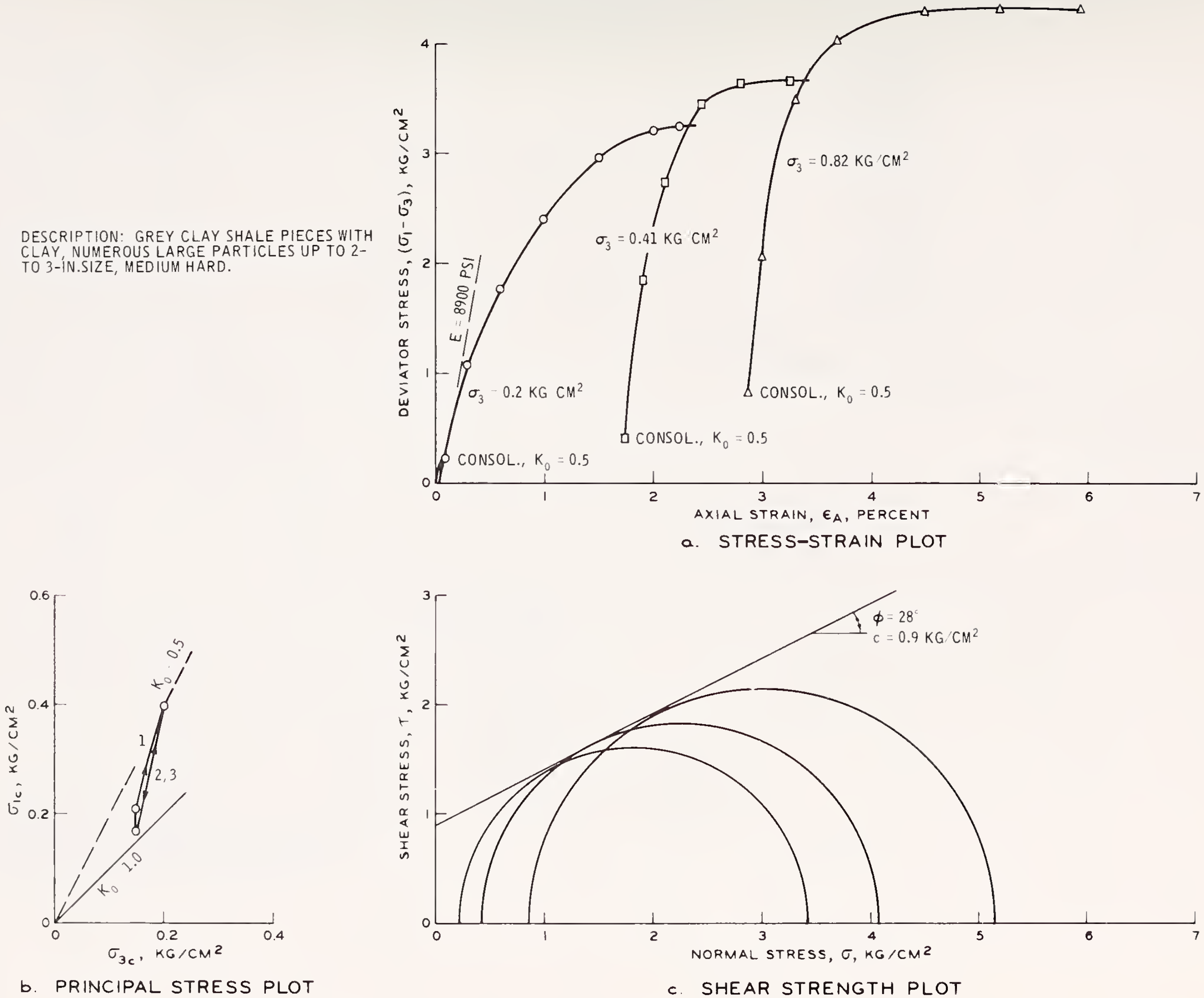
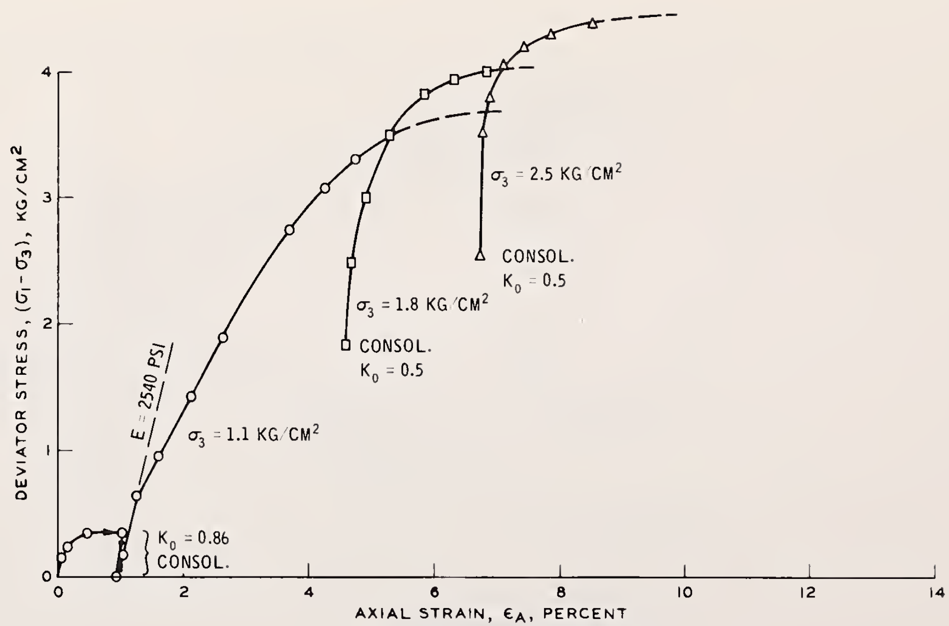
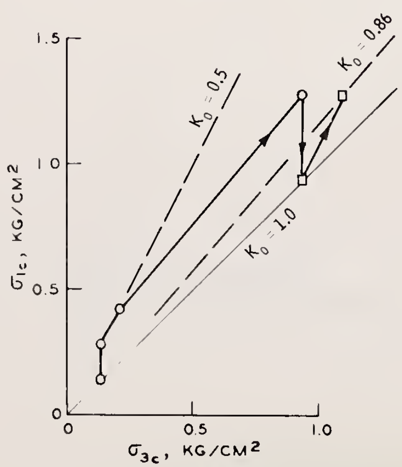


Figure 63. OH I-74 at SR 128, sample No. 1, depth: 6.1-7.4 ft

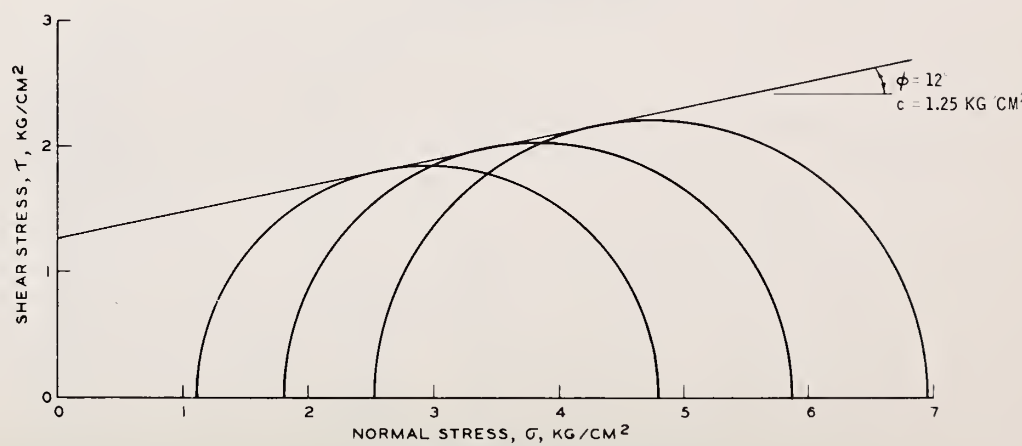
DESCRIPTION: BROWN CLAY WITH SMALL 1/2 IN. TO 1 IN. (FEW) PIECES OF GREY TO MOSTLY BROWN CLAY SHALE, FRIABLE, MEDIUM HARD. ONE 2 IN. PIECE OF BROWN SHALE ON ONE SIDE. SAMPLE WAS DENSE. SHEAR PLANE EVIDENT ON SAMPLE SIDE AFTER TEST.



a. STRESS–STRAIN PLOT



b PRINCIPAL STRESS PLOT



c. SHEAR STRENGTH PLOT

Figure 64. OH I-74 at SR 128, sample No. 7, depth: 20.2-21.2 ft

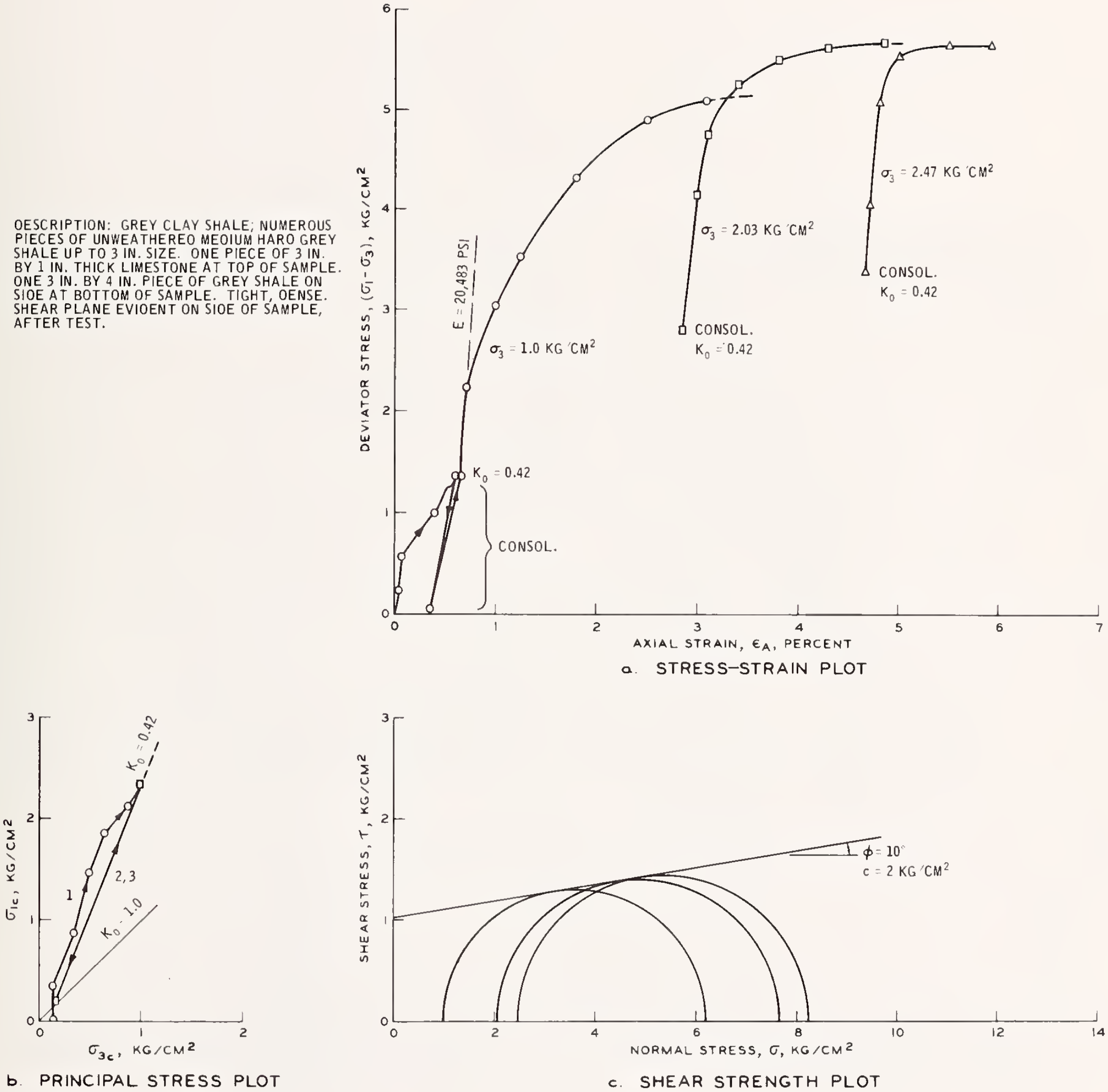


Figure 65. OH I-74 at SR 128, sample No. 12, depth: 33.6-34.9 ft

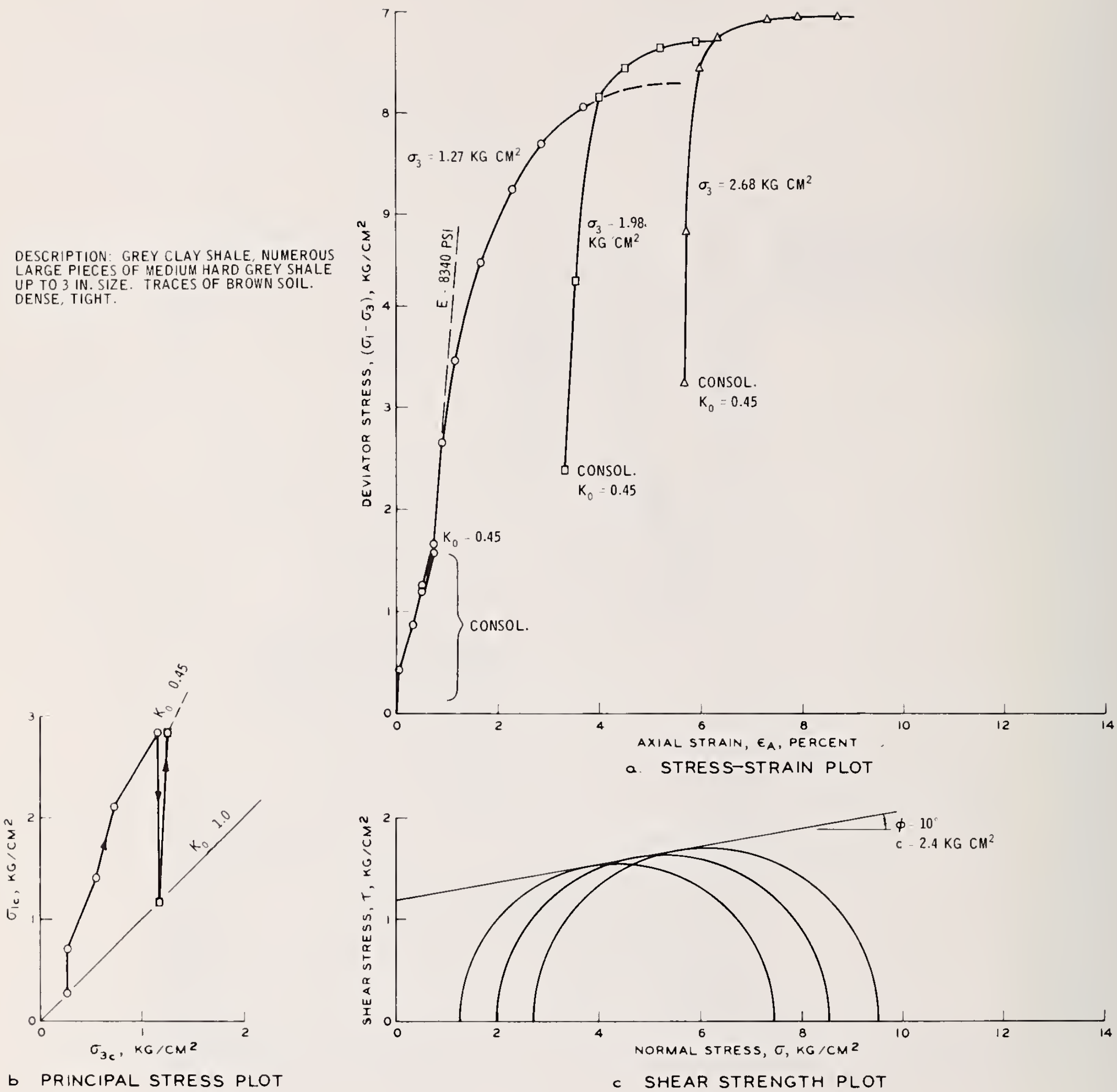


Figure 66. OH I-74 at SR 128, sample No. 15, depth: 41.4-42.4 ft

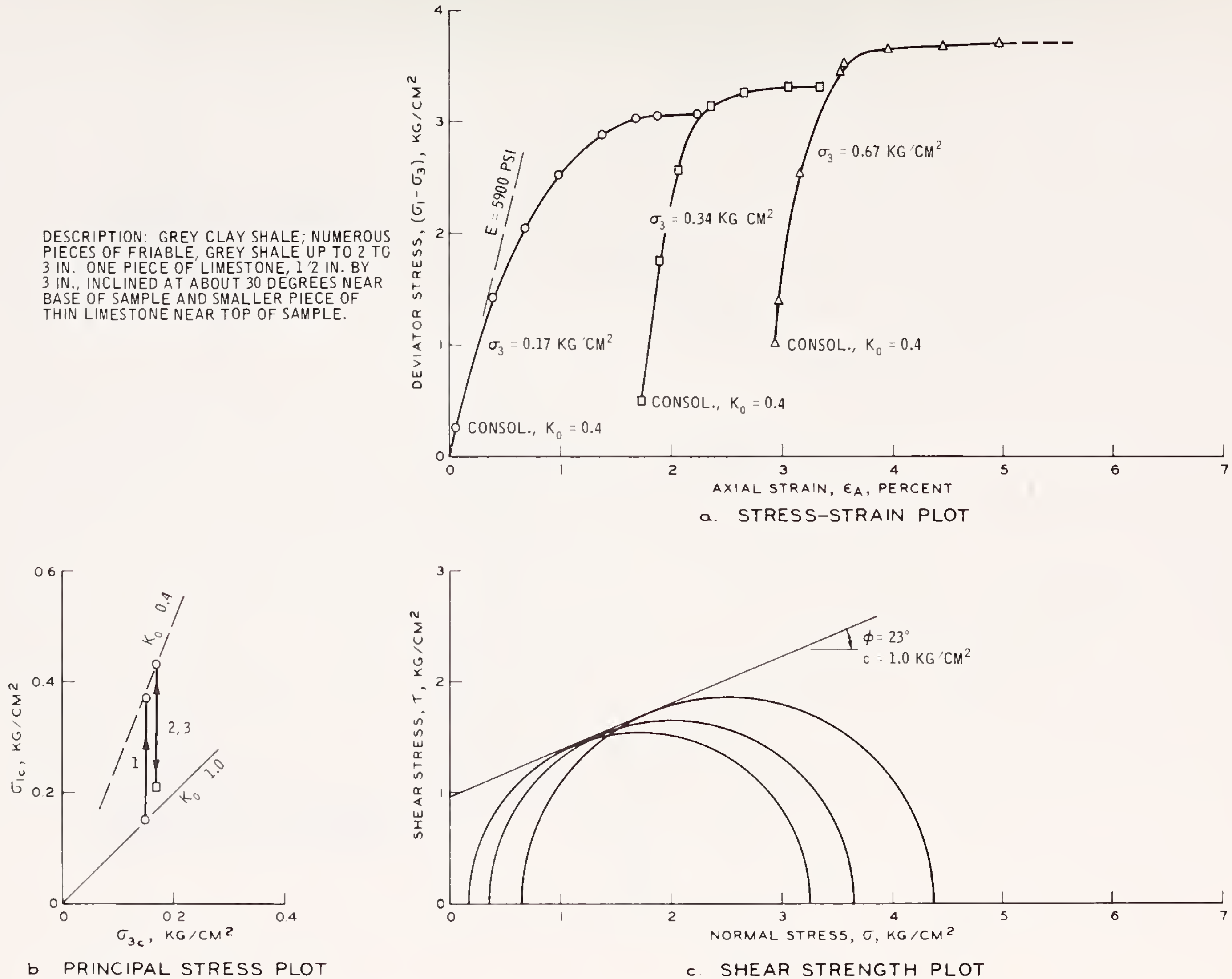


Figure 67. OH I-74 at W'N RD, sample No. 1, depth: 6.2-7.4 ft

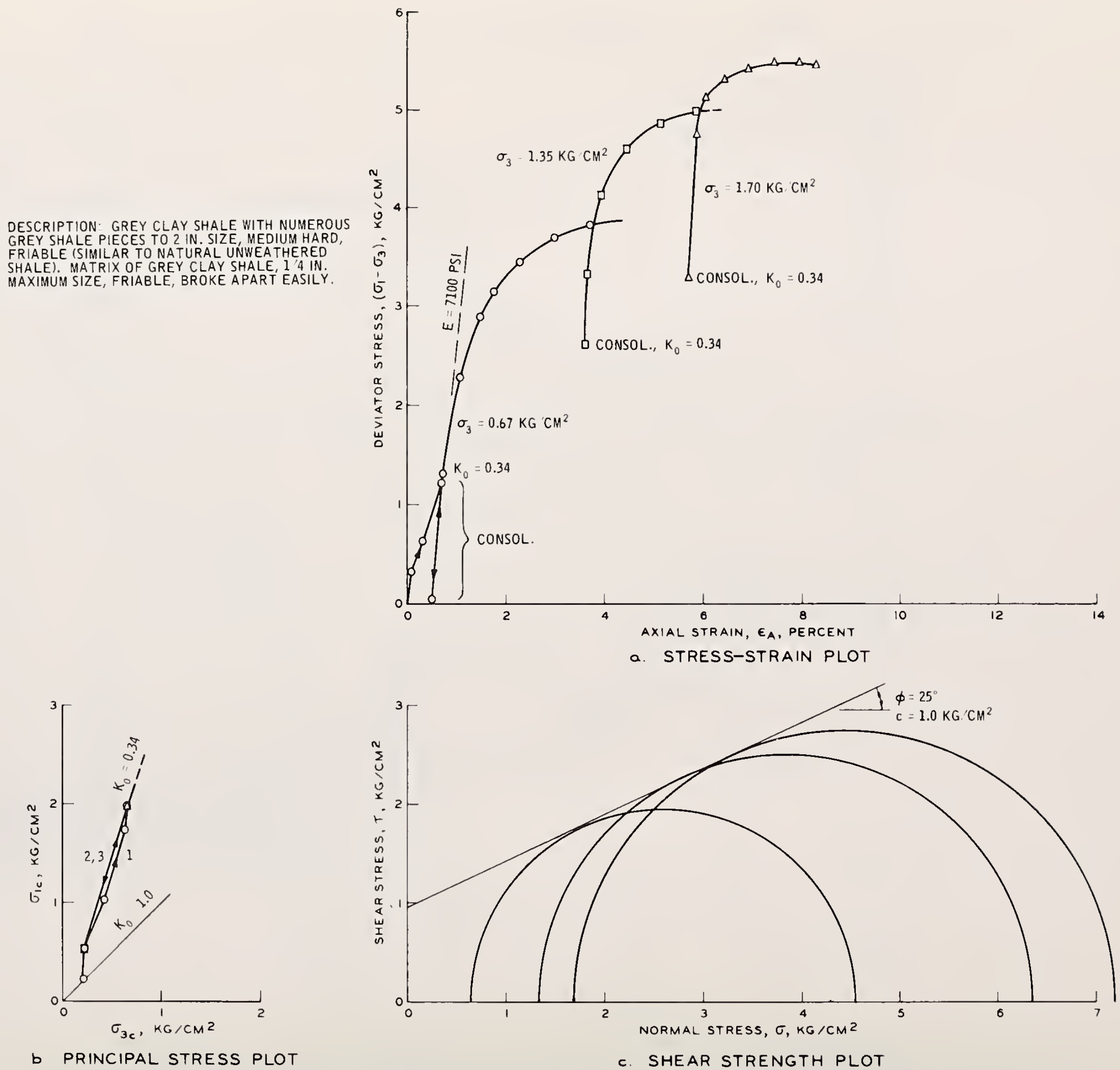


Figure 68. OH I-74 at W'N RD, sample No. 11, depth: 30.0-31.3 ft

DESCRIPTION: GREY WEATHERED COMPACT SHALE; CONTAINS SHALE AND LIMESTONE PIECES UP TO 3 IN. MAXIMUM SIZE. SHALE PIECES WERE CUBICAL IN SHAPE AND CRUMBLY TO HARD.

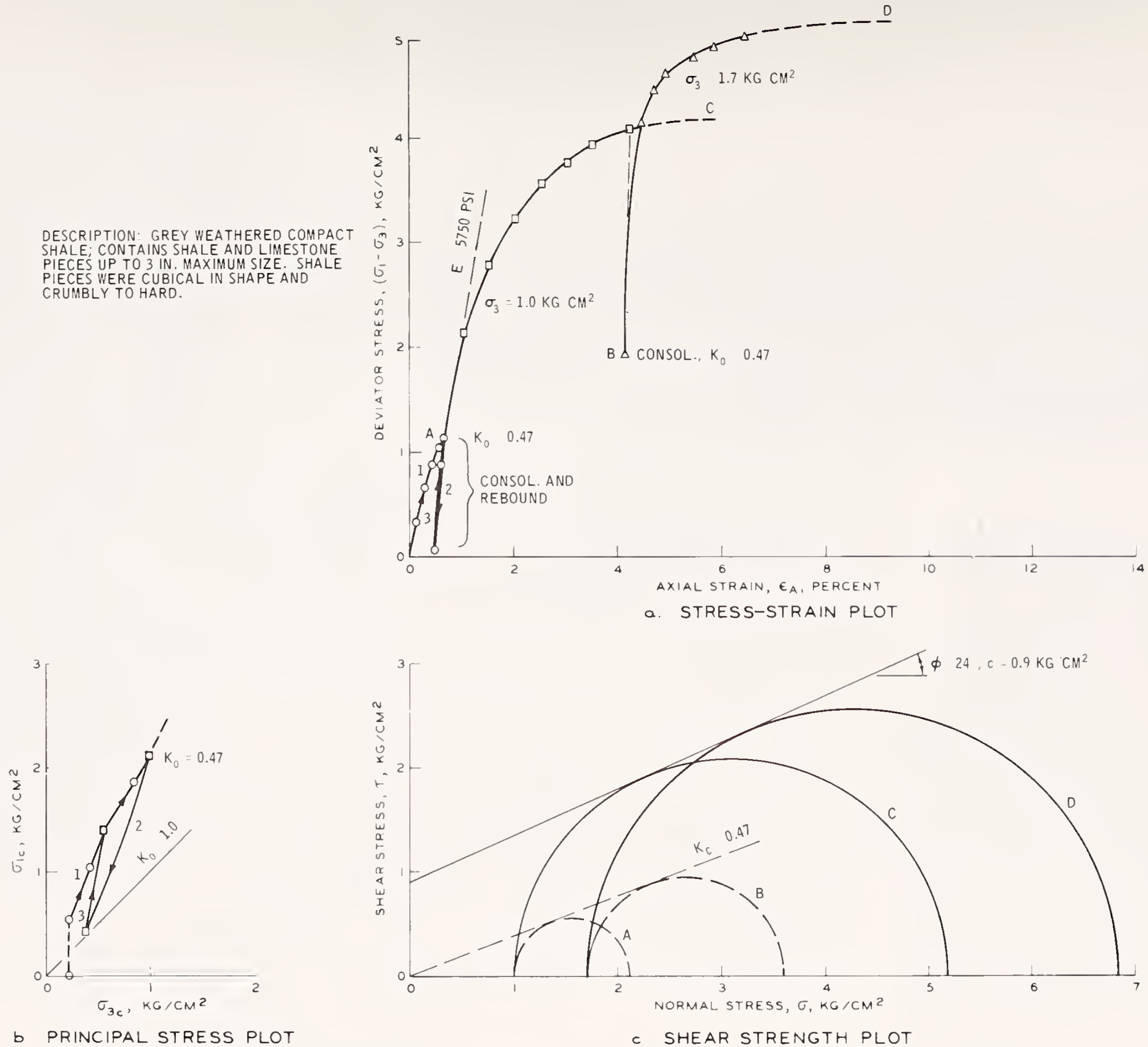


Figure 69. IN I-74, sample No. 10, depth: 30.3-31.4 ft

DESCRIPTION: TAN SANDY CLAY (CL), HARD CONTAINED AT LEAST ONE 2-1/2-IN. LIMESTONE PIECE.

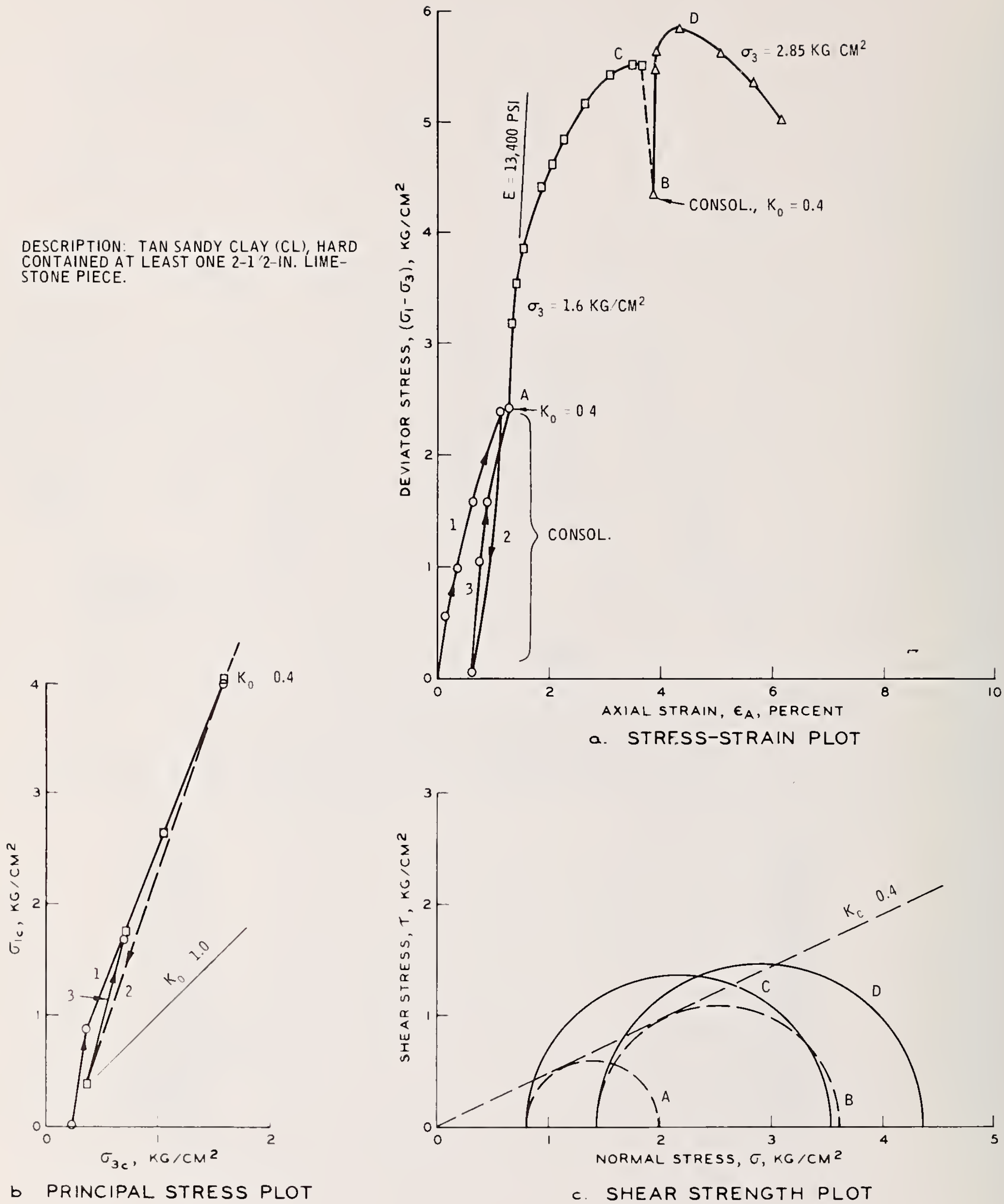
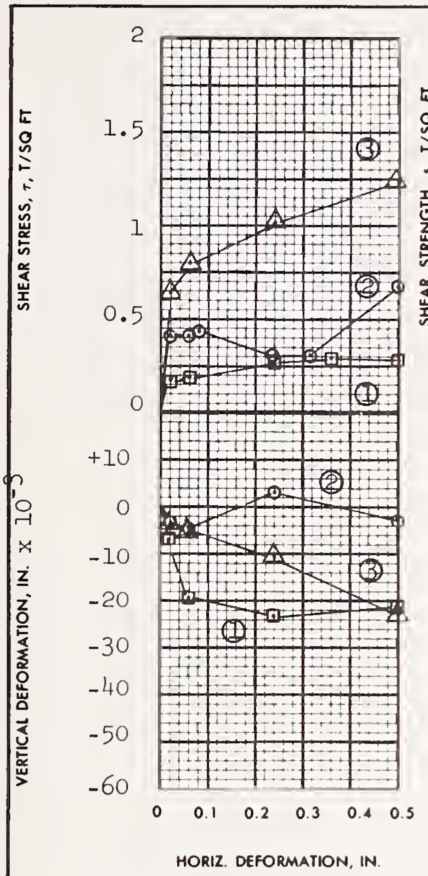
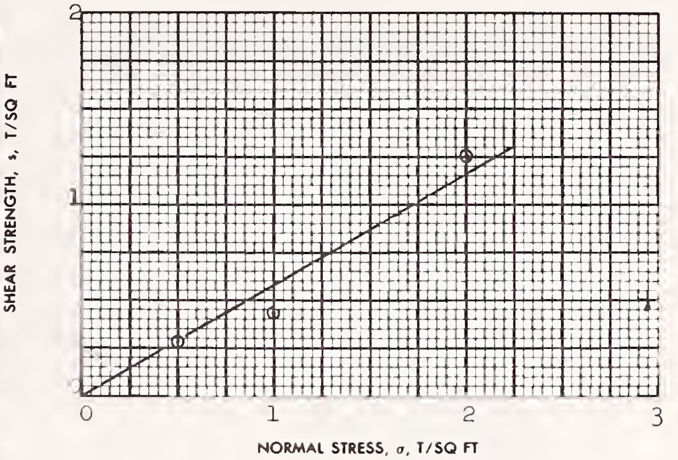


Figure 70. IN I-74, sample No. 22, depth: 60-61 ft





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	11.9%	9.8 %	9.7%	%
	VOID RATIO	e_o	0.607	0.581	0.564	
	SATURATION	S_o	52.9 %	45.5%	46.4 %	%
	DRY DENSITY, LB/CU FT	γ_d	104.0	106.6	107.8	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	<1	4	<1	
FINAL	WATER CONTENT	w_f	15.8%	14.6 %	15.0%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	0.5	1.0	2.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.29	0.43	1.25	
ACTUAL TIME TO FAILURE, MIN		t_f	180	480	2670	
RATE OF STRAIN, IN./MIN			.00019	.00019	.00019	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

SHEAR STRENGTH PARAMETERS

ϕ' = 30°

TAN ϕ' = 0.577

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

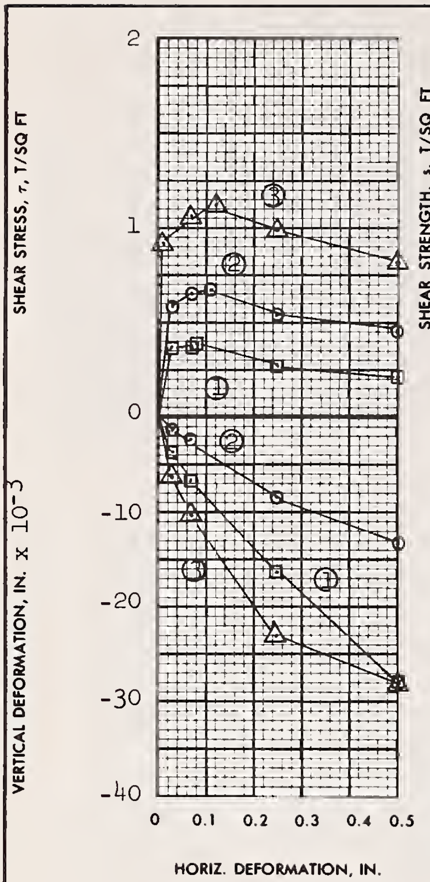
☒ CONTROLLED STRAIN

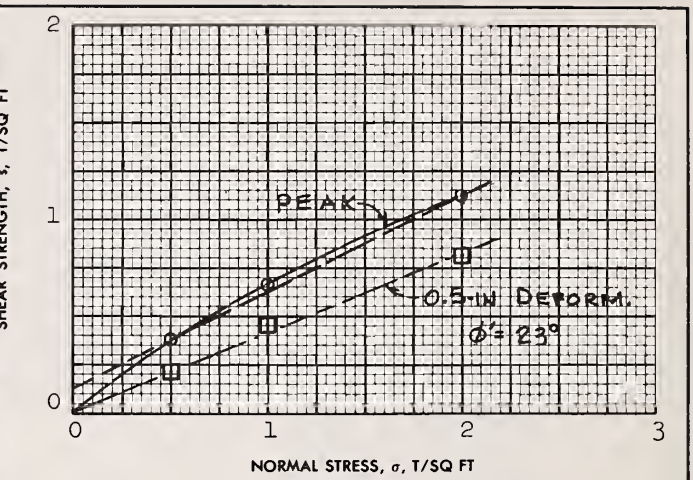
TYPE OF SPECIMEN	UNDISTURBED	3.01	IN. SQUARE	0.616	IN. THICK
CLASSIFICATION	PLASTIC CLAY(CH), GRAY;SHALE FRAGMENTS UP TO 1/2"				
LL	PL	PI		G_s 2.70(est)	

REMARKS

PROJECT	COMPACTED SHALE EMBANKMENT STUDY		
AREA	KENTUCKY I-75, Sta 678+55		
BORING NO.	1	SAMPLE NO.	6
DEPTH EL	21 ft	DATE	8 Nov. 1976
BWG	DIRECT SHEAR TEST REPORT		

Figure 71. Kentucky I-75, sample 6





PEAK SHEAR STRENGTH PARAMETERS*

ϕ' = 25°

TAN ϕ' = 0.466

c' = 0.15 T/SQ FT

* For σ > 0.5TSF

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

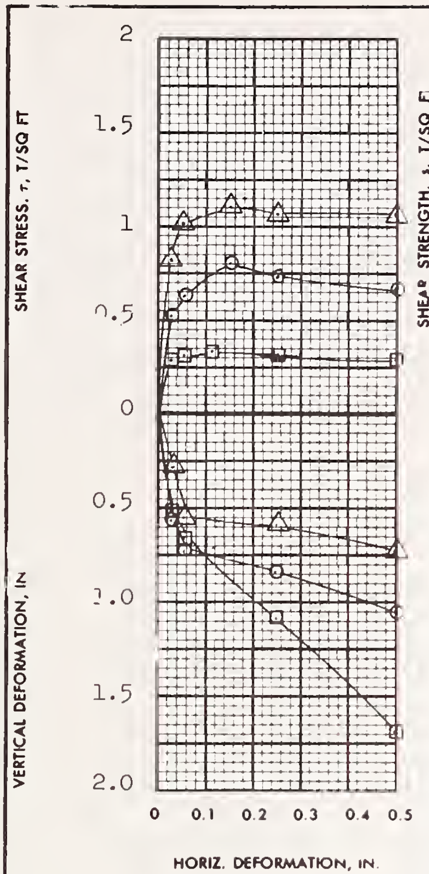
TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	22.3%	17.8%	18.6%	%
	VOID RATIO	e_o	0.569	0.505	0.572	
	SATURATION	S_o	100+ %	95.2%	87.8%	%
	DRY DENSITY, LB/CU FT	γ_d	107.4	112.0	107.2	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	1	<1	1	
FINAL	WATER CONTENT	w_f	23.7 %	20.2%	20.6%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	0.5	1.0	2.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.38	0.67	1.12	
ACTUAL TIME TO FAILURE, MIN		t_f	420	630	660	
RATE OF STRAIN, IN./MIN			.00019	.00019	.00019	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

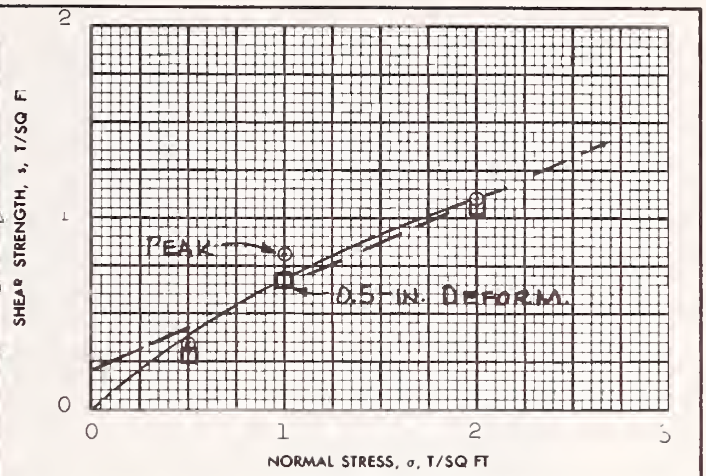
TYPE OF SPECIMEN	UNDISTURBED	3.01 IN. SQUARE	0.616 IN. THICK
CLASSIFICATION	PLASTIC CLAY(CH), LIGHT BROWN		
LL	PL	PI	G_s 2.70(est)

REMARKS

PROJECT	COMPACTED SHALE EMBANKMENT STUDY
AREA	KENTUCKY I-75, Sta 678+55
BORING NO. 1	SAMPLE NO. 11
DEPTH EL 36.5 ft	DATE 16 Nov. 1976
BWG	DIRECT SHEAR TEST REPORT

Figure 72. Kentucky I-75, sample 11





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	10.4 %	12.6 %	11.5 %	%
	VOID RATIO	e_o	0.405	0.403	0.436	
	SATURATION	S_o	69.3 %	84.4 %	71.2 %	%
	DRY DENSITY, LB/CU FT	γ_d	120.0	120.1	117.4	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	2	1	1	
FINAL	WATER CONTENT	w_f	%	%	%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	0.5	1.0	2.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.34	0.80	1.10	
ACTUAL TIME TO FAILURE, MIN		t_f	690	900	930	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

SHEAR STRENGTH PARAMETERS *

ϕ' = 24°

TAN ϕ' = 0.445

c' = 0.2 T/SQ FT

* for σ > 1 tsf

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TYPE OF SPECIMEN UNDISTURBED 3.01 IN. SQUARE 0.616 IN. THICK

CLASSIFICATION PLASTIC CLAY(CH),gray

LL PL PI G_s 2.70(est)

REMARKS

PROJECT COMPACTED SHALE EMBANKMENT STUDY

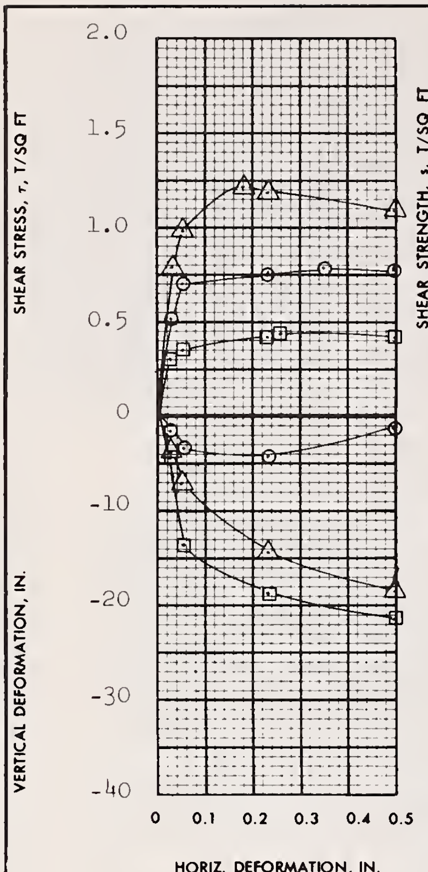
AREA INDIANA I-74, Sta 435+83

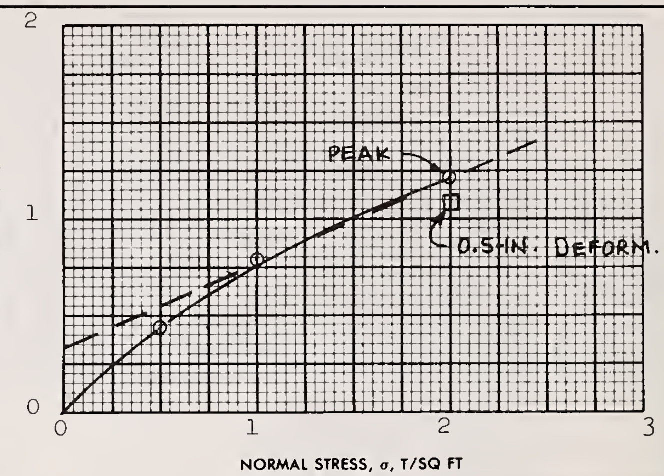
BORING NO. 1 SAMPLE NO. 6

DEPTH EL 21 ft DATE 22 Sept. 1976

BWG DIRECT SHEAR TEST REPORT

Figure 73. Indiana I-74, sample 6





SHEAR STRENGTH PARAMETERS *

ϕ' = 24°

TAN ϕ' = 0.445

c' = 0.34 T/SQ FT

* for σ > 1 tsf

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	7.7 %	12.3 %	10.9 %	%
	VOID RATIO	e_o	0.339	0.378	0.375	
	SATURATION	S_o	61.3 %	87.8 %	78.5 %	%
	DRY DENSITY, LB/CU FT	γ_d	125.9	122.3	122.6	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	4	2	1	
FINAL	WATER CONTENT	w_f	%	%	%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	0.5	1.0	2.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.43	0.78	1.22	
ACTUAL TIME TO FAILURE, MIN		t_f	1530	1860	960	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED		3.01 IN. SQUARE	0.612 IN. THICK
CLASSIFICATION PLASTIC CLAY(CH),gray ;with concretions to 1/3 in.			
LL	PL	PI	G_s 2.70(est)

REMARKS

PROJECT COMPACTED SHALE EMBANKMENT STUDY

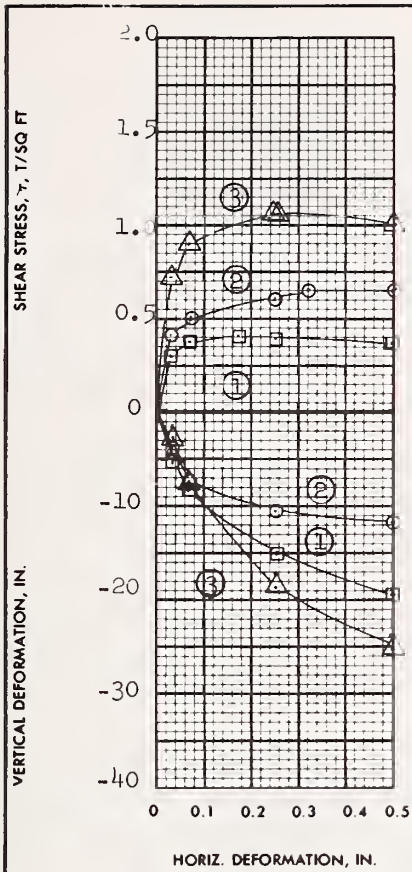
AREA INDIANA I-74, Sta 435+83

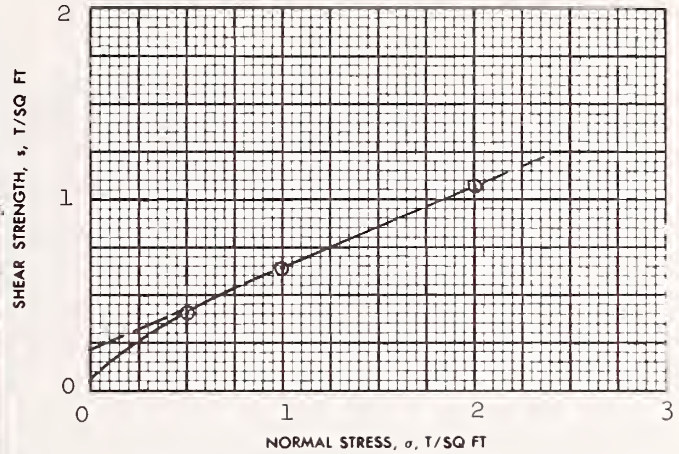
BORING NO. 1 SAMPLE NO. 16

DEPTH EL 45.5 ft DATE 12 Oct. 1976

BWG **DIRECT SHEAR TEST REPORT**

Figure 74. Indiana I-74, sample 16





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	12.0%	14.2%	12.5%	%
	VOID RATIO	e_o	0.390	0.421	0.379	
	SATURATION	S_o	83.1 %	91.1%	89.0%	%
	DRY DENSITY, LB/CU FT	γ_d	121.2	118.6	122.2	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	<1	<1	<1	
FINAL	WATER CONTENT	w_f	23.2%	17.7%	16.1%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	0.5	1.0	2.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.40	0.65	1.06	
ACTUAL TIME TO FAILURE, MIN		t_f	1050	1830	1380	
RATE OF STRAIN, IN./MIN			.00019	.00019	.00019	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

SHEAR STRENGTH PARAMETERS*

ϕ' = 23°

TAN ϕ' = 0.424

c' = 0.2 T/SQ FT

* for σ > 1 tsf

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TYPE OF SPECIMEN	UNDISTURBED	3.01	IN. SQUARE	0.616	IN. THICK
CLASSIFICATION	PLASTIC CLAY(CH), GRAY				
LL –	PL –	PI –		G_s	2.70 EST

REMARKS

PROJECT	COMPACTED SHALE EMBANKMENT STUDY
AREA	INDIANA I-74, Sta 435+83
BORING NO. 1	SAMPLE NO 20
DEPTH EL 55 ft	DATE 18 Oct. 1976
BWG	**DIRECT SHEAR TEST REPORT**

Figure 75. Indiana I-74, sample 20

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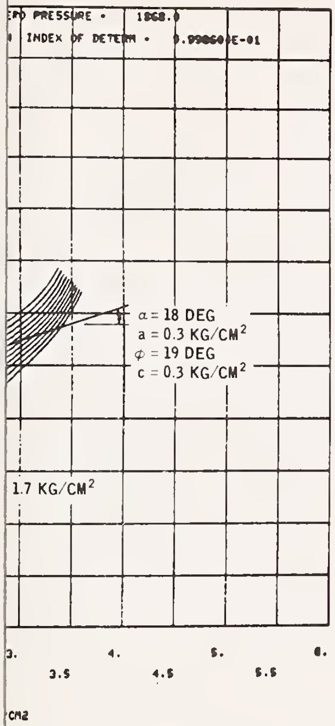
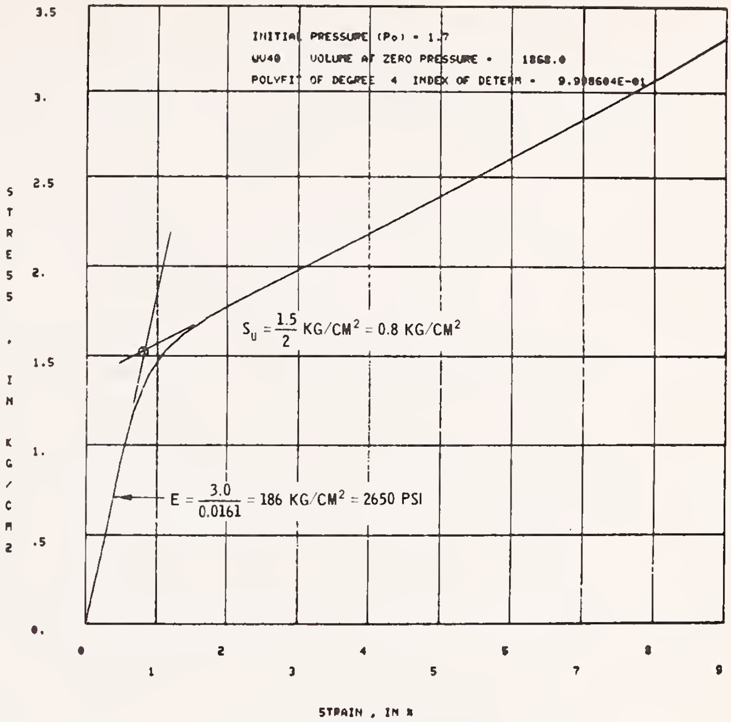
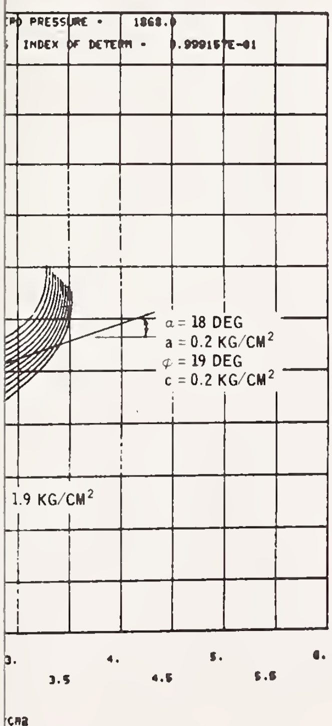


P–Q) PLOT

E POLYNOMIAL FIT

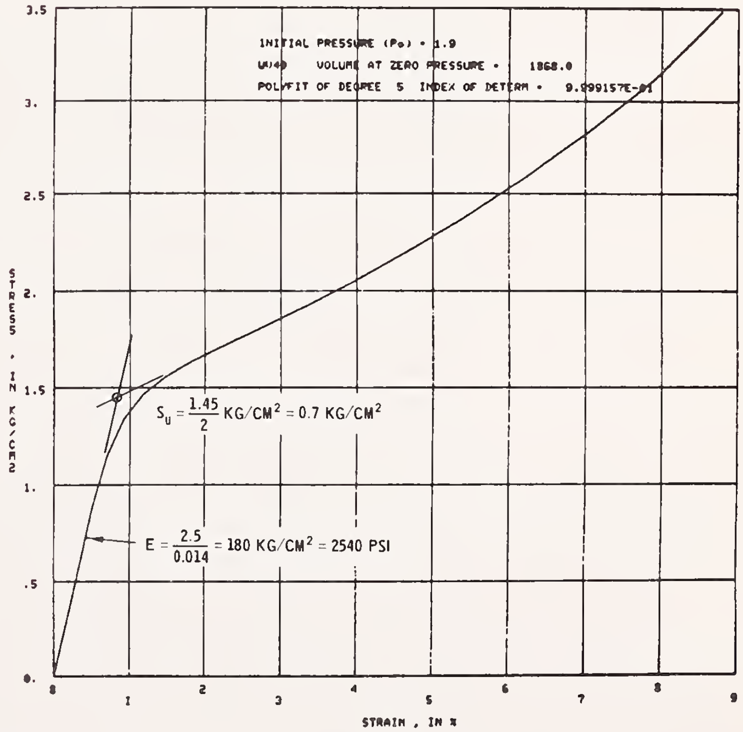


c. STRESS–STRAIN PLOT



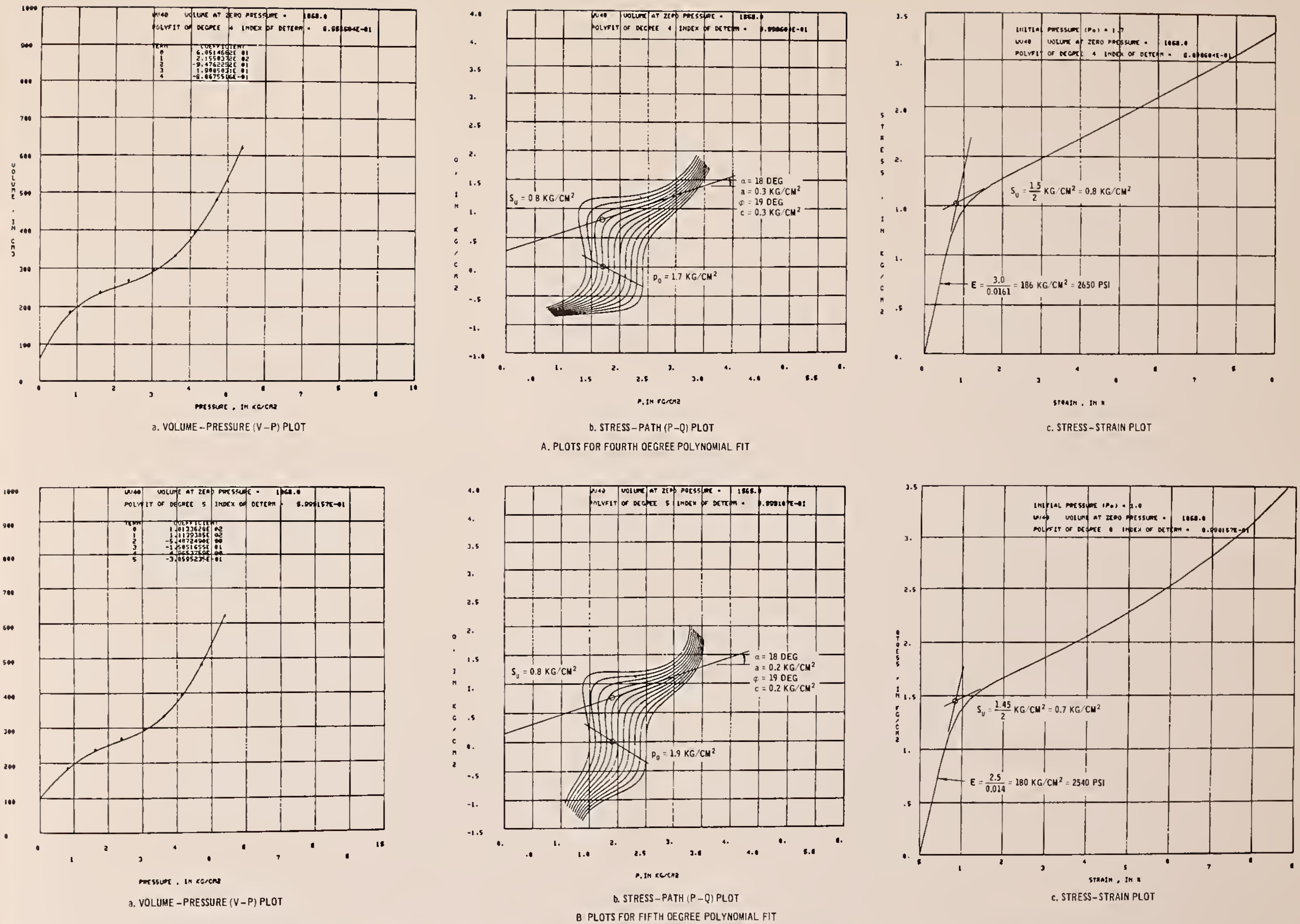
ATH (P–Q) PLOT

DEGREE POLYNOMIAL FIT



c. STRESS–STRAIN PLOT

depth (WV 40), West Virginia U. S. 460 embankment



a. VOLUME-PRESSURE (V-P) PLOT

b. STRESS-PATH (P-Q) PLOT

c. STRESS-STRAIN PLOT

A. PLOTS FOR FOURTH DEGREE POLYNOMIAL FIT

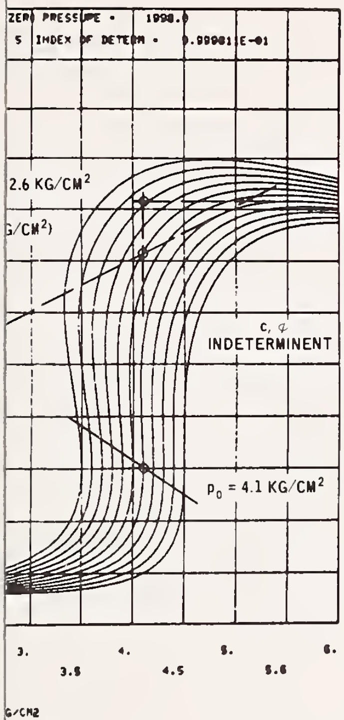
a. VOLUME-PRESSURE (V-P) PLOT

b. STRESS-PATH (P-Q) PLOT

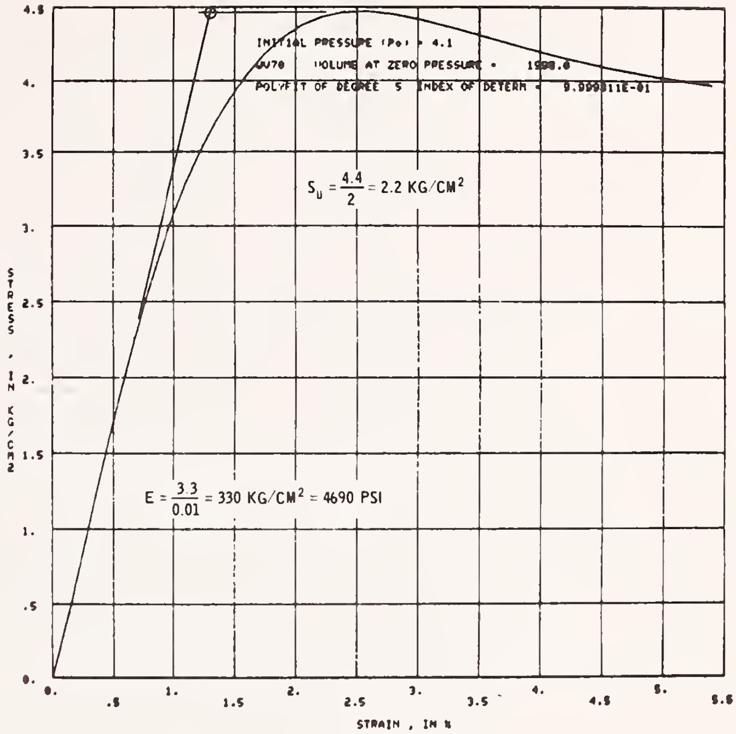
c. STRESS-STRAIN PLOT

B PLOTS FOR FIFTH DEGREE POLYNOMIAL FIT

Figure 46. Example of pressuremeter data plots, 40-ft depth (WV 40), West Virginia U. S. 460 embankment

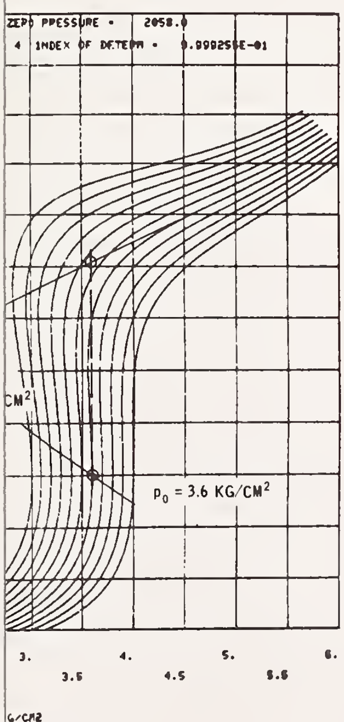


TH (P – Q) PLOT

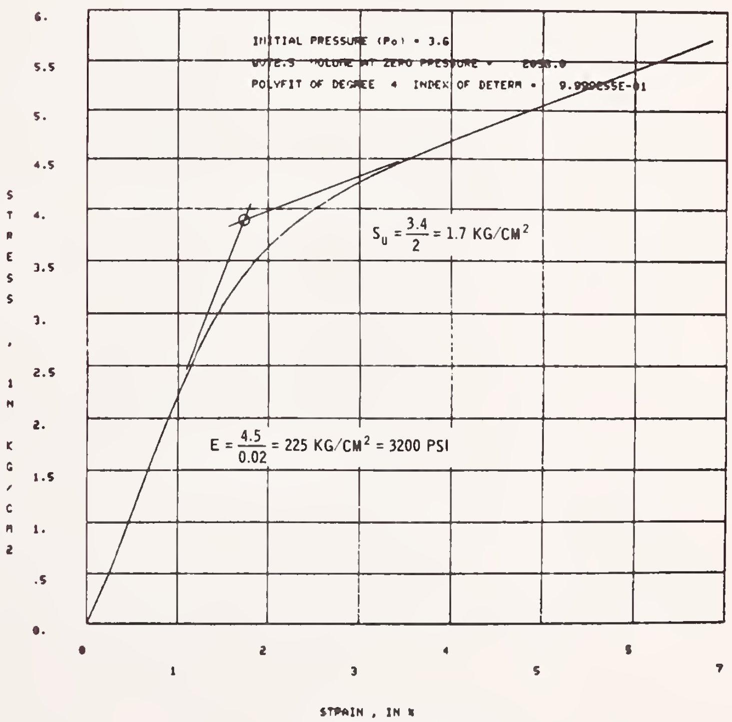


c. STRESS – STRAIN PLOT

R TEST AT 70 – FT DEPTH (WV 70)



H (P – Q) PLOT



c. STRESS – STRAIN PLOT

TEST AT 72.5 – FT DEPTH (WV 72.5)

ınd 72.5-ft depths, West Virginia U. S. 460 embankment

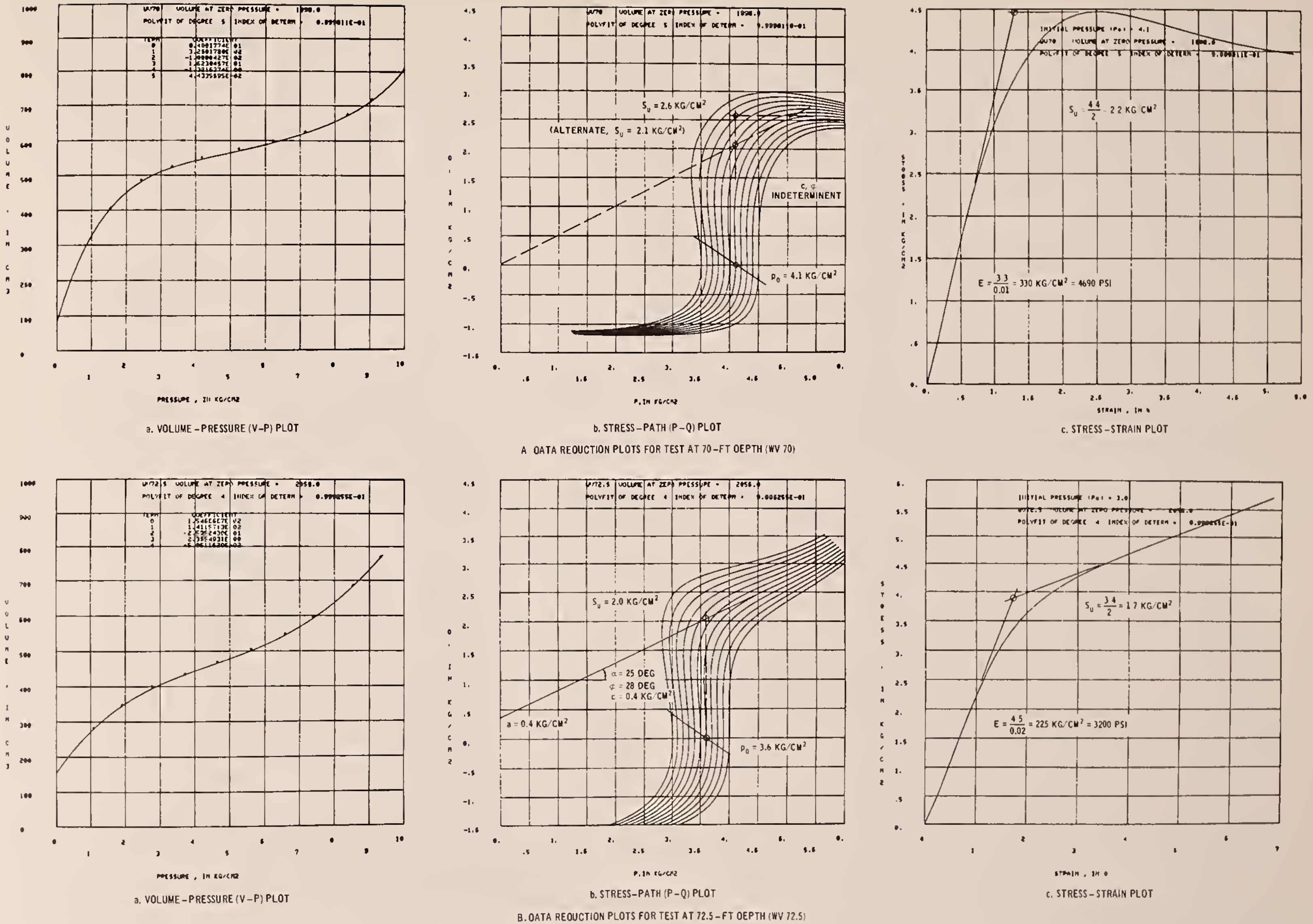


Figure 47. Example of pressuremeter data plots, 70- and 72.5-ft depths, West Virginia U. S. 460 embankment

FEDERALLY COORDINATED PROGRAM OF HIGHWAY RESEARCH AND DEVELOPMENT (FCP)

The Offices of Research and Development of the Federal Highway Administration are responsible for a broad program of research with resources including its own staff, contract programs, and a Federal-Aid program which is conducted by or through the State highway departments and which also finances the National Cooperative Highway Research Program managed by the Transportation Research Board. The Federally Coordinated Program of Highway Research and Development (FCP) is a carefully selected group of projects aimed at urgent, national problems, which concentrates these resources on these problems to obtain timely solutions. Virtually all of the available funds and staff resources are a part of the FCP, together with as much of the Federal-aid research funds of the States and the NCHRP resources as the States agree to devote to these projects.*

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems connected with the responsibilities of the Federal Highway Administration under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by keeping the demand-capacity relationship in better balance through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements which affect the quality of the human environment. The ultimate goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge of materials properties and technology to fully utilize available naturally occurring materials, to develop extender or substitute materials for materials in short supply, and to devise procedures for converting industrial and other wastes into useful highway products. These activities are all directed toward the common goals of lowering the cost of highway construction and extending the period of maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural designs, fabrication processes, and construction techniques, to provide safe, efficient highways at reasonable cost.

6. Prototype Development and Implementation of Research

This category is concerned with developing and transferring research and technology into practice, or, as it has been commonly identified, "technology transfer."

7. Improved Technology for Highway Maintenance

Maintenance R&D objectives include the development and application of new technology to improve management, to augment the utilization of resources, and to increase operational efficiency and safety in the maintenance of highway facilities.

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* The complete 7-volume official statement of the FCP is available from the National Technical Information Service (NTIS), Springfield, Virginia 22161 (Order No. PB 242057, price $45 postpaid). Single copies of the introductory volume are obtainable without charge from Program Analysis (HRD-2), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

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R&D