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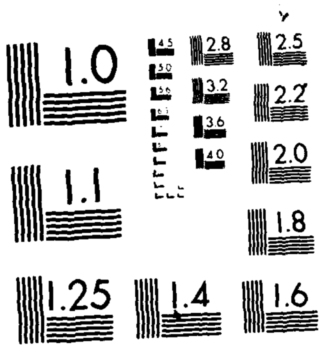

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MOHICANVILLE REINFORCED DIKE NO. 2 DESIGN MEMORANDUM

by

Jack Fowler, Roy E. Leach
John F. Peters, Raymond C. Horz

Geotechnical Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

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19 ABSTRACT *(Continue on reverse if necessary and identify by block number)*

A 24-ft high, 1,100-ft long reinforced embankment was successfully constructed on a very soft foundation that consisted of about 16 ft of peat and about 60 ft of soft clay. Prior to construction, a 1,200-ft long, 3-ft wide, 30-ft deep geomembrane and bentonite slurry trench was designed and constructed to control underseepage beneath the embankment. The embankment, which was a saddle dike for a flood control reservoir, was constructed with slopes of 1V to 3H with a clayey sand gravel fill material. Conventional limit equilibrium and finite element analyses were conducted prior to construction to determine necessary embankment tensile reinforcement to prevent a potential failure. Several analyses were conducted where woven polyester and kevlar geotextiles were favorably considered, but in the final analysis, a steel wire mesh was selected because of the very high modulus of the steel and the very low embankment movements allowed in the design. It was determined that varying the reinforcement modulus from low to high values significantly reduced the horizontal and

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19. ABSTRACT (Continued).

vertical displacements of the embankment. More importantly, the high modulus was needed to ensure that the full working load would be developed in the reinforcement before mobilization of the foundation shear resistance. Loads measured in the steel wire mesh, pore pressure, and settlement measurements in the embankment and foundations were within the values predicted during the design. Successful completion of the embankment to design height would not have been possible without the use of reinforcement.

PREFACE

This report describes the design and construction techniques for Mohicanville Dike No. 2, which is a flood control dike built as part of the Mohicanville Dam and Reservoir project and is located near Mohicanville, Ohio.

This project was conducted by the US Army Engineer District, Huntington (ORH), Huntington, West Virginia, and the US Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, during the period Jan 82 to Sep 83.

Concept formulation and general supervision of the research and design were conducted by Dr. J. Fowler, Geotechnical Laboratory (GL), WES, with the assistance of Dr. J. M. Duncan, Virginia Polytechnic Institute, Blacksburg, Virginia; Mr. S. A. Collins, Law Engineering Testing Company, Atlanta, Georgia; and Mr. L. W. Franks, ORH.

Onsite research and technical guidance were conducted by Mr. J. A. Coffman, Jr., Chief, Geotechnical Branch, ORH, Mr. D. P. Hammer, Chief, Geotechnical Branch, Ohio River Division (ORD), and Mr. C. R. Fondelier, ORD. District Engineer for ORH during this period was COL J. W. Devens.

This report was written by Dr. J. Fowler, Mr. R. E. Leach, Dr. J. A. Peters, and Mr. R. C. Horz. General supervision was provided by Mr. G. B. Mitchell, Chief, Engineering Group, Soil Mechanics Division (SMD), Mr. C. L. McAnear, Chief, SMD, and Dr. W. F. Marcuson III, Chief, GL.

COL Allen F. Grum, USA, was the previous Director of WES. COL Dwayne G. Lee, CE, is the present Commander and Director. Dr. Robert W. Whalin is Technical Director.

CONTENTS

* Construction drawings 1-16 are on microfiche and are enclosed in an envelope in the back cover of this report.

MOHICANVILLE REINFORCED DIKE NO. 2

DESIGN MEMORANDUM

PART I: INTRODUCTION

Background

1. Mohicanville Dike No. 2 is a flood control dike built as part of the Mohicanville Dam and Reservoir project located in the Muskingum Watershed of Ohio, construction drawings 1 and 2. Constructed in 1937, the dike was originally designed to be constructed to the same elevation as the Mohicanville Dam, el 978.0, but foundation failures during construction in the peat and soft clay foundation finally led to the decision to stop construction. The pool of record in 1969 alerted the Huntington District (HD) to the need to upgrade the flood control system. A study was begun to design a dike that would raise the present crest elevation from approximately 965.0 to the elevation of the Mohicanville Dam, el 978, (after settlement of the dike has occurred). After initial subsurface investigations by the HD in 1970, Law Engineering Testing Company (LETCO) was retained in 1980 to complete a detailed analysis of slope stability and seepage. This included the field and laboratory testing necessary to determine material design strength and insitu design conditions. The LETCO report "Mohicanville Dikes, Embankment Reanalysis Report" was completed in January 1982. A major conclusion of the report was that no acceptable factor of safety could be obtained for raising the dike without some type of reinforcement. The Waterways Experiment Station (WES) was contacted in the latter part of 1982 and asked to determine by finite element and conventional analyses the type of reinforcement needed and the placement requirements. The dike is scheduled to be constructed in 1984.

Purpose

2. This report is the second of two design reports, the first one by LETCO, which discusses design parameters, stability analyses, construction techniques, and expected behavior of the dike during and after construction. The purposes of these studies are to (a) design a 23-ft-high flood control

reinforced earth dike on a soft foundation; (b) to describe construction techniques for raising this dike on a soft foundation; and (c) to design an instrumentation monitoring system which will provide the information necessary to monitor the safety of the dike both during and after construction. This experience may be of some value in design and construction of future projects on soft foundations.

Scope

3. This report contains a brief review of the soils and foundation studies made in connection with the LETCO embankment reanalysis study, a description of pertinent features of design and construction, an evaluation of embankment behavior and measurement techniques, and construction plans and specifications. Those elements of embankment design which are fully described in the LETCO report are briefly summarized in this report.

PART II: STATEMENT OF PROBLEM

4. The problem simply stated is that the foundation material is too "soft" to support the embankment, i.e. the strength of the underlying peat and soft clay will not support the full height of the embankment without failing unless some type of reinforcement is added to the embankment. The reinforcement is designed to force the embankment to behave as a semirigid body and settle vertically with a minimum of horizontal displacement. During initial construction (1936) the embankment had been raised approximately 6 ft when cracks appeared and a major shear failure occurred. After a brief halt in construction, fill placement resumed and 8 days later the embankment began cracking again and construction was stopped. An attempt was made to dewater the foundation before the next construction season (1937), but when construction resumed in May, cracks began to form and in August another major failure occurred in the same area and construction was terminated at approximately 15 ft above original grade. Through settlement, the crest height at the present time is approximately 7 ft above the original ground surface.

5. Displacement section construction and/or stage construction are alternative construction methods that have been addressed, but were eliminated in favor of a reinforced embankment. The embankment is to be built to an elevation that allows for anticipated consolidation. Construction borrow measurements showed 142,000 cu yds were placed while current cross sections exhibit only 91,000 cu yds above original ground. In the failed area near the center of the dike (sta 9+00) for one volume of material above grade there was a corresponding 4 to 6 volumes below grade due to rapid consolidation, shear failures, and flow in the foundation materials. This type of construction is unacceptable for the new dike. Stage construction without reinforcement would be a lengthy construction process because only 2 to 3 ft of embankment could be constructed before construction would have to be halted to allow for consolidation and a corresponding gain in foundation strength.

6. The problem of designing a reinforced embankment on a soft foundation is relatively new. Uncertainties existed about relying solely on conventional analysis for a relatively large embankment whose failure could produce a loss of the reservoir. To supplement the conventional analysis, a

finite element analysis was conducted to model the expected behavior of the soil and reinforcement. No factor of safety is actually calculated with a finite element analysis, but the behavior of the embankment and foundation can be modeled. A conventional limit equilibrium method and a finite element analysis were used as complimentary design tools.

PART III: SOIL INVESTIGATION

Subsurface Investigation

7. As stated in the LETCO report the subsurface borings were drilled in time segments: (a) prior to 1935 for the original design; (b) in 1970 by the Huntington District, Corps of Engineers, for embankment reanalysis; and (c) in 1980 by LETCO for embankment reanalysis. The boring logs appear in the LETCO report along with their interpretation of data. The location of the 26 borings drilled for the reanalysis report are shown in construction drawing 9. During the boring program 66 undisturbed samples were recovered, 6 vertical inclinometer casings and 2 piezometers (1-inch open tube Casagrande) were set, and vane shear tests were performed between undisturbed samples. In 1982-1983 the Huntington district collected additional bag samples for laboratory tests at WES. Altogether 20 test pits were excavated for the subsurface investigation to depths of up to 8 ft to obtain jar and bag samples from potential borrow areas. A detailed discussion of the sampling program up through the LETCO report can be found in their report. No further detailed discussion is needed for the 1982-1983 field sampling program, but the soil properties obtained will be discussed in Appendix A.

Laboratory and Field Tests

8. Samples obtained during the subsurface investigation were tested in the laboratory to determine visual classification, water content, grain size distribution, Atterberg limits, loss on ignition, triaxial shear strengths, permeability, consolidation characteristics, and water content-density relationships. All the tests were conducted in accordance with the Laboratory Soils Testing Manual, EM 1110-2-1906, except for the loss on ignition test which followed ASTM D2974-71. Vane shear strength tests and falling head permeability tests were performed at the site. The results of the above mentioned LETCO tests are summarized in Tables 1 through 4. Results of the soil strength tests are shown in Appendix A.

9. Falling head permeability tests were conducted in the field in the piezometers installed for the LETCO study. Also field vane shear tests were conducted in undisturbed sample holes between sample points. Results of the permeability tests are summarized in Table 1 and results of the vane shear tests are listed on the borings (LETCO report) and plotted in Appendix A.

10. Further tests were conducted at the WES on bag samples collected from test pits in the dike and potential borrow areas with the results are summarized in Appendix A. These tests were performed in accordance with EM 1110-2-1906.

Subsurface Conditions

Geologic Properties

11. The subsurface conditions are discussed extensively in the LETCO report, but only a brief summary will be contained in this report. The area of the shear failures that occurred during construction is still evident between stas 7+00 and 11+50, as shown by the toe bulge that extends upstream approximately 360 ft upstream in construction drawing 4. The existence and extent of the failure zone is evidenced in the sections and profiles shown in construction drawing 9. These sections were used to determine design sections used for analyses.

Engineering Properties

12. Embankment. The embankment is constructed from material derived from glacial till composed of gravelly sandy clay (CL) with zones of gravelly clayey sands (SC) and minor amounts of sand (SP), silt (ML), silty sand (SM), and clayey gravel (GC). Results of the standard penetration tests show that the embankment material is softer and/or less compacted near the peat layer compared to the upper material. As reported by LETCO a minimum number of strength tests were conducted because the embankment is to be excavated down to el 960 before new construction is started. Also large deformations of the embankment and discontinuities during the original construction would cause the soil mass strengths to be lower than the values obtained from small triaxial specimens. The results of the triaxial tests

run by LETCO are listed in Tables 1 through 4 and shown in Appendix A. The value obtained from the total stress envelope (R Tests) for cohesion (0.25 tsf) was reduced by LETCO to 0.09 tsf for design to account for the unquantifiable strength reduction of the soil mass strengths. The fraction angle ϕ was 30 deg.

13. To supplement these design values, the WES conducted additional triaxial tests on soil obtained by mixing bag samples obtained from test pits in the embankment and proposed borrow areas. Laboratory tests performed on the samples included visual identification, grain size analysis, water content, Atterberg limits, compaction, and triaxial shear. The triaxial specimens were compacted at 95 percent of standard maximum laboratory density and 3 percent wet of optimum. All the laboratory test results are shown in Appendix A.

14. Peat. As described in the LETCO report, the peat was generally varied from the upper portion being fibrous peat to the lower portion being amorphous peat. Engineering properties determined for the peat are listed in Tables 1 through 4.

15. To obtain a larger statistical base for the strength tests, a number of 1-point strength tests were conducted and are summarized as total stress strengths in Figure A8. The basis for obtaining design strengths in this manner is described in the LETCO report:

> "On this figure, the shear strengths from Q tests have been plotted at their average insitu effective stress, using a K_o' of 0.5. The data was plotted in this manner since the existing insitu confining stress governs the available undrained strength of these saturated samples. Plotting the Q tests in this manner also allowed combining both unconsolidated Q and consolidated R or $\bar{R}$ peat data on one graph. The R and $\bar{R}$ test data have been plotted in the conventional manner, with p equal to the effective confining stress plus one half the total deviator stress."

16. Effective stress strength parameters for the peat were obtained from $\bar{R}$ and S tests and are shown in Figure A9. Also shown in this plot are the in situ vane shear results plotted versus the insitu average effective stress as described above. LETCO warns that the high effective stress strength friction angle of 32 deg is supported in the literature, but cautions against its use at higher confining pressures.

17. Consolidation test results are shown in Figure 1 for samples from beneath the crest, at the toe, and beyond the toe. The data obtained for the peat were difficult to analyze for secondary consolidation; therefore, the consolidation plots are for total consolidation occurring at the end of each load increment.

18. LETCO's permeability values determined from both field and laboratory data are shown for individual borings in Table 1. Plots of the permeability data versus height of fill are shown in Figure 2. The permeabilities associated with the different layers as determined for use in this report are discussed in Appendix C.

19. Foundation Clay. The foundation clay is a medium plasticity silty clay (CL) with thin zones of high plasticity clay (CH) and organic clay (OL and OH). The engineering properties determined from laboratory tests are listed in Tables 1 through 4 and summary plots of the soil strengths are shown in Appendix A.

20. Total stress strength parameters from the Q and vane shear tests shown in Appendix A are plotted by the procedure discussed for the peat above with the exception that K_o' is set equal to 0.6. The design envelope that was chosen for undrained strength is shown on a composite plot in Figure A10. The design envelope that was chosen for the effective strength tests is shown on a composite plot also in Figure A10.

21. Consolidation tests for the clay are shown in Figure 3 for samples in the areas beneath the crest, at the toe, and beyond the toe. Average curves were selected for design. The design permeability for the clay was determined from laboratory tests and is listed in Table 1.

PART IV: FABRIC-REINFORCED EMBANKMENT DESIGN

General

22. A reinforced earth embankment is subject to the same failure mechanisms as a normal embankment on soft soils: (a) rotational slope/ foundation failure; (b) foundation displacement; and (c) horizontal splitting and spreading. Design concepts to prevent failure require large reinforcement tensile forces and small strains. Forces calculated at failure are resisted by reinforcement tensile forces while spreading displacements are controlled by the reinforcement tensile modulus. Friction between the soil and reinforcement is large enough to prevent sliding of the embankment on the reinforcement or of the reinforcement on the foundation. Construction techniques can be used to develop reinforcement tensile forces at small strains. Design criteria are still being refined, but at present the work by the late Dr. Haliburton is compiled as the state-of-the-art in a manual for the FHWA entitled "Use of Engineering Fabrics in Transportation-Related Applications" December, 1981.

23. The reinforced embankment design is based primarily on the criteria that the reinforcement should supply sufficient resistance to provide a factor of safety of 1.3 against mobilizing the full resistance of the foundation soil. The reinforcement force, needed to provide the required factor of safety, is determined from conventional limit equilibrium analyses, Appendix D. To achieve the desired factor of safety, however, the reinforcement must be sufficiently stiff to take up the additional force before excessive deformations occur. The required reinforcement stiffness is determined from finite element analyses described in Appendix C. In addition, the finite element analyses provide information on embankment deformation, construction-induced pore pressure and rates of consolidation.

Slope Stability Analysis and Determination of Working Force

24. To analyze the reinforced embankment, a conventional circular arc limit equilibrium slope stability analysis was conducted to determine the unbalanced moment without reinforcement. For the embankment to fail in the

rotational mode the reinforcement must tear or separate; therefore, the reinforcement strength can be added to the resisting forces (Haliburton). This assumes the following:

a. Reinforcement tensile strength and soil shear strength are mobilized simultaneously, and

b. The critical failure location will be the same for the non-reinforced and the reinforced embankments

End of Construction Case

25. The Fellenius circular arc method of slope stability analysis contained in the WES program SA10478 was used to determine the critical failure surface and minimum factor of safety with no reinforcement as shown in Figure 4. Using soil properties adopted from Law's report a minimum FS of 0.89 was calculated which is well below a Corps recommended value of 1.3. The resisting moment needed for a FS of 1.3 can be calculated and the required reinforcement tensile strength determined using the equation recommended by LETCO and shown in Appendix D. The calculated value of required tensile strength is 32.4 kips/ lin ft. This value was the maximum required for all the cases checked by conventional analysis.

Required Reinforcement Stiffness

26. Finite element analyses were performed for one and two layer reinforcement systems for reinforcement stiffness corresponding to polyester, Kevlar, and steel. It was found that a stiffness of 12,000 ton/ft would be needed to mobilize the required reinforcement force. Also, it was determined that the two layer reinforcement system was not efficient as it did not significantly increase the total reinforcement mobilized. Therefore, the only reinforcement material that can meet both strength and stiffness requirements is steel mesh.

Long Term Stability

27. The two long term stability cases analyzed by LETCO were rechecked using the 1 ft higher embankment being considered for design. The two cases were: Case II - upstream stability for rapid drawdown from maximum pool and

Case VI - downstream stability for steady seepage at maximum pool. Assuming the soil strengths and phreatic surface adopted by LETCO and shown in Figure 5, a FS of 1.07 was calculated for Case II, sudden drawdown. This FS , 1.07, is for a 1 ft higher embankment than the LETCO analysis. Assuming the conditions shown in Figure 6, a FS of 1.45 was calculated for Case VI, steady seepage. In both cases the LETCO and the WES analyses give similar results. In both cases an adequate FS was computed without the use of reinforcement.

PART V: EMBANKMENT INSTRUMENTATION

Evaluation of Embankment Behavior

28. Predictions of embankment performance based on detailed finite element analyses are presented in Appendix C. The analyses were based on idealized subsurface conditions and were primarily intended to supplement the limited equilibrium design computations; however, results of the finite element analysis can also be used for comparison with field measurements of the instrumentation.

29. Predictions of performance are shown on Figures 7 through 9. For purpose of prediction, it is assumed that the dike will be constructed in one season without the eight month halt in construction that was assumed for the design analyses. Separate predictions were made for instrumentation locations near sta 9+00, where the soft clay extends to el 880 ft and other locations where the soft clay only extends to el 930 ft.

a. Reinforcement Force: The maximum reinforcement force occurs at the point of maximum settlement which is at or near the embankment centerline. The maximum predicted force of 18 tons/ft should be observed at the completion of construction. A reduction in force should be observed after significant consolidation has occurred. The reinforcement force (after 11 yrs) was computed to be 10 tons/ft. The reduction in reinforcement force with distance from the centerline was computed to have a maximum value of 0.3 ton/ft^2 at about 50 feet from the centerline (see Figure C12). This value roughly corresponds to the resistance at that location of the clay fill to initial pullout of the steel reinforcement; therefore, slipping between the reinforcement and fill may occur if the reinforcement force gradient exceeds 0.3 ton/ft^2 or if the maximum gradient occurs further from the centerline. The ultimate slip resistance should be sufficient to resist complete pullout.

b. Vertical Displacement: Vertical displacement will occur during and after embankment construction. The maximum settlement that will occur by the end of construction is predicted to be approximately 1.0 ft at the embankment centerline. Subsequent consolidation should cause an additional two feet of settlement, of which about 20 percent (0.4 ft) should occur within one year of completion of construction. Beyond a distance of approximately 60 ft from centerline, upward movement may be observed. The heave is greatest at the dike toe and should be on the order of 0.2 ft.

c. Lateral Displacement: Significant lateral displacement should occur only during embankment construction. Little or no lateral displacement is predicted during consolidation. Near the embankment toe at the reinforcement level, the maximum spreading should be less than one inch, which should occur during the first construction season if the dike is constructed to only el 978 ft. Within the foundation, lateral spreading of up to 0.4 ft at end of construction is predicted. The maximum horizontal movement is predicted to occur about 60 ft from centerline, just above the top of the peat layer.

d. Pore Pressure: Pore pressures induced in the peat and the clay by initial embankment loading (to el 978) are anticipated to increase at a maximum rate of 1.7 ft of piezometric head for each foot of fill. The final five feet of embankment construction (to el 983) should induce a maximum of 3 ft additional head at embankment centerline. The highest rate of consolidation should occur at the embankment centerline. Consolidation at end of construction should continue at a two percent reduction of maximum induced pressure head per month. A reduction of 20 percent of the maximum induced head is predicted for the year following the end of construction. The head reduction at the embankment centerline after construction will be accompanied by a small rise in head near the embankment toe to create an overall reduction of the piezometric gradient. Excess pore pressures are not expected to dissipate beneath the outer portions of the embankment until a significant reduction in pore pressure is achieved at the centerline.

Instrumentation Program

30. Several types of instrumentation will be used to monitor embankment performance during and after construction and to verify design procedures. Settlement plates, inclinometers, and surface monuments will be used to measure horizontal and vertical movements of the embankment surface, the foundation, and along the reinforcement. The displacements measured by the strain gages placed on the steel wire will determine the forces existing in the reinforcement. Piezometers in the foundation will measure the pore water pressure during and after construction, the effectiveness of the slurry trench, and the effectiveness of the downstream sand blanket drain.

All the instrumentation to be used are listed in Table 5 and construction drawing 14 with the locations shown in construction drawings 10 through 13. Details of the instrumentation are shown in construction drawings 10, 15, and 16. All instrumentation will be installed by instrumentation and soil technicians experienced with the units being used.

Settlement

Inclinometers

31. Inclinometers are as follows:

   a. Vertical movement (settlement) of the horizontal inclinometer casing will be monitored by use of a Digitilt - type inclinometer with a sensitivity of 1 part in 10,000 similar to Model 50329. The inclinometer is an instrument which determines the relative movement of a plastic casing with respect to a fixed point in the casing (the exposed end for a horizontal installation).

   b. Vertical movement (settlement) of the vertical inclinometer casing joints will be monitored by use of a settlement probe similar to Sinco Model 50801. The probe is lowered with a surveyor's chain into the casing until it hooks the bottom of the casing providing a direct measure of settlement at that point. This device will only be used to obtain an initial reading. Metal rings attached to the outside walls of the inclinometer pipe will be monitored with a Sinco Model 50819 Sondex settlement probe. Details are shown in construction drawing 10.

   c. Vertical inclinometer casing will be installed at 4 stations along the dam (construction drawing 13) to determine settlement and horizontal movement of the section during and after construction. The bottom of the casing will be founded at el 880 (which should experience no movement). Each coupling will be installed with a 6 inch gap between the casings to monitor settlement as shown in detail on construction drawing 10. Where the casing goes through the welded wire, 2 ft of steel casing will be placed around the PVC casing to prevent damage to the casing.

   d. Horizontal inclinometer casing will be installed at 4 stations along the dam (construction drawings 10 through 13) to measure settlement at the interface of the reinforcement and the foundation. The measurements will be used to determine the effectiveness of the steel wire installed at el 960. Each installation will be continuous from the slurry trench to the downstream toe, wrapped with filter fabric, and extend through the sand blanket. Details are shown on construction drawings 15 and 16.

Settlement Plates

32. Settlement plates will be installed at 4 stations (construction drawings 10 through 13) to determine the settlement occurring at the elevation of the reinforcement. The settlement plates will be founded on the steel wire and details are shown on construction drawing 10.

Bench Marks

33. To assure a system of precise monitoring of structural movements, four control points or bench marks will be established. Second order surveys will be performed. The specific locations of the bench marks, off project control network, will be determined prior to construction to select the most efficient geometry and assure an unobstructed view of all project features.

Horizontal Deflections

Inclinometers

34. Horizontal movement of the vertical inclinometer casing will be monitored by use of a Digitilt - type inclinometer similar to Model 50325 with a sensitivity of 1 part in 10,000. All movement is referenced to the bottom of the casing which is not expected to move. Details are shown on construction drawing 10. Surveys should be conducted on the top of the casing. The vertical casing as discussed above are located as shown on construction drawings 10 through 13.

Surface Monuments

35. A grid of surface monuments will be established as the dike is constructed to monitor settlement and spreading of the dike at approximately every 100 ft station as shown on construction drawings 10 through 13. These monuments will define significant movement that occur at locations other than the 4 more heavily instrumented sections. A row of surface monuments will be installed outside each dike toe along with 4 permanent monuments for control prior to construction. Details are shown on construction drawing 10. Levels will be taken at instrumented sections when readings are made.

Pore Water Pressure

36. Piezometers will be used to measure the pore pressure buildup in the foundation due to the dike fill placement. These measurements will be used to determine the safety of the dike during construction and to check design assumptions. Piezometer locations are shown in construction drawings 10 through 13. To avoid a time lag (response time before a piezometer indicates a change in hydrostatic head) in the data from the piezometers installed in the soft clay foundation, a pneumatic piezometer similar to the Sinco Model 514177 and an electric piezometer similar to a Geonor vibrating wire piezometer will be installed. Both units feature high sensitivity and low diaphragm displacement. This system allows flexibility in locating terminal stations, i.e., beyond the downstream toe. Details of the pneumatic and electric piezometers and terminal boxes are shown on construction drawing 15. To provide a check of the pore water pressures measured by the pneumatic and electric piezometers, Casagrande open tube piezometers will be installed in the peat layers near the pneumatics. Each piezometer will be installed in separate holes spaced a minimum distance of 4 ft apart. All cable ends and tubes will be terminated in a centrally located metal building downstream of the construction activity. Details of the Casagrande piezometers are shown on construction drawing 15.

Strain in the Reinforcing Wire

37. Strain gages will be attached to the reinforcing wire at 6 stations, as shown on construction drawings 10 through 13, to determine the actual stresses occurring in the wire. These measurements will be used to verify design assumptions. The strain gages at stas 6+55 and 9+00 will be placed at 5 ft increments out to 70 ft from the dike centerline to determine the maximum strain. Details of the gages are shown on construction drawings 15 and 16. Two strain gages each will be located on the centerline at stas 4+75, 8+00, 11+00, and 12+20 to determine maximum load in the reinforcement at the center of dike.

38. The cable from the strain gages to the terminal boxes located downstream (construction drawing 15) will be placed side by side and the fill will be hand tamped for protection. All cable ends will be terminated in a centrally located metal building.

PART VI: INSTRUMENTATION INSTALLATION

General

39. This section contains guidelines for the installation of the instruments in Mohicanville Dike No. 2. These instruments should be installed by experienced personnel using approved techniques. Engineer Manuals 1110-2-1908 and 1110-2-4300 were used as references for the installation of these instruments.

Bench Marks

40. All of the bench marks will be installed in a 10-inch-diameter hole to a depth that will be determined at the site. A 1-1/2-inch-diameter pipe will be grouted within the hole, with the monument attached at the top of this pipe. A 10-inch x 10-inch encasement will surround the inner pipe above grade.

Inclinometers

41. The specially grooved plastic casing is installed vertically in a 4-1/2-inch or larger borehole. The casing will be installed to el 880 and extended 5 ft above the ground surface after the dike has been degraded and reconstructed to approximately el 962. A hole must be dug down to the steel wire and a hole cut in the wire only large enough to allow the drill rig to penetrate the reinforcement layer. Thereafter the casing will be installed in 5 ft increments in advance of the dike fill. Hand tamping and mounding of the soil around the casing will be required. Construction equipment will not be allowed closer than 3 ft to the existing casing. Hand digging will be required to clean the soil away. Because of the expected movement of the casing, frequent observations of the vertical alignment will be made. Installation details are shown on construction drawing 10 and locations are listed in Table 5 and construction drawing 14 and are shown on construction drawings 10 through 13.

42. Installation of the horizontal inclinometer casing at stas 4+55, 6+55, 9+00, and 12+20 should begin immediately after the dike has been degraded to el 960 and before the steel wire reinforcement is placed. The ends of the casing should be surveyed in the trench and an initial reading should be made after the trench is filled. Thereafter observations will be taken according to the schedule shown in Table 6 or more frequently as needed. Installation details are shown on construction drawings 15 and 16. Locations are listed in Table 5 and construction drawing 14 and are shown on construction drawings 10 through 13.

Piezometers

43. The closed system piezometers that will be installed will be the pneumatic type with a 3 tube reading arrangement connected to a console downstream of the dike. A total of 24 pneumatic piezometers and 8 electrical piezometers will be installed, according to the instructions in EM 1110-2-1908, Part I, to measure pore pressure buildup in the foundation. The open tube piezometers will be the Casagrande type with a porous high density polyethylene tube set in the peat and clay and attached to small half-inch riser pipes that will exit through the fill to the surface. The riser pipes must advance with the construction and hand tamping and mounding around the pipes will be required. Specific locations of each piezometer are listed in Table 5 and construction drawing 14 and are shown on construction drawings 10 through 13. Details are shown on construction drawings 15.

Settlement Plates

44. A total of 12 settlement plates will be installed on the steel wire reinforcement at the locations shown on construction drawings 10 through 13 and listed in Table 5 and construction drawing 14. Installation details are shown on construction drawing 15. The riser pipes will be extended up with the fill to insure that movement (settlement) of the reinforcement can be monitored.

Surface Monuments

45. A total of 52 surface monuments will be installed at locations shown on construction drawings 10 through 13. The monuments located outside the construction area should be installed before construction begins. The other monuments should be installed as the construction on the dike progresses. These monuments allow continuous monitoring of horizontal or vertical movement of the dike slope and the original ground surface near the toe. The monuments should be replaced immediately if they are damaged. Installation details are shown on construction drawing 10.

Strain Gages

46. Sixty-six strain gages will be installed on steel rods, the same diameter as the reinforcing wire, at WES and then will be transported to the site. After the reinforcing is in place, a rod the same length as the gage assembly will be removed and replaced by the instrumented rod. The strain gages will be installed in a Wheatstone bridge to compensate for temperature and bending. After the gages are installed hand tamping will be required to bring the fill level up 1 ft above the gages and associated cables. Gage locations are shown on construction drawings 10 through 13 and installation details are shown on construction drawings 15 and 16.

Observation Schedule

47. Data collection and reduction will be done in a manner compatible with short and long term data requirements. An observation schedule, shown in Table 6, should be followed to monitor construction, but modifications should be made if problems arise. Short term requirements necessitate immediate reduction of instrumentation data to compare with design assumptions and evaluate the safety of the embankment as construction is proceeding.

PART VII: MOHICANVILLE DIKE CONSTRUCTION DETAILS

Description of Dike Fill Material Borrow Location and Construction Details

48. The location of the borrow sources to be used in dike construction is shown on construction drawings 3 and 16. Borrow from this area will also be used to construct turnouts and haul roads leading from the source to the dike. Materials in the borrow area consist of a gravelly sandy clay that has been tested by the Corps and found to be suitable. These materials were found to be very similar to the existing dike materials and should require only a minimum amount of conditioning for compaction. Compaction in 8 inch lifts to 95 percent of standard maximum density with a self-propelled sheepsfoot roller (Caterpillar 225 or equivalent) will be required. Number of passes will be determined in the field after HD personnel determine the compaction effort required. Additional water may be required during the compaction effort. A water truck must be available not only to add water during compaction, but to control dust on all construction haul roads. In the event the fill material becomes too wet the material must be scarified and disced to the proper moisture content before compaction.

49. Prior to dike construction approximately 30,000 cu yds of existing dike fill will be excavated to el 960 ft. These materials from required excavation will be placed in 8 to 12 inch lifts and compacted with the sheepsfoot roller. Each lift will be scarified, prior to placing the next lift, with a 36-inch-diameter tooth disc. All trafficked areas will be disced before compaction with the sheepsfoot roller. All areas where the embankment is to be constructed shall be stripped of topsoil. These surfaces will be compacted and scarified before new fill material is placed and compacted. In the event wet and soft areas are found above el 960 during excavation of the existing fill material hydrated lime (CAOH) shall be mixed into the soil to improve equipment mobility. Approximately 5 percent by dry weight will be required to stabilize the wet clayey soils. Hand held wacker compactors will be used to compact fill material around the settlement plates and their riser pipes, inclinometer casings, and piezometers riser tubes. A vibratory steel wheeled roller will be used to compact the first 12 inch lift of fill before the sheepsfoot roller may be used. All fill

material will be required to be excavated to el 960. The dike will be treated to el 959 over the entire base area with 5.0 percent by dry weight of hydrated lime mixed into the fill material to protect the reinforcement against corrosion. After the reinforcement is placed an additional foot of fill material treated with hydrated lime will be placed and compacted over the wire.

50. The membrane above el 960 will be folded downstream and the edge covered with 3 to 4 inches of fill material to weight it down to prevent winds from blowing around prior to placement of embankment reinforcement steel. During excavation of the existing dike material, every effort should be made to not damage the slurry trench membrane with the excavation equipment.

51. One 4 cu yd capacity rubber tire or track mounted front end loader will be used to excavate the existing dike materials, and load the 12 cu yd dump trucks for transport to the construction areas. Two track mounted dozers (D-6 Cat) will be used to construct haul roads and to spread the dike fill materials.

52. Prior to placement of the welded steel wire reinforcement at el 960, the ground surface will be compacted with the sheepsfoot roller and then scarified with the disc to a depth of at least 3 inches to provide a good bonding surface between the wire and soil. The first 1 ft of fill above the wire will be compacted with a vibratory steel wheel roller in one 12-inch lifts. After 1 ft of cover has been placed and compacted over the reinforcement by the method above, use of the sheepsfoot roller may be initiated with lift thicknesses of 8 inches. Compaction testing for quality control of every 1500 to 2000 cu yds of fill material will be required.

Dike Construction Procedure

53. The proper construction sequence and procedure can not be overemphasized because the desired effect of the reinforcement can not be achieved unless specific sequential construction procedures are followed. The fill for the dike will be placed along the embankment centerline and spread equally toward each embankment toe in an attempt to smooth out and cover any area where the steel wire may try to bend. The fill must be spread in a 12 inch layer immediately over the reinforcement and compacted

to the proper density. The outward movement of the dike fill during the spread operation will stretch the wire away from the dike centerline and help maintain it in an horizontal plane. The embankment fill should be constructed in essentially horizontal layers from toe to toe over the entire embankment width.

54. The welded wire reinforcement will be delivered in rolls with minimum widths and lengths of 8 ft and 160 ft, respectively. Specifications, wire costs and placement cost for the welded wire are shown in Table 7. The 160-ft-long rolls of wire will be unrolled with a large steel roller. Adjacent edges of the wire will be connected with double wraps of No. 16 tie wire every 3 ft. In addition to the tie wire, 2-ft-long U-shaped rebars will be driven into the soil joining the edges of the welded wire every 15 ft. The U-bar shall be fabricated by bending a 4-ft-long piece of No. 4 rebar on a 2 inch radius and driven in a staggered pattern.

55. The long direction of the welded wire reinforcement roll is to be at right angles to the embankment alignment. Welded wire placement location in the embankment is between stas 2+65 and 13+50 of the site plan as shown on construction drawing 4. Each roll of welded wire should be rolled out perpendicular to the longitudinal axis with half of the roll on each side of an established dike centerline. Any manufacturing defects or damage from construction equipment or installation technique should be documented and replaced or repaired. After wire placement embankment fill may be placed on the wire and spread with the dozers. The dozers and/or dump trucks should never be allowed directly on the wire. At least 12 inches of embankment fill should be maintained between the tracks or wheels and the wire.

56. Placement of a continuous 60-ft-wide by 3-ft-deep sand blanket drain will be constructed along the base of the downstream portion of the embankment (construction drawings 10 through 12). Approximately 8000 cu yds of concrete sand will be required to construct the sand blanket toe drain. A gradation curve for the sand drain material is shown in Figure D2. These materials are available from local sand and gravel operators and are referred to locally as concrete sand. Placement of the sand blanket will require enough water to flood the sand so that a minimum relative density of 80 percent may be obtained. If the 80 percent relative density cannot be obtained by flooding the vibratory steel wheeled roller will be used to achieve the specified density.

57. The construction procedures outlined in this section are necessary assure proper dike performance. The reinforcement and fill placement sequence is extremely important; this construction technique is new and the contractor will probably not have prior experience with earth reinforcement. Corps personnel must be on site at all times to monitor and inspect all construction activity.

PART VIII: REQUIRED EQUIPMENT, MATERIAL QUANTITIES, AND ESTIMATED CONSTRUCTION COST

General

58. The experimental nature of this project is such that the potential need to alter construction sequence during embankment construction and instrumentation installation must be realized. This project will be divided into the following separate contracts:

a. Welded steel wire delivery and placement contract.

b. Concrete sand blanket material purchase and delivery contract.

c. Equipment rental contract.

d. Borrow area fill material purchase and delivery contract.

Welded Steel Wire and Placement Contract

59. The welded steel wire and placement contract will include 136 rolls of welded steel wire, 8-ft-wide, 160-ft-long delivered, placed, anchored and tied together. The warp direction of the welded wire fabric will be made up of deformed wire with a cross sectional area of 0.12 in.^{2} per wire, welded on two inch centers. The fill direction will include deformed wire with a cross sectional area of 0.045 in.^{2} per wire welded on 6 inch centers. The wire mesh will be henceforth referred to as 2X6-D12/D4.5. All welded wire will have a yield strength of 70,000 psi. Table 7 includes the wire properties, widths, lengths, weights, anchors, tie wire and approximate cost of materials and placements. Each roll of welded wire will arrive at the construction site rolled up. The contractor will feed each roll through a steel wire unbending machine that will take the bend out of each roll so that the wire will lay flat on the ground surface. The ground surface will be excavated to el 960 prior to wire placement and scarified for good bonding between the wire and soil. The wire will be placed perpendicular to the longitudinal axis, tie wired and anchored in place as previously described.

Concrete Sand Contract

60. Eight thousand cu yds of filter 1 concrete sand, Figure 8, will be purchased for the 3-ft-deep, 60-ft-wide, 1200-ft-long sand blanket that will be constructed within the downstream portion of the embankment as shown on construction drawings 10 through 12. The bottom of sand blanket will be at el 960, the top of the blanket will be at el 963 with the welded wire at el 960. Sand meeting the specification may be purchased from local sand and gravel operators. These materials as previously described must be compacted to a minimum of 80 percent relative density. Gradation and compaction curves are shown in Figure A1. The sand materials will be delivered to the construction site as directed by the Corps to cover the welded wire and must not be stockpiled in large quantities. Flooding of the sand blanket with water from the water truck must be accomplished in 12 to 18 inch layers soon after the sand is delivered to achieve the required density. Once the 3-ft-thick blanket is constructed to the proper density, dike fill material must be used to cover the sand blanket. Construction equipment not used in placement should not be allowed to traffic over the sand drain.

Equipment Rental Contract

61. Before placement of the welded wire reinforcement the existing dike material should be removed by an equipment rental contract at the direction of by HDO staff. Assuming that a rental contract will be used, rental construction operations may be subdivided into three phases.

- a. Phase I - mobilization, borrow haul road and turn around construction
- b. Phase II - embankment construction
- c. Phase III - demobilization, seeding and mulching

62. Mobilization will include borrow haul road and turn around construction plus time to ready his equipment and 'transport it into the work areas. Demobilization will include seeding and mulching and time to remove his equipment from the construction site. Approximately two weeks will be required to begin setting up equipment for wire placement after initiation of the welded wire contract. This set up time can be concurrent with construction of haul roads, turnaround areas, and excavation of existing dike

materials. All activities must be coordinated by WES and HD personnel for instrumentation installation, scheduling of material placement, compaction, and testing. A bar chart of construction and instrumentation activities is shown in Figure 10.

63. Table 8 lists all the equipment and hours needed for the rental contract construction in a format which may be used to develop bid advertisement specifications. Two front end loaders are specified: one with tracks and one with rubber tires (in the event there are mobility problems in removing material down to el 960). A foreman should be available at all times during the rental contract to coordinate construction activities with the Corps.

64. Approximately 30,000 cu yds of existing dike fill material will be required to be removed prior to placement of the welded wire fabric at el 960 between stas 2+65 and 13+50. Assuming that the contractor can excavate, haul and place 2000 cu yds per day, it will require about 15 days to remove the existing dike material. The dike material will be removed to either the north and/or south abutments or transported around the outside of the dike reinforcement on the berm and placed on the welded wire. After the dike material is removed to el 960 the front end loaders can be demobilized. Once the dike fill material has been constructed to el 962 above the wire reinforcement, the steel wheel roller may be released. The wacker compactors will be used to compact the fill material around settlement plates, inclinometer tubes and piezometers and will be used throughout the job. Laborers will be used to operate the wacker compactors, place material around all the instrumentation, and perform other miscellaneous tasks.

Borrow Area Contract

65. The borrow material purchase and delivery contract will require approximately 117,000 cu yds from two borrow pits located within a half mile of the construction site as shown on construction drawings 3 and 16. About 43,000 cu yds of fill material will be required between stas 0+00 to 2+65 to construct the north abutment to el 983 and relocate the county road. About 4,000 cu yds is required to construct the south abutment to grade from sta 13+50 to 14+50. Approximately 19,342 sq yds of welded wire reinforcement fabric and 100,000 cu yds of fill material are required to construct the

reinforced wire section between stas 2+65 and 13+50. A total of about 117,000 cu yds of fill material will be required from the borrow area, but a total 147,000 cu yds of fill will be handled during dike construction. These data along with the sand blanket material data and number of rolls of welded wire reinforcement are shown in Table 9.

66. It is estimated the 30,000 cu yds of existing dike material above el 960 may be removed in 15 days with the rental contract. The remaining 117,000 cu yds can be excavated and transported to the construction site under the borrow contract at about 1800 cu yds per day; about 60 days would be required to complete this work. It is estimated that it will cost about $3 per cu yd to excavate and transport borrow material to the dike. This work will cost a total of about $351,000. This total time including 3 days for mobilization and 3 days for demobilization, seeding, and mulching will be about 82 days or 4 months to complete the project. The total cost for the project is tabulated for each phase of the project in Table 10. The total cost of construction is estimated to about $876,590.

Table 1

Mohicanville Dike No. 2 Summary of Permeabilities

Boring No.	Depth	Classification	Type Test	K_v(cm/sec)
		Clay		
UD-21	9-11	CL	R	$1.3 * 10^{-5}$
				$1.8 * 10^{-6}$
UD-21	14-15	CL	R	$4.0 * 10^{-7}$
UD-22	19-21	CL	R	$6.5 * 10^{-7}$
UD-22	28-30	CL	R	$1.1 * 10^{-7}$
				$8.4 * 10^{-8}$
				$7.8 * 19^{-8}$
UD-23	14-16	OL	R	$7.9 * 10^{-5}$
				$2.1 * 10^{-5}$
UD-25	39-41	OH	R	$3.4 * 10^{-7}$
				$1.8 * 10^{-7}$
				$1.1 * 10^{-7}$
UD-26	29-31	CL	R	$1.3 * 10^{-7}$
UD-27	20-22	CH - OH	R	$6.3 * 10^{-7}$
		Peat		
UD-27	4-6	PT	Laboratory falling head	$1.7 * 10^{-7}$
				$2.5 * 10^{-7}$
UD-27	9-11	PT	Laboratory falling head	$2.6 * 10^{-7}$
				$2.9 * 10^{-7}$
				$2.3 * 10^{-7}$
D-18	7-12	PT	Field falling head	(K_h) $3.7 * 10^{-5}$
UD-27	6-11	PT	Field falling head	(K_h) $9.2 * 10^{-5}$
		Fill		
UD-21	9-11	CL	R	$6.5 * 10^{-6}$
				$9.3 * 10^{-7}$
UD-21	14-16	CL	R	$2.1 * 10^{-7}$

Table 2

Engineering Physical Properties

Material	90% Range			Average		
	Dry (PCF)	e	Wc (%)	Dry (PCF)	e	Wc (%)
Peat (Centerline)	16.0-31.7	3.135-7.190	116.6-305.8	30.0	4.369	181.8
Peat (Toe)	8.6-18.6	4.035-14.222	269.0-635.9	14.9	7.798	390.6
Clay (Centerline)	73.7-96.3	0.776-1.348	28.5-49.7	82.6	1.059	39.1
Clay (Toe)	40.0-96.3	0.822-2.178	29.2-108.1	77.7	1.234	48.4
Embankment Fill	109.1-116.8	0.464-0.567	14.7-16.3	113.0	0.514	15.7

Table 3

Summary of Strength Test Data

Embankment Fill
Total P-Q

Boring No.	Depth (ft)	P (TSF)	Q (TSF)
	Q Tests		
UD-1	Unknown	0.44	0.44
UD-24	39-41	3.58	1.03
	$\bar{R}$ Tests		
UD-21	9-11	1.73	0.73
		1.49	0.99
		0.96	0.71

Embankment Fill
Effective P'-Q

Boring No.	Depth (ft)	P (TSF)	Q (TSF)
	$\bar{R}$ Tests		
UD-21	9-11	1.02	0.50
		0.74	0.46
		0.64	0.44

(Continued)

Table 3 (Continued)

Foundation Peat
Total P - Q

Boring No.	Depth (ft)	0.75 $'_{v}$ (TSF)	P (TSF)	P (TSF)
		Q Tests		
UD-25	21-23	0.69	1.96	0.56
UD-21	24-26	0.78	1.96	0.46
UD-26	9-11	0.15	0.69	0.24
UD-1	22-24	0.69	0.67	0.67
UD-26	11-12	0.15	0.66	0.16
UD-22	9-11	0.15	0.44	0.09
UD-27	9-11	0.02	0.44	0.09
UD-23	9-11	0.02	0.42	0.07
UD-22	6-7	0.14	0.28	0.08
UD-22	4-6	0.13	0.26	0.11
UD-27	4-6	0.01	0.26	0.11
UD-2	9-11	0.02	0.13	0.13
		R and $\bar{R}$ Tests		
SI-5	7-9		1.34	0.34
UD-25	21-23		1.27	0.57
SI-1	10-12		1.10	0.35
SI-4	9-11		0.95	0.35
SI-6	5-7		0.82	0.44
SI-5	7-9		0.76	0.26
SI-1	10-12		0.57	0.20
UD-23	9-11		0.51	0.22

(Continued)

Table 3 (Continued)

Boring No.	Depth (ft)	P' (TSF)	Q (TSF)
	Foundation Peat Effective P' - Q'		
UD-27	9-11	1.40	0.40
UD-37	4-6	0.68	0.28
SI-5	7-9	0.54	0.34
SI-6	5-7	0.46	0.44
SI-4	9-11	0.42	0.32
SI-1	10-12	0.37	0.35
SI-5	7-9	0.29	0.26
SI-1	10-12	0.24	0.19

(Continued)

Table 3 (Continued)

Foundation Clay
Total P - Q

Boring No.	Depth (ft)	0.80 $'_v$ (TSF)	P (TSF)	Q (TSF)
		Q Tests		
UD-25	59-61	1.36	3.94	0.34
UD-21	49-51	1.24	3.24	0.39
UD-25	49-51	1.16	2.98	0.33
UD-21	39-41	1.03	2.60	0.35
UD-21	29-31	0.83	2.33	0.68
UD-21	34-36	0.93	2.25	0.30
UD-25	34-36	0.85	2.23	0.38
UD-25	29-31	0.75	2.04	0.34
UD-26	29-31	0.41	1.59	0.09
UD-23	29-31	0.40	1.51	0.21
UD-27	24-26	0.16	1.16	0.11
UD-26	19-21	0.20	1.07	0.17
UD-23	19-21	0.19	0.99	0.14
		$\bar{R}$ Tests		
UD-25	39-41		3.67	1.17
UD-22	28-30		2.15	0.65
UD-25	39-41		1.99	0.74
UD-22	28-30		1.23	0.48
UD-25	39-41		1.17	0.54
UD-23	14-16		0.97	0.37
UD-26	29-31		0.94	0.32
UD-27	20-22		0.90	0.40
UD-22	19-21		0.89	0.51
UD-22	28-30		0.71	0.33
UD-23	14-16		0.57	0.27
UD-23	14-16		0.38	0.23

(Continued)

Table 3 (Concluded)

Foundation Clay Effective P' - Q'			
Boring No.	Depth (ft)	P' (TSF)	Q (TSF)
UD-25	39-41	2.19	1.17
UD-22	28-30	1.18	0.61
UD-25	39-41	1.12	0.73
UD-22	28-30	0.82	0.43
UD-25	39-41	0.75	0.55
UD-22	19-21	0.68	0.42
UD-27	20-22	0.67	0.39
UD-26	29-31	0.56	0.31
UD-22	28-30	0.53	0.29
UD-23	14-16	0.49	0.37
UD-23	14-16	0.36	0.26
UD-23	14-16	0.25	0.25

Table 4

Summary of Consolidation Time Rate: C_v

Foundation Clay

Test Type	Boring No.	Depth (ft)	Load (ksf)	C_v (sq. ft/day)
$\bar{R}$	UD-22	28-30	0.75	0.20
			1.50	0.17
			3.0	0.26
$\bar{R}$	UD-25	39-41	1.25	0.32
			2.50	0.14
			5.0	0.10
$\bar{R}$	UD-26	29-31	1.25	0.26
Consolidation	UD-23	24-26	1.0	0.40
			2.0	0.46
			4.0	0.27
			8.0	0.26
Consolidation	UD-25	34-36	4.0	0.19
			8.0	0.08
Consolidation	UD-26	19-21	2.0	0.48
			4.0	0.40
			8.0	0.30
Consolidation	SI-6	28-30	0.5	0.17
			1.0	0.18
			2.0	0.16
			4.0	0.16
			8.0	0.16

Design value C_v = 0.20

Table 5

Location of Instrumentation

Instrument	Dam Station					Location, ft	Elev, ft	Total
Settlement Plates	4+75	6+55	9+45	12+20		+10, ±60	690	12
Piezometers (pneumatic)	"	"	"	"		+10	950	16
	"	"	"	"		+10	935	
	"	"	"	"		+10	925	
	"	"	"	"		+10	910	
		"	"			±62	945	8
		"	"			±62	930	
Piezometers (Casagrande)	"	"	"	"		+10	935	4
(electrical)	"	"	"	"		+10	925	4
(Casagrande)		"	"			±62	945	4
(electrical)		"	"			±62	930	4
(Casagrande)*			"			-150	947	2
(Casagrande)*	"					-75	948	
Slope Indicators (vertical)		"				+60	900	8
					6+55	+118*	904	
					6+55	-105*	905.5	
					9+50	-80*	870	
					9+80	+65*	866.5	
					10+50	-93*	891.5	
					12+15	+128*	909	
			"			+132	867	
Slope Indicators (horizontal)	"	"	"	"		-90 to 90	959	
Surface Monuments	"	"	"	"		+5	98[illegible]	
	3+00	7+50	11+00	14+00		+30		
						+105		
Strain Gages (transverse)		6+55	9+45			℄ and at increme[illegible] to ±70		
(longitudinal)	4+75	8+00	11+00	12+20		±5		

* Presently installed

Table 6

Observation Schedule

Instrument	During Construction	After Construction
Bench marks	Monthly for 6 months, if no movement occurs check semiannually	Semiannually until project is considered stable
Surface monument	When installed, then every 3 days or after each 2 feet of embankment fill, more frequently if adverse conditions develop	Weekly for 6 months, then review
Inclinometer, settlement plates, strain gages	When installed, then every 3 days or after each 2 feet of embankment fill, more frequently if adverse conditions develop	Weekly for 6 months, then review
Piezometers	Daily	Twice a week for 3 months, then review

Table 7

Mohicanville Dike No. 2 Welded Wire

Specification and Placement Cost

Wire Size: Welded Wire 2 * 6 - D12/D4.5	
Yield Strength: 70,000 psi	
Rolls (8 ft * 160 ft) Required: 144	
Weight per Roll (8 ft * 160 ft = 1280 sq ft * 2.75 lb/sq ft): 3520 lb	
Total Weight 136 rolls * 3520 lb/roll: 478,720 lb	
Cost (delivered) $0.25 lb * 478,720:	$119,680
U-Shaped Anchors, 2 ft long No. 4 bar with 2-in. Radius, 12 per roll * 136 rolls: 1632	
Cost of U-Shaped Anchors at $4.00 ea * 1632:	6,528
No. 6 Tie Wire spaced on 3-ft centers, 1 ft wire per location * 160-ft rolls * 135 seams ÷ 3 ft spacing: 7200 ft	
No. of Rolls of Wire Required, 7200 ft : 100-ft roll: 72	
Cost of No. 6 Wire, $4.00 per roll * 72 rolls:	288
Wire Placement Cost, 20 days labor and equipment rental:	50,000
Subtotal	$176,496
Contingency, 10%	17,650
Total	$194,146

Table 8

Items Needed for Rental Contract Construction

Bid Item No.	Quantity	Description	Stand by Rates	Total Hours for Quantity
1	2	Large-track dozer with blade and operator, D-6 or equivalent	3 day-mob. 15 day-remove exit mat 60 day-spread borrow	24 120 480
2	1*	4-cu yd rubber-tire, front-end loader and operator	3 day-mob. 15 day-exist dike mat	24 120
3	1*	4-cu yd track, front-end loader and operator	3 day mob. 15 day-exist dike mat	24 120
4	3-6*	Dump truck and operator, 12-cu yd struck capacity, shore wheel base, tandem axle	15 day-exist mat	720
5	1	Road grader and operator Catapiller 12 or equivalent	3 day-mob. 15 day-exist mat 60 day-spread borrow	24 480
6	1	Water truck and operator, equipped with power sprayer bar and minimum 2000-gal capacity tank	15 day-exist mat 60 day spread borrow	120 480
7	1	Self-propelled sheepsfoot roller and operator equipped with dozer blade	15 day-exist mat 60 day-compact borrow	120 480
8	1-2*	Seeding and mulching	3 day-mob., $1500/acre	24
9	2-6*	Laborer, common	55 day	2640
10	1	36" serated disic	75 day-exist mat	600
11	2	Wacker compactor	75 day-exist mat	600
12	1	Self propelled steel wheel roller and operator	75 day-exist mat	600
13	10	Seeding and mulching	$1500/acre	
14	1	Foreman	3 day mob.	24
			13 day-exist mat 65 day-spread 3 day-demob.	120 520 24

* Quantity of this item will vary depending upon particular phase of the project. The Government will give 24-hr notice when to change (add or delete) number of items in use as contemplated.

Table 9

Material Quantities

Item No.	Description	Quantity
1	Existing dike material above el 960 between sta 2+65-14+50 to be excavated and moved to each abutment	-30,000
	Material required to relocate county road and complete north abutment between sta 0+00-2+65	43,000
	Material required to complete dike section from el 960 el 983 between sta 2+65-13+50	100,000
	Material required to complete dike south abutment from el 966 to el 983 between sta 13+50 to 14+50	4,000
2	Contract borrow material fill for dike	117,00
	Total Material to be handled	147,000 cu yd
3*	Welded steel wire dike reinforcement fabric 478,720 lb or 19,342, sq yd	
4	Concrete sand blanket material	8,000 cu yd

* Furnished fabric rolls (labeled with lengths and width) deformed welded steel wire, 2 * 6 - D12/D4.5, 136 rolls 8 ft wide and 160 ft long.

Table 10

Summary of Estimated Construction Costs by

Project Construction Phase

1. Equipment Rental

Mobilization; Haul Road and Turnaround Area Construction (3 working days)

Contractor-Furnished

(3 working days)

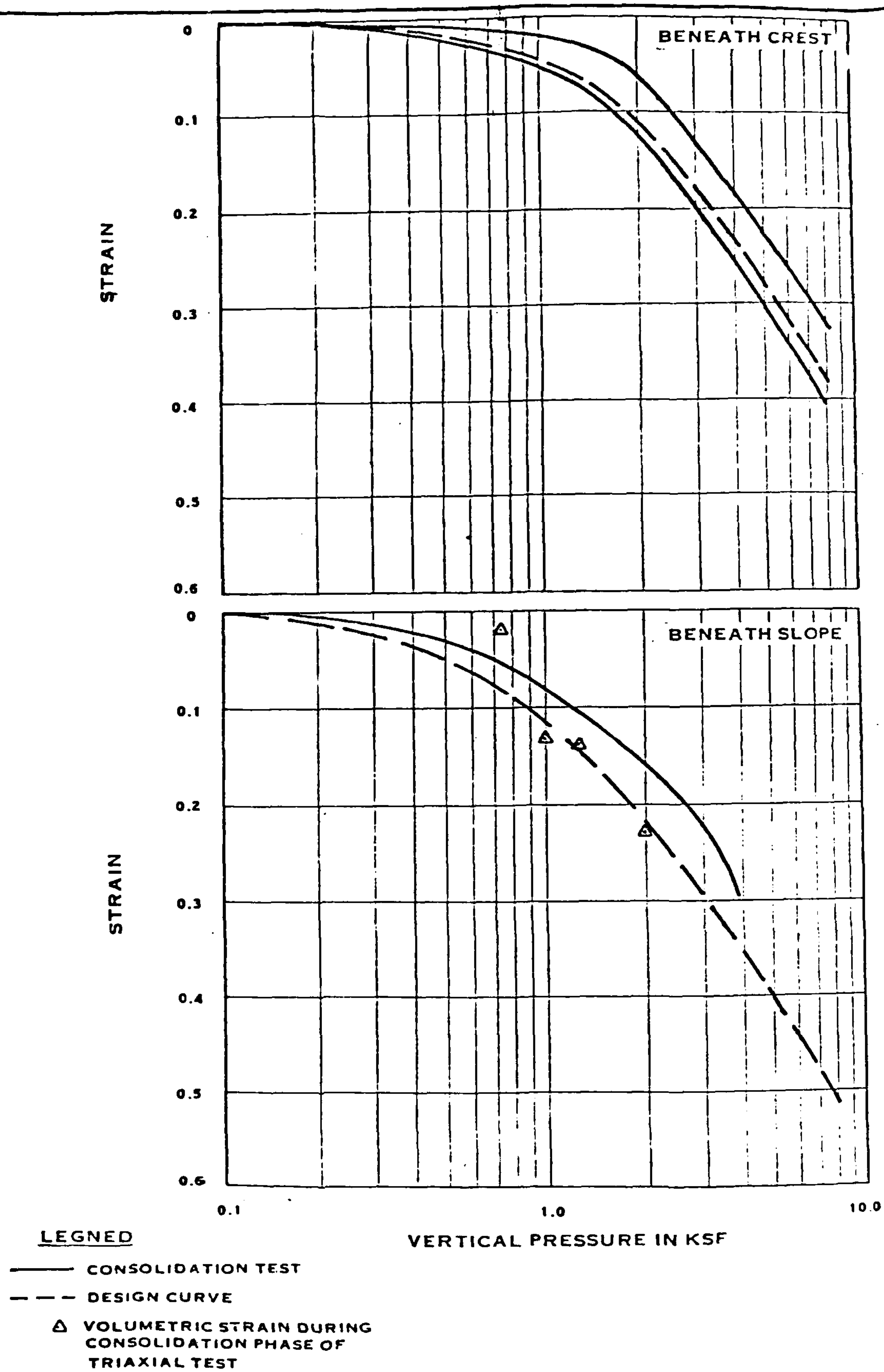
1 dozer * $55/hr * 8 hr/day * 3 days - - - - - - - - - -	1,320
1 dozer * $80/hr * 8 hr/day * 3 days - - - - - - - - - -	1,920
1 Front-end Loader * $60/hr * 8 hr day * 3 days - - - -	1,440
2 Laborers * $12/hr * 8 hr/day * 3 days - - - - - - - -	288
Mobilization, Lump Sum (1 day)	1,000
Subtotal	4,968

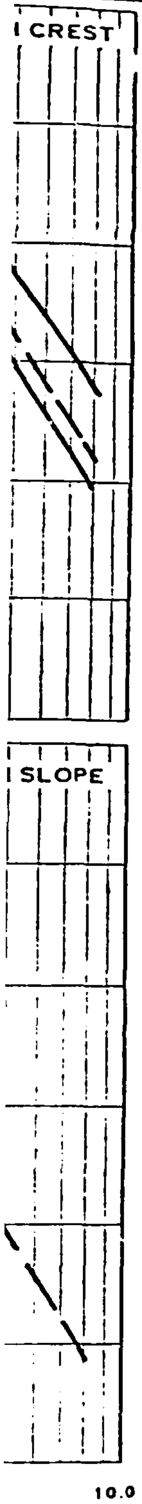
Excavate 30,000 cu yd existing dike material and spreading and compaction 117,000 cu yd of borrow material

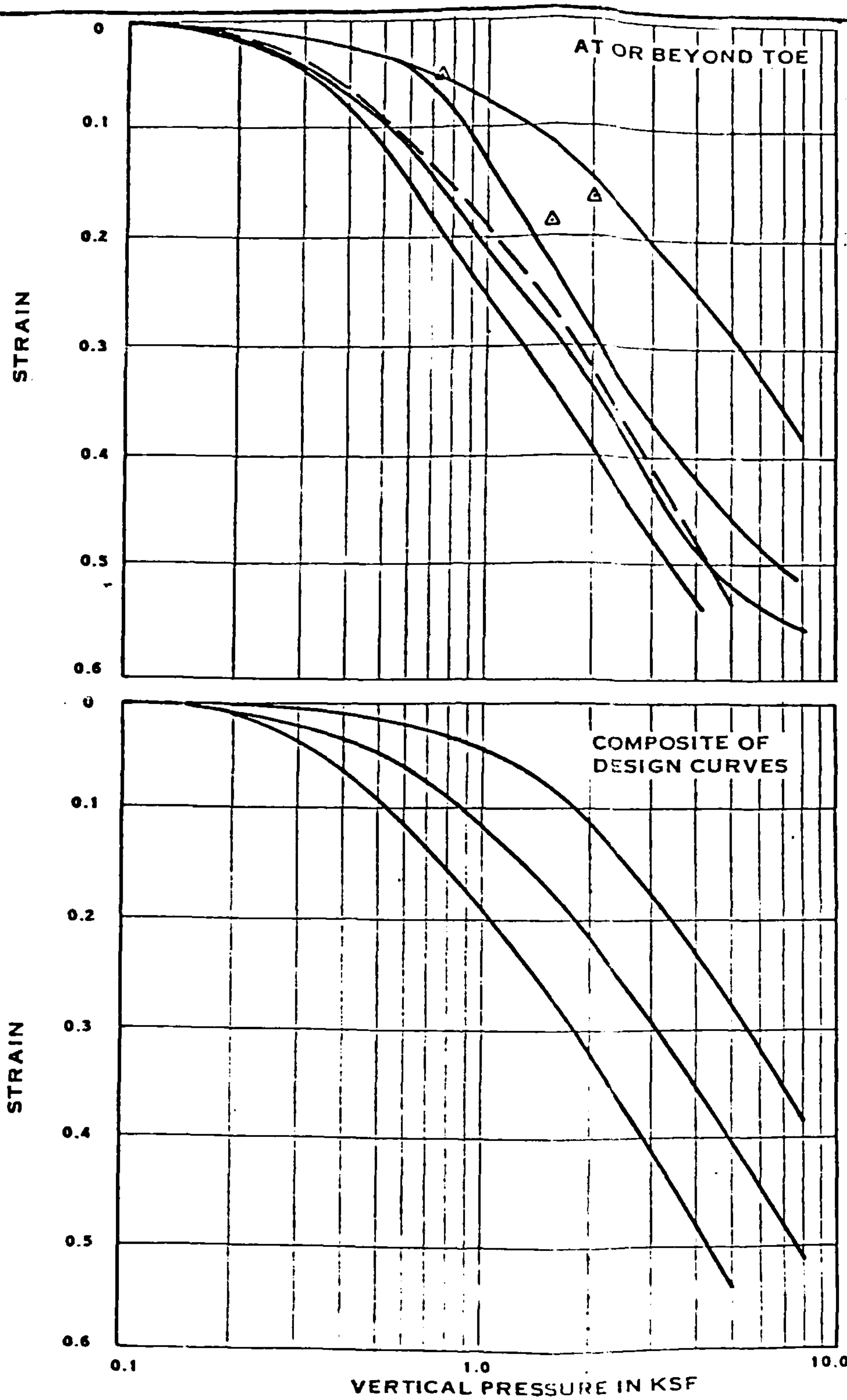
Contractor-Furnished

1 dozer * $55/hr * 8 hr/day * 75 days - - - - - - - -	33,000
1 dozer * $80/hr * 8 hr/day * 75 days - - - - - - - -	48,000
1 Front-end Loader * $60/hr * 8 hr/day * 15 days - -	7,200
4 Dump Trucks * $35/hr * 8 hr/day * 15 days - - - -	16,800
1 Road Grader * $50/hr * 8 hr/day * 75 days - - - -	30,000
1 Water Truck * $40/hr * 8 hr/day * 75 days - - - -	24,000
1 Sheepsfoot Roller * $50/hr * 8 hr/day * 75 days -	30,000
6 Laborers * $12/hr * 8 hr/day * 55 days - - - - - -	31,680
1 Steel weld roller $40/hr * 8 hr/day * 15 days - -	4,800
1 36" diameter disc $30/hr * 8 hr/day * 75 days - -	18,000
1 Wacker compactor $30/hr * 8 hr/day * 75 days - -	18,000
Demobilization - Contractor Furnished Demobilization, Lump Sum (1 day)	1,000
Seeding and Mulching $1,500 per acre about 10 acres, 3 days	15,000

Rental Contract Estimated Cost - - - - - - - - - - - - - -	282,450
2. Concrete Sand - 8000 cu yd * $5.00/cu yd - - - - - - - - -	40,000
3. Borrow Fill Material - 117,000 cu yd * $3.00/cu yd	351,000
Seeding and Mulching $1,500 per acre, about 6 acres	9,000
4. Welded wire and placement - - - - - - - - - - - - - - - - -	194,146
Total Project Cost	876,590


BENEATH CREST
BENEATH SLOPE
STRAIN
STRAIN
0
0.1
0.2
0.3
0.4
0.5
0.6
0.1
1.0
10.0
VERTICAL PRESSURE IN KSF
LEGNED
CONSOLIDATION TEST
DESIGN CURVE
Δ VOLUMETRIC STRAIN DURING CONSOLIDATION PHASE OF TRIAXIAL TEST





U.S. ARMY CORPS OF ENGINEERS
HUNTINGTON DISTRICT

HUNTINGTON WEST VIRGINIA

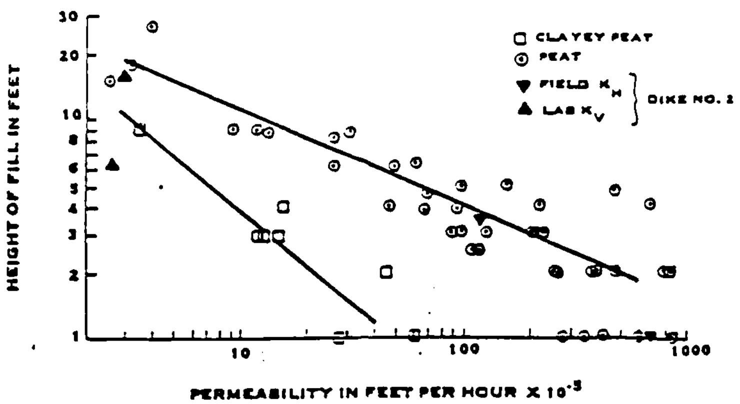
LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA

MUSKINGUM RIVER WATERSHED
OHIO

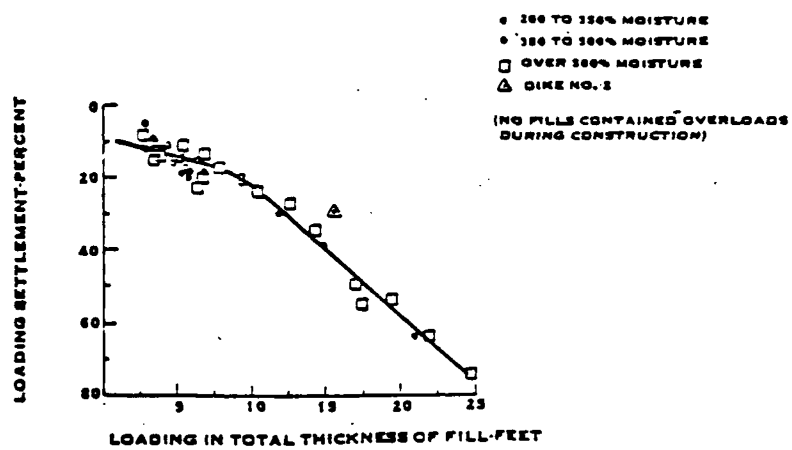
MOHICANVILLE DAM
DIKE NO. 2

SUMMARY OF CONSOLIDATION
TEST DATA
PEAT

EXHIBIT NC
FIGURE 1



a.) VARIATION OF PERMEABILITY WITH LOADING, DATA FROM ALL PROJECTS



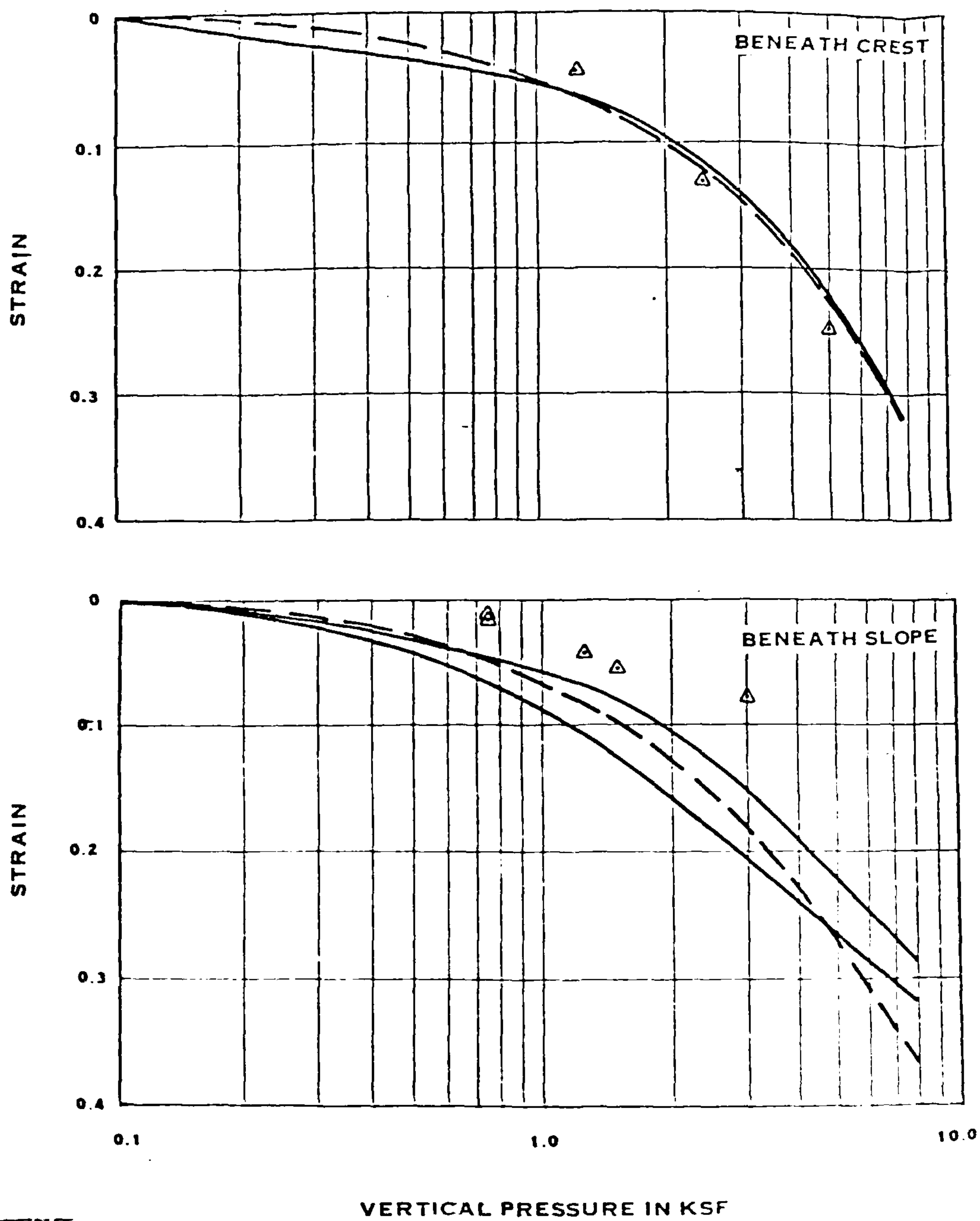
b.) RELATIONSHIP OF LOADING, SETTLEMENT, AND MOISTURE CONTENT OF PEAT

SOURCE: WEBER, WILLIAM G. JR., "PERFORMANCE OF EMBANKMENTS CONSTRUCTED OVER PEAT." JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION, ASCE, VOL. 95 NO. SM1, JAN. 1969

U.S. ARMY CORPS OF ENGINEERS HUNTINGTON DISTRICT HUNTINGTON WEST VIRGINIA	MUSKINGUM RIVER WATERSHED OHIO MOHICANVILLE DAM DIKE NO. 2
LAW ENGINEERING TESTING COMPANY MARIETTA, GEORGIA	COMPARISON OF SETTLEMENT AND PERMEABILITY DATA FIGURE 2

EXHIBIT NO. 32

Figure 2.

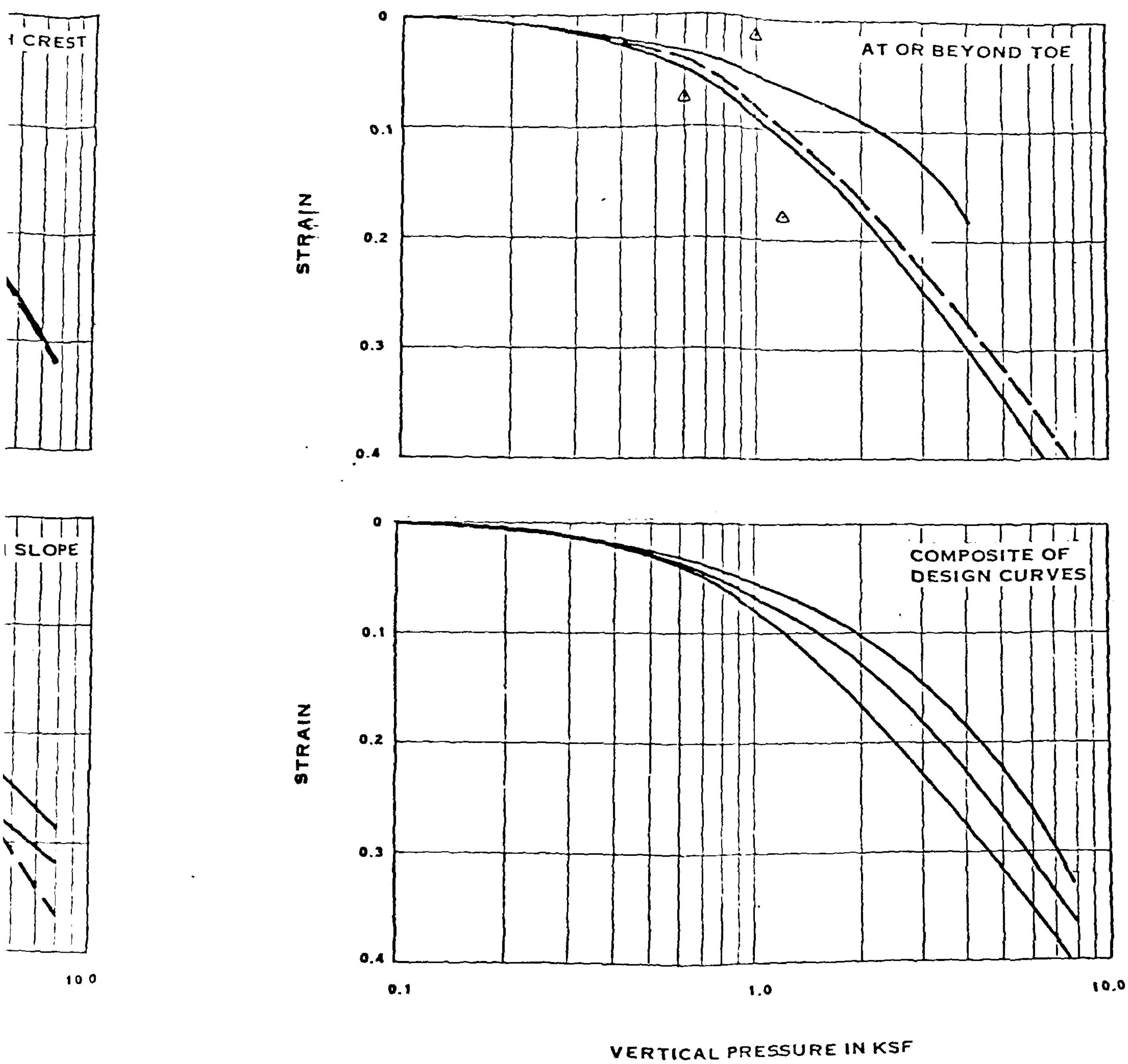


LEGEND

CONSOLIDATION TEST

DESIGN CURVE

VOLUMETRIC STRAIN DURING CONSOLIDATION PHASE OF TRIAXIAL TEST



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HUNTINGTON WEST VIRGINIA

LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA

MUSKINGUM RIVER WATERSHED
OHIO

MOHICANVILLE DAM
DIKE NO. 2

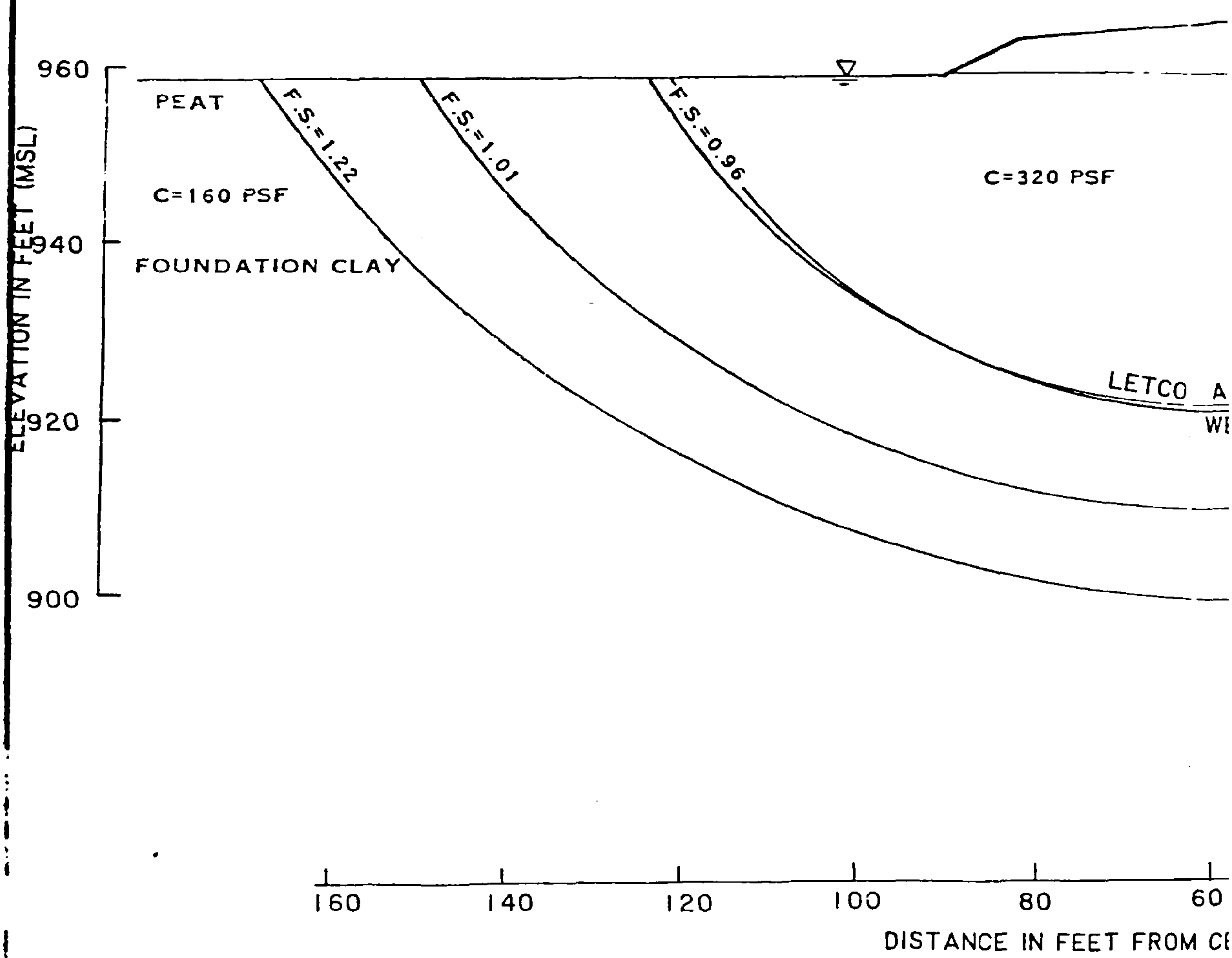
SUMMARY OF CONSOLIDATION
TEST DATA
CLAY

FIGURE 3

EXHIBIT NO. 38

ARC DATA

(X,Y)	LOW TANGENT	F.S
60,993	921	0.96
60,1013	910	1.01
55,1033	900	1.22



LINES IN FOUNDATION CLAY ARE CONTOURS OF EQUAL COHESIVE UNDRAINED STRENGTH

STATION 9 + 15

WES CALCULATIONS:

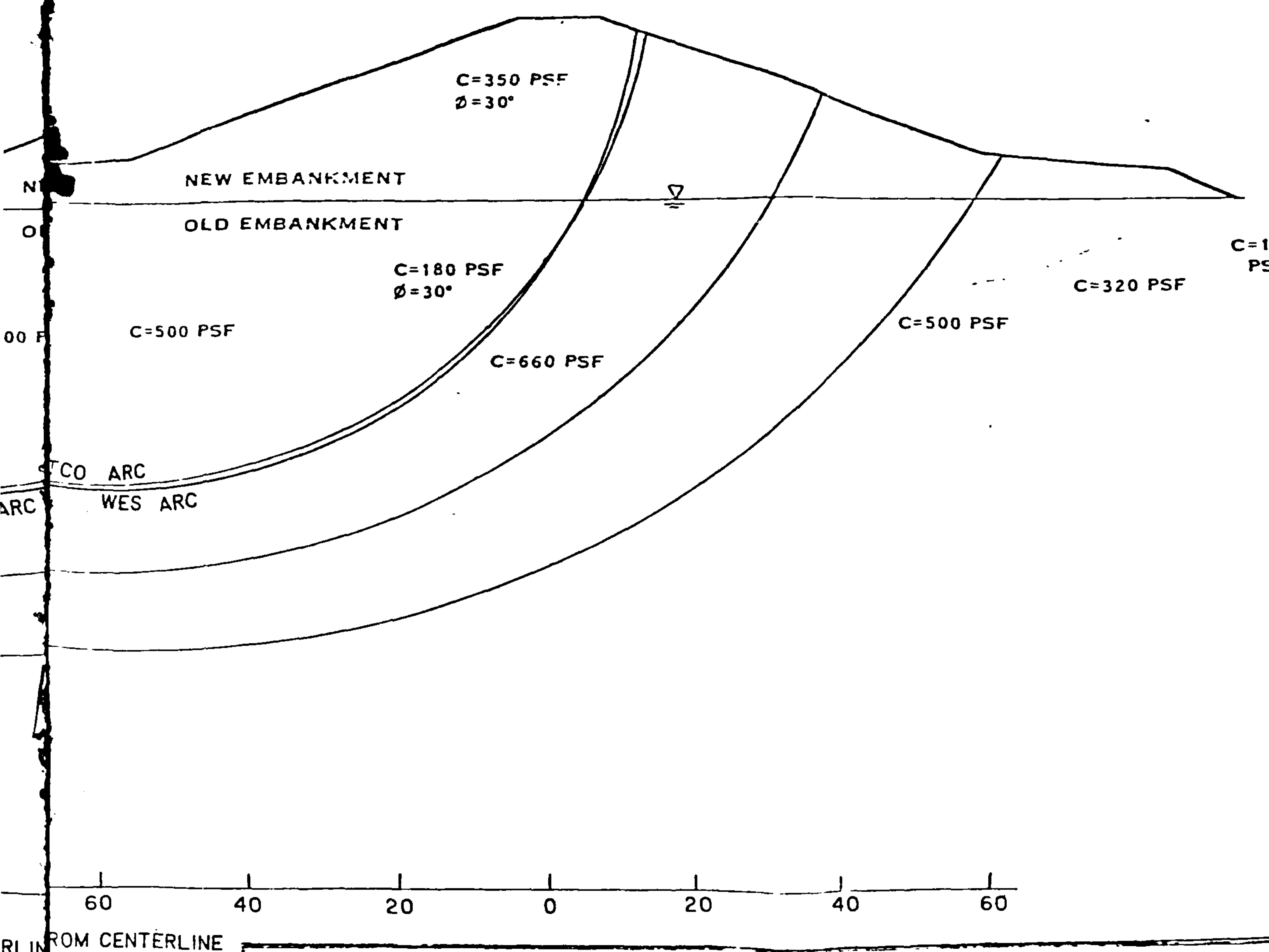
WES ARC FOR DIKE 1 FT HIGHER

M_A=7266 k, FT

M_R=6499 k, FT

R =70 FT

NOTE: THE F.S. VALUES SHOWN DO NOT INCLUDE REINFORCEMENT. REINFORCEMENT IS ADDED TO BRING EACH CIRCLE TO F.S. = 1.3 (SEE EXHIBIT 63)



HUNTINGTON DISTRICT

HUNTINGTON WEST VIRGINIA

LAW ENGINEERING TESTING COMPANY

MARIETTA, GEORGIA

MUSKINGUM RIVER WATERSHED

OHIO

MOHICANVILLE DAM

DIKE NO. 2

UPSTREAM SLOPE STABILITY

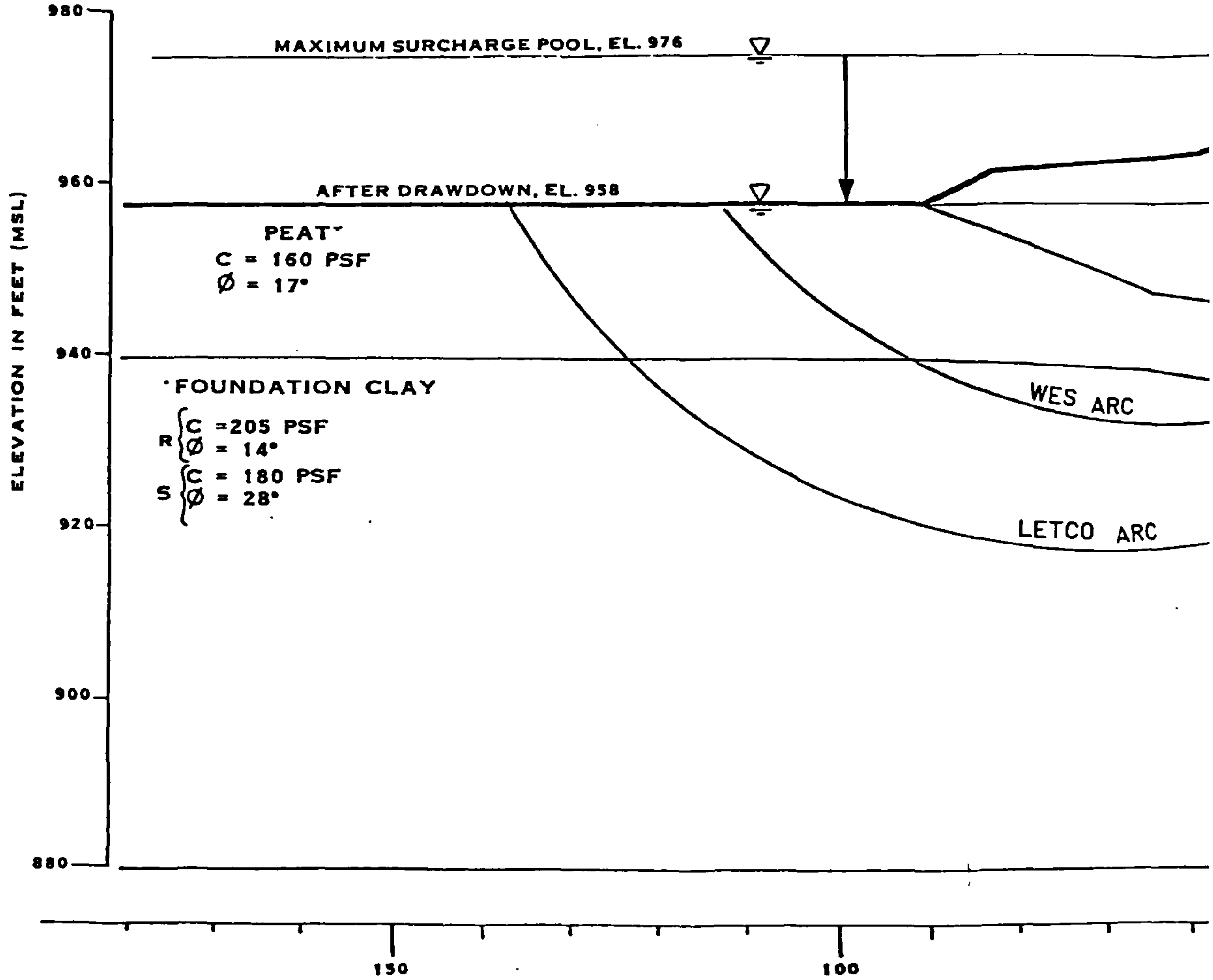
END OF CONSTRUCTION - CASE I

EXHIBIT

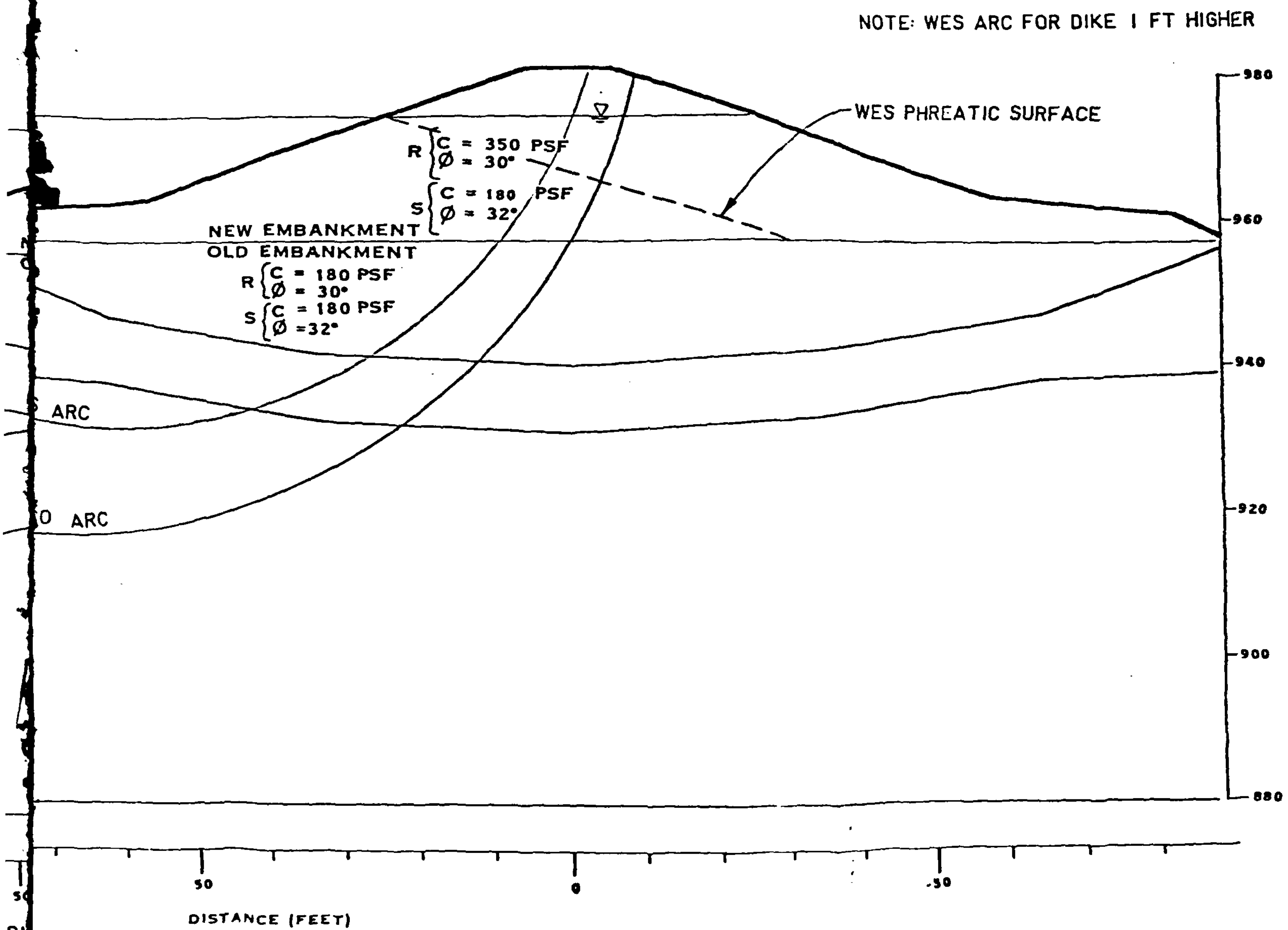
FIGURE

ARC DATA

X,Y	LOW TANGENT	F.S.
70,998	918	1.07



CORPS STABILITY PROGRAM



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HUNTINGTON WEST VIRGINIA

LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA

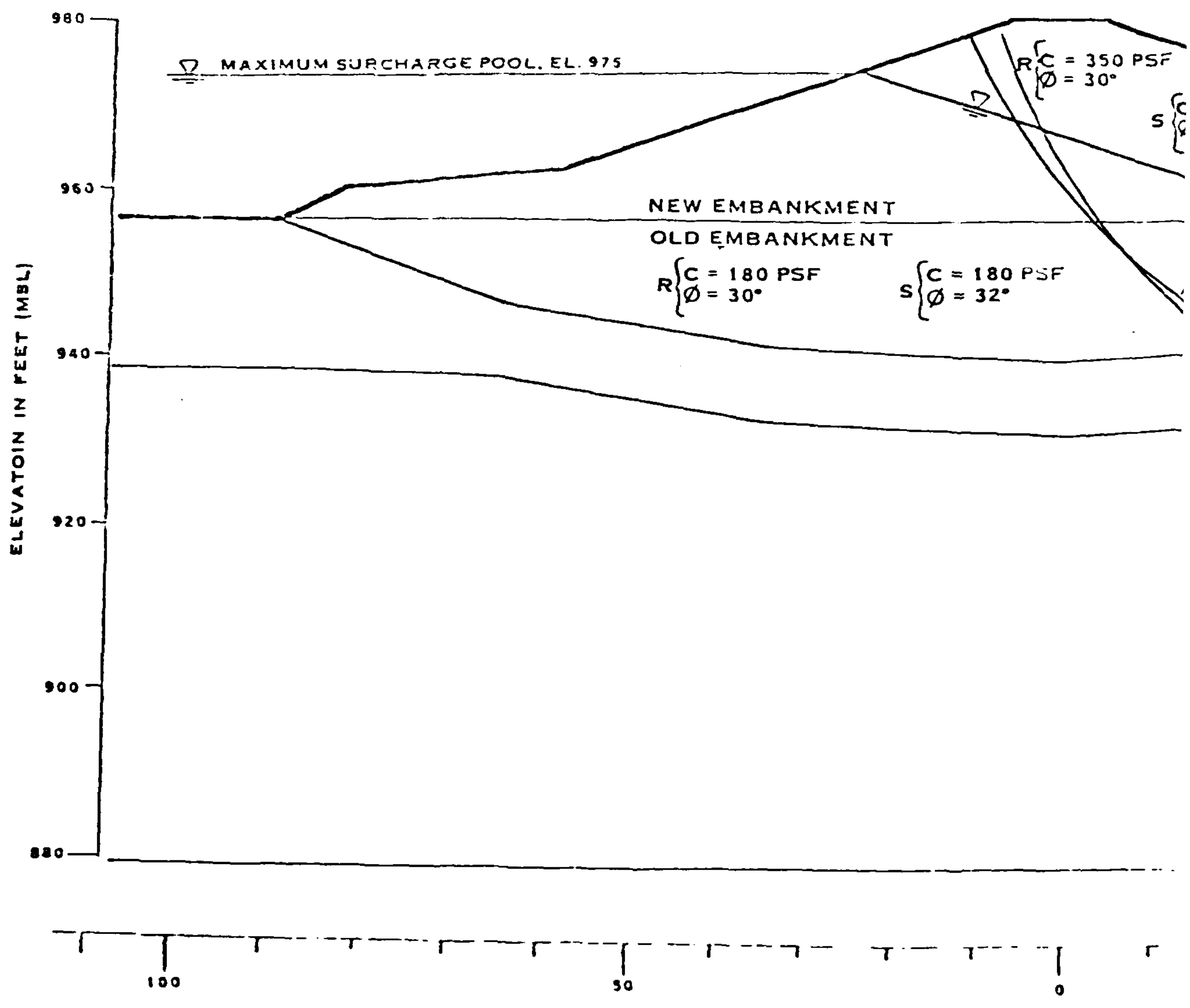
MUSKINGUM RIVER WATERSHED
OHIO
MOHICANVILLE DAM
DIKE NO. 2

UPSTREAM SLOPE STABILITY
SUDDEN DRAW DOWN-CASE II
SATURATED EMBANKMENT

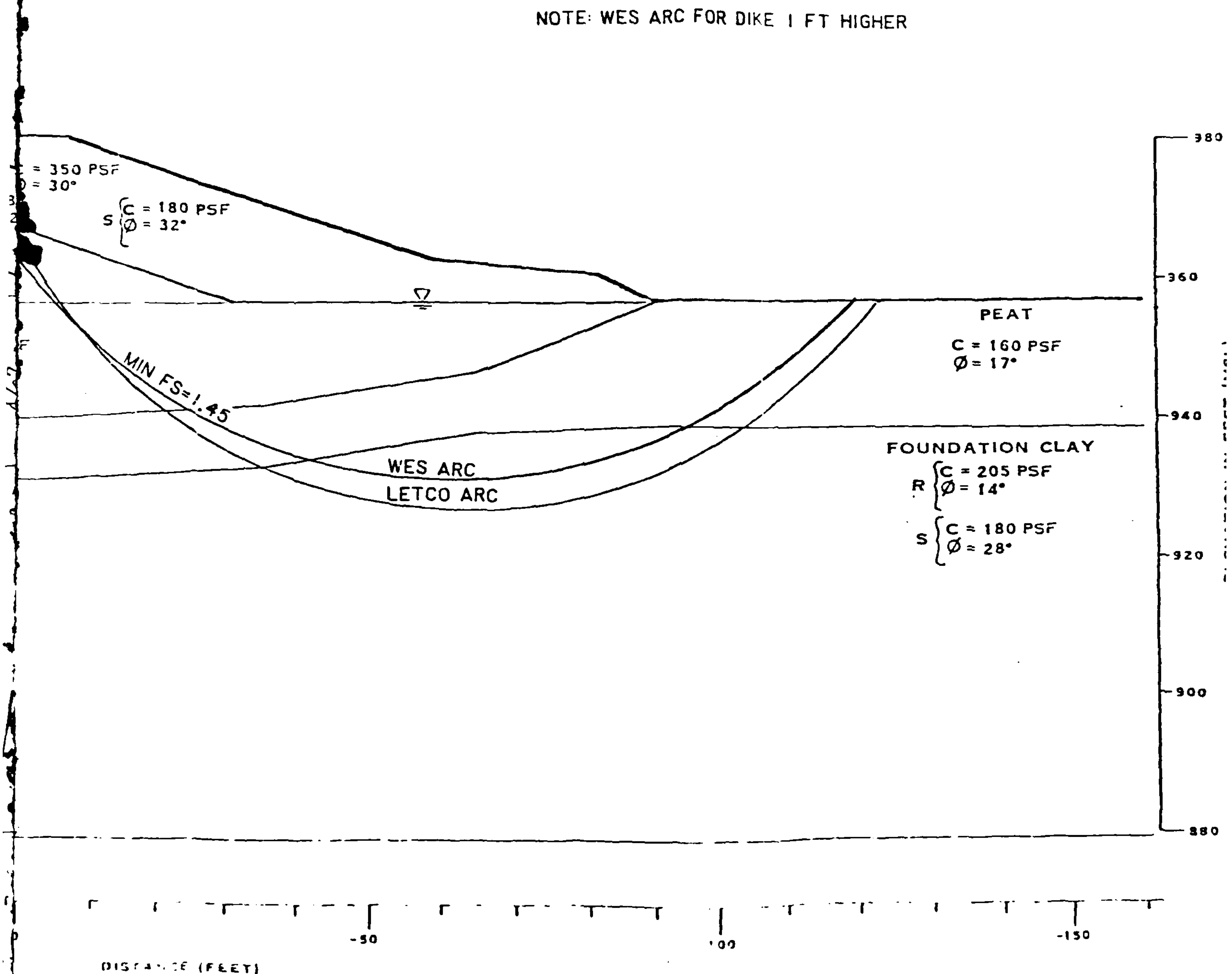
FIGURE 5

EXHIBIT N

X,Y	LOW TANGENT	F.S.
-65,1003	928	1.44



CORPS STABILITY PROGRAM



U.S. ARMY CORPS OF ENGINEERS
HUNTINGTON DISTRICT

HUNTINGTON WEST VIRGINIA

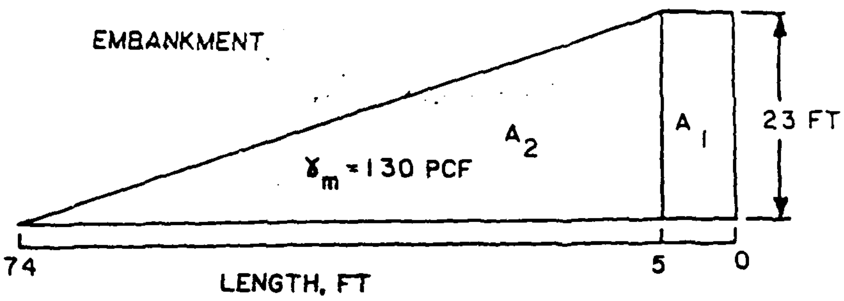
LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA

MUSKINGUM RIVER WATERSHED
OHIO
MOHICANVILLE DAM
DIKE NO. 2

DOWNSTREAM SLOPE STABILITY
MAXIMUM SURCHARGE POOL-CASE VI
STAURATED EMBANKMENT

FIGURE 6

EXHIBIT NO



ASSUME A RIGID EMBANKMENT

AREA $A_1 = 5 \times 23 = 115.0\ ft^2$

$A_2 = 1/2 \times 69 \times 23 = \underline{793.5}\ ft^2$

Total $908.5\ ft^2$

WEIGHT OF DAM

$W_T = A_T\ \gamma_m$

$= 908.5 \times 130$

$= 118100$ P/lin ft

$W_T = 118.1$ kips/lin ft

PRESSURE AT FOUNDATION

$q = W_T$/EMB. LENGTH

$= 118.1/74$

$q = 1.596$ KSF

FOUNDATION STRENGTH

$q_d = 5.5\ c$

$= 5.5\ (0.5)$

$= 2.75$ KSF

ASSUME AVERAGE COHESION FOR THE PEAT AND CLAY - c = 0.5 KSF

FACTOR OF SAFETY

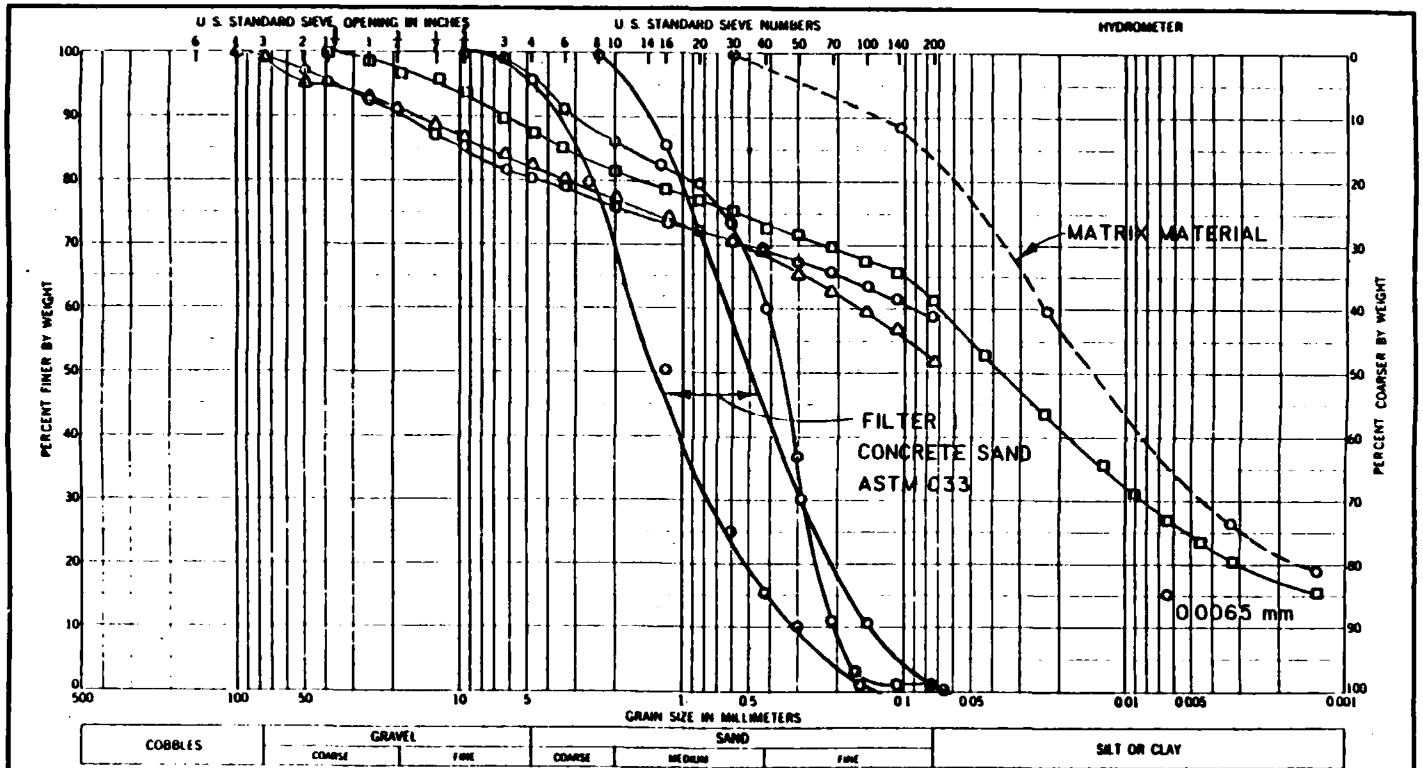
$FS = q_d/q$

$= 2.75/1.596$

$= 1.72$

PRESSURE BENEATH A RIGID STRUCTURE DECREASES WITH DEPTH, THEREFORE A MINIMUM FS IS CALCULATED USING THE FULL EMBANKMENT PRESSURE

Figure 7. Bearing capacity analysis



Sample No	Elev or Depth	Classification	Nat w %	LL	PL	PI
Borrow Composite 1 = △		Gravelly Sandy Clay (CL) Brown		32	16	16
Borrow Composite 2 = ○		Gravelly Sandy Clay (CL) Brown		29	16	13
Old Dike Composite = □		Gravelly Sandy Clay (CL) Brown		29	15	14
PULLOUT TEST SAND = ○		Sand (SP) V. Light Tan; trace of Gravel**				
Borrow Composite 2: G_S=2.76, G_A=2.76, G_B=2.35						

Project
Area
Boring No
Date

GRADATION CURVES

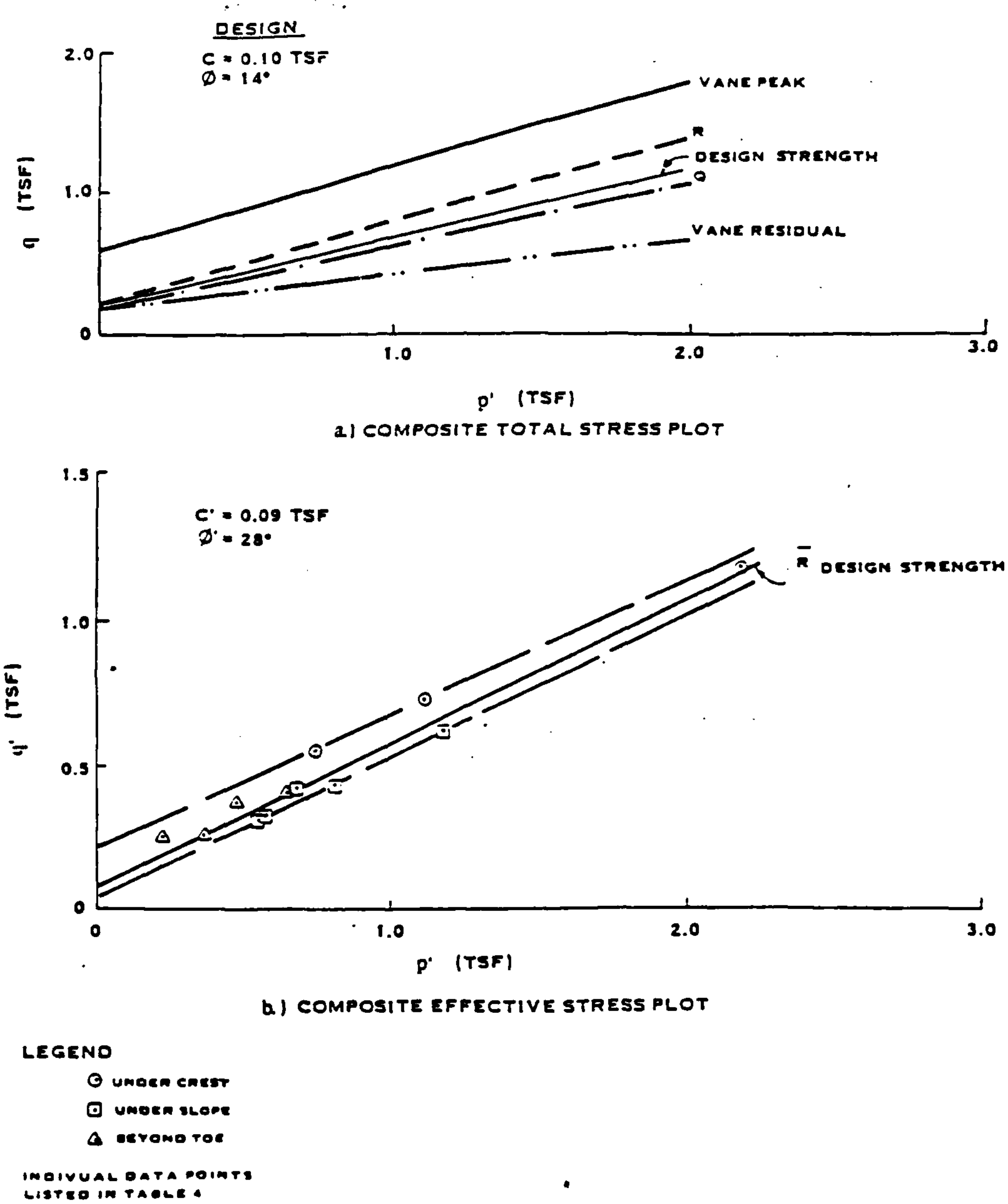
ENG FORM 1 MAY 63 2087

**RELATIVE DENSITY

γ_D = 108.9 max

γ_D = 100.1 min

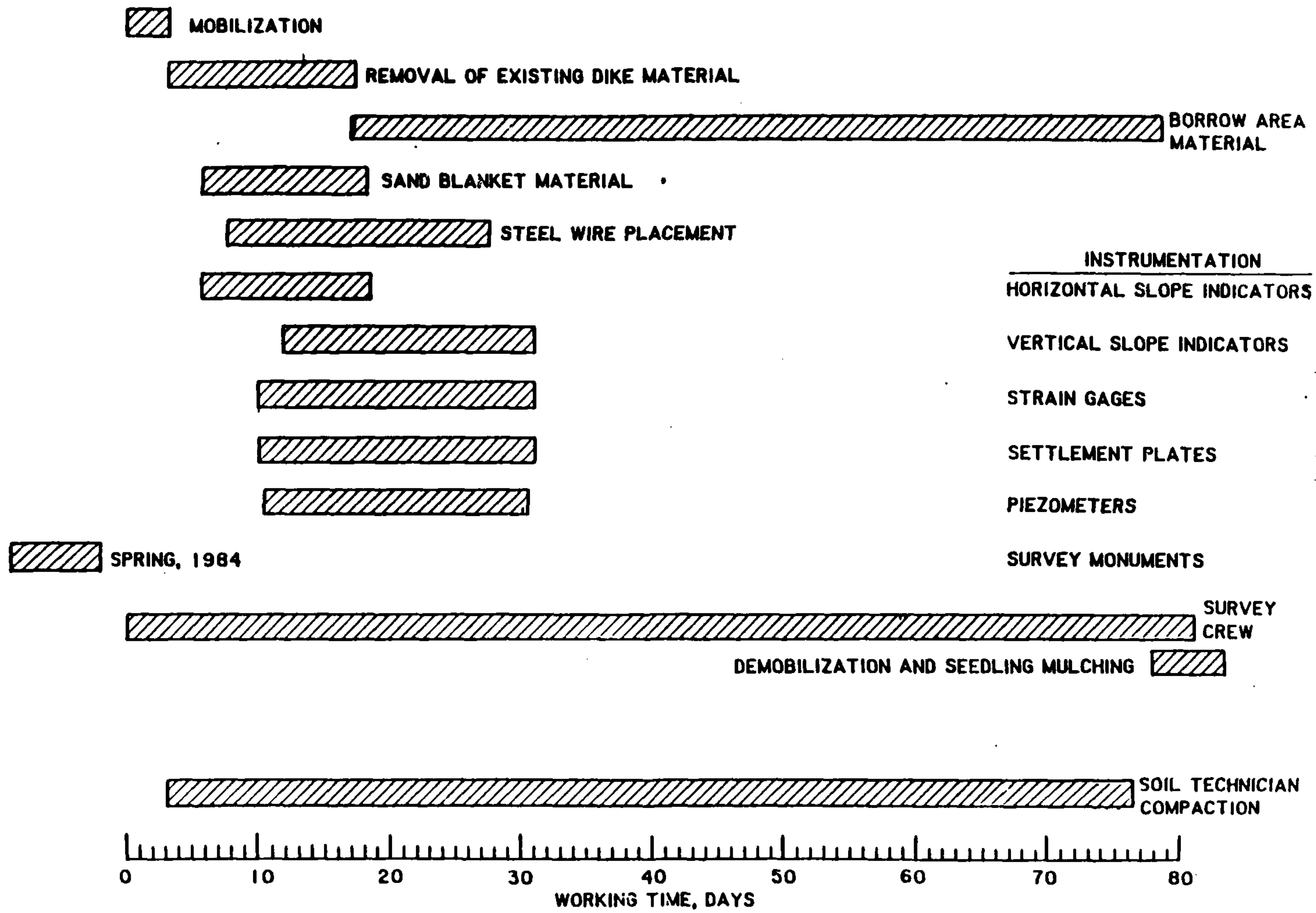
Figure 8.



U.S. ARMY CORPS OF ENGINEERS HUNTINGTON DISTRICT HUNTINGTON WEST VIRGINIA	MUSKINGUM RIVER WATERSHED OHIO MOHICANVILLE DAM DIKE NO. 2
LAW ENGINEERING TESTING COMPANY MARIETTA, GEORGIA	STRENGTH OF FOUNDATION CLAY Figure 9

EXHIBIT NO

Figure 9.



CONSTRUCTION BAR CHART

Figure 10.

APPENDIX A: LABORATORY TESTS

Embankment Characterization Tests

1. Figure A1 presents gradation, Atterberg limits, specific gravity, and relative density data for the three clays and one sand used in the laboratory testing program. All tests were performed in accordance with EM 1110-2-1906.* The sample designated Old Dike Composite was taken from the existing dike, and the two borrow composite samples were taken from the proposed borrow areas. Except for the fraction retained on the 1-1/2-in. sieve for the two borrow samples, the three samples are nearly identical in gradation and Atterberg limits. Thus, the three clay (embankment) materials were used interchangeably for testing. Figure A2 presents Standard Compaction data on one of the clays, Borrow Composite 1.

Soil Corrosiveness Tests

Procedure and results

2. Results of the soil and water chemistry and resistivity tests to evaluate soil corrosiveness potential are presented in Table A-1. Tests on the saturated paste extract and the 1:1 soil-water paste were conducted by procedures given in Black.** Tests performed on the 1:9 soil-water slurry were conducted by procedures given in Standard Methods. The pH from a 1:1 soil-water ratio are the soil pH values referred to in the discussion of results.

3. The soil resistivity test was performed on a sample of borrow soil from which the material retained on the 3/4-in. sieve had been removed. The specimen was compacted in a rigid walled test chamber having an inside diameter of 4.00 in. Specimen height was 7.00 in. The specimen was saturated

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* Headquarters, Department of the Army. 1970. Laboratory Soils Testing, Engineer Manual EM 1110-2-1906, Washington, D. C.

** Black, C. A., Ed. in Chief. 1975. Methods of Soil Analysis, No. 9 in the Series Agronomy, American Society of Agronomy, Madison, Wisc.

American Public Health Association. 1980. Standard Methods of Examination of Water and Wastewater, Publication Office, 15th Edition, Washington, D.C.

using tap water and back-pressured to assure 100 percent saturation. Specimen after saturation are given in Table A-1. Resistivity measurements were taken until the resistivity became constant and remained constant overnight.

Discussion of results

4. Several experts in the field of corrosion were contacted and several references consulted to evaluate the potential corrosiveness of the soil at the project site. Test results were sent to Dr. Ashok Kumar at the U. S. Army Construction Engineering Research Laboratory for evaluation, and in telephone conversation with Dr. Kumar he stated that the resistivity and pH values indicated that the soil should be considered corrosive. He recommended using the thicker of the two meshes being considered at that time (0.329 in. diameter longitudinal wire size) and estimated that perforation of the metal reinforcement would take place in 20 years. He said, however, that predictions of metal loss due to corrosion were very poor.

5. Army Technical Manual TM 5-811-4* was consulted as well as a recent report on soil corrosiveness by King.** The report by King presents a nomogram for predicting the pitting rate of steel for a given soil resistivity and pH. Assuming that pitting takes place simultaneously from opposite sides of the wire, and using a resistivity of 27 ohm-meters, and a pH of 6.5, the nomogram predicts that perforation of 0.391 in diam. (D12) wire would take place in 27 years. This prediction has a 30 percent likelihood of underestimating the rate of corrosion, based on comparisons made by King of field data with nomograph predictions. King also reported that in a study of corrosion over a 15 year period the highest value for rate of corrosion was found to be 0.1 mm per year for general corrosion. This would indicate that the reinforcement being proposed would last well over 10 years if general corrosion were the mode of deterioration.

6. General conclusions from these reports are given as follows:

a. Good compaction tends to reduce the corrosion hazard by reducing the porosity of the soil. This increases resistivity and reduces oxygen diffusion.

* Headquarters, Department of the Army. 1962. Electrical Design: Corrosion Control, TM 5-811-4, Washington, D. C.

** King, R. A. 1977. "A Review of Soil Corrosiveness with Particular Reference to Reinforced Earth," Supplementary Report 316, Transport and Road Research Laboratory, Crowthorne, Berkshire, England.

b. Applying fertilizer to the embankment, to promote grass growth, for example, may greatly increase salt concentrations and corrosiveness of the soil.

c. All organic matter should be kept away from the area of the reinforcement.

d. Interconnecting rolls of reinforcement may increase corrosion due to non-homogeneities in soil from one part of embankment to the other causing long line corrosion.

e. A transition from a saturated soil of low oxygen content to an unsaturated soil of high oxygen content may accelerate corrosion. The transition between the embankment soil and the sand drainage blanket could accelerate corrosion.

f. Carbonate ions tend to reduce corrosion rate by forming an adherent scale on the metal surface. High calcium carbonate concentrations in the soil surrounding the wire mesh may thus result in a lower corrosion rate.

Strength and Consolidation Tests

7. Results of soil strength and consolidation tests performed at WES are shown in Figures A3 through A6. Results of tests conducted by Law Engineering Testing Company (LETCO) and used by WES in the design analysis are shown in Figures A7 through A11.

8. The Q and R tests performed at WES were conducted in accordance with EM 1110-2-1906.

9. The void ratio-pressure curve for embankment soil shown in Figure A6 was obtained on a 2.8-in.-diam triaxial specimen which had been back-pressure saturated and then consolidated isotropically in increments. The void ratio at the end of each consolidation increment was calculated from the volume change during the preceding increments and the initial specimen volume.

Pullout Tests

Apparatus

10. A large direct shear apparatus, normally used to perform direct shear tests on aggregate mixtures, was modified to perform pullout tests on the wire mesh. The apparatus consists of a 2 piece shear box having 24 in. square inside dimensions with the shear plan located at the mid-height.

Normally, the upper half of the shear box is pulled relative to the stationary lower half. However, for this testing program, both the upper and lower halves of the box were held stationary, and the wire mesh specimen was pulled in the plane between the upper and lower box. Figure A12 shows a schematic of the apparatus. The thickness of the soil specimen below the wire mesh was approximately 3.50 inches and the thickness of soil above the surface was approximately 3.90 inches including the depth of soil in which the wire mesh specimen was embedded. Three quarter inch thick plywood strips separated the upper half of the shear box from the lower half, and provided a space through which the longitudinal reinforcement rods passed. Three quarter inch thick plywood blocks were also placed in the spaces between the longitudinal reinforcement wires to minimize the amount of soil that was displaced out of the box as the wire was pulled. A pan surrounding the shear box could be filled with water to inundate the test specimen.

11. The pullout force was generated by two parallel 30-ton capacity screw jacks driven through a continuously variable transmission with constant speed motor, or for the higher rates a variable speed motor and reduction gearing. The total range of pullout rate achievable was 0 to .250 in. per minute. The shear loads were measured by two 50,000 lb capacity electronic load cells, one mounted on each screw jack. Load cell output signals were electrically balanced and summed to indicate total pullout load applied to wire mesh. Maximum pullout load error was 0.02 tsf.

12. The normal force was applied with a 14-in. diameter steel flat jack having a one inch range of movement. The jack was pressurized from a bottled gas cylinder. Pressure to the flat jack was controlled by a non-bleeding, non-relieving pressure regulator which was manually bled to control pressure overshoot. The gas pressure applied to the flat jack was monitored throughout the test using a bourdon tube pressure gage. Pressure remained essentially constant throughout the test although slight adjustments were made from time to time. The vertical force generated by the flat jack was transmitted to the specimen through an adjustable steel spacer cylinder, and a one inch thick, buttressed aluminum pressure plate.

13. Four, .001 in. per division, dial gages were mounted on the pressure plate at each corner to measure vertical deformations. Dial gages were monitored during the test to evaluate the degree of specimen consolidation

or swell. Horizontal displacement of the wire mesh during pullout was measured with a rotary potentiometer and verified with a horizontally mounted dial gage. Maximum displacement measurement error was less than 0.015 in. Outputs from the potentiometer and the load cells were fed to an x-y type recorder. Figure A13 is a photo of the assembled apparatus.

14. For all the tests, the wire mesh longitudinal wires (wires parallel to the direction of pull) were 30-1/2 in. long. For the test on embankment soil at 1.6 tsf normal pressure, the transverse wires (wires perpendicular to the direction of pull) were 18 in. long, whereas on all the other tests, the transverse wires were 20 in. long. The diameters of both the longitudinal and transverse wires are given in Table A-2. The wires were welded at the intersections between the longitudinal and transverse wires, and each longitudinal wire was welded at one end to a bar which was attached by pins to the pulling mechanism. The wire mesh was positioned at the start of the test in such a way that the mesh could be pulled about 4 in. before a cross wire would touch the side of the shear box. In addition, the arrangement of the mesh was such that as wire was pulled out of the box during the test, the same length of wire was being pulled in. Thus the same area of wire remained in contact with soil throughout the test. Figure A14 shows the positioning of the wire mesh in the shear box. Figure A15 is a photograph of the partially assembled apparatus with the wire mesh in place.

Procedure

15. To minimize the amount of soil required for each test, the thickness of soil in the upper and lower boxes was limited to about 3.5 inches. While the embankment material has particles up to 4 inches maximum, the maximum particle size of the test soil was limited to 1-1/2 inches to minimize the possibility of particles contacting the top or bottom of the shear box and the wire mesh at the same time and causing nonuniform load distribution against the wire. This required removal of about 5 percent of the total material.

16. The desired initial specimen condition fo· the test on embankment soil at 1.6 tsf normal load was 95 percent of standard compaction maximum dry density, and 1 percent wet of optimum water content. For the tests at 0.6 tsf and 0.2 tsf normal load, the desired water content was changed to 2 percent wet of optimum water content. Since the compaction test on embankment soil was performed on material from which the +3/4 inch particles had

been removed, the density and water content was corrected to allow for the presence of material passing the 1-1/2 inch sieve and retained on the 3/4 inch sieve. Maximum dry density from the compaction curve is 116.7 pcf and optimum water content is 14.0 percent. Corrected for the presence of 4 percent material passing the 1-1/2 inch sieve and retained on the 3/4 inch sieve, the desired test conditions were:

$_d$ = 112.2 pcf for 95 percent of maximum dry density

w = 14.6 percent for 1 percent wet of optimum water content

w = 15.6 percent for 2 percent wet of optimum water content

17. For the embankment material, quantities of air-dry soil sufficient to fill the lower and upper parts of the shear box at the desired density were weighed out and sufficient water added and mixed with the soil to reach the desired water content. The soil was then allowed to cure for a minimum of two days in sealed containers.

18. After curing, soil was compacted to the desired level in the lower half of the shear box using a hand compaction rammer and by applying rammer blows to a 1-ft square plastic plate resting on the soil. After the soil had been compacted in the lower half of the shear box to the desired density and height, the wire mesh was placed on the soil surface, the top half of the shear box was bolted into position and the upper layer of soil compacted in the box to the desired density and height. Assembly of the apparatus was then completed and the normal load applied. Dial gage readings were taken to monitor the extent of specimen vertical movement prior to shear and the values used in calculating the specimen dry density at start of shear.

19. For the test performed on embankment soil at 1.6 tsf normal load, the soil specimen was inundated and allowed to soak for 2-1/2 days before the start of shear. To facilitate the uptake of water to the specimen in this test, layers of thick, felt-like filter fabric were placed between the soil specimen and the bottom and top bearing surfaces of the shear box.

20. For the tests at 0.2 tsf and 0.6 tsf normal load, which were not inundated, pull was begun about 1/2 hr after the application of normal load.

21. For the test on sand, the initial dry density was 107.0 pcf, which for the sand tested was a relative density of 48 percent. In this test oven dried sand sufficient to produce the desired density was compacted in the shear box in the same manner as the embankment soil. After compaction, the

sand specimen was inundated and allowed to consolidate for about 1/2 hr before the start of pull.

Results

22. Test results for the four pullout tests are presented in Table A-2 and in Figures A16 and A17. Figures A18 and A19 show the condition of the pullout zone after the test on embankment soil at 1.6 tsf. In evaluating the results on embankment soil it should be noted that there were substantial differences in test conditions between the tests performed at 1.6 tsf and the tests at 0.2 and 0.6 tsf normal load. The diameter and type of wire used for the test at 1.6 tsf was different from the wire used for the other tests. Finally, the rate of pull for the test at 1.6 tsf was considerably greater than for the other tests on embankment soil. Due to an erroneous initial rate setting, a rate of .157 hr/in. was used for approximately 7 minutes and then reduced to the lower rate of .923 hr/in. However, even this rate was approximately 10 times faster than the rates of pull (shown in Table A-2) for the other tests on embankment soil. Also, the test at 1.6 tsf was inundated and allowed to soak prior to pull. Consequently the soil was at a significantly higher water content than for the other two tests.

23. The test on sand was performed at a faster rate than that used for the embankment soil since the faster rate was not considered to have a significant effect on results.

24. Vertical dial gage readings taken during shear indicated a slight consolidation took place which was less than .10 inch for all the tests.

Soil-Fabric Friction Tests

Apparatus

25. A standard 3-in.-sq direct shear apparatus of the type used in the WES soils laboratory was modified to perform the soil fabric friction tests. The modification consisted of replacing the lower half of the shear box with a solid aluminum platform with a clamping bracket at one end to hold a fabric specimen. A schematic of the apparatus is shown in Figure A20. The apparatus holds a fabric specimens 4 in. wide by 4.5 in. long. Shims were placed under the platform when necessary to keep the direction of pull parallel with the surface of the fabric for specimens of different thickness.

26. Normal loads were applied to the specimen by a pneumatic actuator controlled by a bleeding regulator and bourdon tube pressure gage. Shear loads were applied by a gear reduction drive and a single speed electric motor. The rate of pull was fixed at 3.0 inches per hour and the specimens were pulled for a distance of 1/2 inch.

27. Load and displacement measurements were made by electronic load cells and linear potentiometers inputting to a strip chart recorder. Load values were accurate to .02 tsf, and displacement values were accurate to 0.005 in.

Procedure

28. Since the maximum thickness of soil specimen that could be accommodated in the test apparatus was 1/2 inch, the soil used was embankment material from which material retained on the No. 4 sieve had been removed. It was desired to prepare specimens at 95 percent of maximum dry density of standard compaction for which compensation had been made for the removal of material between the 3/4 inch and No. 4 size. The desired density of the minus No. 4 material was calculated to be 107.1 pcf and the compensated water content at approximately 1 percent wet of optimum was 17.0 percent.

29. For each test, a fabric specimen was attached to the platform, and a soil specimen was compacted into the apparatus. Assembly of the apparatus was then completed and the normal pressure applied. The specimen was inundated and allowed to reach equilibrium overnight. The shear box containing the soil specimen was then pulled across the fabric surface.

Results

30. The results of the soil fabric friction tests are presented in Figure A21. Also included in these data is a direct shear test on soil only. The rate of two and five specimens of each fabric were tested and the results averaged to determine the fabric strength.

Results

31. Table A-3 summarizes the strength results and other pertinent data for the candidate reinforcement fabrics. Also included for comparison are data on steel mesh reinforcement and composite strip reinforcement. All fabrics except Stabilenka 200 and 400 were special order or experimental items and the suppliers stated that the properties could be varied to suit specific requirements. With the exception of the Stabilenka fabrics, none of the fabrics tested had strengths close to that of the manufacturers'

claim. The test strengths averaged about 80 percent of the manufacturer's predicted strengths. It should be noted that the strengths for these high-strength fabrics are normally predicted on the basis of yarn strength multiplied by the number of yarns per unit width and not on actual fabric tests. Shear for this test was the same as that for the soil-fabric friction tests. Consequently the results for the soil alone should be considered as representing an undrained or partially drained condition.

Fabric Tensile Strength Tests

Apparatus

32. An Instron Model 1116, 50,000 lb capacity testing machine was used to apply load to the test specimens. Fabric specimens were held with Instron Model G-61-11F webbing capstan grips, capable of holding specimens up to 2 inches wide. Elongations were measured by a device consisting of two spring loaded clasps placed on the fabric between the grips and fitted on each end. LVDT's having ±250 mil range measured the relative movement between the clasps. The outputs of the LVDT's were averaged and displayed on a digital voltmeter calibrated to indicate thousandths of an inch movement. Readings from the voltmeter were manually recorded on the load indication strip chart. The space between the clasps at the start of test was 3 inches. Figure A22 shows the assembled test setup.

Procedure

33. Fabric strips approximately 3 inches wide by 52 inches long were cut from the sample, and then reduced to a width of 2 inches by ravelling away the outer yarns on each side. The outer yarns were protected from further ravelling by taping the edges of the fabric with low strength plastic tape, and the last 2 inches at each end of the specimens were expoxyed to eliminate any possibility of yarn slippage. Specimens were pulled at a rate of 1 in./minute, resulting in an elongation rate of about 2 percent per minute.

Table A-1

Mohicanville Soil and Water Chemistry and Resistivity Results

	Borrow Soil	Dike Soil	Ditch Water
		1:1 Soil-Water Ratio	
pH	6.7	7.0	6.1
		Saturated Paste Extract	
pH	8.0	8.0	
Resistance, ohm-cm	1395	2003	
Specific Conductance, mhos/cm	717	499	
		1:9 Soil-Water Ratio	
Resistance, ohm-cm	5208	7812	2003
Specific Conductance, mhos/cm	192	128	499
Sulfates	82 mg/kg	90 mg/kg	40
mg/1			
Chlorides	25.7 mg/kg	25.7 mg/kg	18
mg/1			
		Compacted Soil	
Soil Resistivity, ohm-meters	27.3		
Dry Density, pcf	112		
Degree of saturation, %	100		
Water Content, %	20.6		

Table A-2

Pullout Test Data Summary

Test No.	1	2	3	4
Soil Tested and Maximum Particle Size, inches	Embankment 1-1/2	Embankment 1-1/2	Embankment 1-1/2	Filter Blanket 3/8
Mesh spacing, long. cross, in.	2 6	2 6	2 6	2 6
Wire size, long. cross	W6 W4	D11.5 D4.5	D11.5 D4.5	D11.5 D4.5
Nominal wire diam., in.	.276 .226	.383 .239	.383 .239	.383 239
Embedded mesh area, ft^2	3.0	3.33	3.33	3.33
Soil dry density, as comp.	112.2	112.0	108.5	107.0
Water content, as comp.	15.0 (desired)	15.6 (desired)	15.6 (desired)	Dry
Soil dry density, start of test	117.1	114.1	109.2	108.3
Water content, after test	17.3*	14.5	15.2	Inundated
Normal pressure, tsf	1.6	0.6	0.2	0.6
Pullout resistance @ 0.1" pull, tsf**	.232	.216	.128	.523
Max. pullout resistance, tsf**	.517	.570	.442	.754
Displacement at max. resistance, in.	2.5	3.2	2.8	2.45
Maximum consolidation during shear, in.	.066	.078	.049	.103
Rate of pull, hr/in.	.157 - .923	5.1 - 10.5	8.9	0.95
Time of test, hr	3.1	29.8	31.6	3.3
Ratio of max. pullout load to normal load	.323	.950	2.209	1.256
Equivalent Ø, deg	17.9	43.5	65.6	51.5

* From small specimen taken in shear zone; may not be representative of entire specimen.

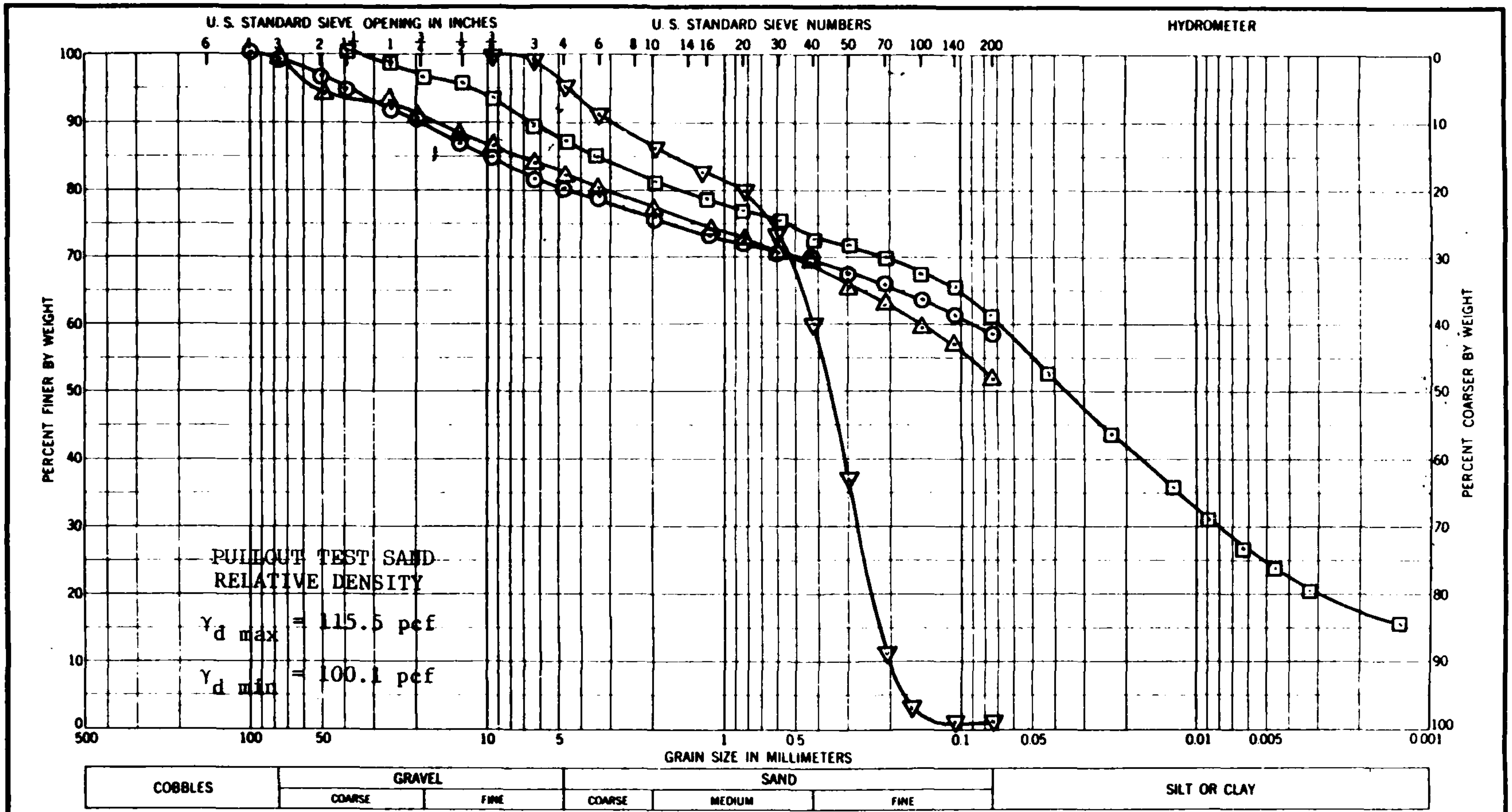
** Pullout load divided by embedded mesh area.

Table A-3

Properties of Candidate Reinforcement Fabrics

Type of Reinforcement	Material Description	Cost Per Sq. Yd., Dollars	Tensile Strength kips/ft	Elong. At Break %	Weight Per 160 ft Roll, lb	Roll Width, ft	Weight, Oz. Per Sq. Yard
Textile Fabric							
Stabilenka 400	Polyester	7.43*	27.6	11.2	434	16	24.4
Stabilenka 200	Polyester	3.81*	15.6	7.2	252	16	14.2
Firestone Type 1	Polyester	3.96	24.0	17.0	149	7	19.1
Firestone PA140WXF	Polyester	7.40	36.1	17.5	261	7	33.6
Owens Corning	Fiberglass	8.00**	29.3	2.7	300	9	30.0
Knytex Proform K29/1950	Kevlar 29	7.50	29.6	2.9	123	11	10.0
Burlington 59384	Kevlar 29	23.00	37.7	3.7	197	12	14.8
Steel							
2 6-W6 * W4	65 ksi yield	3.70	23.4+	0.22	1920		
2 6-D12 * D4.5	70 ksi yield	6.07	50.4+	0.22	3525		
Composite Reinforcement							
Paraweb Strips++	Polyester	7.63	33.0+	Not Available	1284	15	77.0
	Kevlar	15.25	33.0+	Not Available	Not Available	15	Not Available

* Estimate from Law Engineering Testing Co.
** A firm cost was not given for this fabric.
\+ Manufacturer's data.
++ Filament yarn encased in polythylene to make 3.5 in. 3/8 in. strips. Strips bonded to nonwoven fabric.



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
BORROW COMPOSITE 1	△	GRAVELLY SANDY CLAY (CL),BROWN		32	16	16
BORROW COMPOSITE 2	○	GRAVELLY SANDY CLAY (CL),BROWN		29	16	13
OLD DIKE COMPOSITE	□	GRAVELLY SANDY CLAY (CL),BROWN		29	15	14
PULLOUT TEST SAND	▽	SAND (SP), V. LIGHT TAN: TRACE OF GRAVEL				
BORROW COMPOSITE 2: G_s=2.74, G_a=2.74, G_b=2.35						

Project MOHICANVILLE DIKE NO. 2.

Area

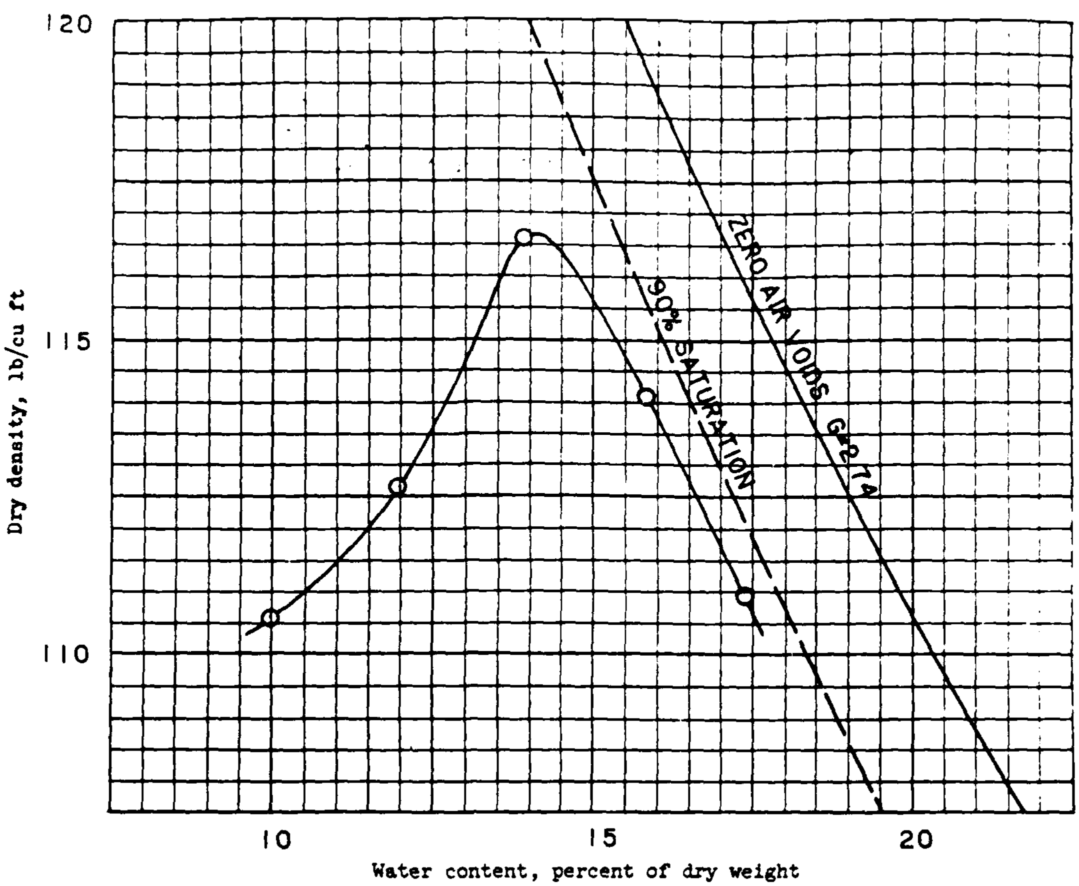
Boring No.

Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

Figure A1. Classification test results



STANDARD compaction test

56 blows per each of 3 layers, with 5.5 lb SLIDING WT

12 inch drop. 6 inch diameter mold

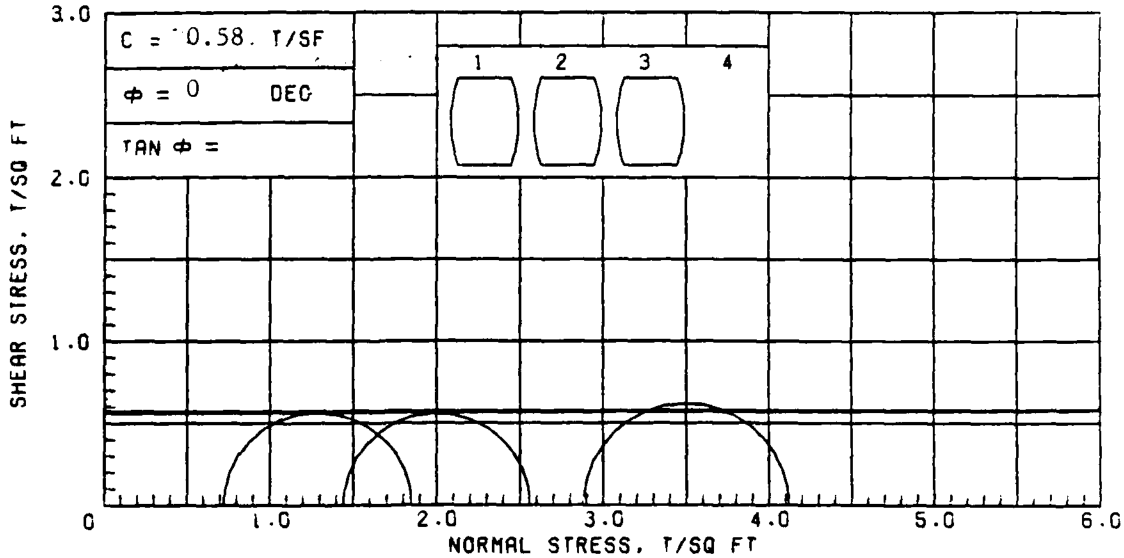
Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
		GRAVELLY SANDY CLAY (CL),	2.74	32	16	18	9
		BROWN	(COMPOSITE)				

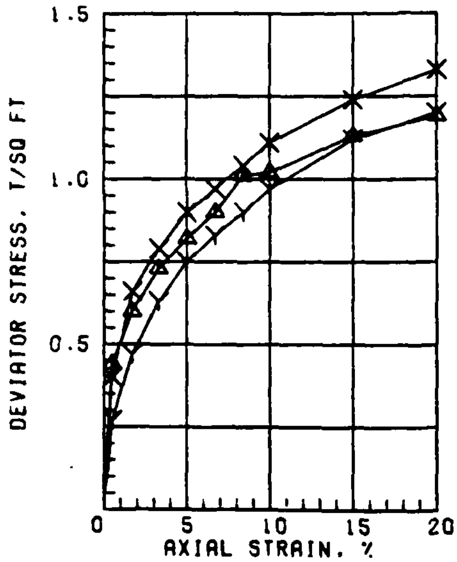
Sample No.				
Natural water content, percent				
Optimum water content, percent	14.0			
Max dry density, lb/cu ft	116.7	110.9 PCF=95% γ_{max}		

Remarks	Project MOHICANVILLE DIKE NO. 2
+3/4 IN. MATERIAL	
REMOVED	Area BORROW COMPOSITE NO. 1
	Boring No. — Date 21 DEC 1982
	COMPACTION TEST REPORT

ENG FORM 1 JUN 65 2091 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

Figure A2. Compaction test results





SPECIMEN NO.		△1	Y2	X3	4
INITIAL	WATER CONTENT, %	16.9	17.0	17.0	
	DRY DENSITY, PCF	110.8	110.5	110.8	
	SATURATION, %	85.2	85.1	85.6	
	VOID RATIO	0.544	0.547	0.544	
BEFORE SHEAR	WATER CONTENT, %				
	DRY DENSITY, PCF				
	SATURATION, %				
	VOID RATIO				
	BACK PRESS., TSF				
MIN PRIN. STRESS, TSF		0.7	1.4	2.9	
MAX. DEV. STRESS, TSF		1.13	1.12	1.24	
TIME TO FAILURE, MIN.		45	68	46	
RATE OF STRAIN INCR.,%					
INITIAL DIAMETER, IN.		2.86	2.86	2.86	
INITIAL HEIGHT, IN.		5.99	5.99	5.99	

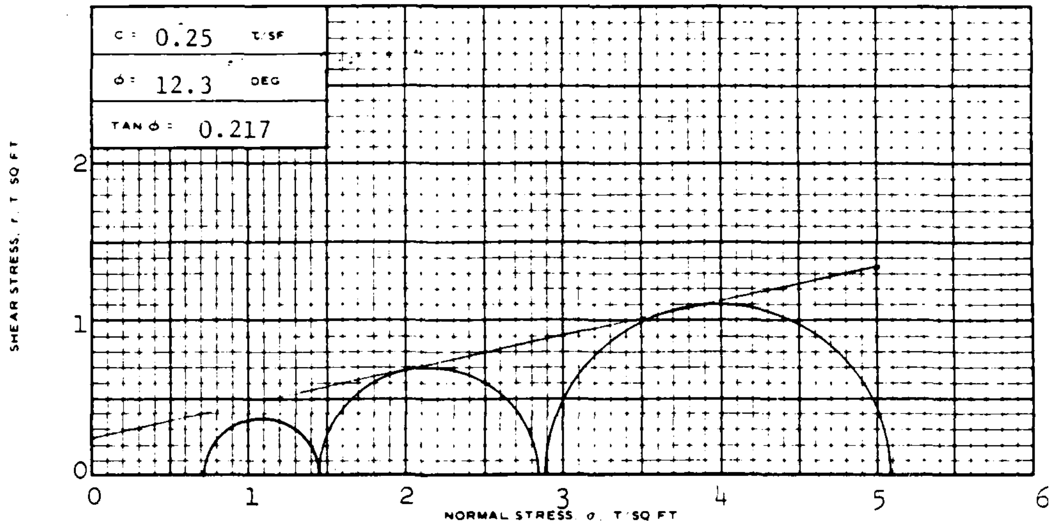
CONTROLLED-STRAIN TEST

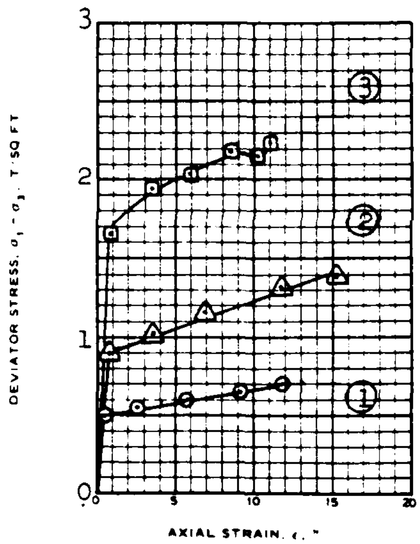
DESCRIPTION OF SPECIMENS: GRAVELLY SANDY CLAY (CL), BROWN

LL 32	PL 16	PI 16	GS 2.74	COMPACTED SPECIMEN	Q TEST

REMARKS:	PROJECT MOHICANVILLE DIKE NO. 2	
	BORROW COMPOSITE	
	BORING NO. -	SAMPLE NO.
	DEPTH/ELEV -	TECH. WJH
+ 1/2 in. MATERIAL REMOVED	LABORATORY USAE WES	DATE 06 APR 83
	TRIAXIAL COMPRESSION TEST REPORT	

Figure A3. Q test results on embankment soils





SPECIMEN NO.			M-1	M-2	M-3
INITIAL	WATER CONTENT, %	w_o	17.1	17.1	17.5
	DRY DENSITY LB/CU FT	γ_{d_o}	109.6	110.3	109.1
	SATURATION, %	S_o	83.5	85.2	84.5
	VOID RATIO	e_o	0.561	0.550	0.567
BEFORE SHEAR	WATER CONTENT, %	w_c	18.3	15.8	16.2
	DRY DENSITY LB/CU FT	γ_{d_c}	112.1	114.1	113.5
	B PARAMETER		0.99	0.99	0.99
	VOID RATIO	e_c	0.526	0.500	0.507
	FINAL BACK PRESSURE, T/SQ FT	u_o	3.60	3.60	3.60
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.72	1.44	2.88
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.71	1.40	2.19
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	90	77	691
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$			
INITIAL DIAMETER, IN.		D_o	2.86	2.87	2.87
INITIAL HEIGHT, IN.		H_o	5.98	5.97	5.96

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS GRAVELLY SANDY CLAY (CL), BROWN

LL 32 | PL 16 | PI 16 | Gs 2.74

TYPE OF SPECIMEN COMPACTED | TYPE OF TEST $\overline{R}$

REMARKS:

PROJECT MOHICANVILLE DIKE NO. 2

BORROW COMPOSITE

BORING NO. | SAMPLE NO.

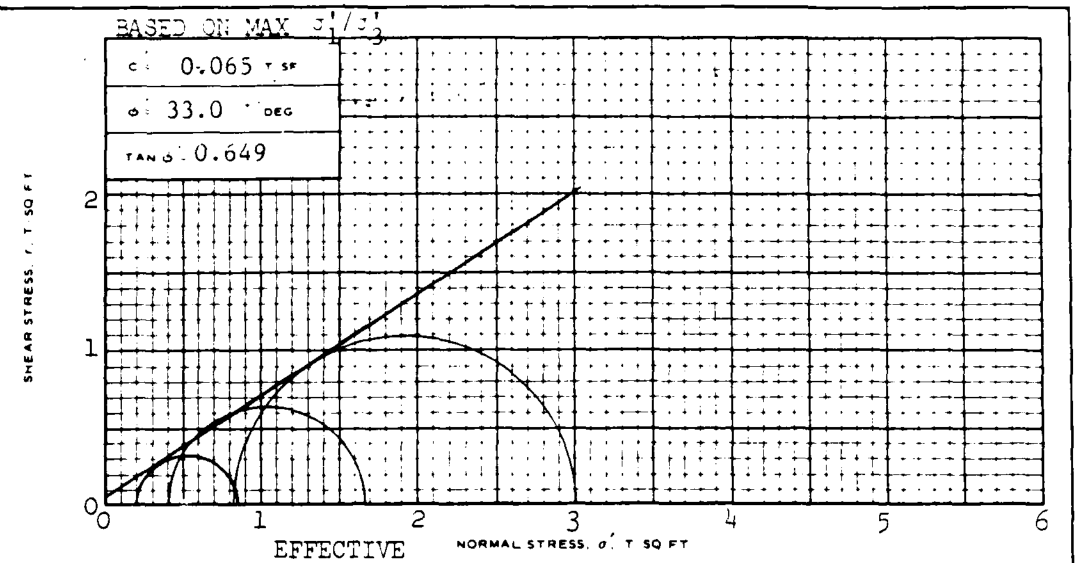
DEPTH/ELEV

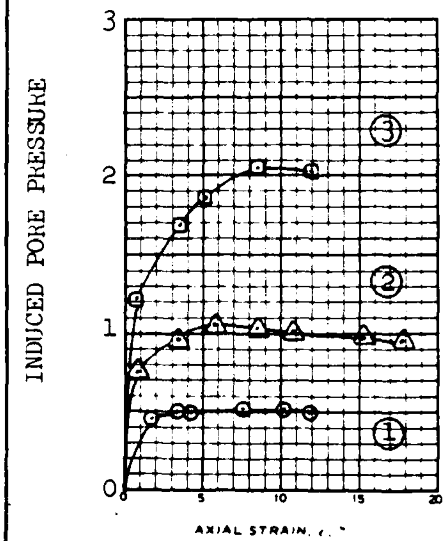
LABORATORY USAEWES | DATE 2 FEB 1983

+ 1/2 in. MATERIAL REMOVED

TRIAXIAL COMPRESSION TEST REPORT

Figure A4. $\overline{R}$ test results on embankment soil





			M-1	M-2	M-3	
SPECIMEN NO			M-1	M-2	M-3	
INITIAL	WATER CONTENT, %	w_o				
	DRY DENSITY LB/CU FT	γ_{d_o}				
	SATURATION, %	S_o				
	VOID RATIO	e_o				
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γ_{d_c}				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.20	0.40	0.83	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.66	1.27	2.19	
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f				
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$				
INITIAL DIAMETER, IN.		D_o				
INITIAL HEIGHT, IN.		H_o				

CONTROLLED-STRAIN TEST			
DESCRIPTION OF SPECIMENS			
LL	PL	PI	Gs
TYPE OF SPECIMEN		TYPE OF TEST $\overline{R}$	
REMARKS:	PROJECT MOHICANVILLE DIKE NO. 2		
	BORROW COMPOSITE		
	BORING NO	SAMPLE NO	
	DEPTH ELEV		
	LABORATORY USAEWES	DATE 2 FEB 1983	
	TRIAXIAL COMPRESSION TEST REPORT		

Figure A5. $\overline{R}$ test effective stresses

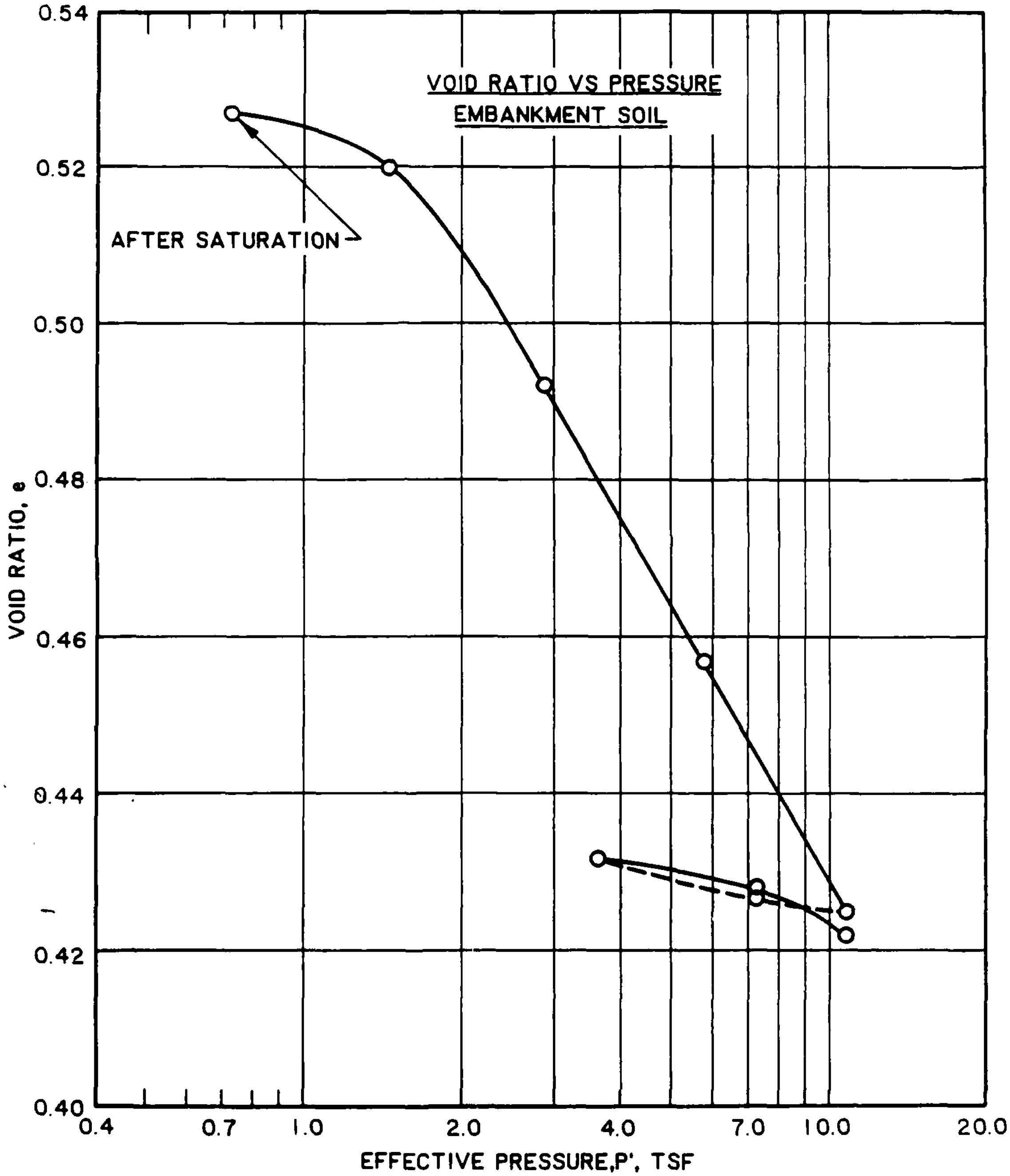
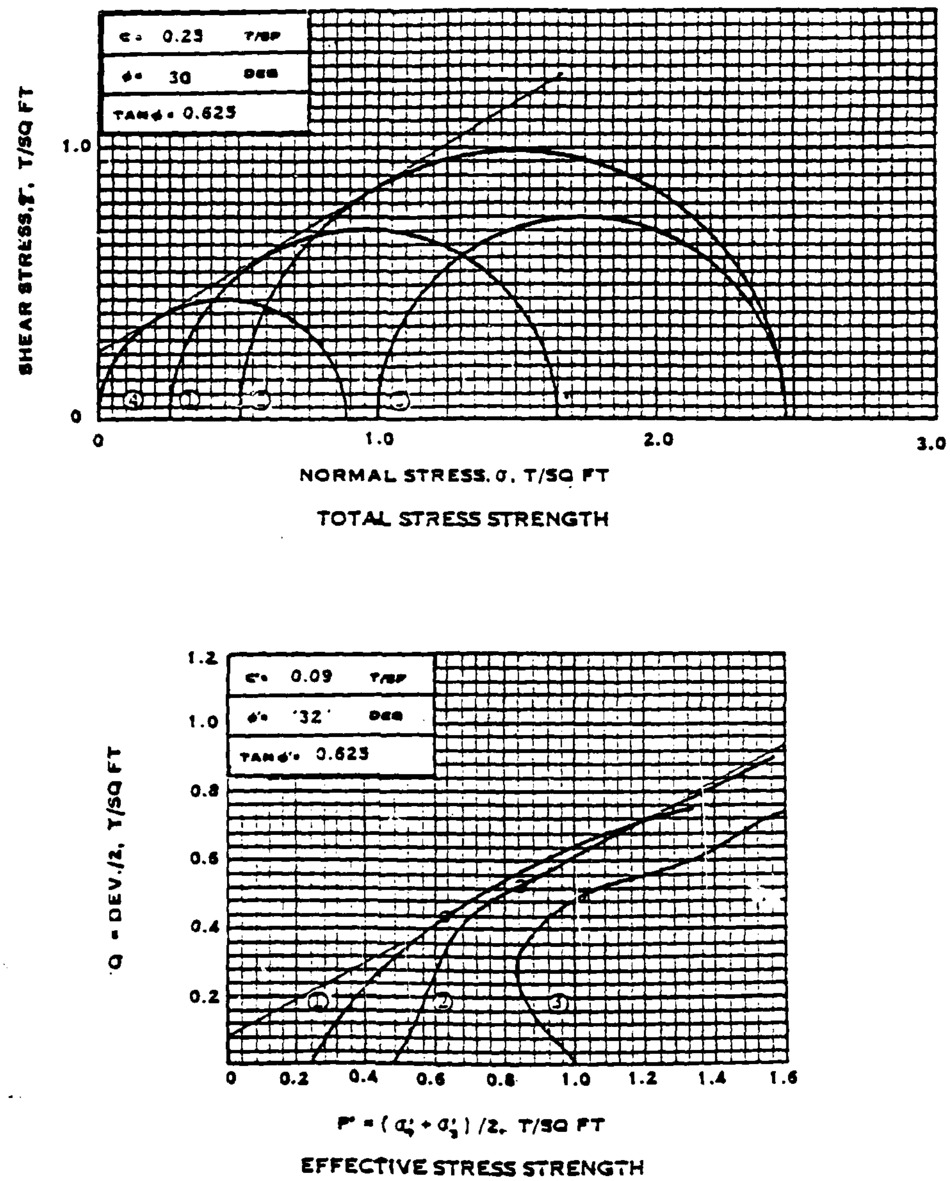


Figure A6. Consolidation test results on embankment soil



TEST 4 IS UU
OTHER TESTS ARE $\overline{R}$

U.S. ARMY CORPS OF ENGINEERS HUNTINGTON DISTRICT HUNTINGTON WEST VIRGINIA	MUSKINGUM RIVER WATERSHED OHIO MOHICANVILLE DAM DIKE NO. 2
LAW ENGINEERING TESTING COMPANY MARIETTA, GEORGIA	STRENGTH SUMMARY EMBANKMENT FILL

Figure A7. Law Engineering strength summary on embankment

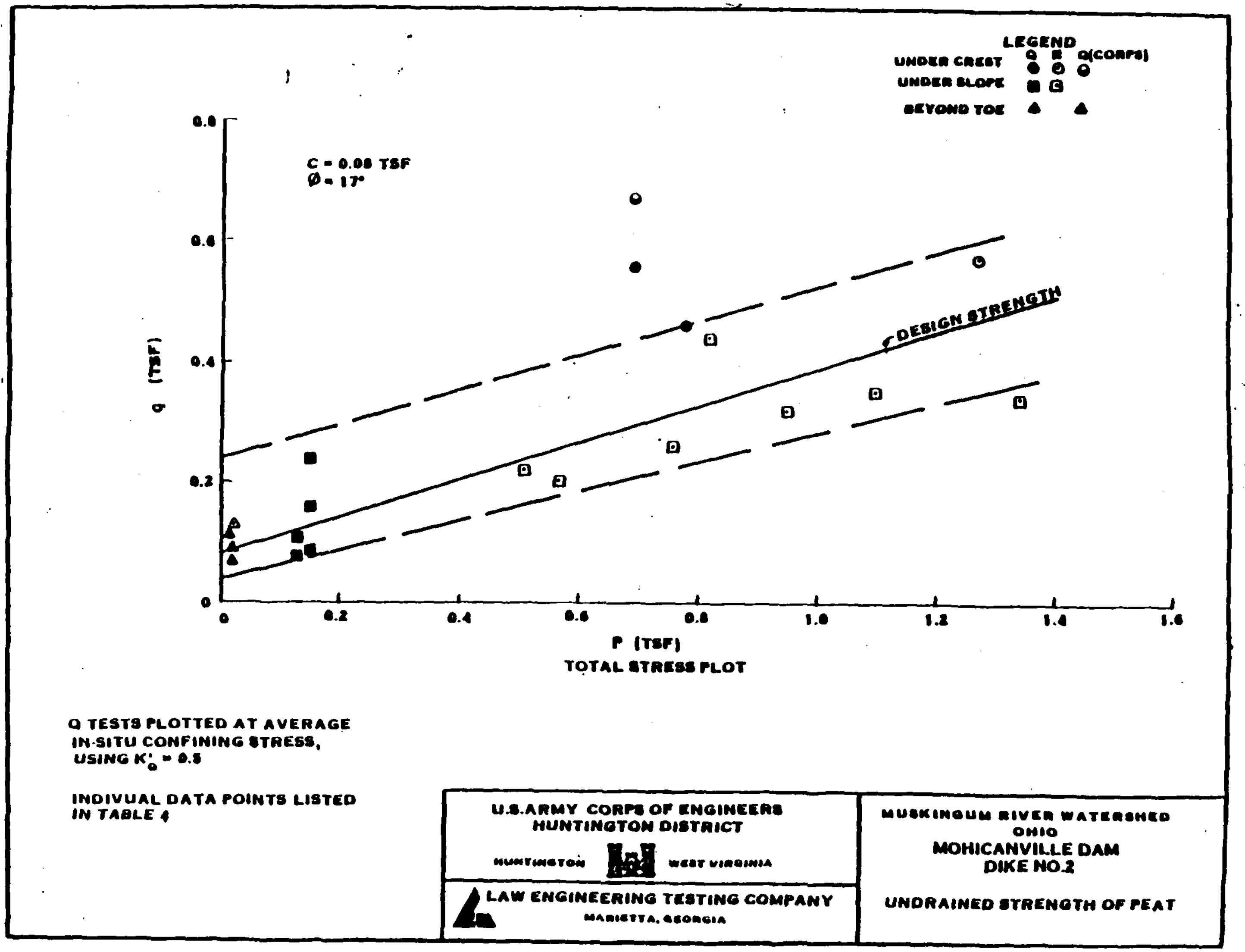


Figure A8. Law Engineering undrained strength of peat

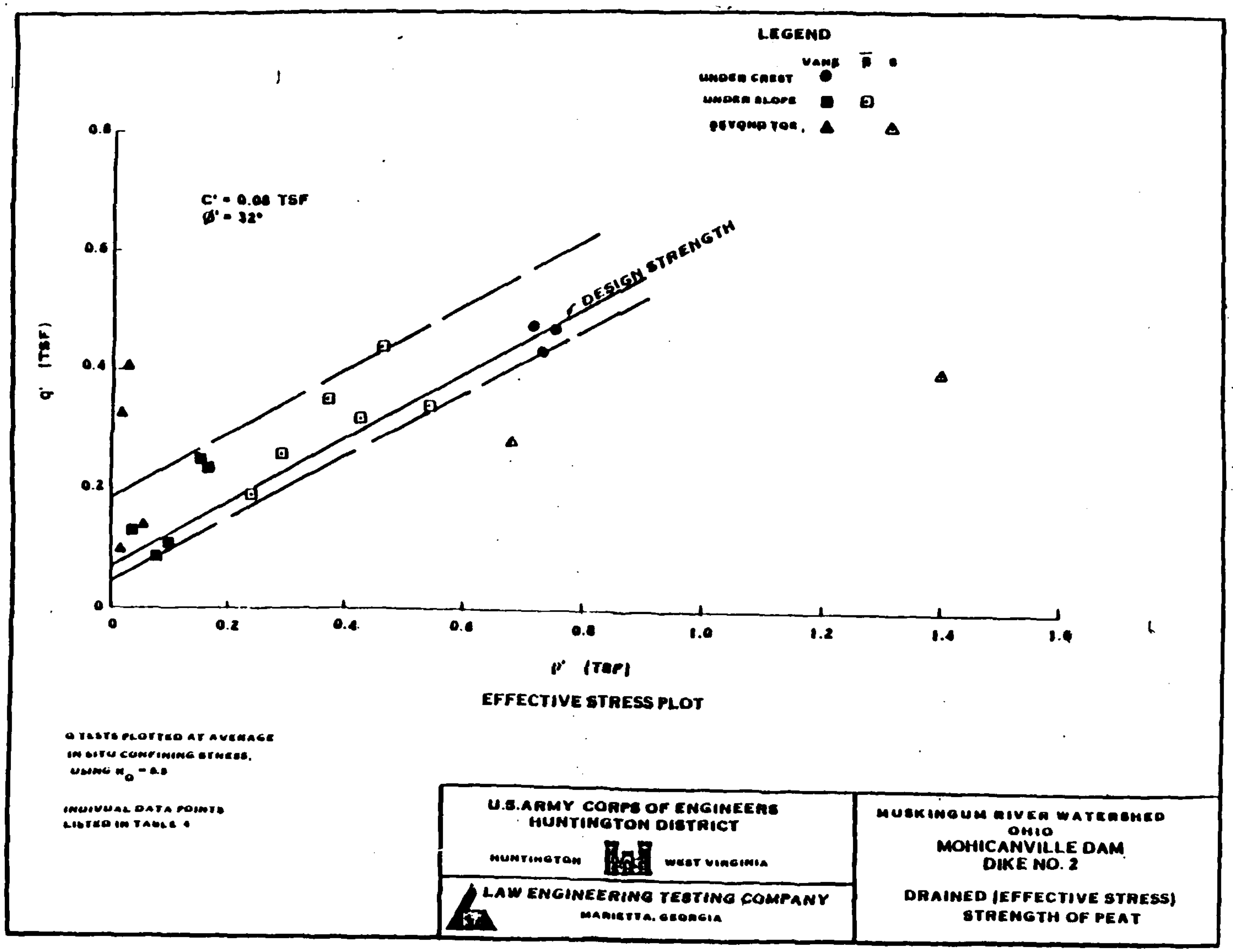


Figure A9. Law Engineering drained strengths of peat

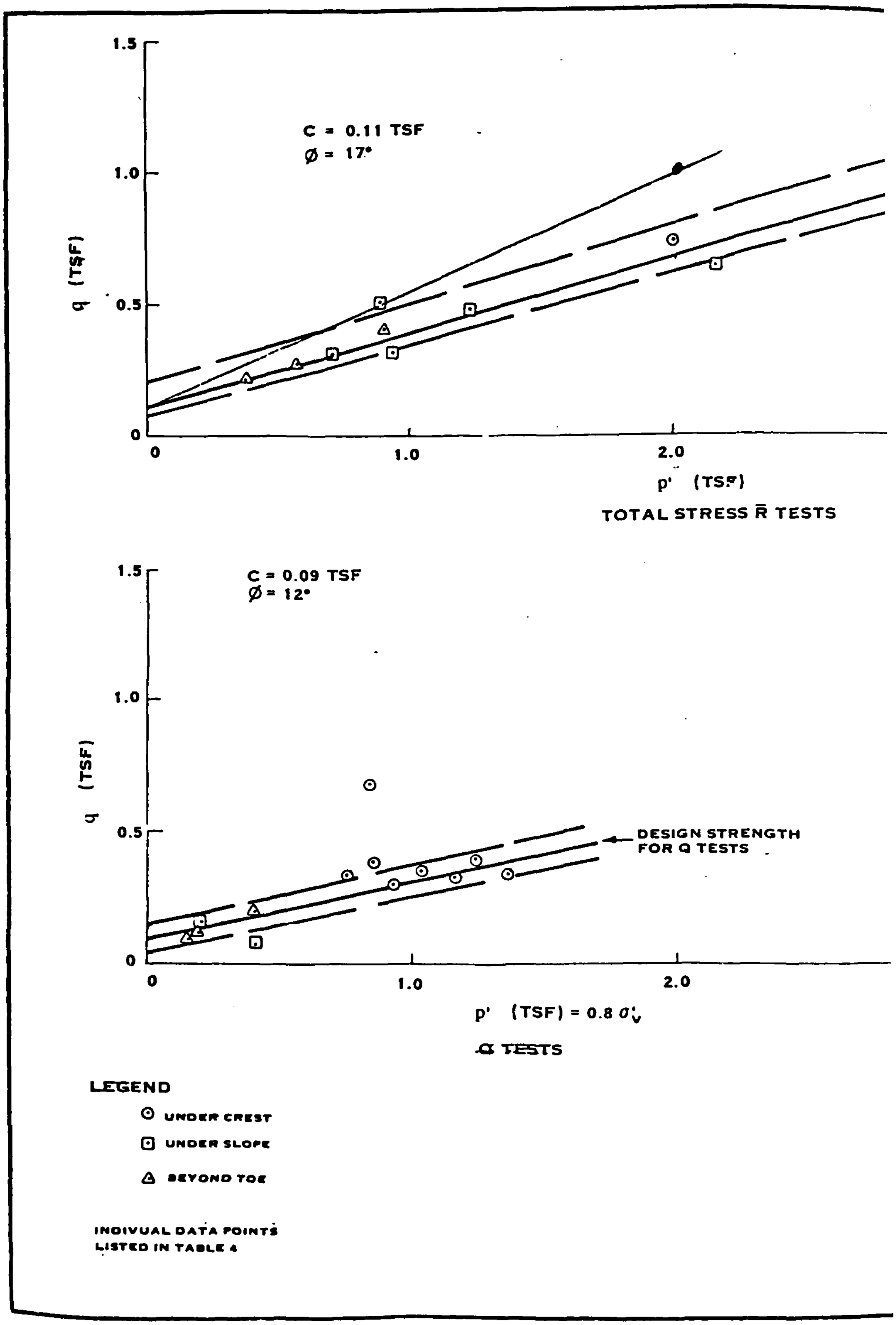
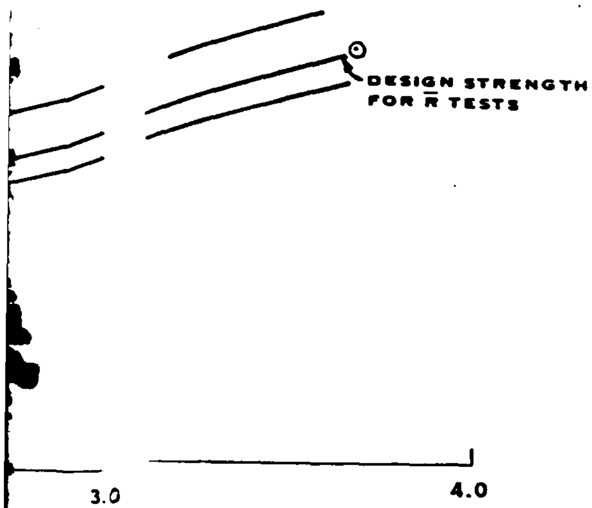
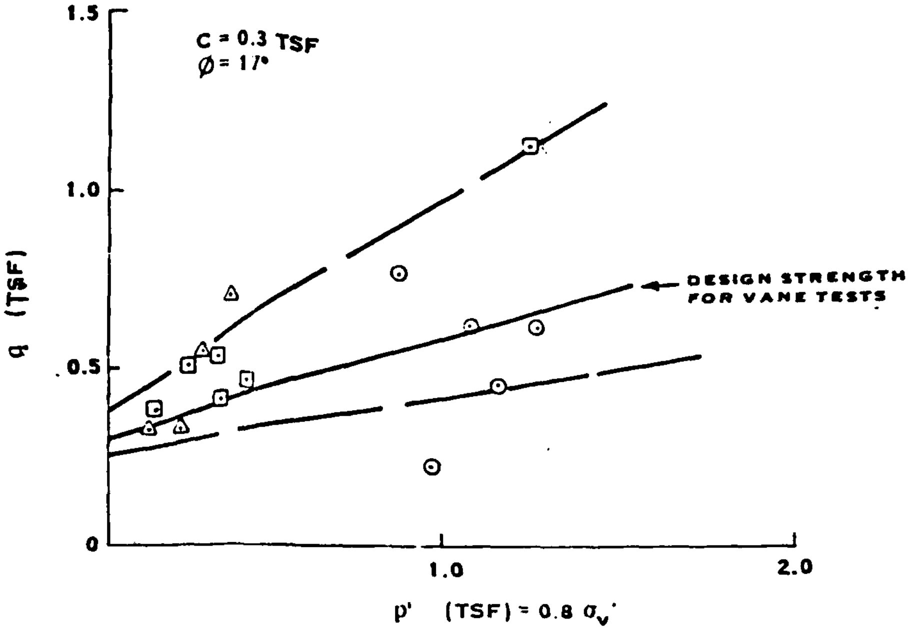
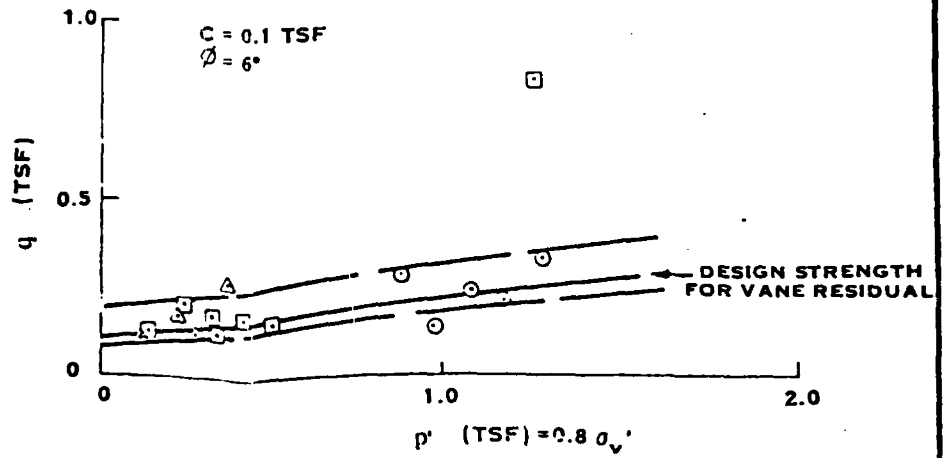


Figure A10. Law Engineering summary of undrained strength data on foundation cla





PEAK VANE SHEAR STRENGTH



RESIDUAL VANE SHEAR STRENGTH

U.S. ARMY CORPS OF ENGINEERS HUNTINGTON DISTRICT HUNTINGTON WEST VIRGINIA LAW ENGINEERING TESTING COMPANY MARIETTA, GEORGIA	MUSKINGUM RIVER WATERSHED OHIO MOHICANVILLE DAM DIKE NO. 2 UNDRAINED STRENGTH OF FOUNDATION CLAY

ay

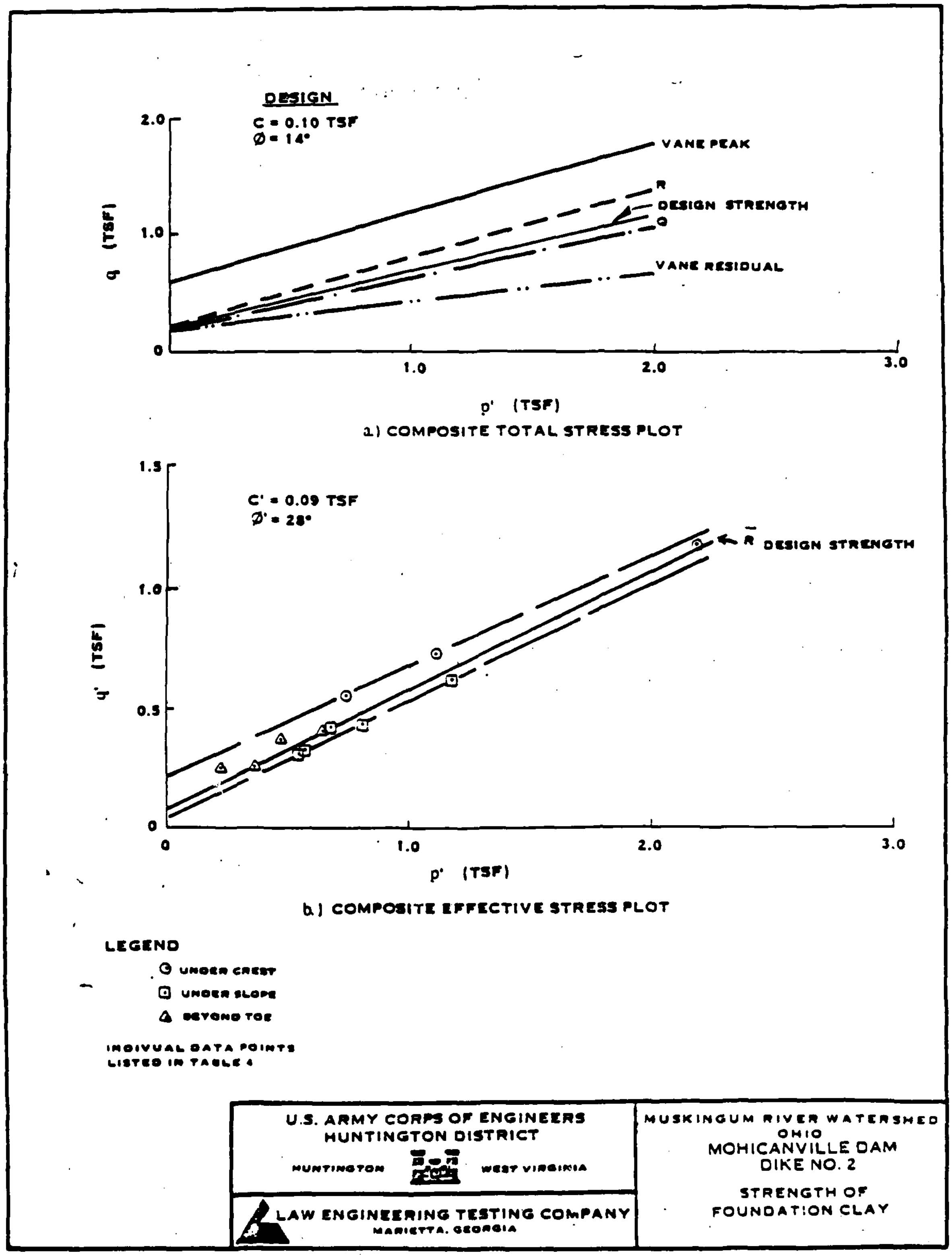


Figure A11. Law Engineering summary of strength of foundation clay

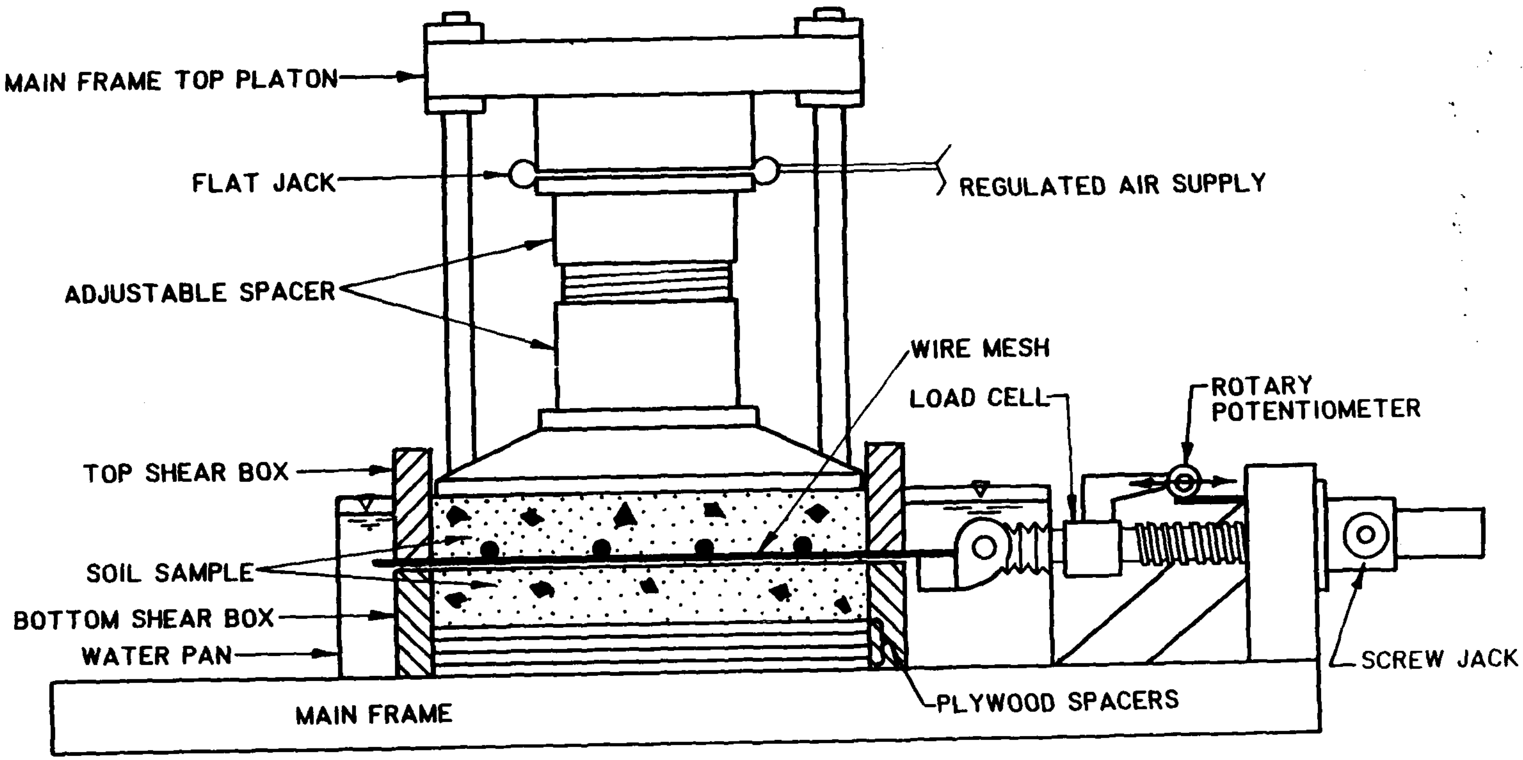


Figure A12. Pullout test apparatus

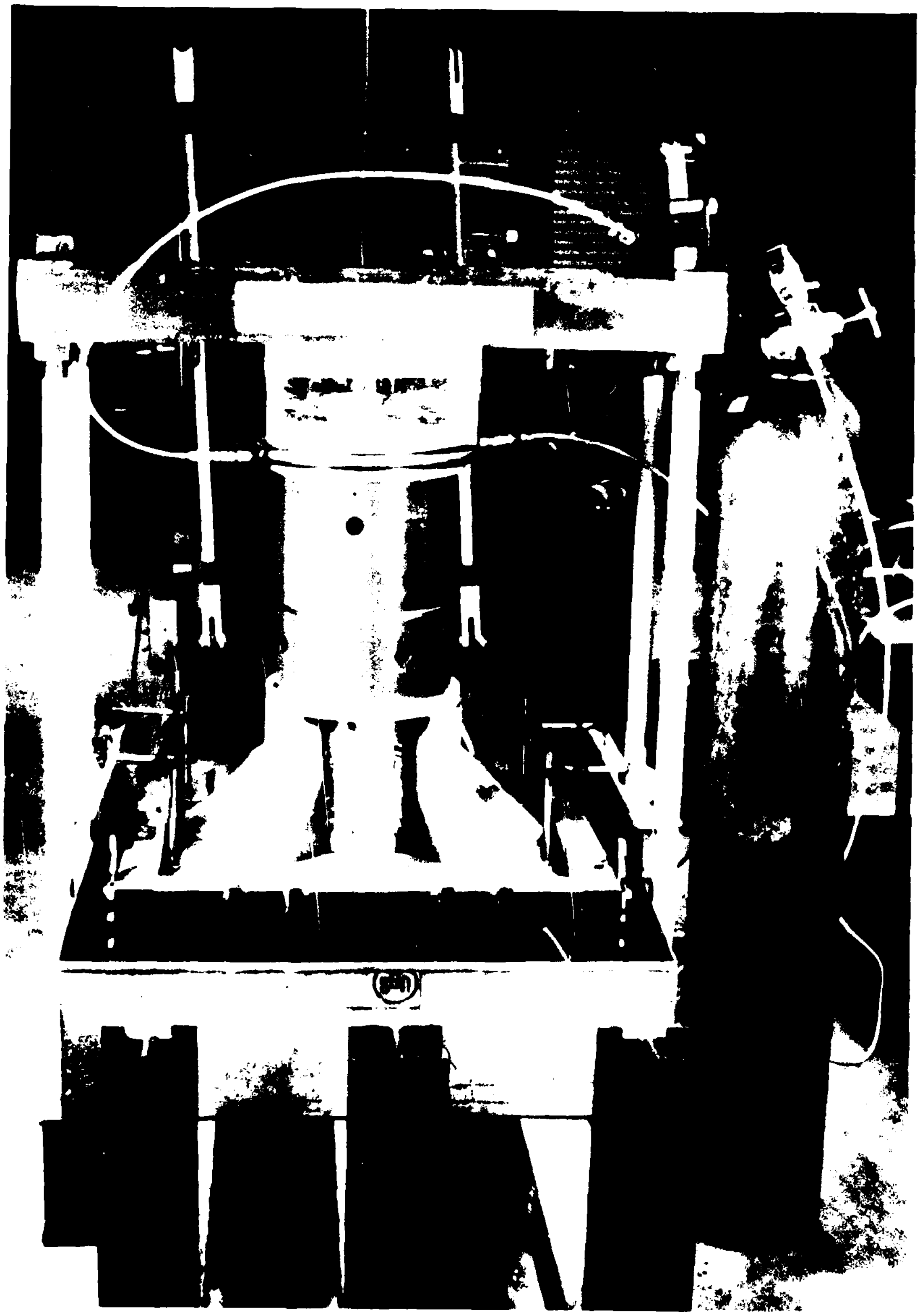


Figure A13. Assembled pullout test apparatus

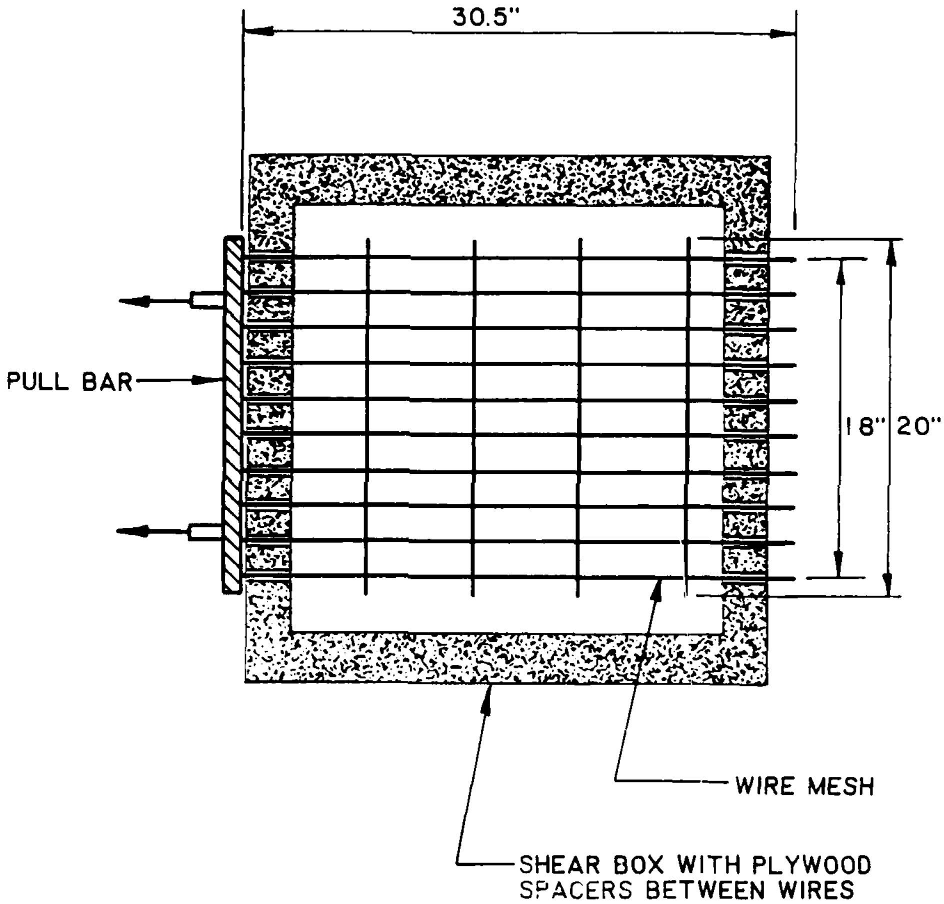


Figure A14. Position of wire mesh in pullout test apparatus

Figure A15. Partially assembled pullout test apparatus
with wire mesh in place

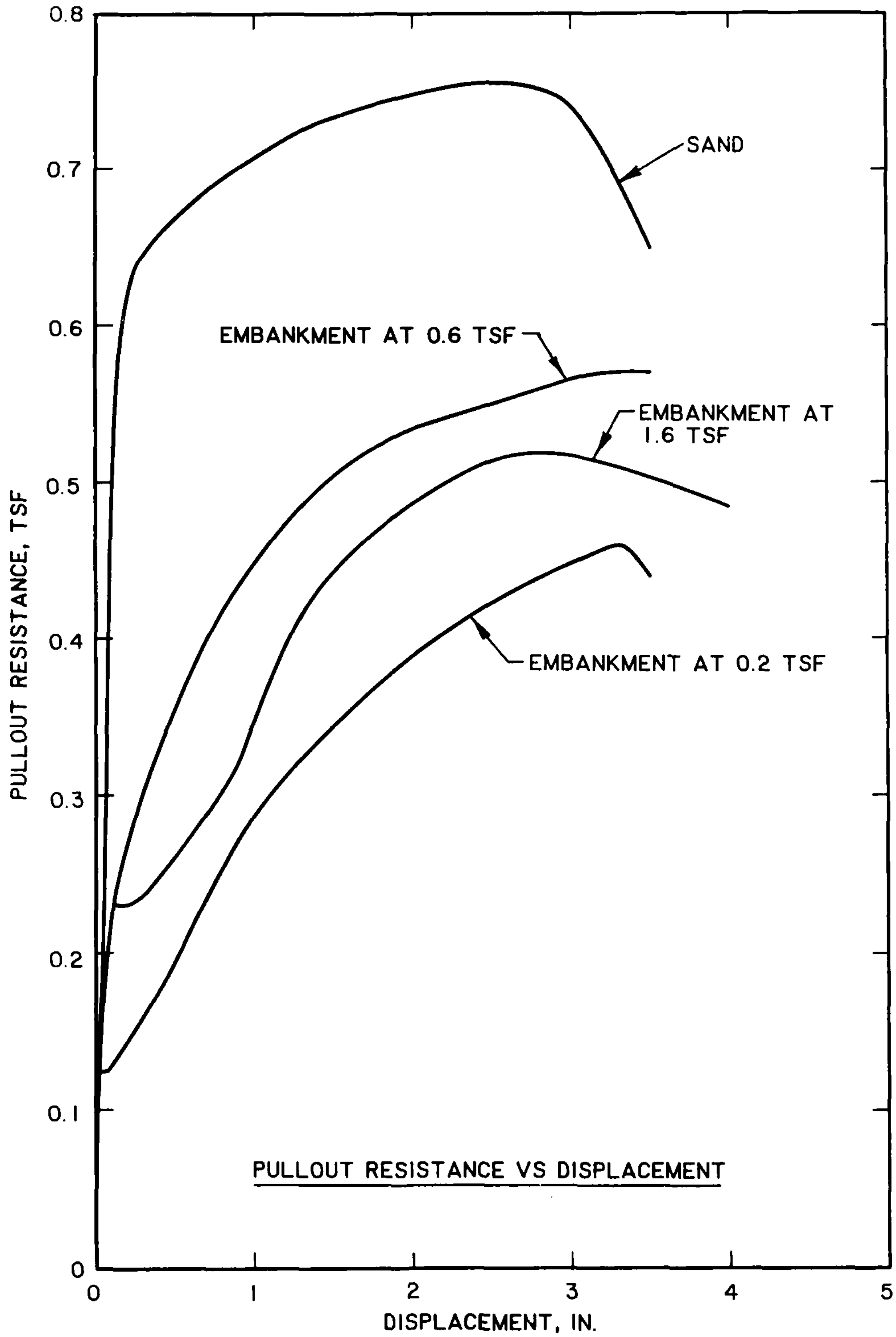


Figure A16. Pullout resistance curves

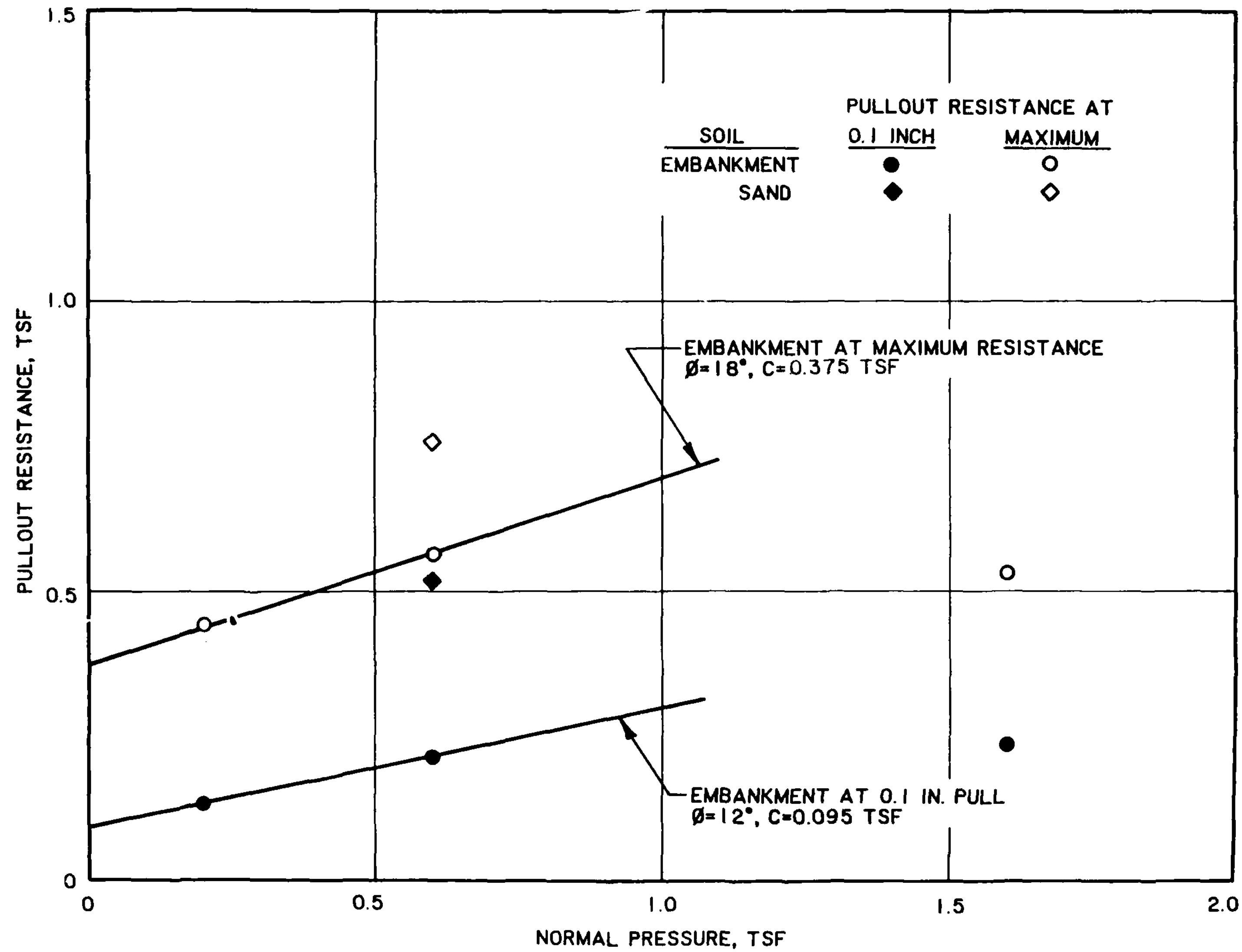


Figure A17. Pullout test results

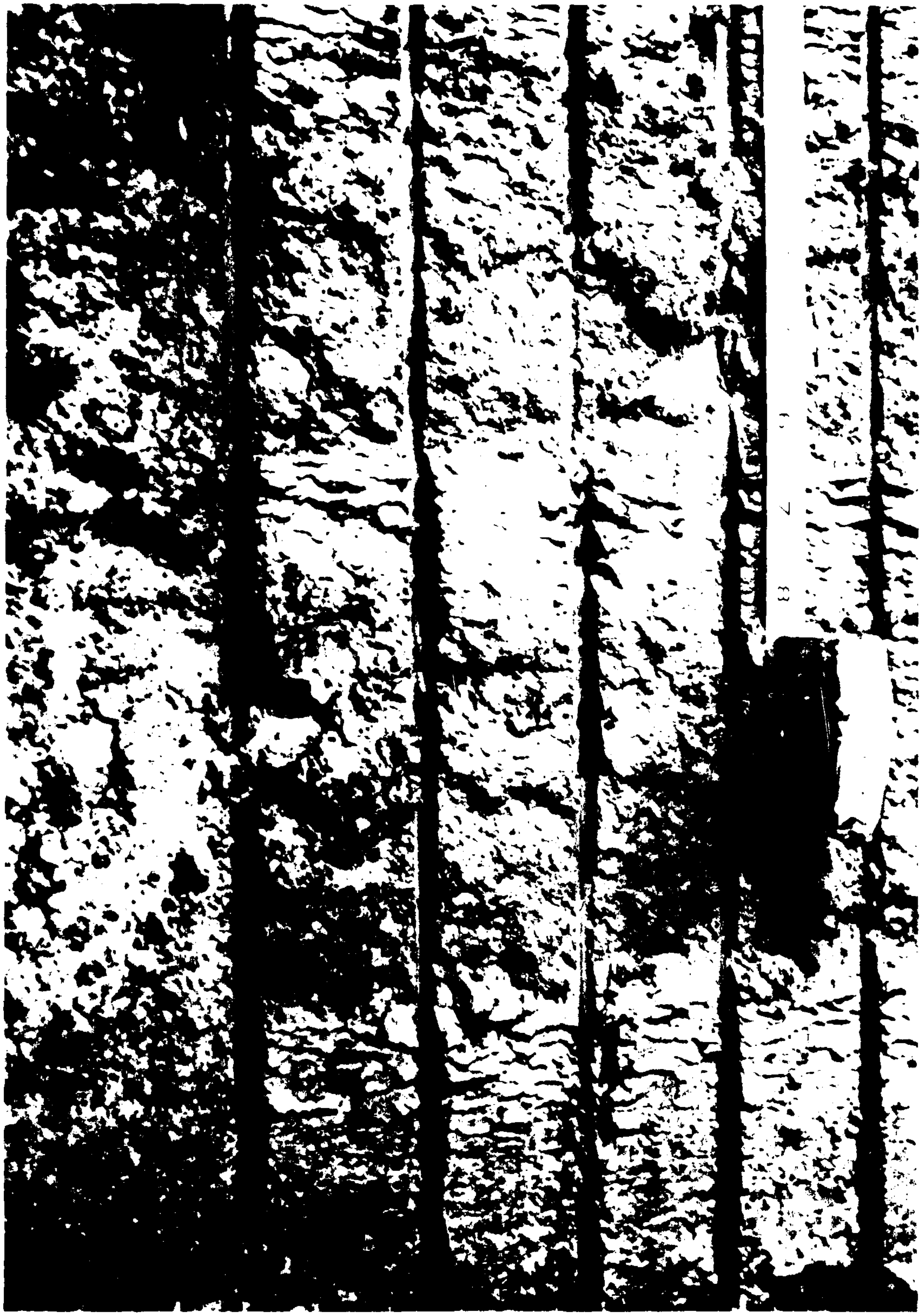


Figure A18. Pullout zone after testing showing lower half of shear box

Figure A19. Pullout zone after test showing upper half of specimen with embedded wire

AD-A173 269 MOHICANVILLE REINFORCED DIKE NUMBER 2 DESIGN MEMORANDUM 2/2
(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION
VICKSBURG MS GEOTECHNICAL LAB J FOWLER ET AL. AUG 86
UNCLASSIFIED WES/MP/GL-86-25 F/G 13/13 NL

END
DATE
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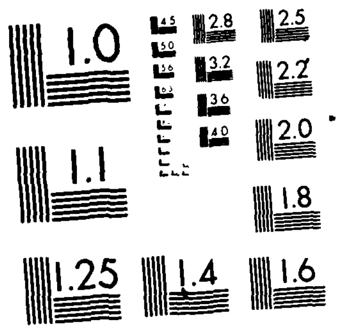
12-86
DTIC



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

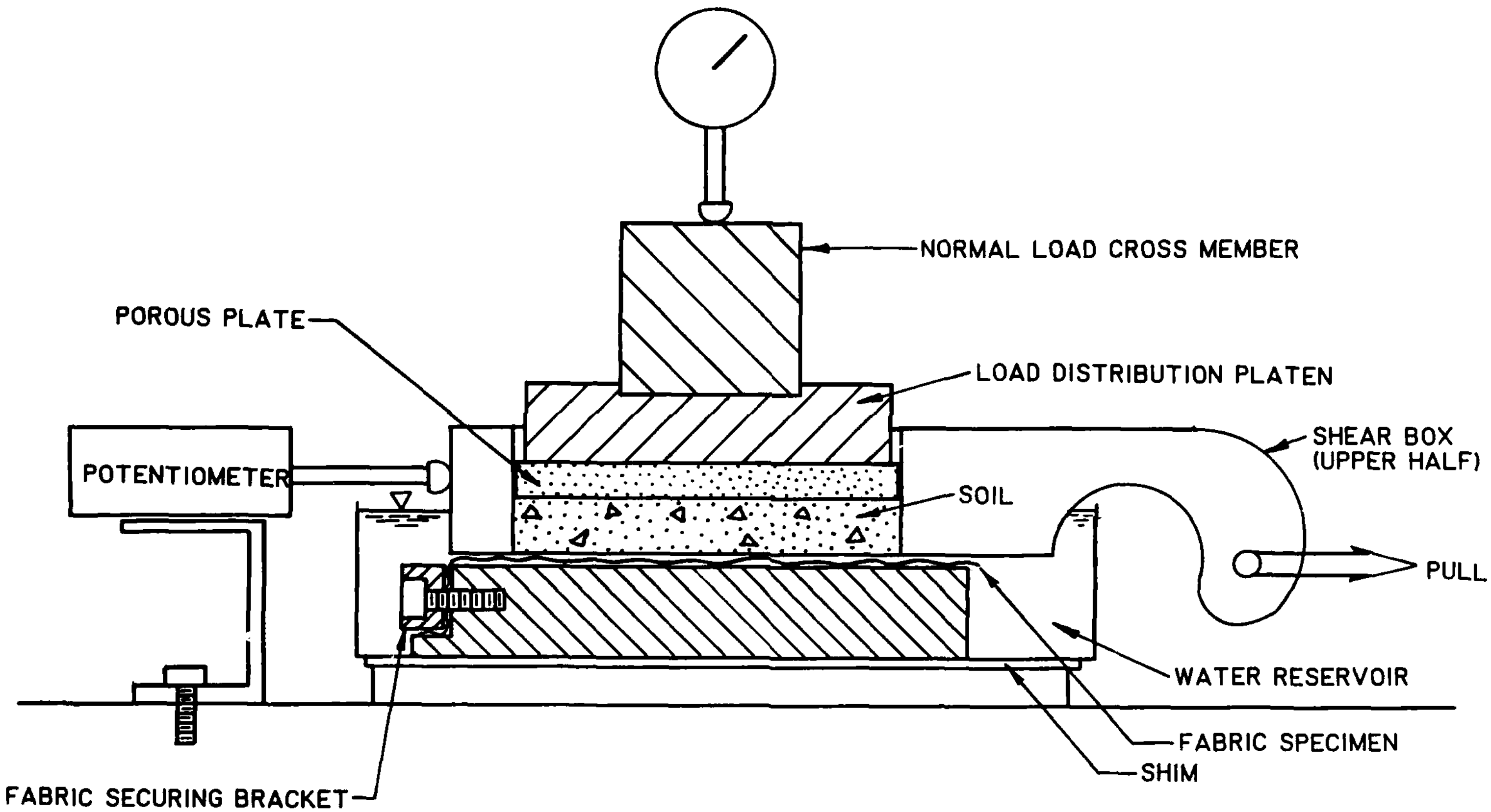


Figure A20. Soil-fabric friction apparatus

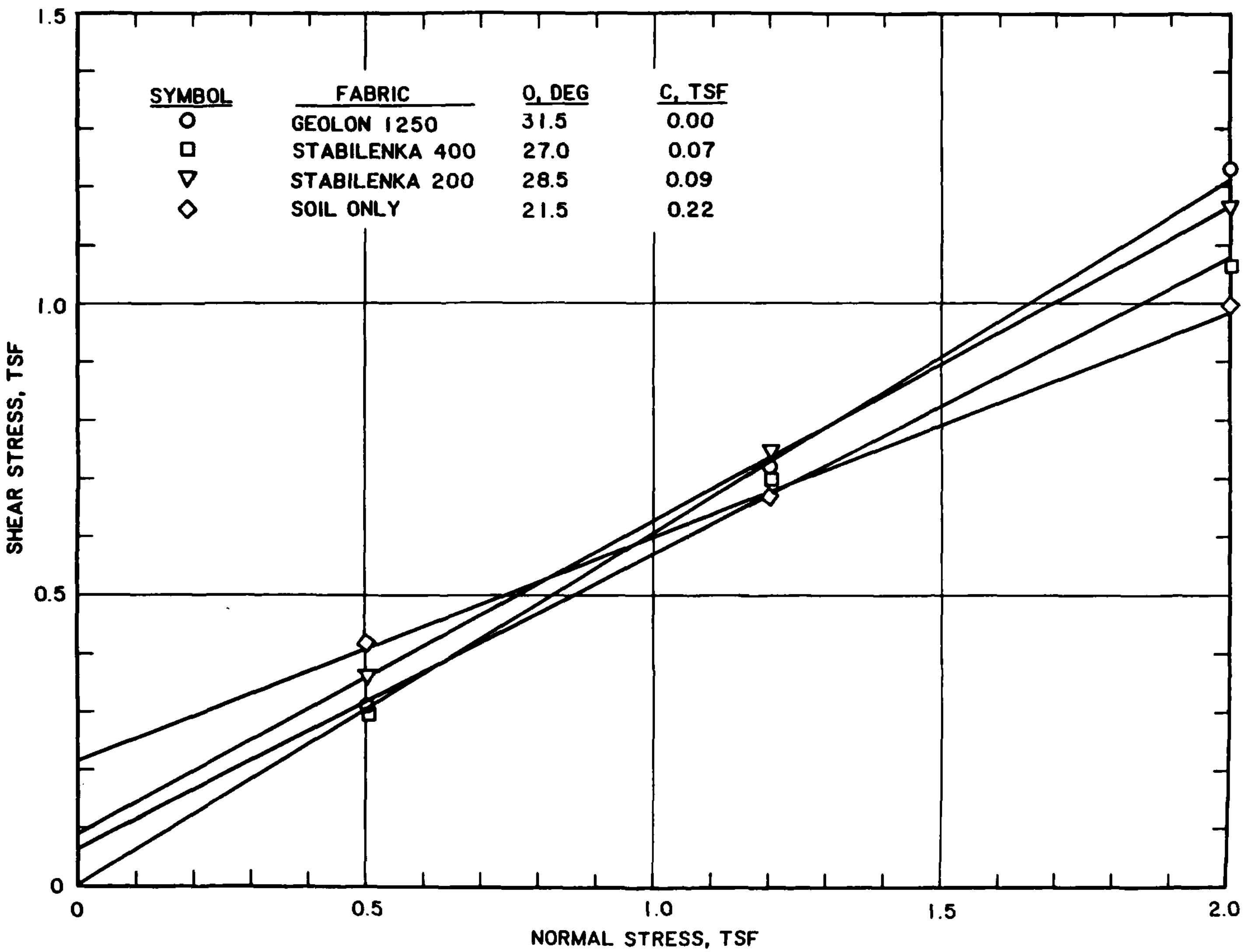


Figure A21. Soil-fabric friction test results

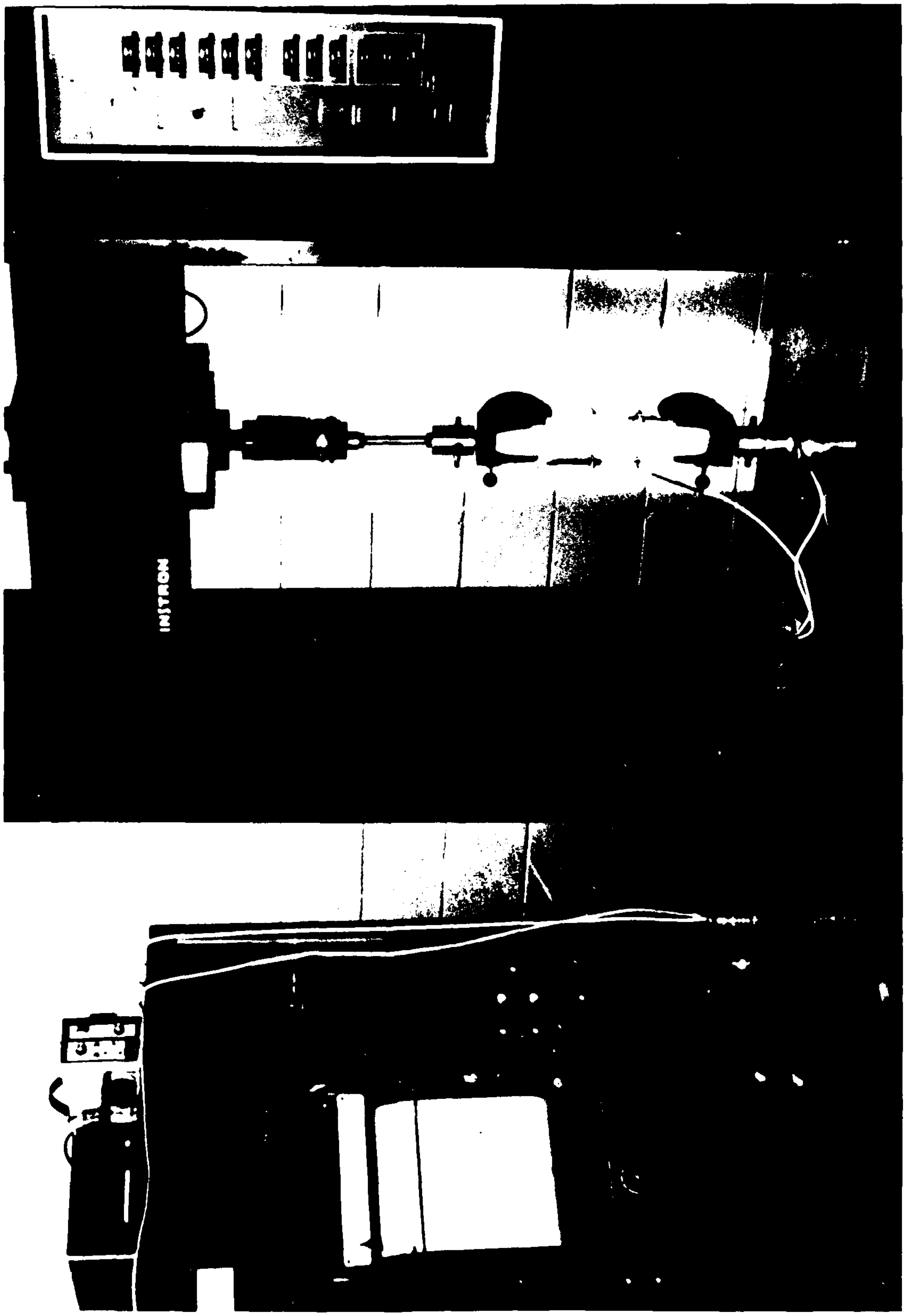


Figure A22. Fabric tensile test apparatus

APPENDIX B: SLURRY TRENCH DESIGN

Scope

1. This Appendix presents design parameters for a lined slurry trench at the Mohicanville Dike No. 2, including dike and trench location, analysis of expected behavior, selection of a geotextile fabric to be placed in the trench, and cost parameters needed to estimate relative costs for conducting the work. This Appendix is the design required to prepare "Plans and Specifications for Mohicanville Dike No. 2 Slurry Trench," which contains specifications and construction drawings required for contract advertisement. Construction drawings from the Plans and Specifications are referenced in this Appendix. A second report, "Embankment Reanalysis, Mohicanville Dikes," Law Engineering Testing Company (LETCO), July 30, 1982, contains parameters pertinent to this report including foundation and embankment soil properties and profiles and sections of the existing embankment. Boring logs are shown in the LETCO report and laboratory soils tests from the report are listed in Table B-1 and shown in Figure B2 through B5.

Design of Slurry Trench

2. The slurry trench is designed as a seepage cutoff for the layer of peat visible on at ground surface outside the limits of the existing dike and extending beneath the embankment, Figure B1. As part of the foundation of the original embankment, the peat and the soft clay beneath it settled and/or flowed out to form the existing foundation conditions shown in section in Figure B1. The embankment is part of a flood control system that is normally dry and any design of a seepage cutoff must consider the effects of drying cracks that could form on the upstream face of the dam. Cracking could also develop due to splitting/spreading of the embankment on a soft foundation. Construction sequence considerations dictate that the slurry trench must be constructed before the proposed embankment is placed; therefore, settlement and horizontal displacement of the proposed embankment must be considered. Total vertical settlements of 38 in. beneath the dam center line and 28 in. near the upstream toe have been projected by the LETCO

report. A finite element study conducted at the Waterways Experiment Station (WES) predicts that during construction there will be vertical settlement of approximately 1 ft at the centerline, heave of approximately 0.2 ft at the toe, and horizontal movements of 0.5 ft within the foundation.

3. The following assumptions were made in designing the slurry trench:

a. Vertical displacements of 5 ft could occur beneath the ℄ of the dike.

b. Horizontal displacements of 0.5 ft could occur at the upstream toe.

c. Existing soil conditions are as shown in Figure B1 for typical sections.

d. Adopted design data are shown in Figures B11 and B14.

e. The trench should extend 3 ft into the clay beneath the peat layer to reduce the hydraulic gradient through the clay beneath the bottom of the trench and form a continuous seal between dike sta 2+65 to sta 14+00. Profiles are shown in the slurry trench plans and specifications.

f. Three typical sections, as shown in Figure 1, are considered:

(1) Cases 1 and 2, sta 6+00 to 11+00 and

(2) Case 3, from sta 2+65 to 6+00 and sta 11+00 to 14+00.

g. A liner will be used in the trench in conjunction with the soil-bentonite backfill.

Selection of soils design data

4. Because of rapid construction of the Mohicanville slurry trench, an undrained analysis ($\phi = 0$ simplification) of slurry wall stability is appropriate. Table B-1 and Figures B2 through B5 are the results of laboratory tests performed by LETCO and are submitted in their report, "Embankment Reanalysis, Mohicanville Dikes." Figures B6 and B7 are the gradation curves (with the Atterberg limits and soil classification listed) and $\bar{R}$ test results, respectively, that were obtained at the WES soils laboratory on a composite sample from test pit bag samples taken from the Mohicanville dike.

5. The two Q tests conducted by Ohio River Division Laboratory (ORDL) on undisturbed samples, Table B-1, indicate undrained strengths of 0.44 tsf (0.88 ksf) for an unknown depth and 1.03 tsf (2.06 ksf) from 39 to 41 ft. These two tests are one-point tests. A Q test was conducted at the WES on a composite recompacted sample from the dike, and the results are shown in Figure B8. The envelope for this test indicates an undrained

strength of 0.65 tsf (1.3 ksf) and $\emptyset = 0$. A R test conducted with the composite sample indicated an apparent cohesion (total stress) parameter of 0.25 tsf and an apparent $\emptyset$ of 12.3 deg as shown in Figure B9.

Selection of a trench liner

6. In a meeting of Corps representatives and their consultants at the WES, 10-11 April 1983, the decision was confirmed to place a liner in the slurry trench to insure the continuity of the seepage cutoff if, in fact, large displacements occur in the foundation during construction and cracks develop in the soil-bentonite trench. The technique for placement of a liner in a slurry trench for this purpose has been developed only recently and specific guidelines are not available. Installation of different thicknesses of high density polyethelene (HDPE) for pond linings is an established engineering practice with numerous examples. Eighty-mil liners have been placed for wastewater ponds (City of Corsicana, Texas) and sewage lagoons (Town of Sundre, Alberta, Canada), but the construction technique allows the HDPE to be rolled into place and seamed. Tears and punctures can be seen and repaired. Installation in a slurry trench requires that the membrane be weighted at the bottom, then lifted and dropped into the trench or slid into the trench from the working surface. Unlike a pond liner, no checks for continuity or punctures can be made successfully after installation. Industry spokesmen have recommended the 100-mil HDPE sheet due to its resistance to punctures and tearing during construction, its high percent elongation (750 percent) before breaking, and its tensile strength at break (3500 psi). Other specifications for the HDPE sheet are shown in the Plans and Specifications for the slurry trench. Although a cost savings on a thinner 80-mil membrane ($1/sq ft for 100 mil; $0.80/sq ft for 80 mil) could be attained, the newness of the engineering application and the importance of the seepage cutoff dictates the use of a substantial membrane (100 mil). Several plants are set up to manufacture the 100-mil sheet and price competition should insure a fair market price.

7. Rolls of the HDPE material can be obtained as wide as 34 ft and 1150 ft long. For the trench, a roll 29 ft by 1150 ft would weigh approximately 17,400 lb. If seams are required to handle the material or to conform to the trench bottom, the seams will be required to match the properties of the material subject to the approval of the Corps.

Slope stability analysis
(Corps Program SAVA104)

8. General. As a design consideration for the proposed dike, the slope stability failure potential of the open slurry trench was checked using the soil properties associated with the three typical sections mentioned above and shown in Figure B1. The slurry in the trench was assumed to have no strength and the resisting forces beyond the trench were replaced by water pressure, but no fluid loss is allowed into the soil. As mentioned earlier the slurry trench will be rapid construction in a gravelly sandy clay (CL); therefore, a short-term undrained analysis should be performed. When modeling a foundation with only cohesive properties ($\phi = 0$ analysis), a force equilibrium analysis yields a high FS, thus a moment equilibrium analysis is preferred. Using the Fellenius circular arc method that satisfies moment equilibrium available in WES Program SAVA104 and the soil strength parameters shown in Figure B10, a minimum FS was computed for arcs exiting as shown in Figure B10. The internal mechanics of the program were not changed to accept either a slurry density greater than that of water (62.4 PCF) or a slurry head above the water table, therefore the FS calculated is considered to be conservative.

9. Case 1. Case 1 is a typical section occurring at sta 9+15 of the old embankment, Figure 1. It is assumed that the old embankment material has completely displaced the peat in this area and that the 30.5-ft depth of the trench will be excavated in a saturated clay. The excavated slurry trench was modeled by replacing the material with water which provides no strength but does provide a pressure acting against the wall. The minimum FS calculated was 1.11, Figure B10. The Corps specifies a 1.3 FS for slope stability at end of construction conditions. In order to obtain a FS of 1.3 the density of the slurry was considered to act against the side of the trench and a resisting moment was calculated. The calculations are shown in Figure B11 and the density required for a FS of 1.3 would be 74 pcf. A hand calculation verifying the circular arc method for Case 1 is shown in Figure B12.

10. Case 2. Case 2 is for a section located at sta 6+35, Figure B1. The section analyzed assumes that the old embankment material is 18.5 ft thick. This material is underlain by peat which has been consolidated to leave a layer only 9 ft thick. Beneath the peat is a lean clay material

into which the trench will be keyed 3 ft. Assuming the trench is filled with water the minimum FS calculated for the full depth including the embankment, peat, and clay was 1.07, Figure B10. To bring the FS up to 1.3, a slurry density of 71 pcf was required as shown in Figure B11.

11. Case 3. Case 3 is for a section located at sta 4+45, Figure B1. This section has a 17.5-ft-thick layer of old embankment, 8 ft of peat and then 5 ft of a soft lean clay. Assuming water fills the trench the minimum FS calculated for the full depth, including the embankment, peat, and clay was 1.15, Figure 5. A slurry density of 74 pcf is required in the trench to raise the FS to 1.3, Figure B11.

Wedge analysis using slurry forces and $\emptyset = 0$ concept

12. General. Nash and Jones (1963), "The Support of Trenches Using Fluid Mud", suggested a theory of trench stability using the wedge method, Figure B13. If the condition $\emptyset = 0$ exists, then $a = 0$ and $r = 45$ deg and an equation for equilibrium can be expressed as:

$$H_{cr} = \frac{4C}{s - sL}$$

where

H_{cr} = critical height

C = cohesion

s = saturated density of the soil

sL = density of the slurry in the trench

This equation gives satisfactory results if (a) the trench is long compared to its depth, and (b) the cohesion value is representative for the trench depth, The $\emptyset = 0$ analysis does not establish a failure surface but implies it is inclined at 45 deg with the horizontal. Although the exact failure plane is not known, the concept is valid if failure is assumed to occur in the trench when the maximum shear stress reaches the maximum shear existing at failure in a triaxial test. The analysis applies only to saturated soils and assumes a slurry level at the top of the trench.

13. One approach to determining shear strength for a saturated soil, confined to an existing effective stress has been put forth by LETCO in the letter of June 17, 1983. LETCO states that R test results would best describe the above parameters and that from these results an undrained

strength profile versus depth can be calculated. By substituting $C = S_u$ and treating the strength as a layered system with $\phi = 0$, the Nash and Jones analysis can be applied. Strength profiles for Cases 1, 2, and 3 calculated in this manner are shown in Figure B14. Results of slurry density versus FS using Nash and Jones' equation are shown in Figure B13.

14. For the results shown in Figure B13, Case 3 would be the worst case if the slurry trench is actually excavated to a depth of 30.5 ft or 13 ft into the foundation material. For Case 3 the FS would be slightly higher than 1.2 for an in-trench slurry density of 74 pcf. For a slurry density of 74 pcf the Case 1 FS is well above the required 1.3 while Case 2 would be 1.25.

Bearing capacity of the trench

15. The bearing capacity of the trench foundation was calculated for the worst case (case 2 with backfill in place) as shown in Figure B15. To account for local shear or punching, 0.67C was used. An FS of 1.47 was calculated which is considered safe in bearing.

Discussion

16. Slurry trench wall stability analyses were made for the end of construction or undrained strength case. A limited number of undrained strength tests had been conducted on the old embankment material. The strength values selected for the old embankment were from WES a total stress plot, Figure B9, and the values for the peat and clay were taken from the LETCO report. The following is a summary of the design results from Figures B11 and B13:

	Failure Plane Exit	Method	FS	Slurry and Water Level	sL
Case 1	Bottom of Trench (all old embankment)	Arc	1.3	Ground Surface	74
		Wedge	1.3	" "	62.7
Case 2	Bottom of Trench	Arc	1.3	" "	71
		Wedge	1.3	" "	76.5
Case 3	Bottom of Trench	Arc	1.3	" "	74
		Wedge	1.3	" "	78.5

These results are considered safe by the Corps standard of a minimum FS of 1.3 for the end of construction case. The circular arc FS was obtained using the WES Program SA10478 which incorporates the Fellenius circular arc method and is modified to satisfy moment equilibrium. The material in (slurry) and beyond the trench (soil) was replaced by water which has no strength but does apply a pressure against the slurry wall. For the wedge analysis the slurry density was accounted for in developing the equation for critical height of the slurry trench. In both cases the WES value of undrained strength (C = 0.5 ksf, Ø = 12.3 deg) was used for the embankment material and values of 0.5 ksf and Ø = 0 were used for the peat and the foundation clay.

17. Density of the slurry in the trench can be expected to increase from 6 to 30 PCF above the initial discharge pipe mix as reported by D. R. Duguid et al., "Slurry Trench Cut-Off for the Duncan Dam," 1970; therefore, the construction FS should be higher because the in-trench slurry density will be higher than the discharge pipe density. The design results show a slurry density of 74 pcf is needed in the trench. Although there is some difference between the slurry density calculated by the two methods if the discharge pipe density is held at 68 pcf, the range of density in the trench should be higher than the a density required to provide a FS of 1.3.

Differences in analyses

18. The WES slope stability Program SA10478 uses the Fellenius circular arc method of moment equilibrium analysis but by using a long arc radius and forcing the arc to exit in the trench and by assuming liquid properties for the material in and beyond the trench, the method is very similar to the wedge method. The layered properties are accounted for but the head differential between the slurry and the water table is assumed to be zero. The Fellenius method (Ø = 0 simplification) as used is applicable for the layered cohesive materials present and the end of construction case.

19. The wedge analysis equation developed by Nash and Jones (1963) is for a homogeneous soil where Ø = 0 and the slurry levels are at the top of the trench. With Ø = 0 the analysis assumes the failure wedge is at a 45 deg angle to the horizontal. For a material that meets the assumptions, it is a simple method to determine critical height for a given slurry density.

Bearing capacity

20. The material in which the trench is founded has consolidated for over 30 years under the load of the old embankment and should have fairly uniform strength at the bottom level of the trench. The values assumed for this depth are adequate to prevent a bearing capacity failure.

Cutoff Location Analysis

21. Initially it was felt that the slurry trench might be at one of three locations: (1) the upstream toe, (2) the centerline of the dam, and (3) 45 ft upstream of the center line. The upstream toe location was cost prohibitive because of several construction features: (a) constructing a working surface over the soft peat, (b) constructing haul roads, (c) obtaining a positive cutoff for the peat layer because it would be necessary to extend a membrane up the face of the embankment, and (d) bearing capacity of the foundation requiring costly light cement-bentonite slurry. The centerline of the dam was dropped from consideration because the dam had to be degraded to construct a working surface and the location required more square feet of pay items (trench wall and membrane). The location 45 ft upstream of the centerline provided a better working surface (no fabric required to support equipment), allowed the backfill and membrane to be stockpiled near the trench, and required a lesser depth of trench than at the centerline. The cost alternatives are listed in Table B-2. After reviewing these costs it was decided at a meeting of Corps representatives and consultants, held at the WES 10-11 April 1983, that the engineering and cost considerations were in favor of the location 45 ft upstream of the centeline.

22. The following cost items obtained from the Corps and industry were used to make the relative cost determination mentioned above:

a. 100-mil HDPE liner costs $1/sq ft in place in the trench (supplied by Gundel Liners).

b. Soil bentonite costs $3 to $5/sq ft of wall, if there is a flat area to mix the backfill and the excavated material can be reused (Vicksburg District).

c. Replacement backfill costs $1/yd if the dam can be degraded (supplied by Huntington District). Negligible compared to items a and b.

d. Trench cap costs $5/yd in place. Negligible compared to items a and b.

e. Building haul roads and a working platform on soft peat. High cost item applicable only to a trench at the toe, $1.50/ft^2$.

f. Mobilization, demobilization, and restoration of the site are job bid items that would be similar for the three locations and were not included in the cost comparisons.

Recommendations

23. It is recommended that

a. For the soil and construction conditions similar to all three cases, the discharge pipe slurry density should be a minimum of 68 pcf and the in-trench density should be a minimum of 74 pcf.

b. If construction conditions change, i.e., slurry level, water table, or working surface, then the FS and cost alternatives should be rechecked.

Table B-1

LETCO

Summary of Strength Test Data

Embankment Fill Total P-Q				
Boring No.	Depth (ft)		P (TSF)	Q (TSF)
		Q Tests*		
UD-1	Unknown		0.44	0.44
UD-24	39-41		3.58	1.03
		$\bar{R}$ Tests		
UD-21	9-11		1.73	0.73
			1.49	0.99
			0.96	0.71

Embankment Fill Effective P'-Q				
Boring No.	Depth (ft)		P (TSF)	Q (TSF)
		$\bar{R}$ Tests		
UD-21	9-11		1.02	0.50
			0.74	0.46
			0.64	0.44

(Continued)

* Ohio River Division Laboratory

Table B1 (Continued)

Boring No.	Depth (ft)	0.75 $'_{v}$ (TSF)	P (TSF)	P (TSF)
		Foundation Peat Total P - Q		
		Q Tests		
UD-25	21-23	0.69	1.96	0.56
UD-21	24-26	0.78	1.96	0.46
UD-26	9-11	0.15	0.69	0.24
UD-1	22-24	0.69	0.67	0.67
UD-26	11-12	0.15	0.66	0.16
UD-22	9-11	0.15	0.44	0.09
UD-27	9-11	0.02	0.44	0.09
UD-23	9-11	0.02	0.42	0.07
UD-22	6-7	0.14	0.28	0.08
UD-22	4-6	0.13	0.26	0.11
UD-27	4-6	0.01	0.26	0.11
UD-2	9-11	0.02	0.13	0.13
		R and $\bar{R}$ Tests		
SI-5	7-9		1.34	0.34
UD-25	21-23		1.27	0.57
SI-1	10-12		1.10	0.35
SI-4	9-11		0.95	0.35
SI-6	5-7		0.82	0.44
SI-5	7-9		0.76	0.26
SI-1	10-12		0.57	0.20
UD-23	9-11		0.51	0.22

(Continued)

Table B1 (Continued)

Foundation Peat Effective P' - Q'			
Boring No.	Depth (ft)	P' (TSF)	Q (TSF)
UD-27	9-11	1.40	0.40
UD-37	4-6	0.68	0.28
SI-5	7-9	0.54	0.34
SI-6	5-7	0.46	0.44
SI-4	9-11	0.42	0.32
SI-1	10-12	0.37	0.35
SI-5	7-9	0.29	0.26
SI-1	10-12	0.24	0.19

(Continued)

Table B1 (Continued)

Foundation Clay
Total P - Q

Boring No.	Depth (ft)	0.80 σ'_v (TSF)	P (TSF)	Q (TSF)
		Q Tests		
UD-25	59-61	1.36	3.94	0.34
UD-21	49-51	1.24	3.24	0.39
UD-25	49-51	1.16	2.98	0.33
UD-21	39-41	1.03	2.60	0.35
UD-21	29-31	0.83	2.33	0.68
UD-21	34-36	0.93	2.25	0.30
UD-25	34-36	0.85	2.23	0.38
UD-25	29-31	0.75	2.04	0.34
UD-26	29-31	0.41	1.59	0.09
UD-23	29-31	0.40	1.51	0.21
UD-27	24-26	0.16	1.16	0.11
UD-26	19-21	0.20	1.07	0.17
UD-23	19-21	0.19	0.99	0.14
		$\bar{R}$ Tests		
UD-25	39-41		3.67	1.17
UD-22	28-30		2.15	0.65
UD-25	39-41		1.99	0.74
UD-22	28-30		1.23	0.48
UD-25	39-41		1.17	0.54
UD-23	14-16		0.97	0.37
UD-26	29-31		0.94	0.32
UD-27	20-22		0.90	0.40
UD-22	19-21		0.89	0.51
UD-22	28-30		0.71	0.33
UD-23	14-16		0.57	0.27
UD-23	14-16		0.38	0.23

(Continued)

Table B1 (Concluded)

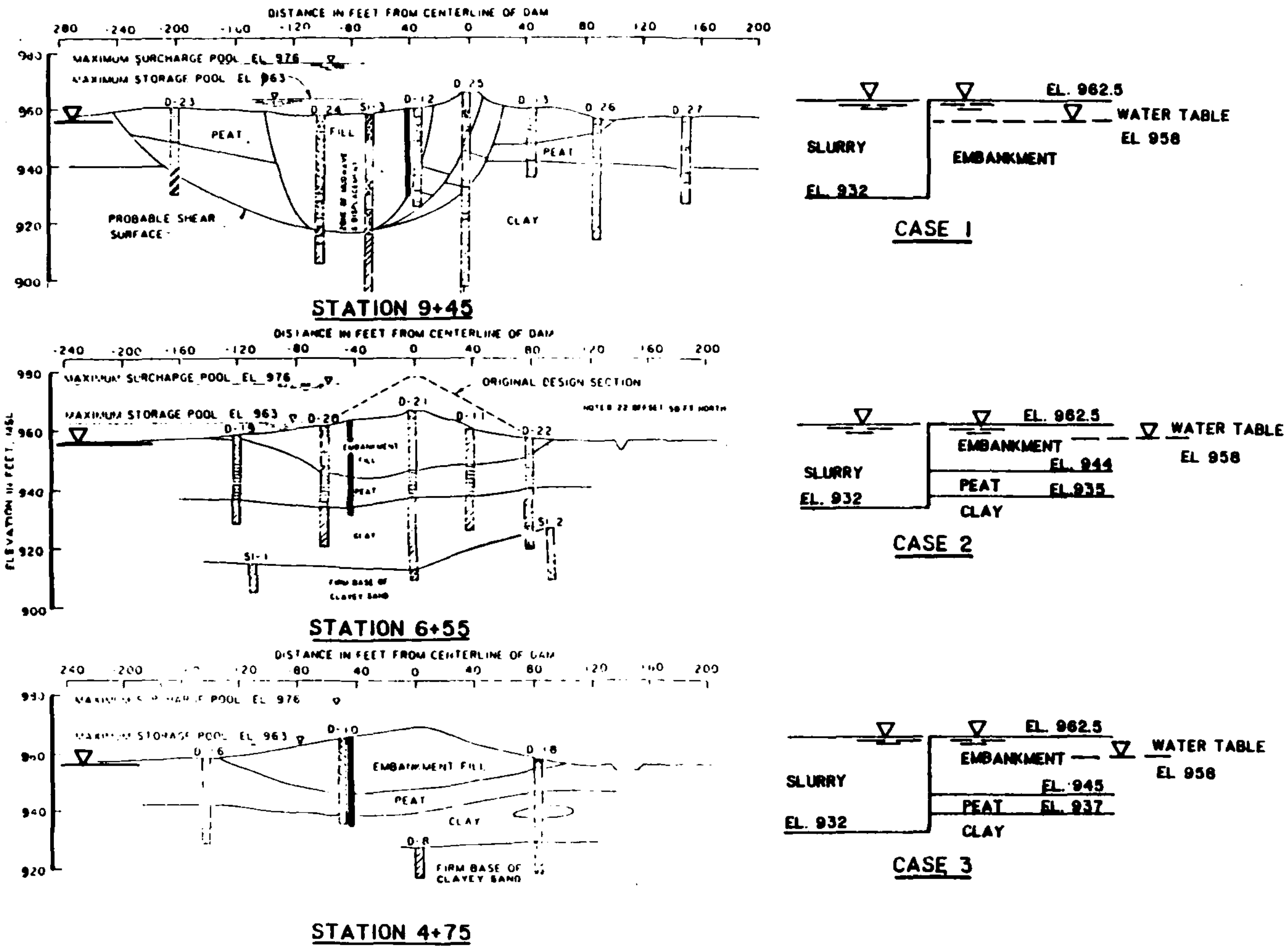
Foundation Clay
Effective P' - Q'

Boring No.	Depth (ft)	P' (TSF)	Q (TSF)
UD-25	39-41	2.19	1.17
UD-22	28-30	1.18	0.61
UD-25	39-41	1.12	0.73
UD-22	28-30	0.82	0.43
UD-25	39-41	0.75	0.55
UD-22	19-21	0.68	0.42
UD-27	20-22	0.67	0.39
UD-26	29-31	0.56	0.31
UD-22	28-30	0.53	0.29
UD-23	14-16	0.49	0.37
UD-23	14-16	0.36	0.26
UD-23	14-16	0.25	0.25

Table B-2

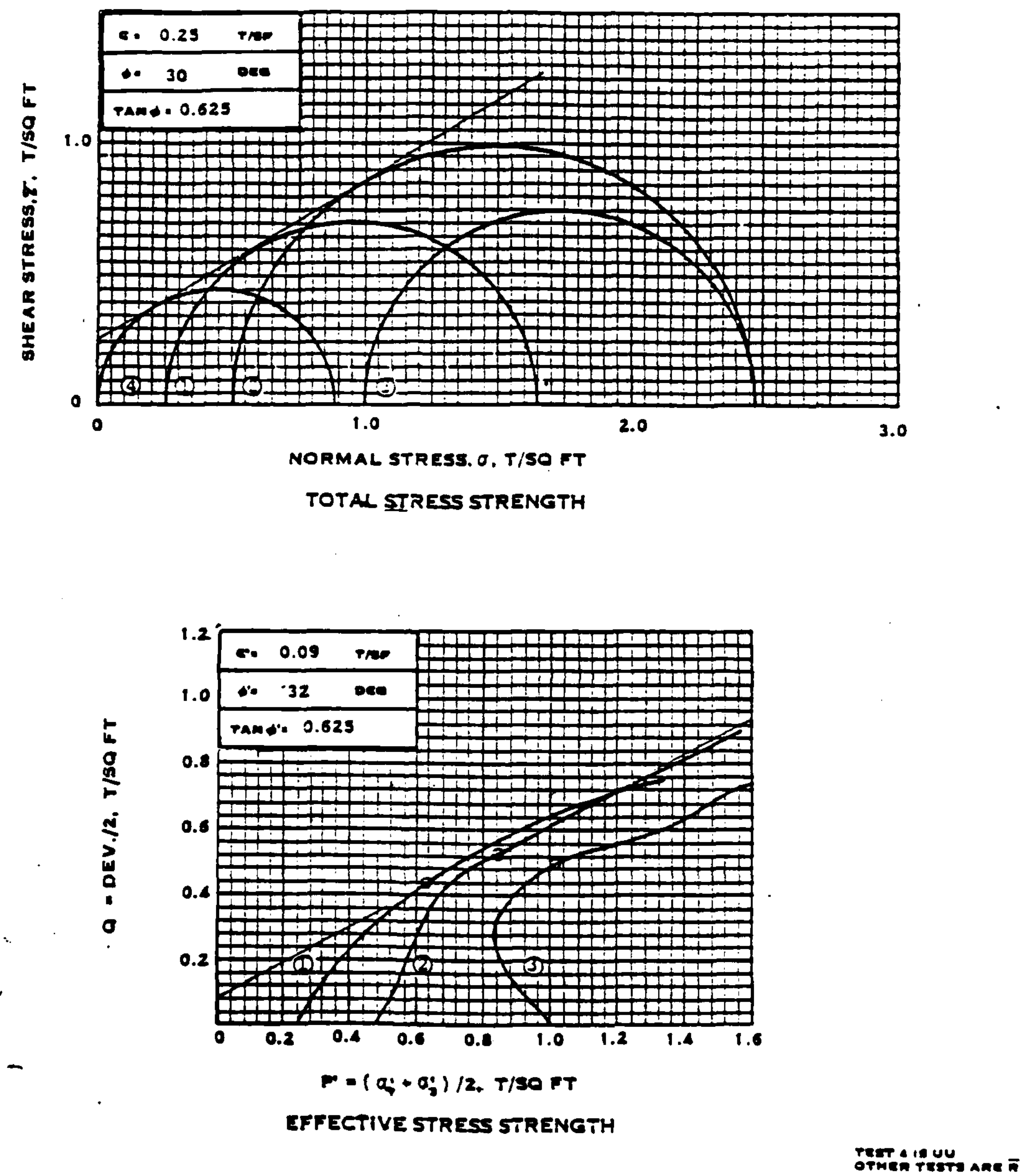
Economic Analysis

	Liner	Cement-Bentonite	Soil Bentonite	Road Fill	Road Fabric	Extra Backfill	Trench Cap
	Depth 1250'	3' 1250'	3' 1250'				
Toe Trench	TR 28' @ 35k Face 95' @ 119^k	280^k	--	17^k	56^k	--	2^k
U/S 45' Dam b	29' @ 36^k	290^k	181^k	5^k	--	2^k	2^k
Dam b Degrade Dam	33' 41^k	330^k	206	$5^k + 14^k$ Degrade Dam	--	2^k	2^k
Relative Cost	Cost $1/Ft² in place	Cost $6 - $8/Ft² dispose back fill, cement	Cost $3 - $5/ Flat area, reuse backfill	Cost $5/yd in place	Cost $1.50/ Ft² in place	Cost $1/yd re- place peat	Cost $5/yd in place
	Total						
Toe Trench	$509K						
U/S 45' Dam b	$226K						
Dam b	$270K						



NOTE: SLURRY TRENCH AT -45 FT FROM CENTERLINE
WATER TABLE ESTIMATED, EL. 958.
FOR ANALYSIS WATER LEVEL SHOWN AT GROUND SURFACE.

Figure B1. Soil profiles and analysis sections



U.S. ARMY CORPS OF ENGINEERS HUNTINGTON DISTRICT HUNTINGTON WEST VIRGINIA	MUSKINGUM RIVER WATERSHED OHIO MOHICANVILLE DAM DIKE NO. 2
LAW ENGINEERING TESTING COMPANY	STRENGTH SUMMARY EMBANKMENT FILL

EXHIBIT NO. 26

Figure B2.

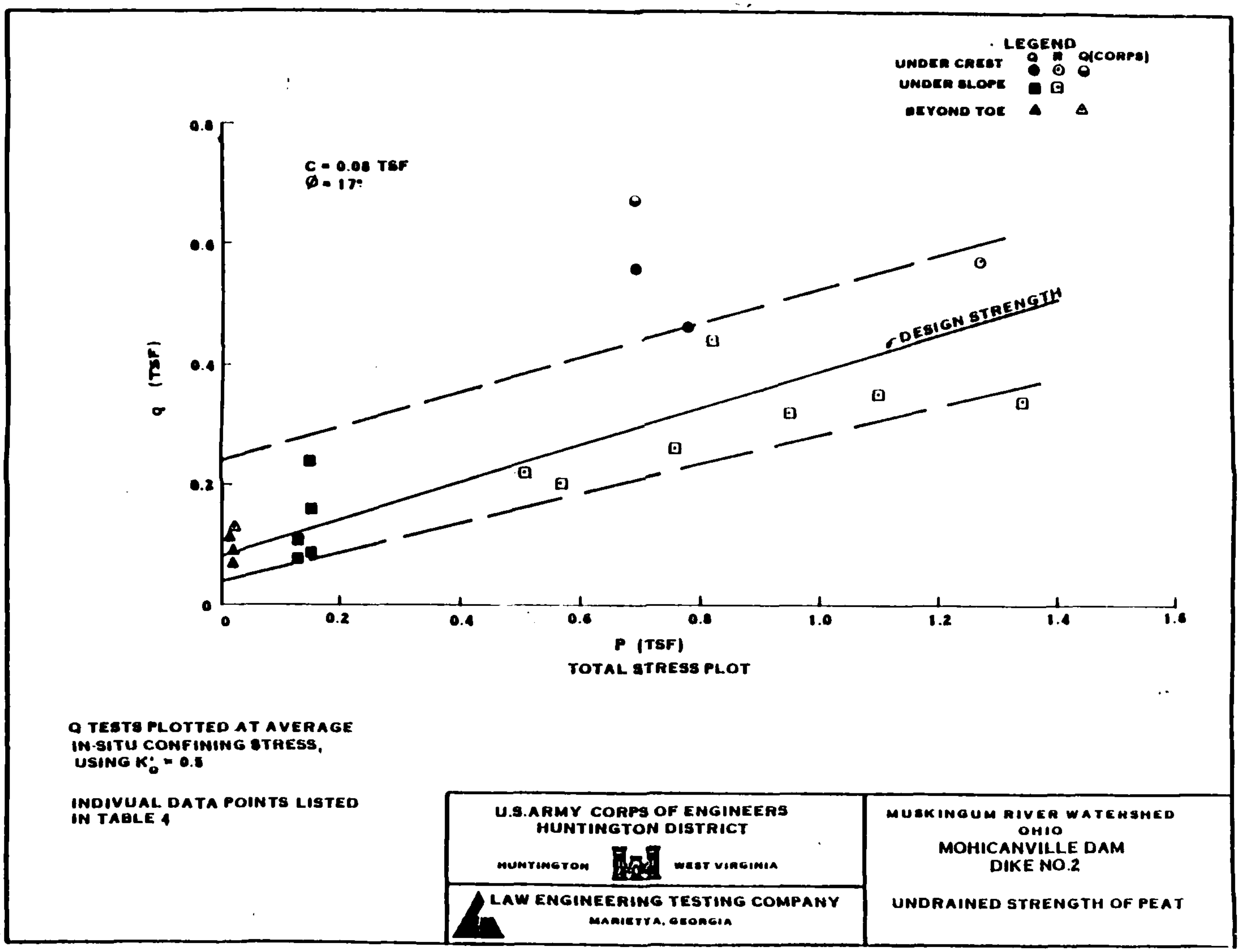
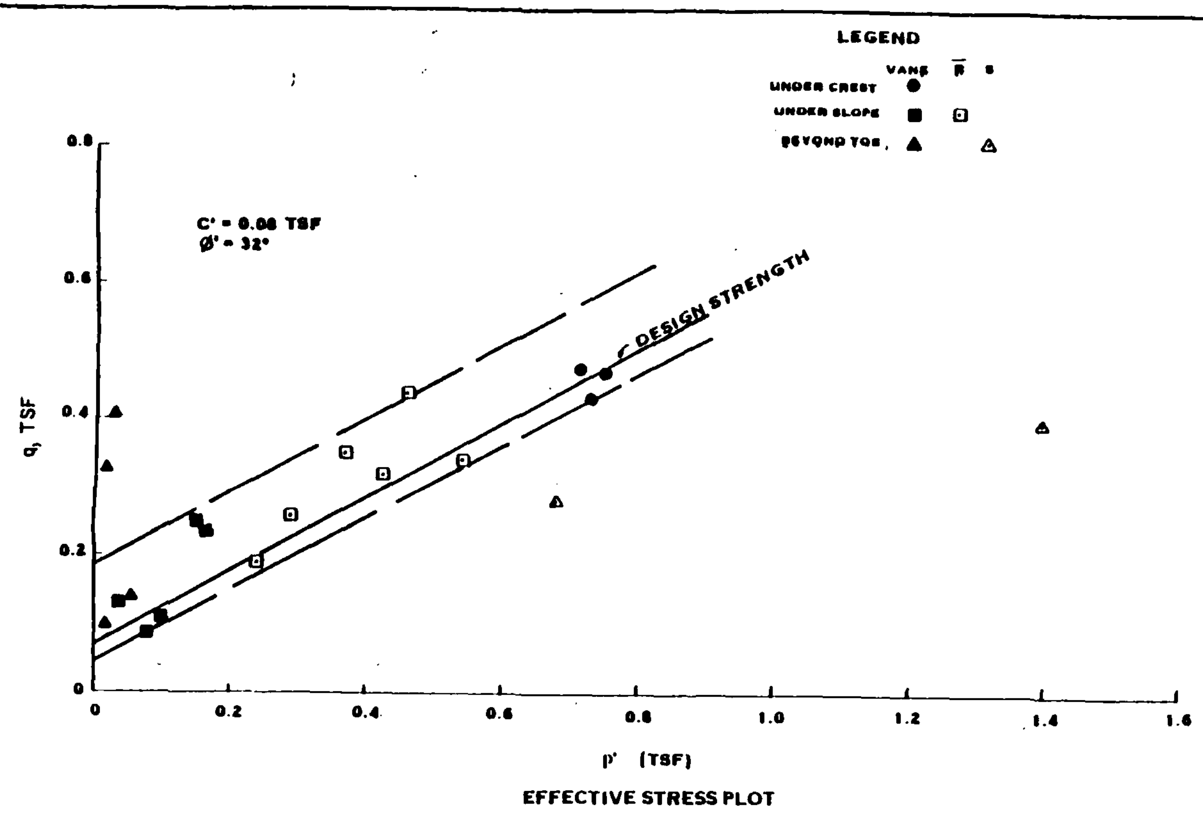


EXHIBIT NO. 28

Figure B3.



Q TESTS PLOTTED AT AVERAGE IN SITU CONFINING STRESS, USING K_O = 0.8

INDIVUAL DATA POINTS LISTED IN TABLE 4

U.S.ARMY CORPS OF ENGINEERS
HUNTINGTON DISTRICT
HUNTINGTON WEST VIRGINIA

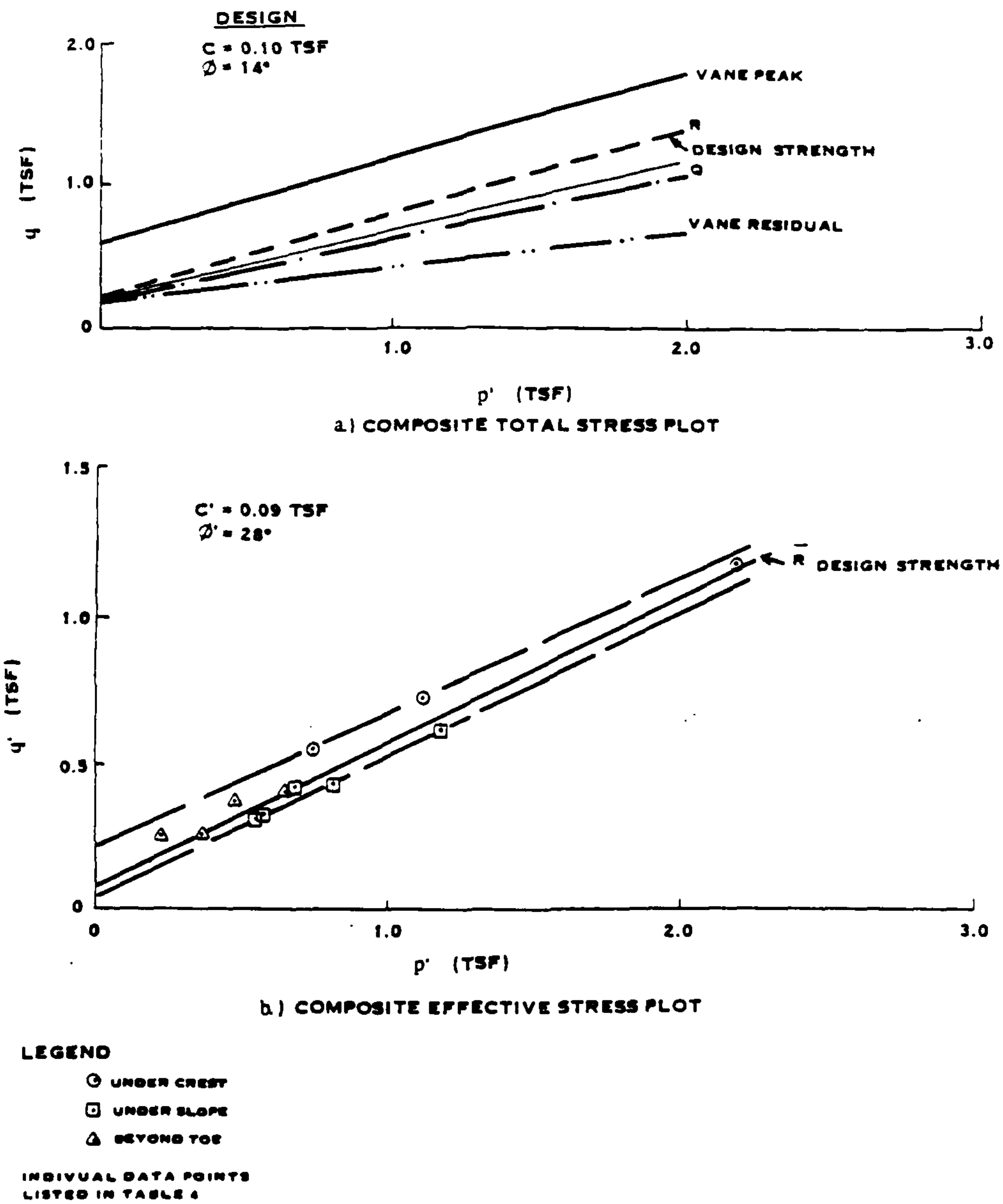
LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA

MUSKINGUM RIVER WATERSHED
OHIO
MOHICANVILLE DAM
DIKE NO. 2

DRAINED (EFFECTIVE STRESS)
STRENGTH OF PEAT

EXHIBIT NO. 29

Figure B4.



a) COMPOSITE TOTAL STRESS PLOT

b) COMPOSITE EFFECTIVE STRESS PLOT

U.S. ARMY CORPS OF ENGINEERS HUNTINGTON DISTRICT HUNTINGTON WEST VIRGINIA	MUSKINGUM RIVER WATERSHED OHIO MOHICANVILLE DAM DIKE NO. 2
LAW ENGINEERING TESTING COMPANY MARIETTA, GEORGIA	STRENGTH OF FOUNDATION CLAY

EXHIBIT NO. 37

Figure B5.

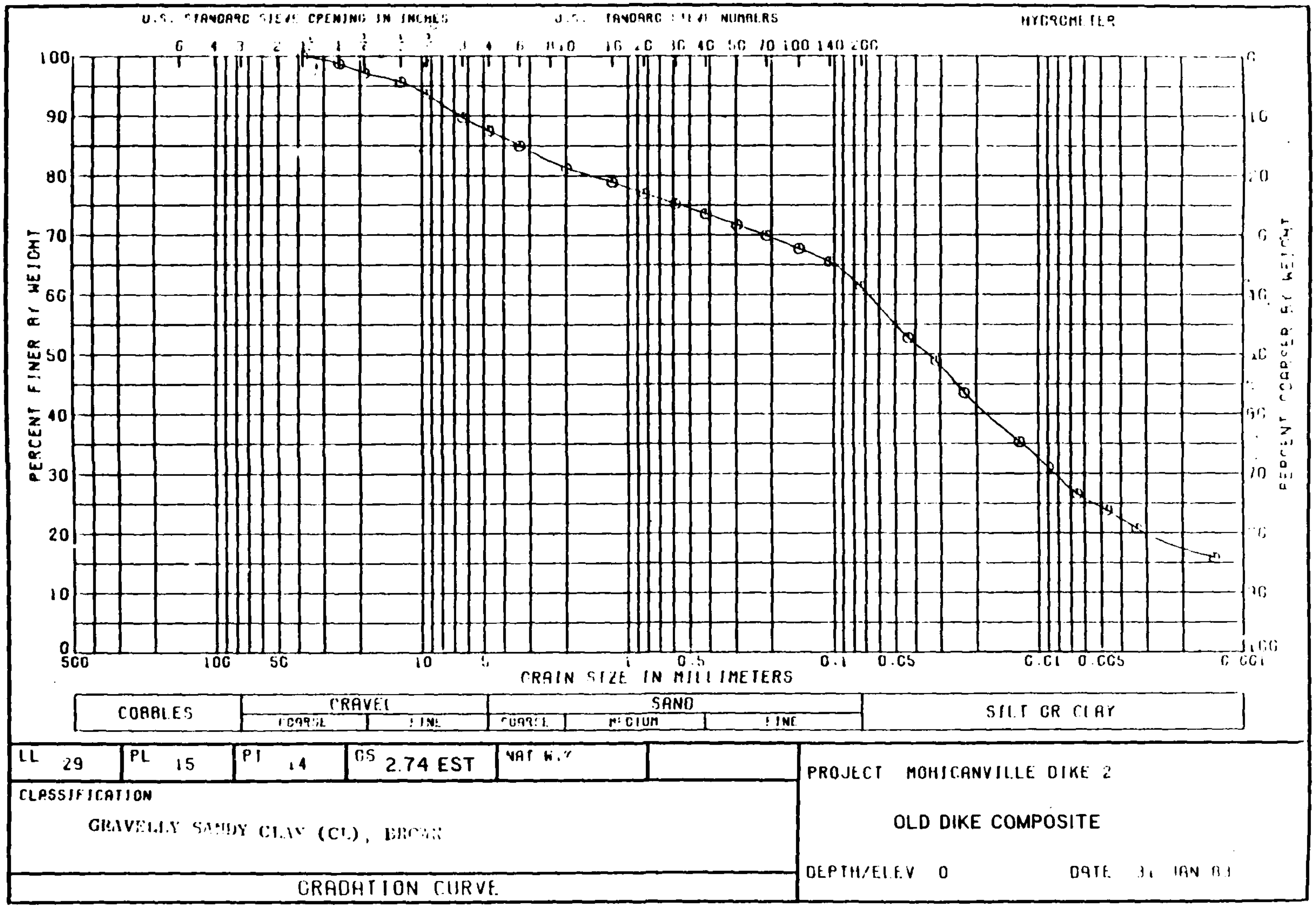


Figure B6. Dike No. 2, soil gradation curve

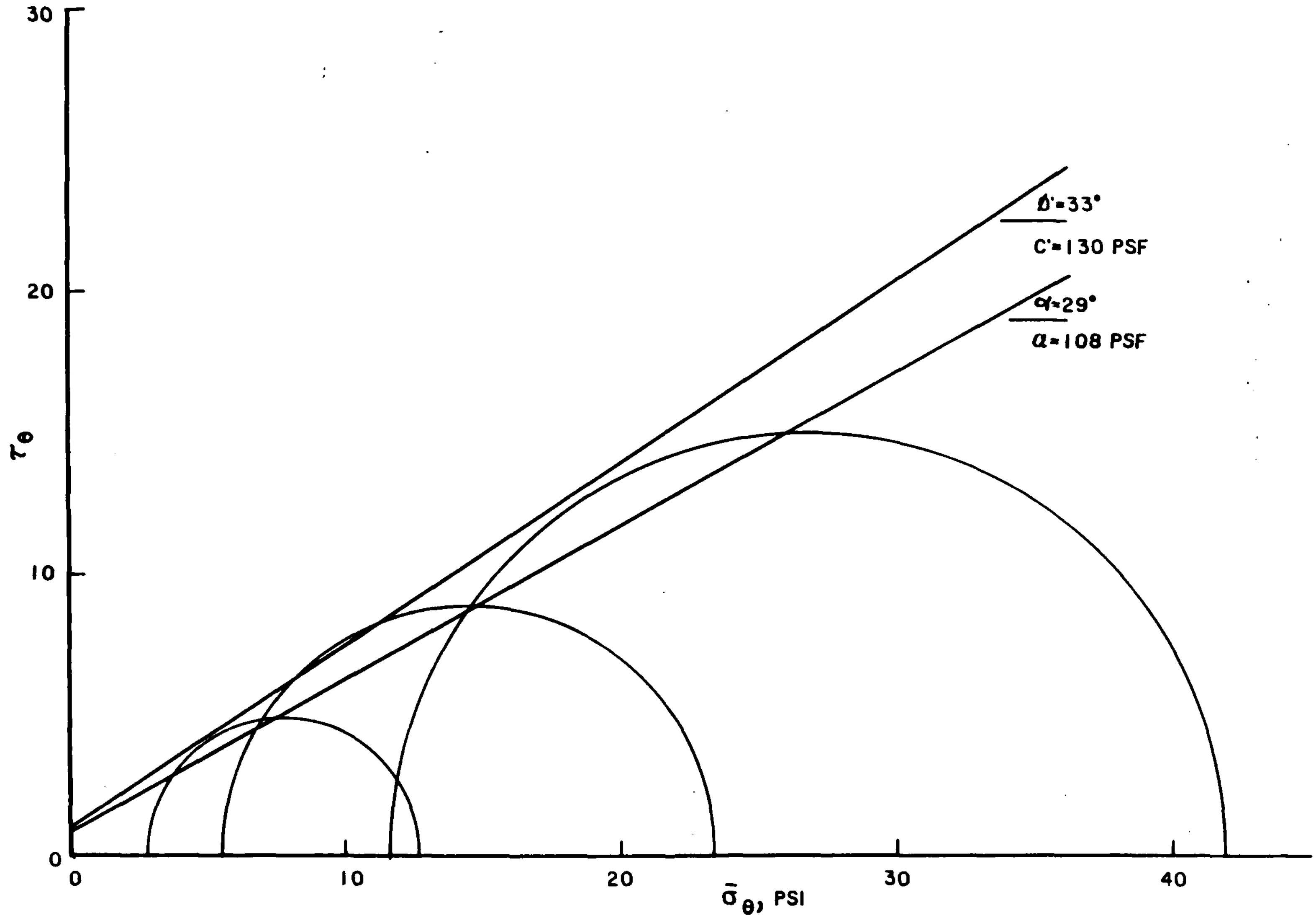


Figure B7. Composite sample $\overline{R}$ tests

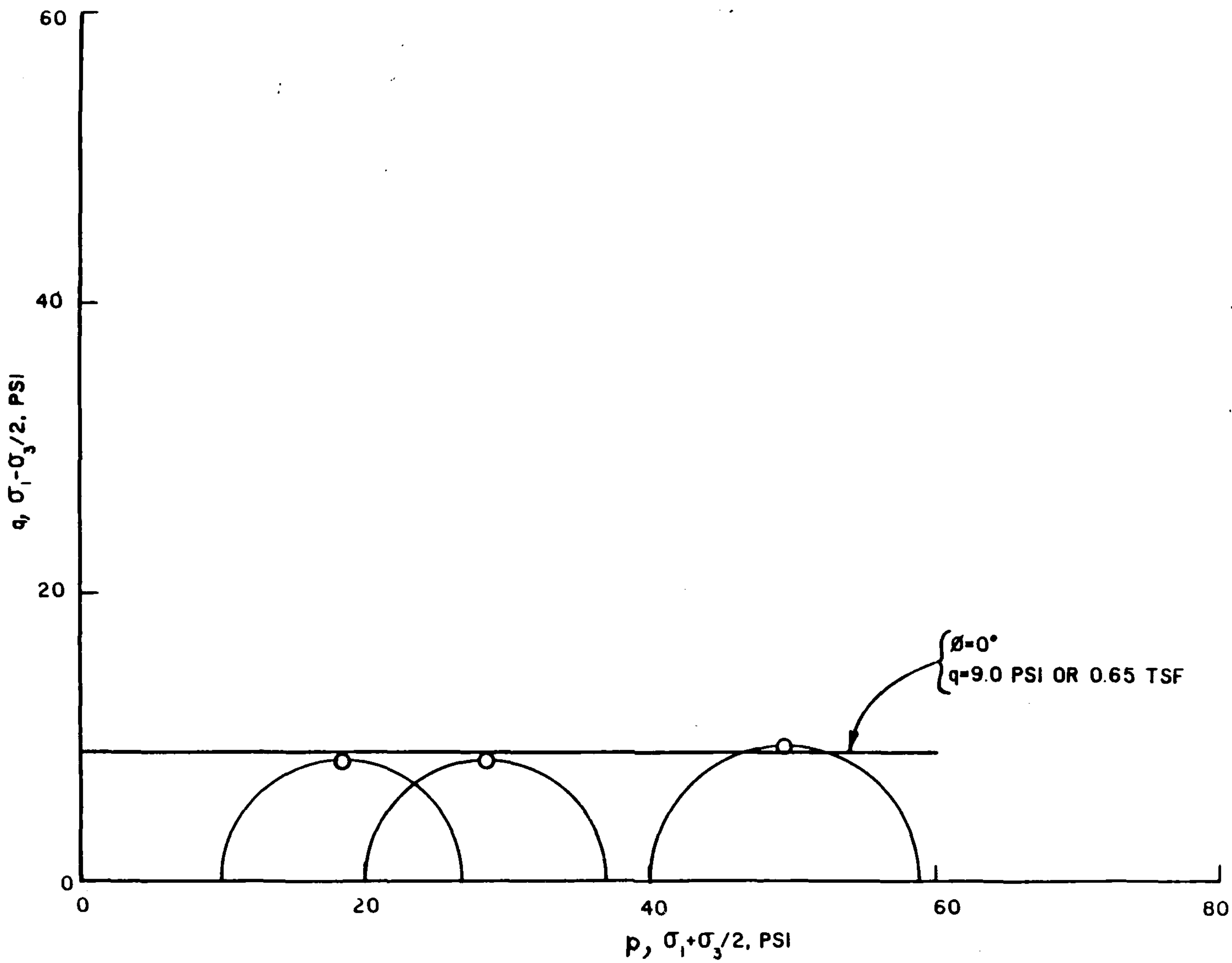


Figure B8. Composite sample Q test

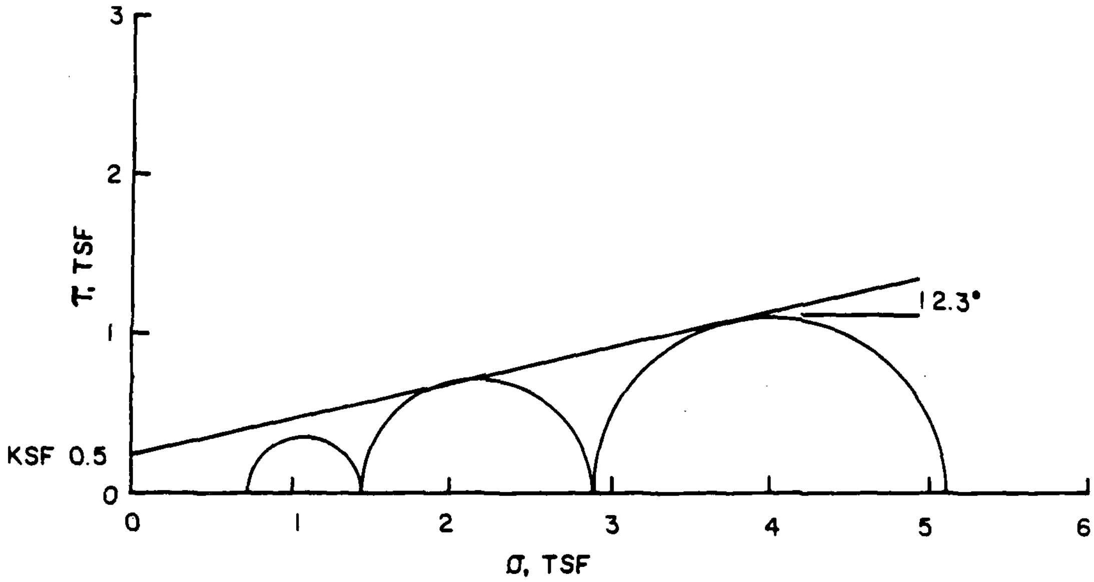


Figure B9. Total stress plot

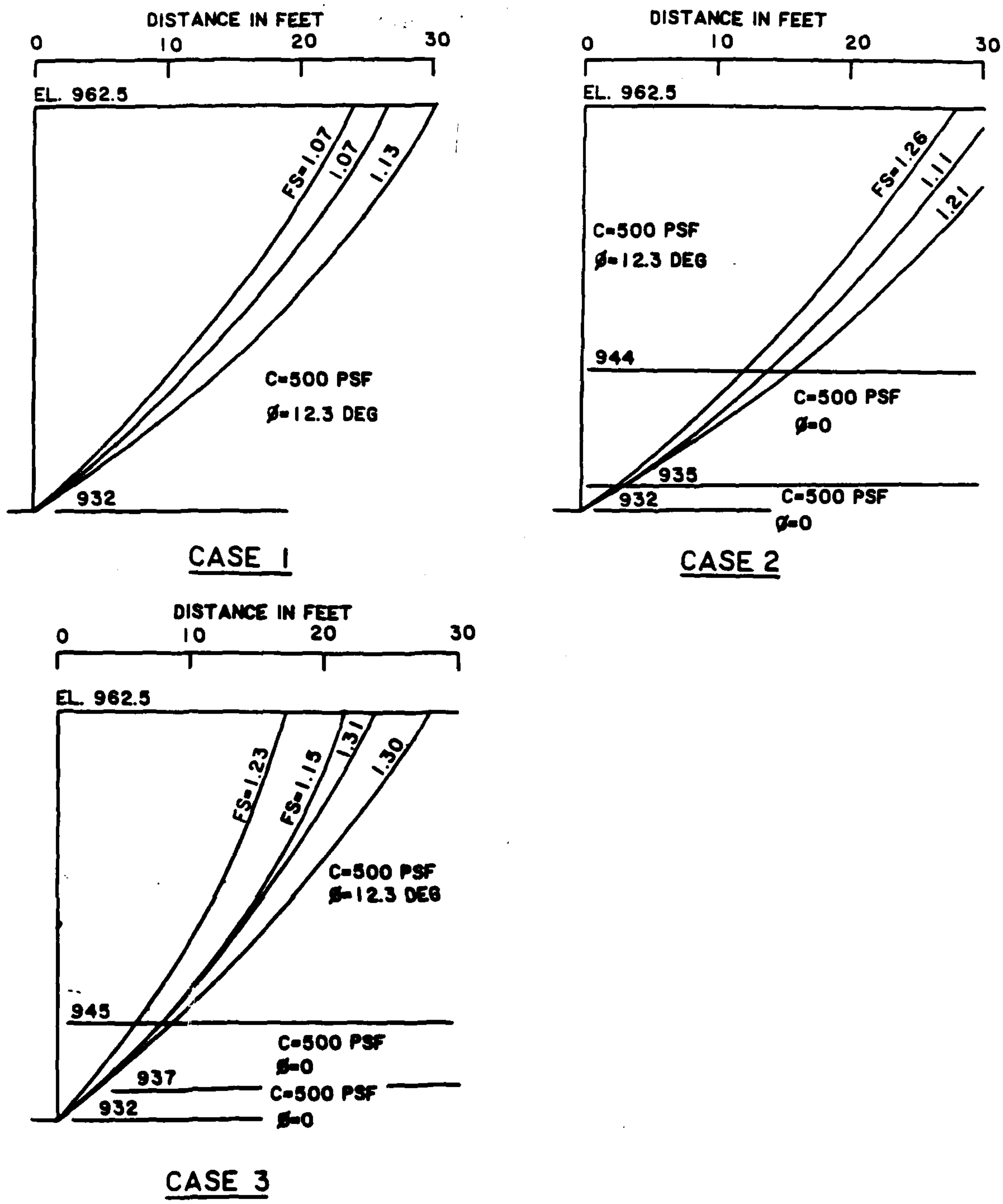
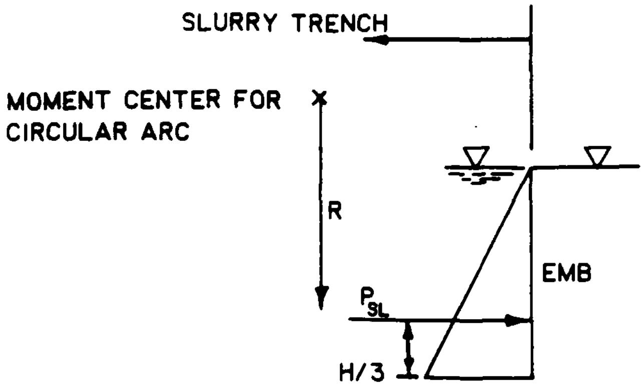


Figure B10. Slope stability computer analysis



Moment from slurry

$$M_{sl} = R\,P_{sl}$$

or

$$M_{sl} = R\,\frac{H^2}{2}\,(\gamma_{sl} - \gamma_w)$$

where

M_{sl} = moment caused by slurry, K-ct

R = moment arm, ft

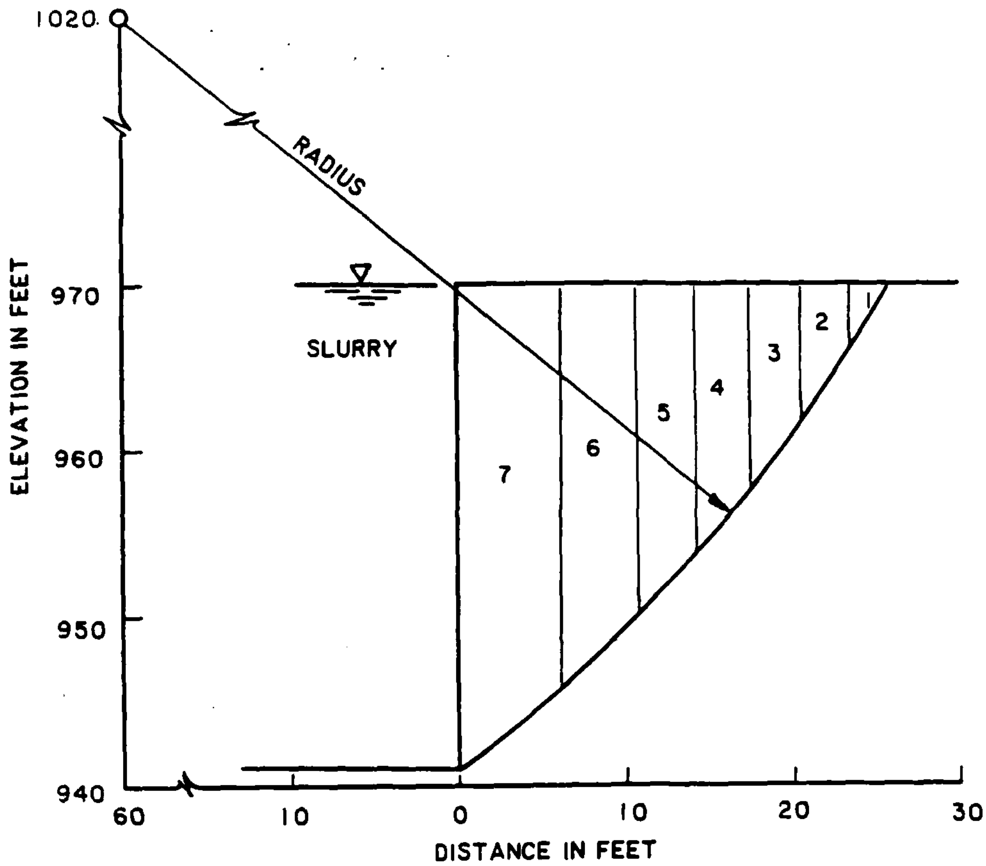
H = Depth of trench, ft

γ_{sl} = slurry density, pcf

γ_w = water density, pcf

Case	M_{res}	M_{act}	M_{sl}	FS	γ_{sl}
1	2487	2227	413	1.3	74
2	1885	1760	395	1.3	71
3	1339	1123	337	1.3	74

Figure B11.



MEASUREMENTS AND WEIGHTS											
SLICE	HORIZONTAL WIDTH (FT)	BASE LENGTH (FT)	SLICE HEIGHT (FT)			AREA OF SLICE (SQ FT)	WEIGHT (KIPS)				
			LEFT	RIGHT	AVERAGE		MOIST	SATURATED	BUOYANT	W, TOTAL SATURATED	W', TOTAL BUOYANT
1	2.4	4.9	4.0	0	2	4.8		.65	.35		.35
2	2.6	5.1	8.0	4.0	6	15.6		2.10	1.13		1.13
3	3.0	5.2	12.0	8.0	10	30		4.05	2.18		2.18
4	3.3	5.4	16	12	14	46.2		6.24	3.37		3.37
5	3.8	5.6	20	16	18	68.4		9.23	4.98		4.98
6	4.3	6.0	24	20	22	94.6		12.77	6.90		6.90
7	6.4	8.1	29	24	26.5	169.6		22.90	12.37		12.37

MOMENTS AND FORCES COMPUTATION									
SLICE	X, MOMENT ARM (FT)	W X, DRIVING MOMENT (K-FT)	MATERIAL PROPERTIES		C L COHESIVE RESISTING FORCE (K)	α	W', CONSOL. TANφ FRICTION RESISTING FORCE (K)	F, TOTAL RESISTING FORCE (K)	F 99.3', TOTAL RESISTING MOMENT (K-FT)
			C (KSF)	φ					
1	84.7	30	0.5	12.3	2.45	58	.04	2.49	247.3
2	82	93	↓	↓	2.55	56	.14	2.69	267.2
3	79.0	172			2.6	53	.28	2.88	286.1
4	75.7	255			2.7	49	.48	3.18	315.9
5	72.5	361			2.8	46	.75	3.55	352.6
6	68.5	473			3.0	44	1.08	4.09	405.3
7	63.1	781			4.05	38	2.12	6.17	612.9
Σ		2165							2487.3

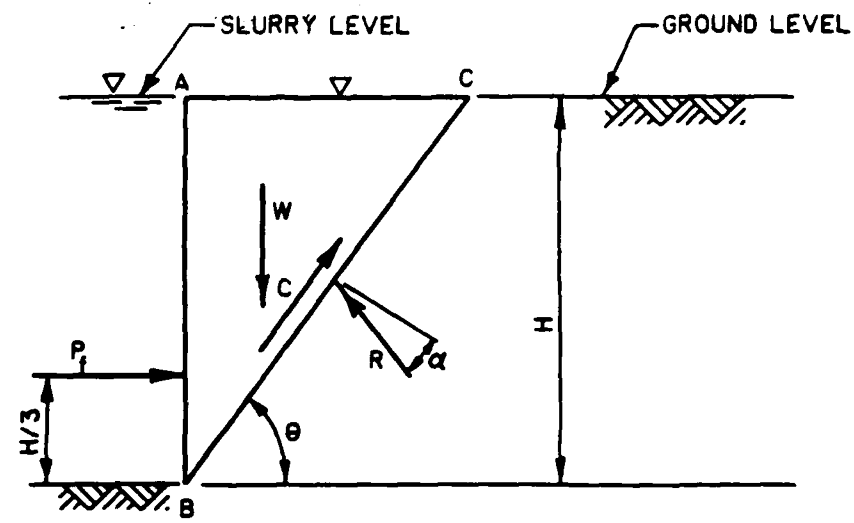
[illegible] DESIGN DATA					
MATERIAL	UNIT WEIGHT (PCF)			STRENGTH	
	γ_M	γ_S	γ_B	C(KSF)	TAN φ
OLD EMBANKMENT	130	135	73	0.5	.217

$$FS = \frac{\text{Resisting Moment}}{\text{Active Moment}}$$

$$= \frac{2487}{2165}$$

$$= 1.15$$

Figure B12. Slope stability calculation check



STABILITY OF A SLURRY TRENCH

NASH AND JONES

$$H_{cr} = 4C/\gamma_s - \gamma_{SL}$$

Then

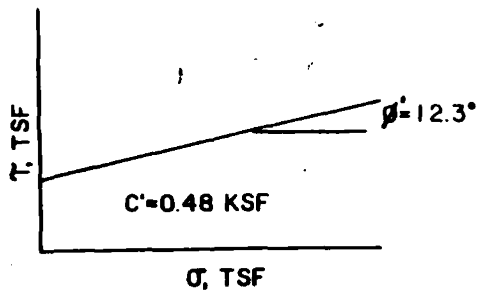
$$\gamma_{sl} = \gamma_s - \frac{4\,\dfrac{S_u}{FS}}{H_{cr}}$$

where

γ_{sl} = slurry density, pcf
γ_s = soil density, pcf
S_u = avg effective stress, psf
H_{cr} = critical depth, ft
FS = factor of safety

γ_s	c'	H_{cr}	FS	γ_{sl}	Case
135	717	30.5	1.2	56.6	1
			1.3	62.7	1
			1.4	67.8	1
135	580	30.5	1.2	71.6	2
			1.3	76.5	2
			1.4	80.7	2
135	560	30.5	1.2	73.8	3
			1.3	78.5	3
			1.4	82.5	3

Figure B13. Slope stability wedge analysis (Nash and Jones,1963)



Avg effective stress on failure plane

Vertical eff. stress = $1.0\ \sigma'_v$

Horizontal eff. stress $\left(@\ K'_o = 0.6\right)$ = $0.6\ \sigma'_v$

Avg eff. stress = $0.8\ \sigma'_v$

Then

$$S = c' + 0.8\ \sigma'_v \tan \phi'$$

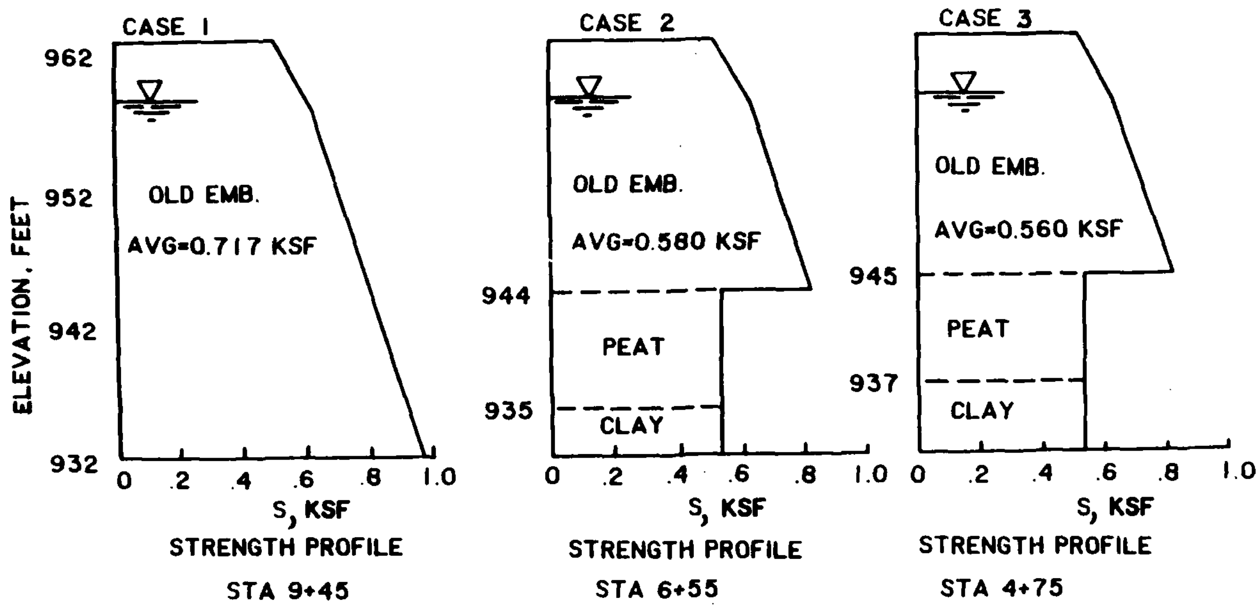


Figure B14. Undrained strength profile for statility analysis

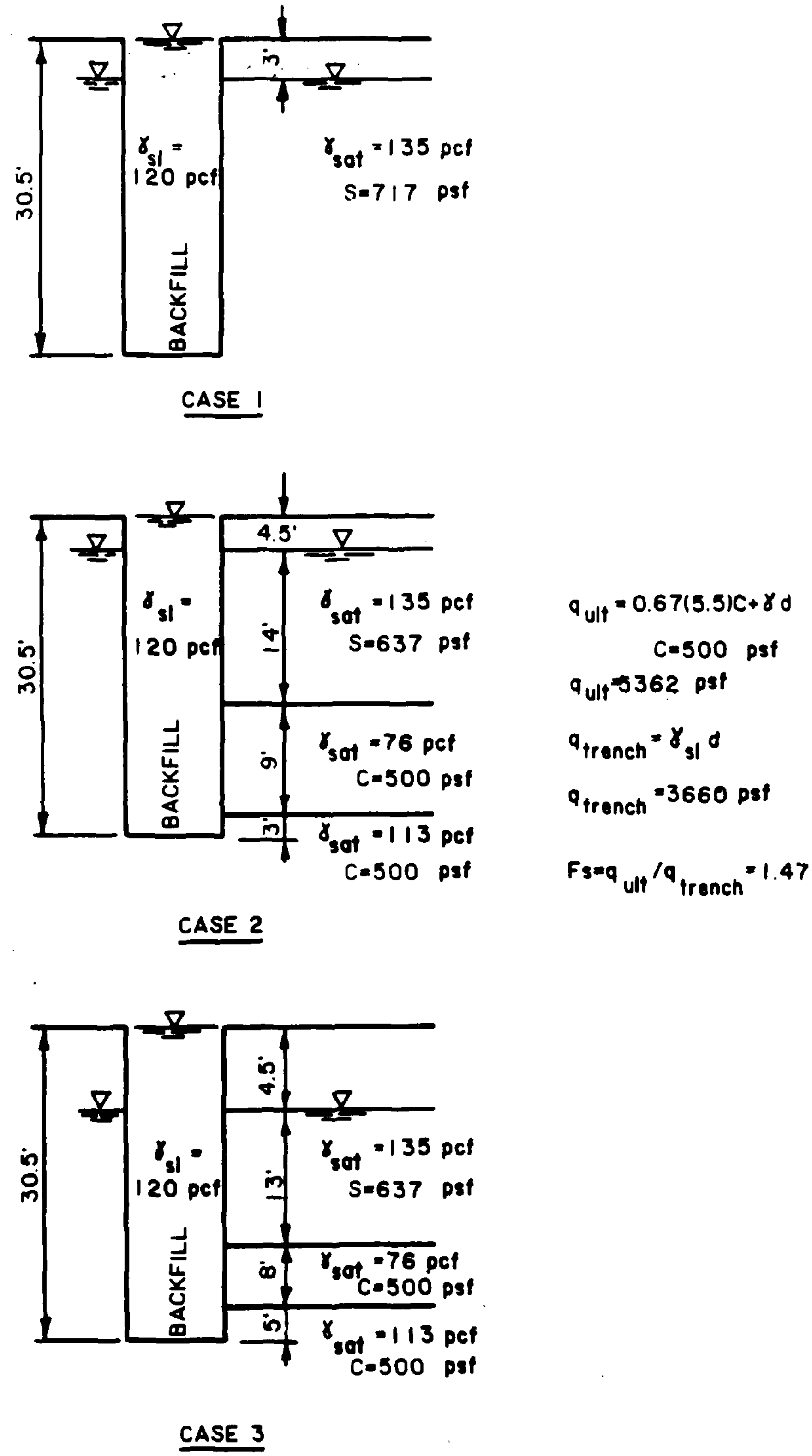


Figure B15. Slurry trench bearing capacity analysis

APPENDIX C: FINITE ELEMENT ANALYSIS OF MOHICANVILLE DIKE NO. 2

Introduction

General

1. Finite element analyses were performed for Mohicanville Dike No. 2 to supplement the results of conventional analyses using limit equilibrium methods. The analysis included thirteen individual computer runs using CON2D, a computer program for analysis of embankment dams(1). One dimensional bar elements were added to CON2D to model the proposed horizontal reinforcing layers. The principal advantage of using the finite element analysis in design is the ability to consider rationally the influences of reinforcement stiffness and layer configuration on fabric force. In addition, the finite element analysis provides estimates of embankment deformation, induced pore pressures and consolidation rate; all items that are subject to field monitoring.

2. A major objective in formulating the finite element analysis was to achieve compatibility with the limit equilibrium analyses. That is, the calculated deformations of an unreinforced section should be large as implied by the factor of safety of 0.89 computed by limit equilibrium methods. Once compatibility is achieved between the limit equilibrium and finite element analyses, the reinforcement design can be based on the following steps:

- a. The limit equilibrium analysis provides the reinforcement force (working force) needed to maintain a factor of safety of 1.3 for the soil. The working force was determined from limit equilibrium analysis to be 16.2 tons/ft.
- b. The finite element analysis is used to determine the reinforcement stiffness and layer configuration to achieve the desired working force within acceptable deformation limits.
- c. Reinforcement type, cross section and configuration is designed to carry an ultimate force equal to 1.5 times the working force.

The approach outlined above has the advantage of emphasizing the better qualities of the limit equilibrium and finite element analyses. The limit equilibrium method has considerable precedence; thus it serves as a means to calibrate the material properties used in the finite element analysis. The finite element analysis provides a relationship between reinforcement and stiffness; its accuracy depends primarily on the accuracy of the material properties assumed.

Description of problem

3. Plane strain two dimensional finite element analyses were performed for the cross section shown in Figure C1. All analyses were performed using the assumption that the cross section was symmetrical. Thus only a half section is shown in Figure C1. Important geometric boundaries are:

Geometric Boundary	Location
Top of dike	el 983, 5 ft from centerline
Beginning of working surface	el 960
Toe of dike	el 958, 90 ft from centerline
General ground surface	el 958
Rigid base layer	el 880
Rigid downstream problem boundary	400 ft from centerline

It was assumed that horizontal displacements were zero at centerline (symmetry assumption) and at the downstream boundary. All displacements were fixed along the base. Excess pore pressures were assumed to be zero at the dike and general ground surface and along the rigid downstream boundary. Two horizontal fabric layers were placed at el 960 and el 964. Analyses for one fabric layer were performed by giving the fabric layer at el 964 a zero stiffness.

4. All analyses were begun with the working surface at el 964. The actual sequence of construction consists of degrading the existing dike to create a working surface and beginning construction from el 960. It was assumed that excavation and replacement of material would have a neutral influence on the foundation, thus the analysis could be started from el 964 ft with no loss in accuracy. No consolidation was given to the rebound which would occur when the dike was removed to form the working surface at el 960. The assumed construction sequence is as follows:

Computation Step	Time from Start of Construction (yr)	Final Elevation Centerline Height above Working Surface) (Ft)	Event
0	0.0	964 (4)	Complete work to el 964
1	0.08	970 (10)	Complete work to el 970
2	0.16	978 (18)	Complete work to el 978
3	0.83	978 (18)	Winter hiatus
4	0.96	983 (23)	Final construction to el 983
5	1.96	--	Consolidation
6	11.96	--	Consolidation

Note: although the coordinates of model points were recomputed at the end of each computation step to account for displacement, the top of the dike was assumed to be finished at el 983 at the end of construction.

5. Analyses were performed using two different finite element meshes shown in Figure C2 a and b, the meshes differed primarily in the manner that the reinforcing was assumed to interact with the soil. In the first mesh no provision was made for slipping between the soil and reinforcement. In the second mesh, interface elements were used to permit slipping between the soil and reinforcement when a specified shear stress was exceeded. If no slippage occurs the analyses are, in principle, equivalent. One advantage of using slip elements is that even if slipping does not occur, the magnitude of forces at the soil-fabric interface and the mode of potential slip could be determined. One disadvantage was the increased cost and complexity added to the analysis. Only two of the thirteen analyses incorporated interface elements. The analyses are summarized on Table C-1.

Determination of Material Properties

6. The CON2D program models the soil as an elastic-plastic material. The properties used to model the soil behavior fall into three categories 1) strength; 2) stiffness; and 3) permeability. Of these, the stiffness and permeability properties were determined directly from field and laboratory test data. The strength properties were first determined from laboratory

test data, then modified as necessary to "soften" the soil to achieve compatibility with the limit equilibrium analysis. The following describes the determination of material properties.

7. Permeability Properties: The permeability of the soil is specified by its principal values in the vertical and horizontal directions (K_v and K_H). The permeabilities for the soft foundation clay and clay fill were estimated from average values presented in the LETCO report. For clay $K_v = K_H = 0.5$ ft/yr (5×10^{-7} cm/sec). For the clay fill $K_H = 1.5$ ft/yr (1.4×10^{-6} cm/sec and K_v 0.15 ft/yr ($.14 \times 10^{-6}$ cm/sec).

8. The permeability for the peat was not so easily specified. The CON2D program uses an empirical scale factor to relate permeability to void ratio:

$$K \quad \frac{e}{e_o}^{3}$$

where e is the current void ratio and e_o is the void ratio for which K_H and k_v are specified. Although the scaling relationship is widely used for clays and sands, it is not correct for peat. As an alternative, a special empirical relationship was developed relating permeability and vertical effective stress using data from a number of sites presented by Weber[2]. These data were shown in the LETCO report to closely match the permeabilities determined for the peat at the Mohicanville site. The horizontal and vertical permeabilities for the peat were assumed to be equal.

9. Stiffness Properties: The stiffness of the soil is directly controlled by the elastic parameters, bulk modulus B and Poisson's ratio , and the parameter which controls the nonrecoverable (plastic) volume change. The bulk modulus is related to the mean effective stress P' by:

$$B = \frac{(1 + e)\, P'}{K}$$

where P' is the average normal effective stress and K is the parameter describing elastic void ratio change. The parameters and K respectively are equal to the slopes of the virgin and rebound segments of compression curves plotted as e versus ln p' . The compression data for the soft clay, peat and clay fill are shown in Figure C3.

10. The value of cannot generally be determined directly from standard soil tests. However, for a given value of bulk modulus, the stiffness of the soil in a plane strain state is controlled by the quantity (1 - 2). Thus, can be used to soften or stiffen the computed stress-strain response to simulate laboratory data. After a few trial computations, = 0.25 was found to be appropriate for the soft clay and peat and = 0.45 was found suitable for the clay fill.

11. Strength Parameters: The maximum shear strength that can be mobilized in an element is controlled by the parameters P_r and M_t . These parameters are related to the traditional triaxial test effective stress parameters ' and c' by:

$$M_t = \frac{6 \sin '}{3 - \sin '}$$

$$P_r = c' \cot '$$

The parameters determined from the strength tests are:

Material	' (deg)	c' (tsf)	M_t	P_r (tsf)
Soft clay	28	0.09	1.11	0.17
Peat	32	0.08	1.29	0.13
Clay fill	32	0.09	1.29	0.14

12. In selecting strength parameters it should be kept in mind that the stress-strain model used in CON2D is formulated in terms of effective stresses; the ability to correctly predict the undrained response depends not only on the accuracy of the effective stress parameters but also on the capability of the model to predict pore pressure. To ensure that the undrained behavior was correctly modeled, the computed undrained response for an individual soil element should be compared to laboratory data.

13. A second factor to be considered is the need for compatibility between the finite element analysis and the limit equilibrium analysis discussed previously. To achieve compatibility, the dike was analyzed for the unreinforced case for a number of trials; the strength parameter of one of the materials was reduced for each trial until an "unstable" condition was

obtained. Two details were checked after each trial. First, the end-of-construction settlement of the dike was used as a measure of instability (Figure C4a). Second, it was attempted to achieve a condition whereby a contiguous group of elements were at a failure state (Figure C4b). For each trial, the reduction in strength needed to cause failure in a group of elements within one material was estimated and a new trial analysis was performed using reduced strength parameters. It was observed that once the failure was induced in a contiguous group of elements, settlements of the unreinforced dike became large. A plot of fill height and centerline settlement is shown in Figure C4. It can be seen that Trial 5 corresponds to the condition whereby the unreinforced dike is marginally stable and is thus considered to be most compatible with the limit equilibrium analysis. The strength parameters for the soft clay, peat and clay fill, after reduction using the above process are:

Material	' (deg)	c' (tsf)	M_t	P_r (tsf)
Soft clay	18	0	0.7	0
Peat	21	0.05	0.8	0.13
Fill	32	0	1.29	0

Comparisons of laboratory data and predicted undrained behavior are shown for strength tests in Figure C5 and stress-strain response in Figure C6. It can be seen that the computed undrained stress strain response gives a conservative estimate of undrained strength but gives a good approximation to the pre-yield stiffness.

14. The properties used for the analysis presented in the remainder of this report are summarized in Table C-2.

Fabric and soil fabric interface properties

15. The reinforcement was modeled as a one dimensional elastic "bar" element. The behavior of the element is completely characterized by its bar modulus which is defined as the force needed to elongate a reinforcement of unit width one strain unit (L/L) . For fabrics, the stiffness can be computed from elongation test data by:

$$K_s = P/(0.01\ b\ )$$

where

K_s = fabric modulus

P = applied force

= percent elongation

b = width of specimen

For the steel mesh, K_s can be computed from

$$K_s = AE$$

where

A = area of steel in direction of applied force per unit width of mesh

E = Young's modulus of steel

The properties of various fabric types are given in Table C-3. Four values of K_s were used in the analyses: zero stiffness to represent no reinforcement, 109 ton/ft to represent the polyester family of fabrics, 625 ton/ft to represent the Kevlar family of fabrics and 12,000 ton/ft to represent steel mesh reinforcement.

16. The behavior of the interface between soil and reinforcement was characterized by a friction angle = 27. degrees and C = 0.07 ton/ft^2. These properties are based on the assumption that slip between fabric and soil occurs as a simple slip mechanism. A second mode of behavior that must be considered is reinforcement pullout. These two modes of soil-reinforcement interaction are illustrated in Figure C7. Both modes of soil-reinforcement interaction may occur. It is important to note that for the wire mesh the pullout mode involves entirely different mechanisms than the slip mode and pullout resistance can not be assumed to be simply the sum of the slip resistances on each side of the fabric. The implications of reinforcement pullout to design are discussed herein under "Potential for Reinforcement Pullout".

Results of Analysis

General outline of behavior

17. The results of the finite element analysis are presented for the following items

a. Settlement and lateral spreading

b. Reinforcement force

c. Potential for pullout of reinforcement from soil

d. Implications of analysis to reinforcement design

e. Predicted trends for assessment of field monitoring data

18. Typical deformation patterns are shown on Figure C8a to d. The effect of the reinforcing to restrain horizontal movements in the embankment is evident. This lateral restraint appears to be the principal mechanism through which load is transferred to the reinforcement.

19. Settlement of the reinforcement, reinforcement force and pore pressure in the peat are shown in Figure C9a to d. The following may be noted

a. The stiffer the reinforcement the greater load it picks up.

b. The force in the reinforcement, lateral spreading and pore pressure increase during rapid embankment construction and decrease during subsequent consolidation settlement.

c. After-construction centerline settlement is little affected by fabric reinforcement stiffness or configurations and is likely to be about two feet (see Figure C9d).

20. The maximum reinforcement force, centerline settlement and centerline pore pressure in the peat are summarized in Table C-4. The results of the analysis are described in more detail in following sections.

Settlement and lateral spreading

21. The settlement consists of two components: 1) immediate settlements related to shear strains caused by construction; and 2) settlements related to volume changes caused by consolidation after construction is complete. The correspondence between reinforcement stiffness and immediate settlement, shown in Figure C10, clearly shows that reinforcement has a moderate influence on settlement. The computed end of construction settlement using steel reinforcement is 1.03 ft which represents only a 46 percent

reduction from the settlement of 1.89 ft for the unreinforced dike. Consolidation settlement is virtually unaffected by type of reinforcement and post construction settlement should be on the order of 2 ft for all reinforcement systems.

22. Unlike settlement, lateral spreading is greatly influenced by reinforcement stiffness (Figure C10). Reinforcement is effective in reducing embankment spreading because it provides stiffness at the base of the embankment level. Thus, embankment spreading corresponds directly to the amount of stretch in the reinforcement. It may be seen from Figure C10 that the use of steel reinforcement reduces the lateral spreading to less than an inch versus nearly one foot of spread computed for the unreinforced case.

Reinforcement force

23. The reinforcement force for the various configurations analyzed are summarized in Table C-4. As already noted, the greater the reinforcement stiffness the greater the load mobilized by the reinforcement. Some features of double-layer reinforcement scheme warrants special comment. First, the two layers are not equally effective in taking up load. For the Kevlar reinforcement, the upper layer takes only 74 percent as much load as the bottom layer. For the steel, the top layer take a mere 24 percent as much as the bottom. Second, using two layers does not necessarily provide twice as much resisting force. For the Kevlar, use of two layers only provides and additional 20 percent reinforcing force while combining two layers of steel provides no additional force.

24. Another important observation is that while a reduction in reinforcement force occurs during consolidation, the amount of the reduction depends on both reinforcement stiffness and soil properties. For example, the reduction in the steel reinforcement force during post construction consolidation is computed to be 35 percent whereas an 15 percent reduction is computed for the Kevlar reinforcement (See Table C-4). This difference is not solely related to reinforcement however, because only a 12 percent reduction is computed for the steel reinforcement using two layers. No reduction in force is computed for steel reinforcement using the severe

foundation conditions represented by trial 6 (Analysis D, Table C-4). One imnplication of the uncertain relationship between consolidation and reinforcement force is that it may not be possible to use consolidation as an effective means of controlling reinforcement or pullout force should they approach the allowable maximum values during construction.

Potential for Reinforcement Pullout

25. The nature of interaction between the soil and reinforcement can be determined from the shear stress on each side of the reinforcement as determined from the interface elements (See Figure C7). Also, the interaction can be inferred from the reinforcement force distribution. A flat distribution indicates that little force is transferred from the reinforcement to the soil, a condition that occurs in the sliding mode. A steep gradient implies a large transfer of load from the reinforcement to the soil; a condition that can only exist for the pullout mode. Finally, the interaction between reinforcement and soil can be inferred from its influence on the stress distribution in the soil. If the pullout mode predominates a sharp discontinuity in the shear stress should be observed (see Figure C11). All of the above features indicate that the sliding mode predominates near the centerline of the embankment, but the pullout mode predominates in its outer two-thirds.

26. The distribution of total pullout resistance required is shown in Figure C12. The maximum pullout required is seen to be 0.31 tons/ft^2. As a reference, note that the average pullout resistance required to transfer to the embankment the required 16.2 tons/ft reinforcement load (determined by limit equilibrium analysis) would be 0.2 tons/ft^2. It should also be noted that the pullout resistance depends on the distribution of reinforcement force. The maximum force computed by the finite element analysis occurs at the embankment centerline in part because a symmetrical cross section was used. Should the maximum reinforcement force occur near the 1/3 point because of non-symmetrical deformation, as implied by the limit equilibrium analysis (see Figure C13), the required pullout resistance could be as great as 0.45 tons/ft^2. The normal force on the reinforcement and the frictional resistance available for pullout resistance is also shown on Figure C12. The pullout resistance is shown computed based on two pullout tests on clay. The strength for 0.1 inch pull represents the pullout force needed to initiate significant movement between the steel mesh and soil. About 3 inches of

pullout is needed to mobilize the ultimate pullout resistance. Therefore, while there appears to be ample ultimate pullout resistance, some slip between the reinforcement and soil could occur.

Conclusions

Implication of Results to Reinforcement Design

27. The results of the finite element analysis clearly suggest that a stiff reinforcement system is needed to mobilize the required force to maintain the factor of safety for the foundation at 1.3. For example, the relationship between mobilized reinforcement force and stiffness shown on Figures C14 shows that to achieve the working force of 16.2 tons/ft, a reinforcement system having a stiffness of at least 12,000 tons/ft is required. From the strength versus stiffness relationship plotted for various reinforcement materials it can be seen that only steel can supply the needed stiffness. This is true even if multiple layered systems are used.

28. The results also suggest that use of multiple layers is not efficient. For the fabrics, use of two layers effectively doubles the stiffness of the system, which is insufficient to achieve the needed force. Use of two layers of steel appears to ineffective because the upper layer takes up little additional load. The combined load of the two layers is the same at the load mobilized in the single layer case; thus, the added layer of steel would not increase the effective stiffness of the system. Importantly, the use of layers does not reduce significantly the maximum force in an individual layer, thus each layer would have to be designed to carry the ultimate reinforcement force.

Expected trends

29. Settlement and pore pressures computed, for the steel reinforcement case, at locations of proposed instrumentation are presented on Figures C15 and C16. The vertical and horizontal deformation profiles previously presented for steel reinforcement in Figure C9d also provide predictions for behavior at proposed instrument locations. Note that these predictions correspond to an idealized cross section that approximates conditions between stations 6+00 and 9+00 where the soft foundation soils extend to the greatest depths. At other locations, where the soft materials are not as thick, deformations and reinforcement force may not be as large as those

predicted. It is anticipated that immediate settlement, lateral spreading and reinforcement force would increase somewhat with increasing thickness of soft clay. Long term consolidation settlement will likewise depend on the thickness of the clay although, because consolidation settlements are expected to be greatest in the peat layer, the peat thickness may control long term settlement. Pore pressures are not expected to be greatly affected by the thickness of the soft layers although the rate of consolidation may be slower in areas where the foundation is most compressible.

30. If the dike is constructed to el 983 in one season, the predictions would have to be revised. In general, the major effect of not having the eight month winter hiatus would be to increase the end of construction pore pressure about 20 percent and the maximum reinforcement force about 15 percent. The end of construction settlement would not differ greatly from that computed by assuming the winter hiatus because the settlement that would have occurred during the eight months of consolidation would probably be offset by a large proportion of shear displacement. The initial rate of consolidation would be somewhat greater if one construction season is used but the time required for complete settlement would still be in excess of 11 years.

References

1. Duncan, J. M., D'Orazio, T. B., Chang, C-S., Wong, K. S., and Namiq, L. T., 1981. "CON2D: A Finite Element Computer Program for Analysis of Consolidation," Report No. UCB/GT/81-01, College of Engineering, University of California, Berkeley.

2. Weber, W. G., 1969. "Performance of Embankments Constructed Over Peat", Journal of The Soil Mechanics and Foundations Divisions, American Society of Civil Engineering, Vol 95, No. SMI, pp 53-76.

Table C-1

Summary of Mohicanville Dike No. 2 Finite Element Analysis

Analysis No.	Mesh no.	Interface Elements	Soil[1] Properties	Fabric Layers	Fabric Type[2]
1	MD3	No	5	0	None
2	MD3	No	5	1	P
3	MD3	No	5	1	K
4	MD3	No	5	1	S
5	MD3	No	5	2	K
6	MD3	No	5	2	S
3a	MD5	Yes	5	1	K
4a	MD5	Yes	5	1	S
A	MD3	No	6	0	None
B	MD3	No	6	1	P
C	MD3	No	6	1	K
D	MD3	No	6	1	S
E	MD3	No	6	2	K
F	MD3	No	7	0	None

(1) Soil property set 5 represent conditions compatible to those assumed for limit equilibrium analyses. Soil property sets 6 and 7 represent more severe conditions than assumed for limit equilibrium analyses

(2) P = polyester-type (KS = 109 ton/ft); K Keular-type (KS = 625 ton/ft); S = Steel mesh (KS = 12000 ton/ft), where KS is the reinforcement stiffness

Table C-2

Properties Used For Finite Element Analysis of Mohicanville Dike No. 2

Property	Material		
	Soft Clay[5]	Peat	Embankment Fill[4]
Physical characteristics:			
Total Density[1] (ton/ft)	0.055	0.035	0.063
Degree of Saturation (%)	100.	100.	90.0
Stiffness Properties:			
(2)	0.2	1.3	0.05
K	0.045	0.2	0.007
(2)	0.25 0.45*	0.25	0.45
Strength Properties:			
M_t	0.7 0.6*	0.8	1.29
P_r (ton/ft^2)	0.0	0.13	0.0
Permeability Properties:			
K_H (ft/yr)	0.5	$K_H = K_v$ $= 10^{-a}$; a = 0.42 +2.35 log $'_v$ (3)	1.5
K_v (ft/yr)	0.5		0.15

(1) Density of water = 0.032 tons/ft^3
(2) Average value used
(3) From data given in reference 2
(4) Properties of pre-existing fill and new embankment differed only in their initial preconsolidation pressure and void ratio.
(5) Values with * were used for trial 6.

Table C-3

Strength and Stiffness Properties of Potential Reinforcement Materials[1]

Reinforcement	Material	Tensile Strength (ton/ft)	Elongation at Break (%)	K_s (ton/ft)
Stabilenka 400	Polyester	13.8	11.2^+	123.
Stabilenka 200	Polyester	7.8	7.2^+	108.
Firestone Type 1	Polyester	12.0	$17.^+$	71.
Owens Corning	Fiberglass	18.0	3.3	546.
Knytex Proform	Fiberglass	18.0	4[2]	450.
Knytex Proform	Kevlar 29	18.0	4[2]	450.
Burlington 59384	Kevlar 49	25.2	2	1260.
Steel Mesh (65 ksi yield)				
W6 * W 4		11.7	0.22	5318.
W 8.5 * W 4		16.6	0.22	7545.
W 12 * W 6		23.4	0.22	10636.

(1) Summarized from Table A3
(2) Estimated

Table C-4

Summary of Results for Finite Element Analysis

Analysis	Reinforcement	Results[1] Computation Step	1	2	3	4	5	6
		Final Elevation (ft)	970	978	978	983	--	--
		Time (yr)	0.08	0.16	0.83	0.96	1.96	11.96
Trial 5 Soil Properties:								
1	None	d =	0.23	0.78	1.05	1.89	*[3]	*
		T =	--	--	--	--	--	--
		u =	0.27	0.64	0.56	0.63	*	*
2	P	d =	0.22	0.76	1.02	1.58	*	*
		T =	0.60	1.9	1.8	2.5	*	*
		u =	0.28	0.66	0.57	0.63	*	*
3	K	d =	0.21	0.70	0.97	1.4	*	*
		T =	2.1	6.5	5.9	8.1	*	*
		u =	0.29	0.68	0.61	0.68	*	*
4	S	d =	0.18	0.54	0.82	1.03	1.40	2.56
		T =	5.6	15.2	12.7	16.0	14.0	10.4
		u =	0.32	0.73	0.62	0.69	0.58	0.22
5	K(2 layers)[2]	d =	0.21	0.65	0.95	1.27	*	*
		T[2] =	1.4/1.6	3.9/4.4	3.3/4.2	4.3/5.8	*	*
		u =	0.27	0.71	0.62	0.70	*	*
6	S(2 layers)[2]	d =	0.19	0.57	0.85	1.08	1.47	2.67
		T[2] =	1.3/4.5	4.1/11.8	2.7/10.6	3.2/13.6	3.4/13.2	2.4/12.4
		u =	0.32	0.76	0.65	0.73	0.60	0.23

(Continued)

(1) d = centerline settlement at elevation 960 (ft)
T = maximum reinforcement force (tons/ft)
u = centerline pore pressure in peat (tons/ft^2)
(2) Force in top layer (el. 964)/force in bottom layer (el. 960)
(3) * denotes analysis not performed

Table C-4 (concluded)

			Results(1)					
		Computation Step	1	2	3	4	5	6
		Final Elevation (ft)	970	978	978	983	--	--
Analysis	Reinforcement	Time (yr)	0.08	0.16	0.83	0.96	1.96	11.96
3a (inter-face element used)	K	d =	0.21	0.67	0.89	1.20	1.53	*
		T =	2.2	6.7	6.2	7.4	7.5	*
		u =	0.29	0.69	0.62	0.71	0.62	*
4A (inter-face element used)	S	d =	0.18	0.54	0.79	0.98	1.09	1.90
		T =	5.6	15.4	13.1	16.1	13.6	12.7
		u =	0.32	0.74	0.65	0.73	0.43	0.20
Trial 6 Soil Properties:								
A	None	d =	0.70	2.75	2.94	5.16	*	*
		T =	--	--	--	--	--	--
		u =	0.23	0.62	0.60	0.69	*	*
B	P	d =	0.68	2.33	2.58	4.52	*	*
		T =	0.7	3.0	2.9	4.3	*	*
		u =	0.23	0.63	0.61	0.72	*	*
C	K	d =	0.66	2.01	2.32	3.3	*	*
		T =	2.8	8.2	8.1	12.6	*	*
		u =	0.25	0.67	0.64	0.73	*	*
D	S	d =	0.61	1.63	1.96	2.61	3.06	5.13
		T =	6.3	16.9	16.1	21.6	21.2	21.9
		u =	0.28	0.68	0.67	0.76	0.70	0.38
E	K(2 layers)(2)	d =	0.66	1.99	2.32	3.16	*	*
		T(2) =	1.4/2.2	3.8/6.6	3.4/6.6	4.4/9.3	*	*
		u =	0.26	0.69	0.65	0.74	*	*

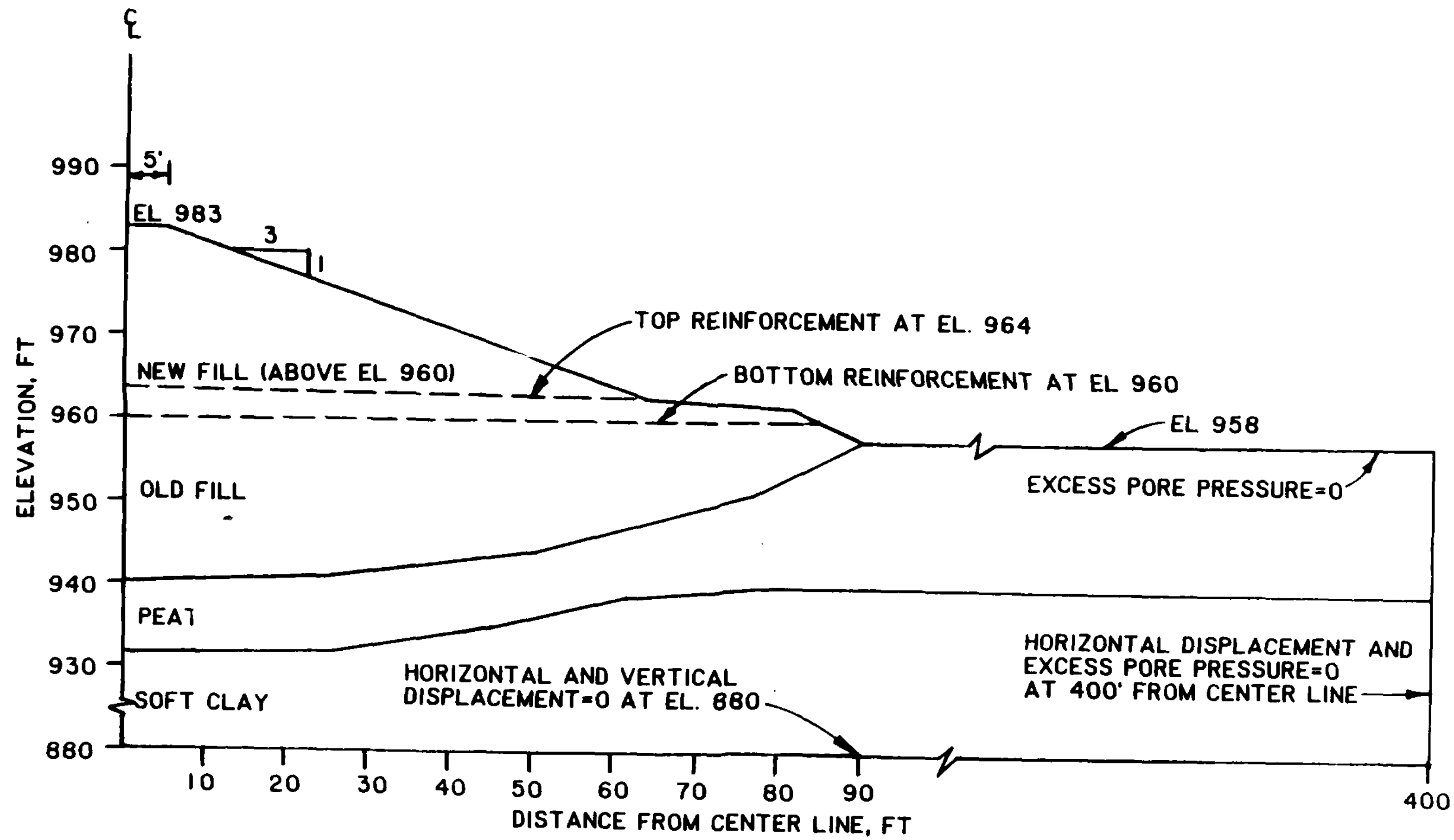


Figure C1. Cross section used for finite element analysis

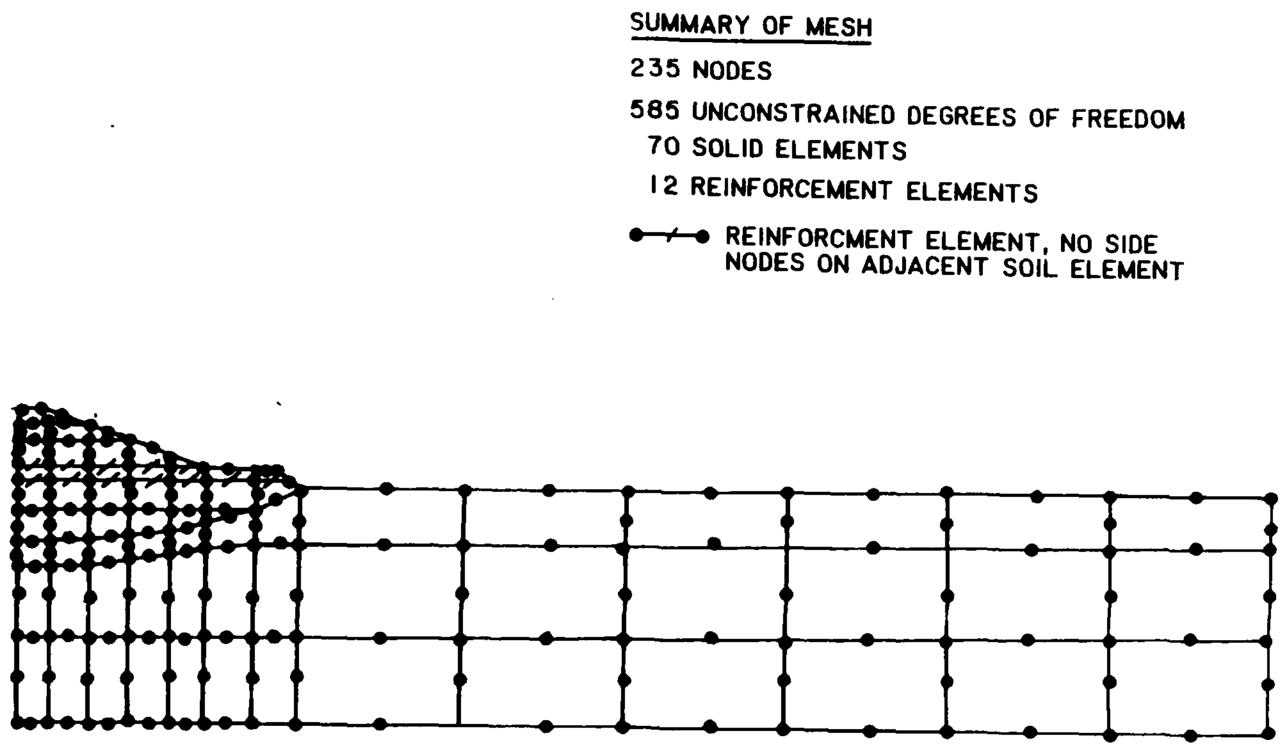


Figure C2a. Mesh used for analysis without interface elements (mesh No. MD3)

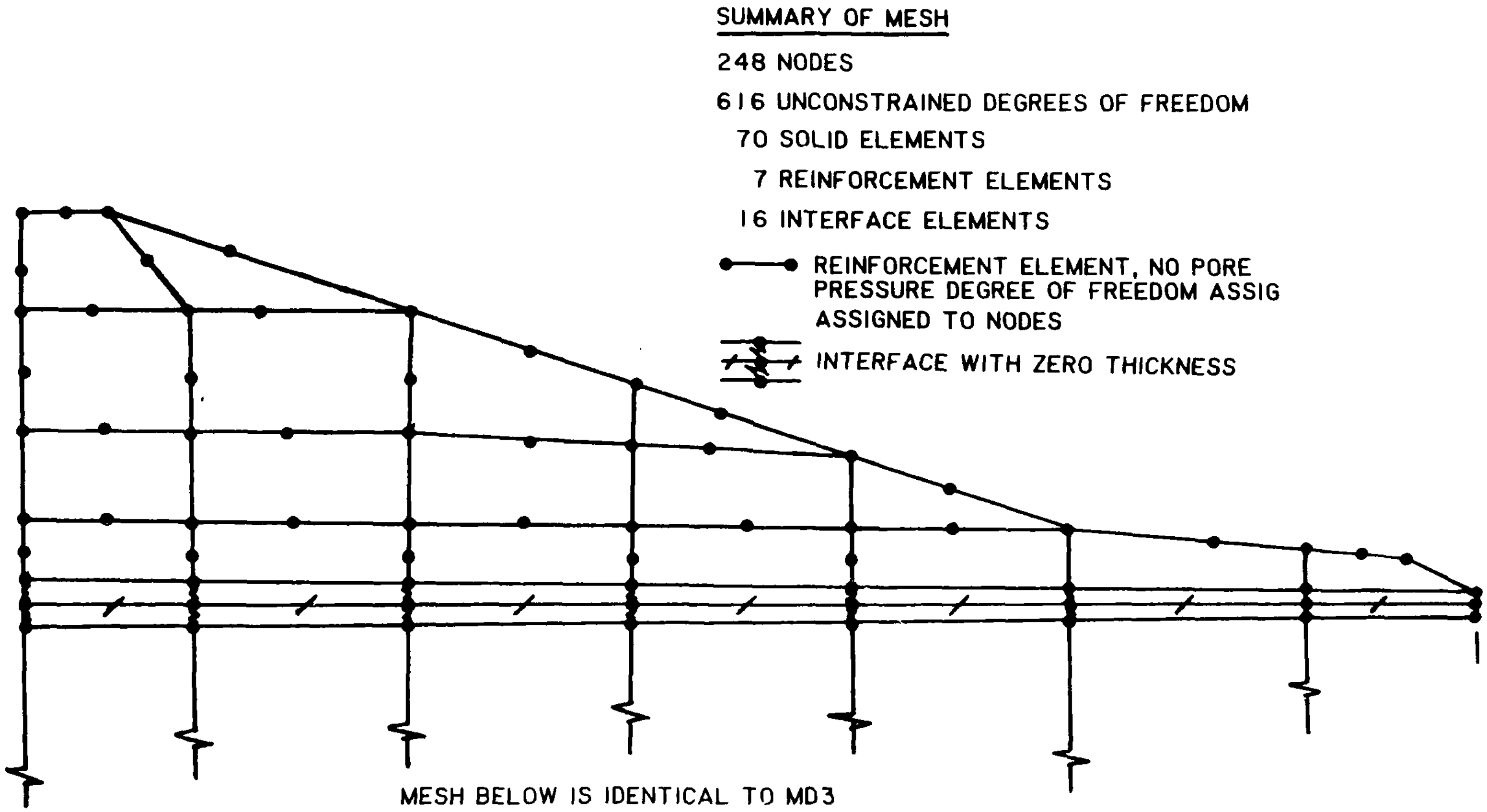


Figure C2b. Mesh used for analysis with interface elements (mesh No. MD5)

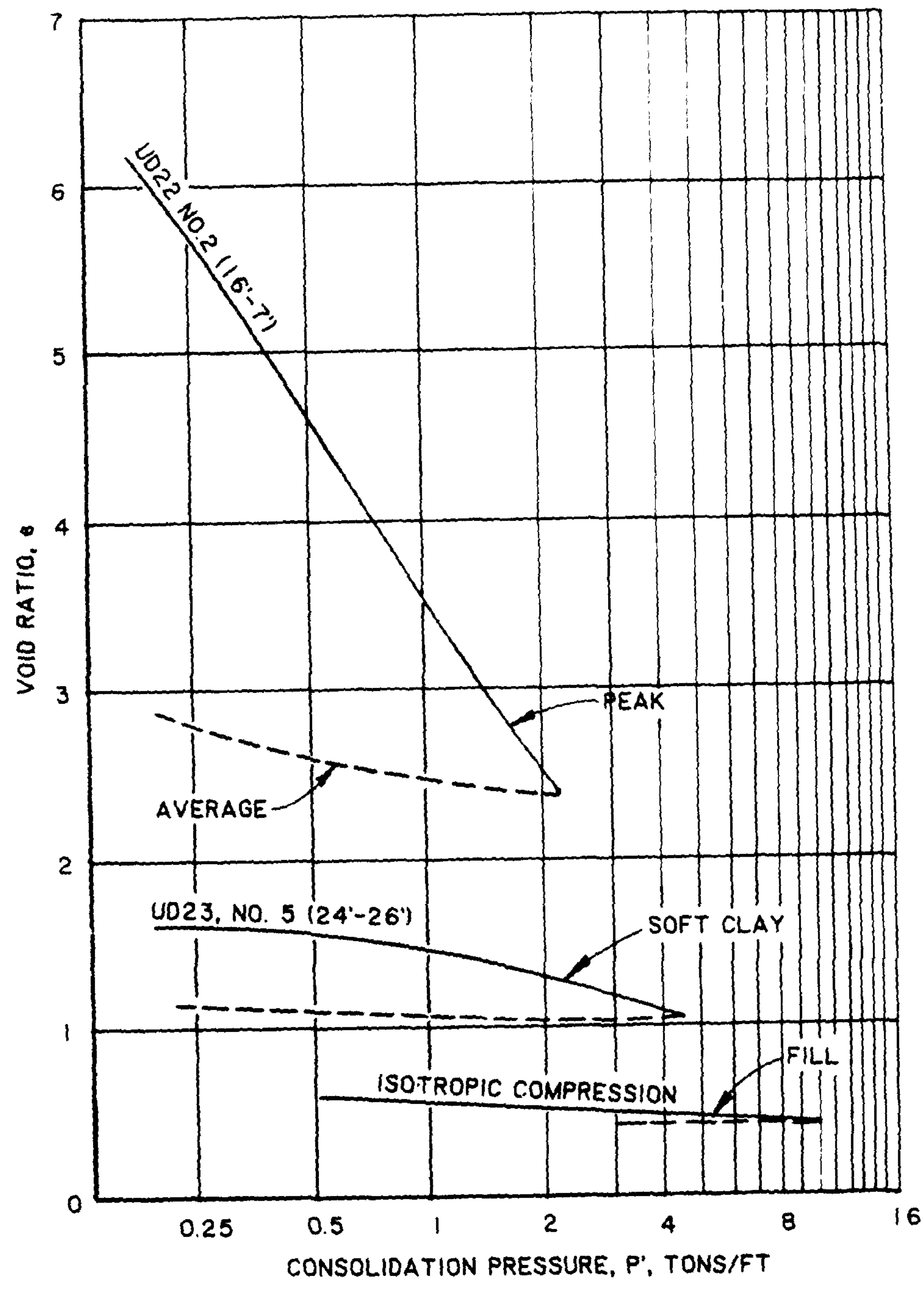


Figure C3. Compression curves for embankment and foundation soils

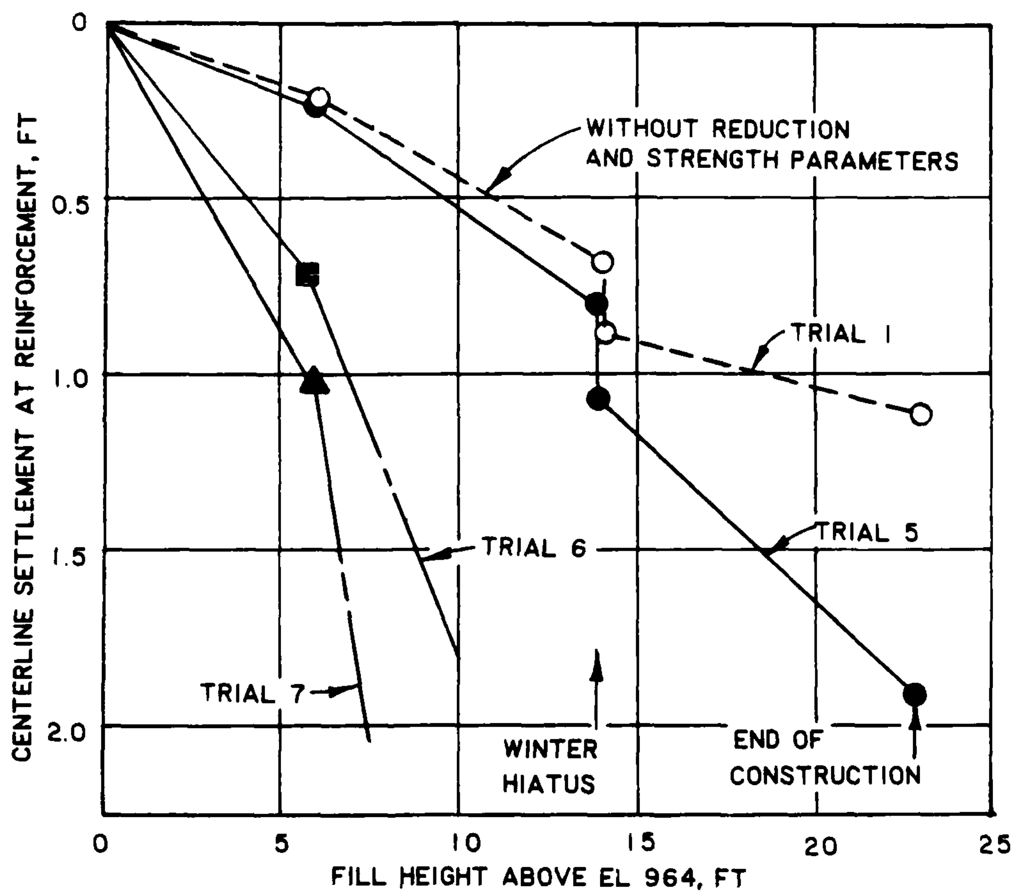


Figure C4a. Settlement at el 960 versus fill height for Trial Nos. 1, 5, 6 and 7.

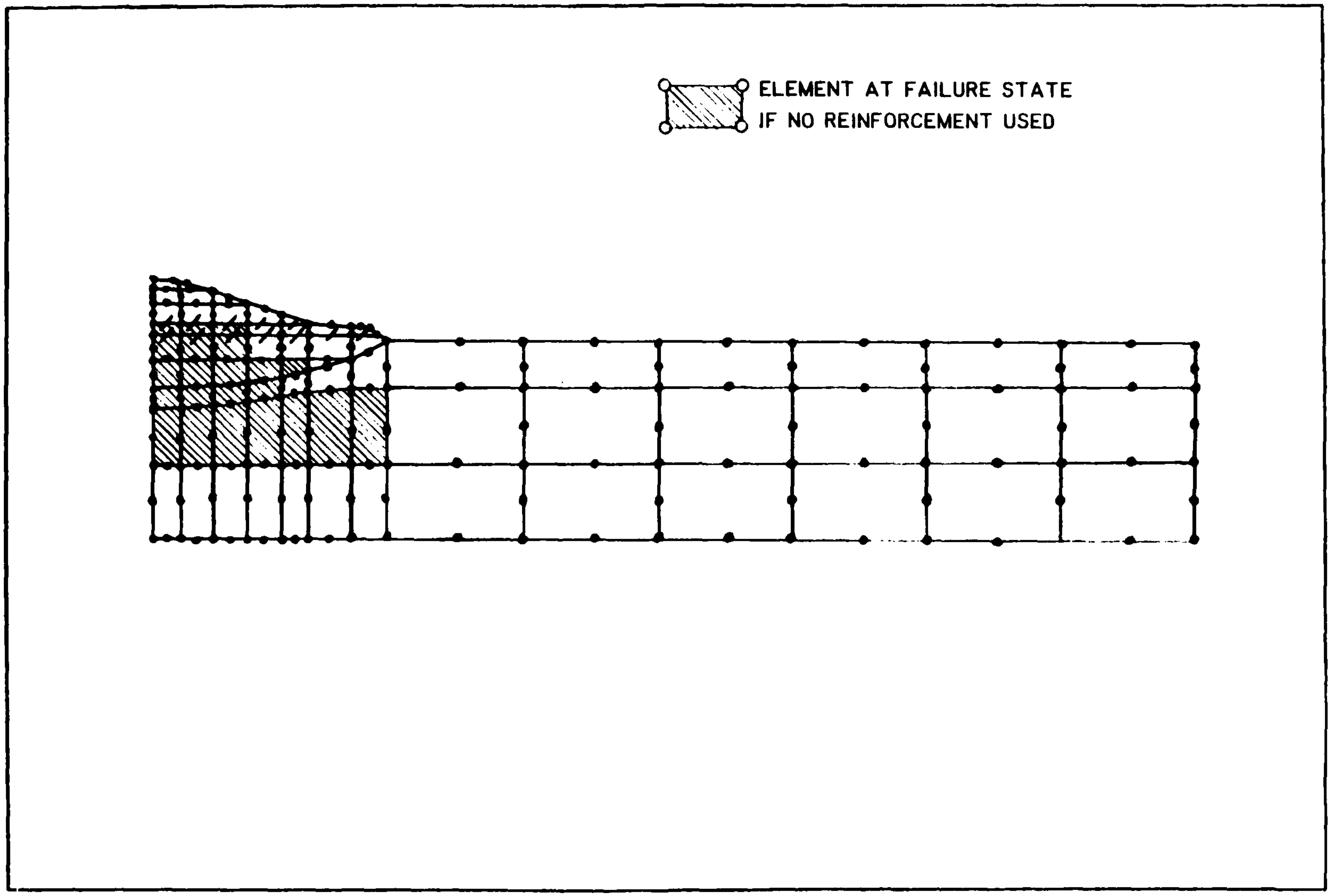


Figure C4b. Elements in state of failure at end of construction for trial 5 without reinforcement

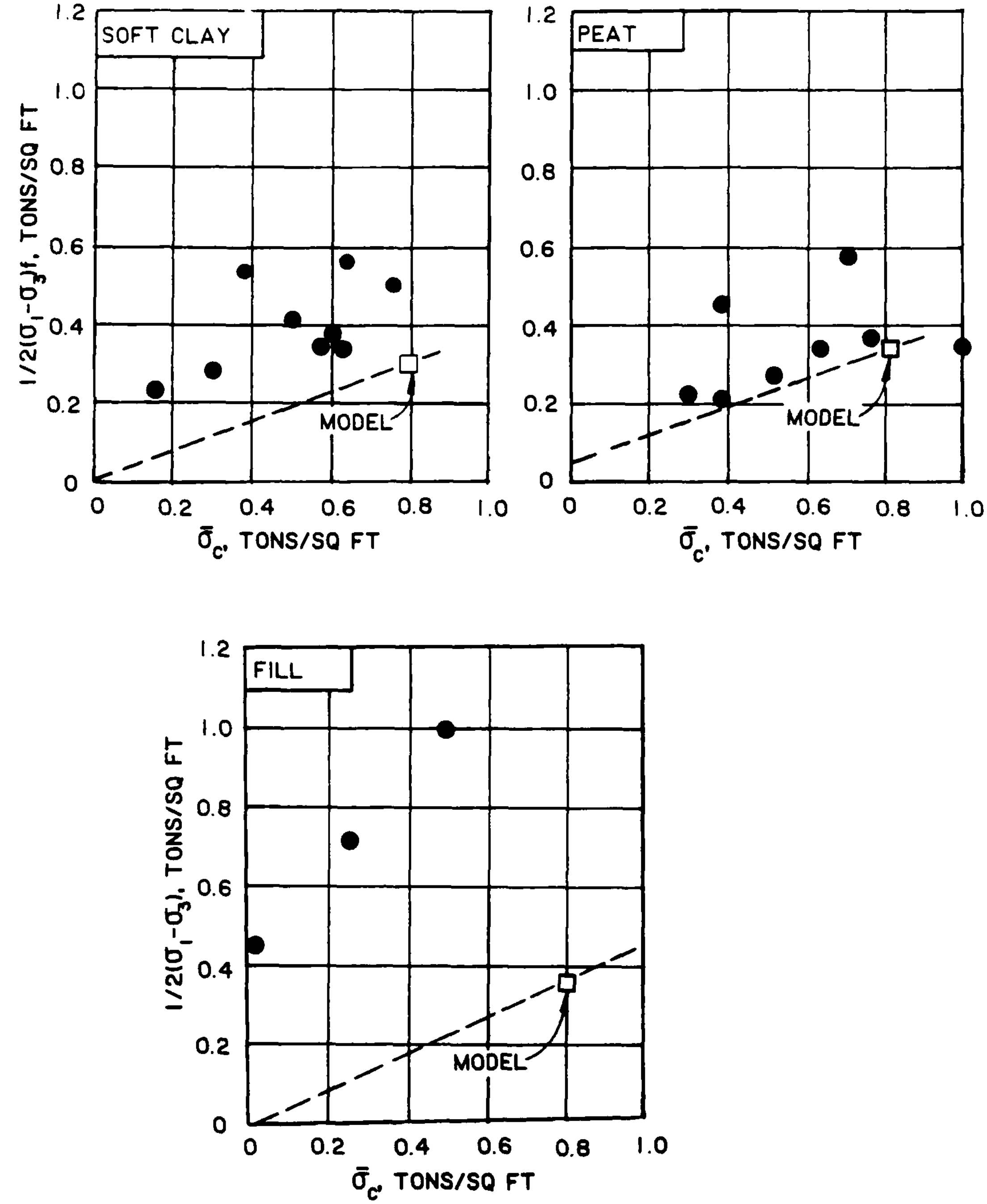


Figure C5. Comparison of laboratory test data from Q and R tests, strength properties used for analysis

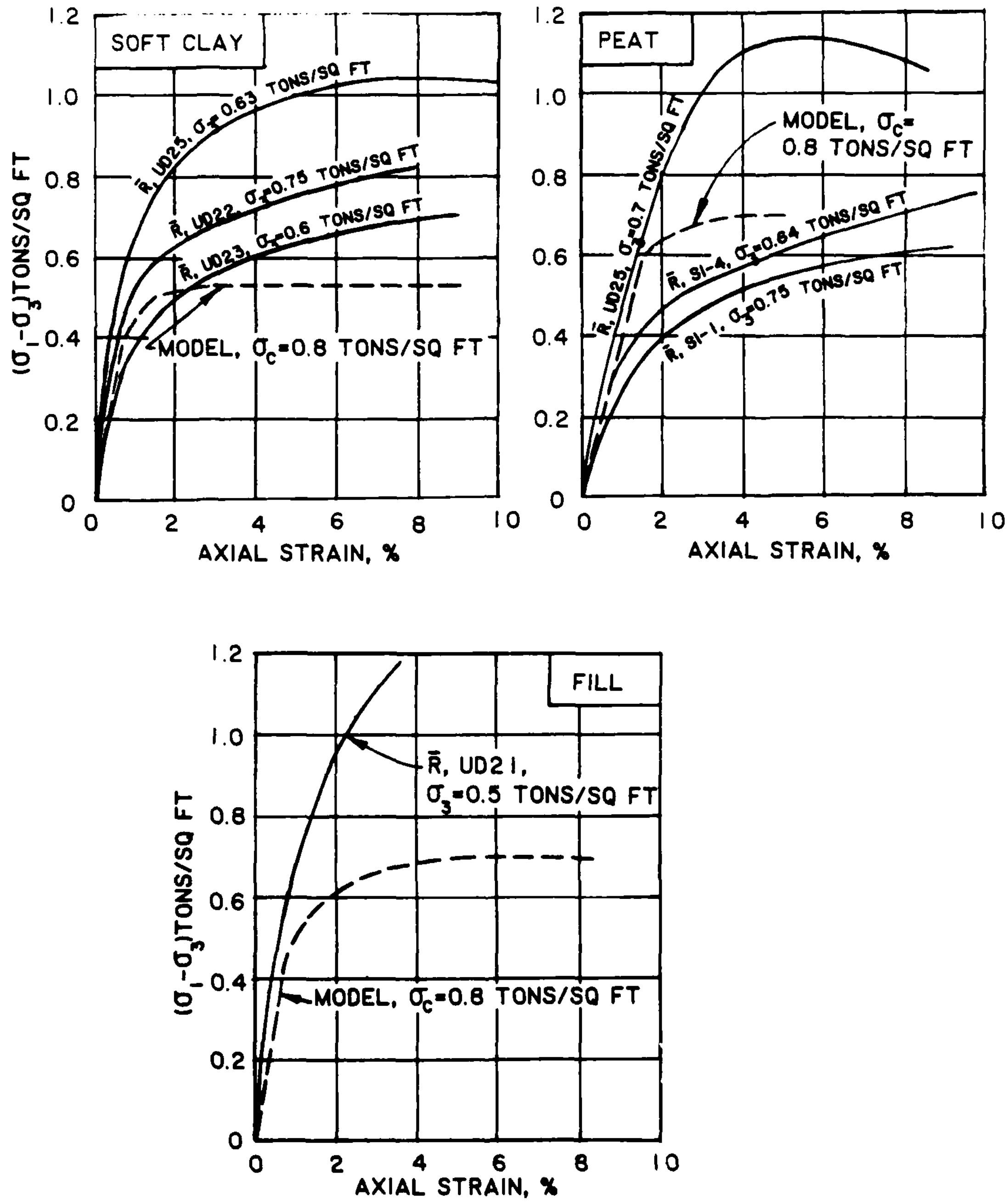


Figure C6. Comparison of stress-strain curves from undrained (R) tests and computed stress-strain curve for one-element simulations

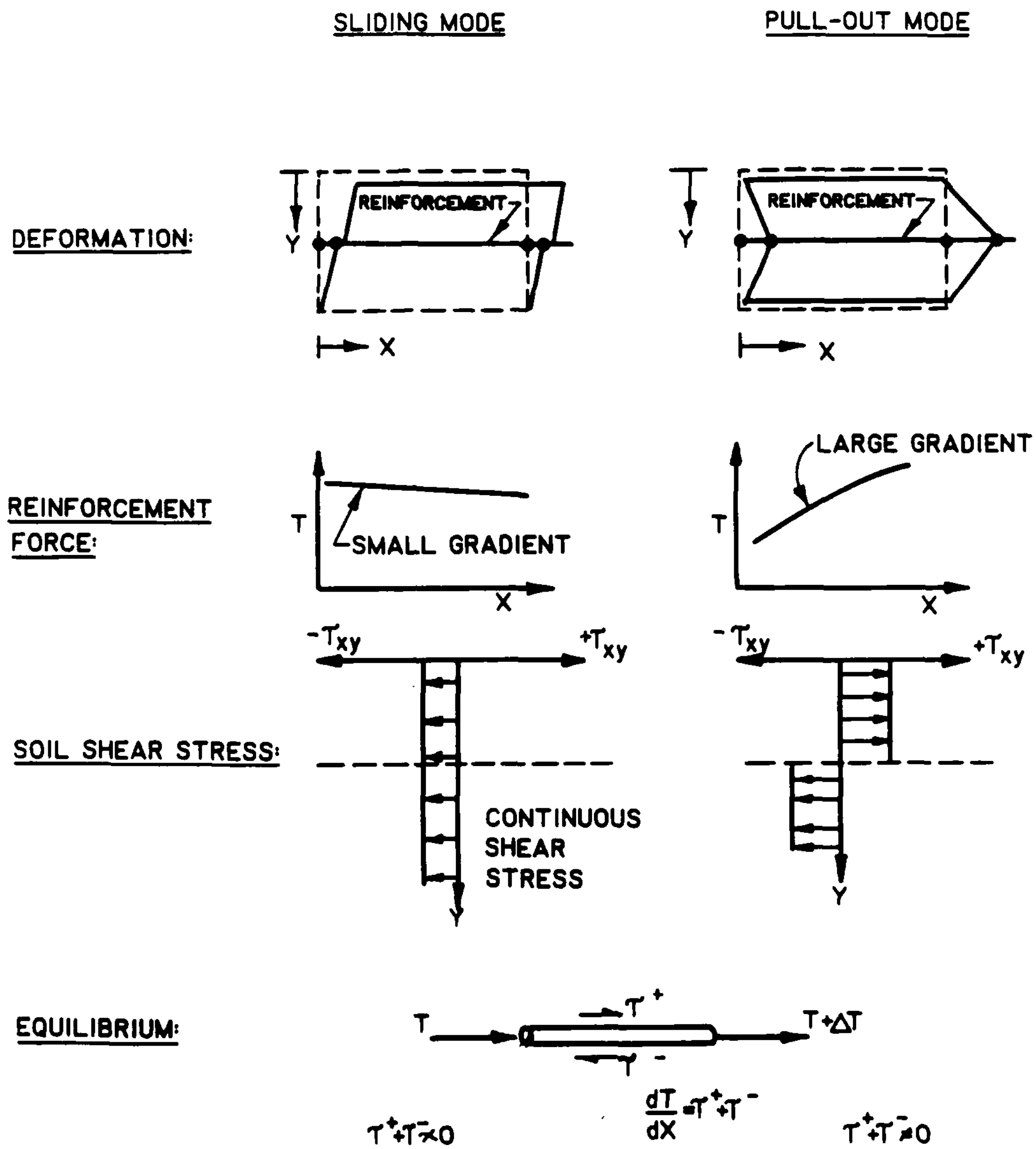


Figure C7. Slip and pullout modes of soil-reinforcement interaction

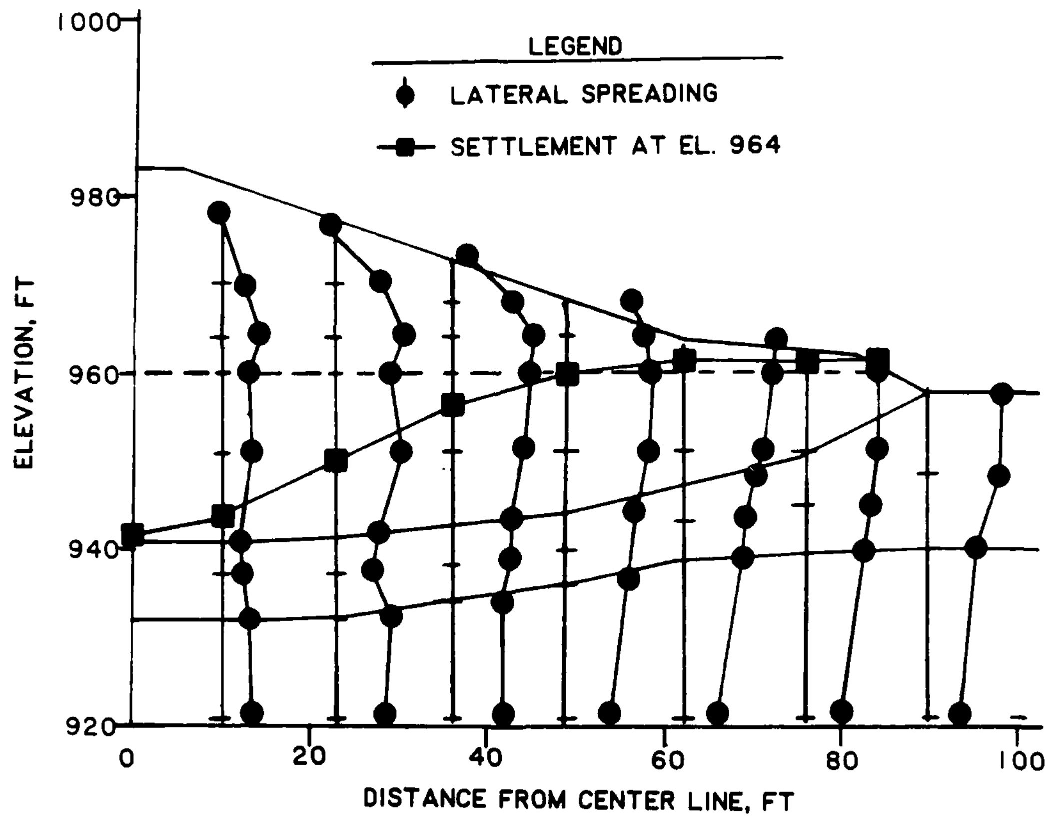


Figure C8a. Settlement and lateral spreading of nonreinforced embankment

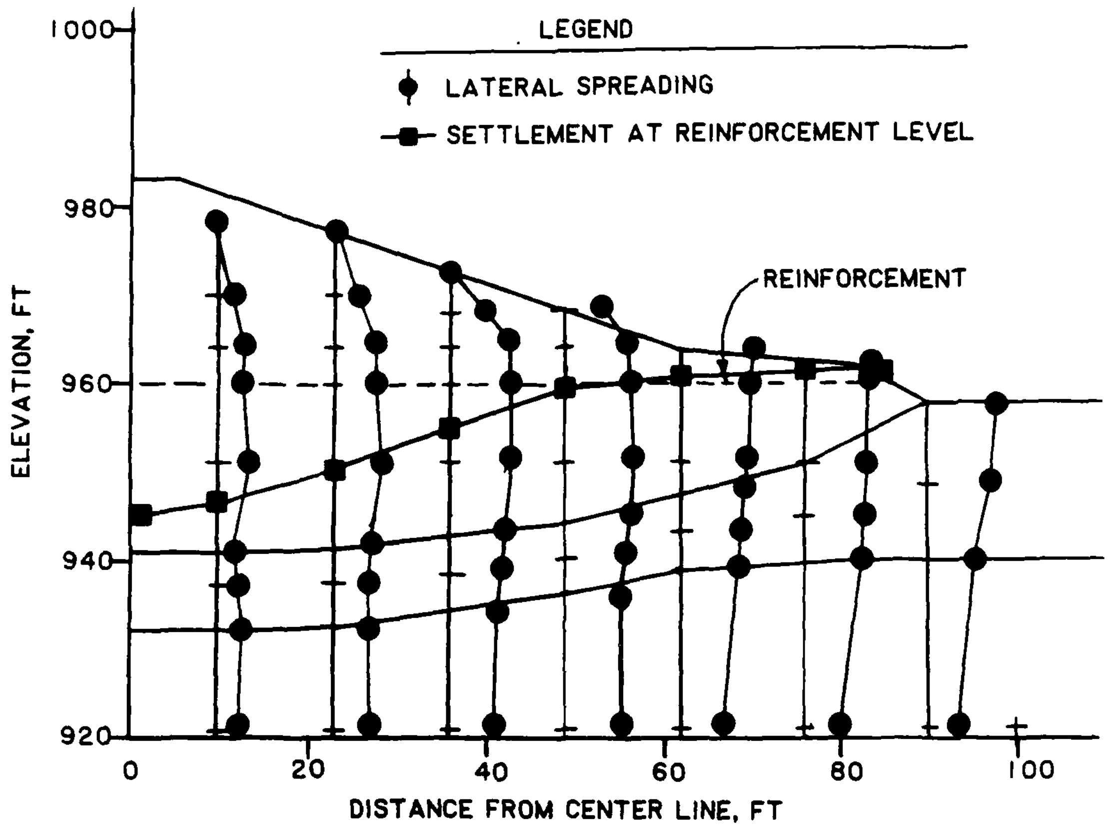


Figure C8b. Settlement and lateral spreading of embankment with K_s = 109 ton/ft

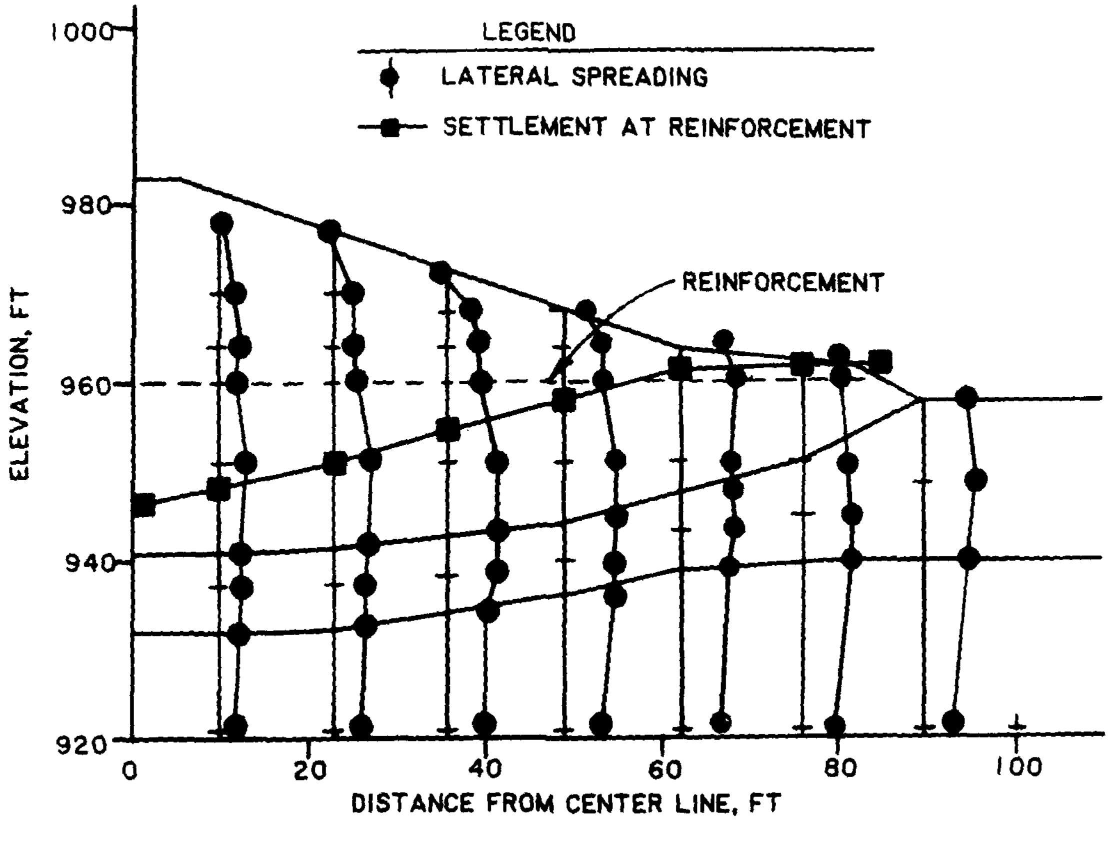


Figure C8c. Settlement and lateral spreading of embankment with K_s = 625 ton/ft

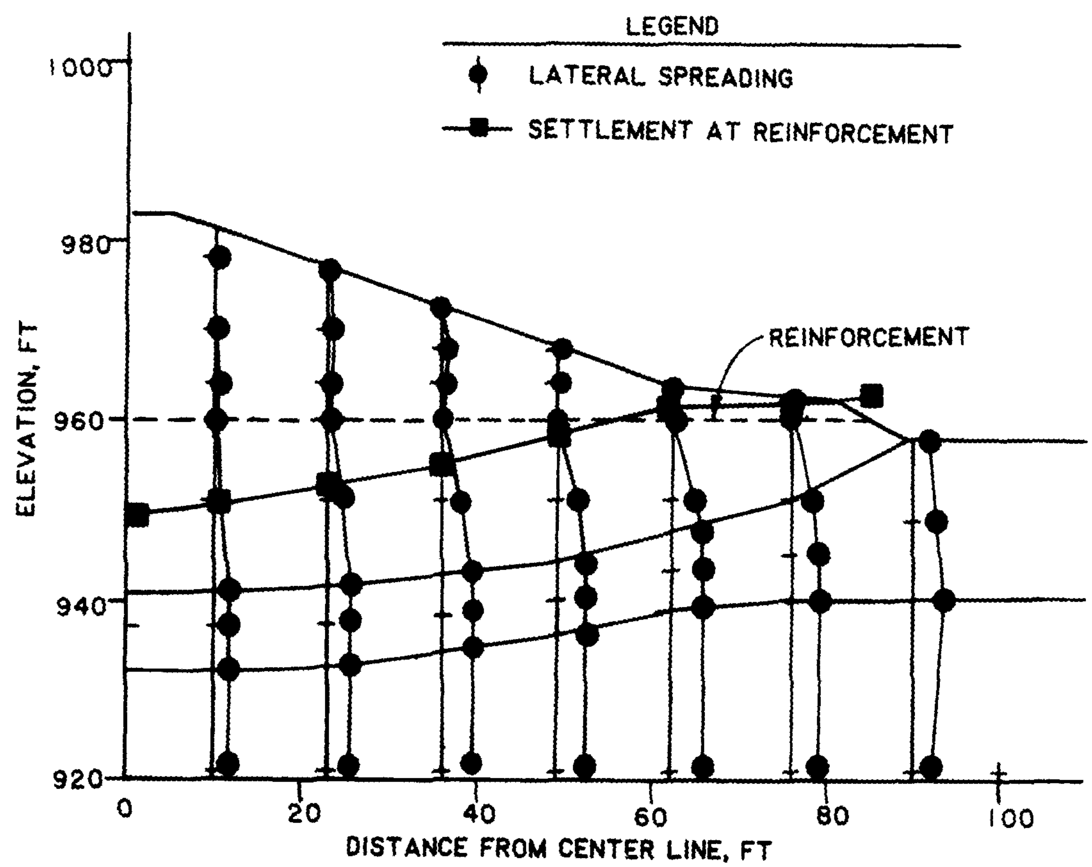
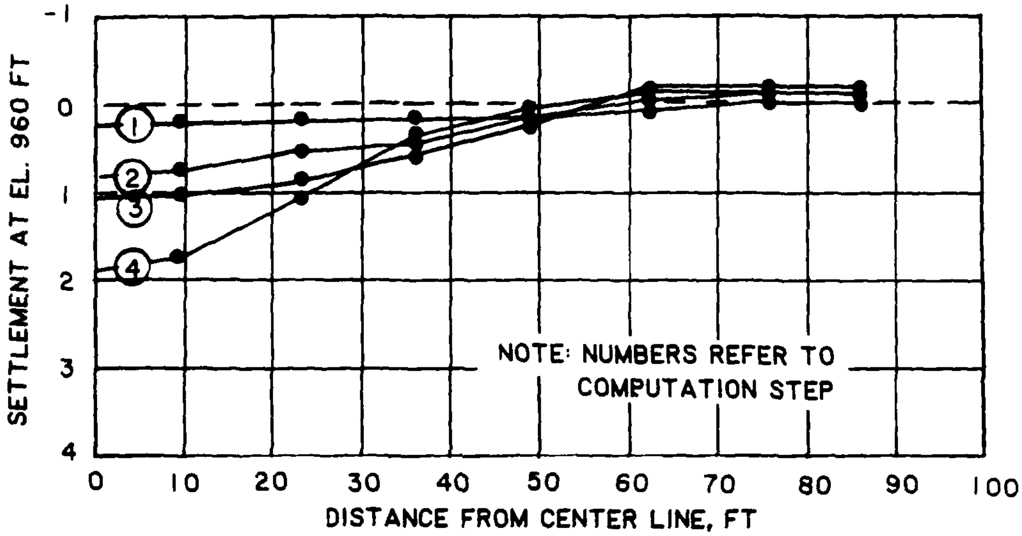


Figure C8d. Settlement and lateral spreading of embankment with K_s = 12,000 ton/ft



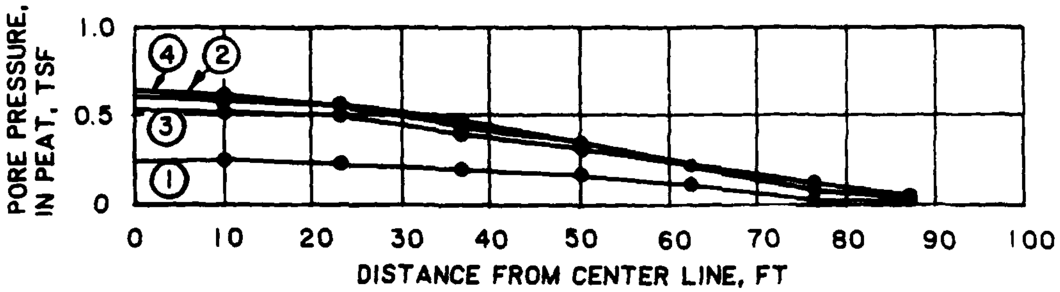


Figure C9a. Settlement, pore pressure profiles for unreinforced embankment for each construction step

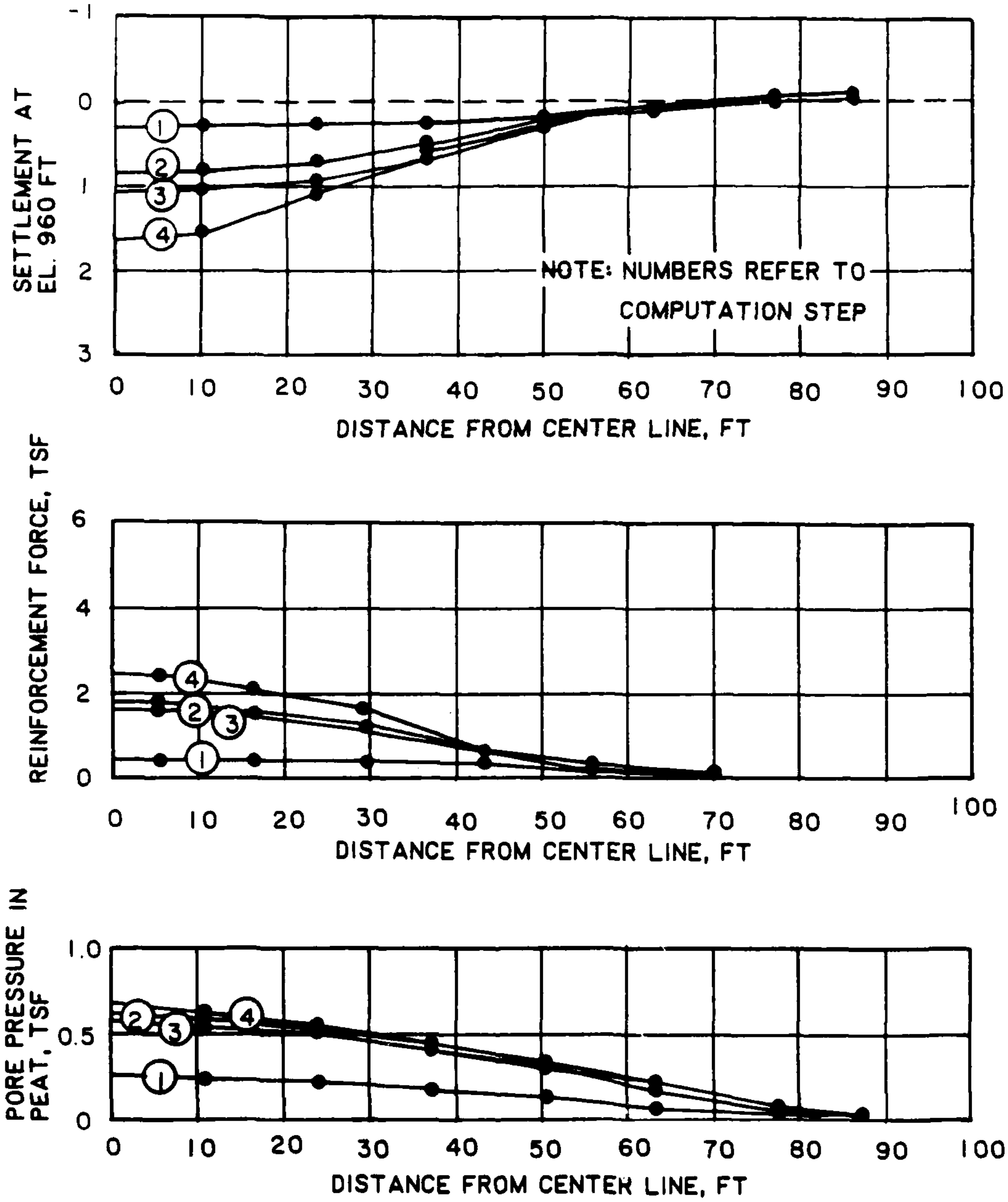


Figure C9b. Settlement, pore pressure, and reinforcement force profiles for K_s = 109 ton/ft

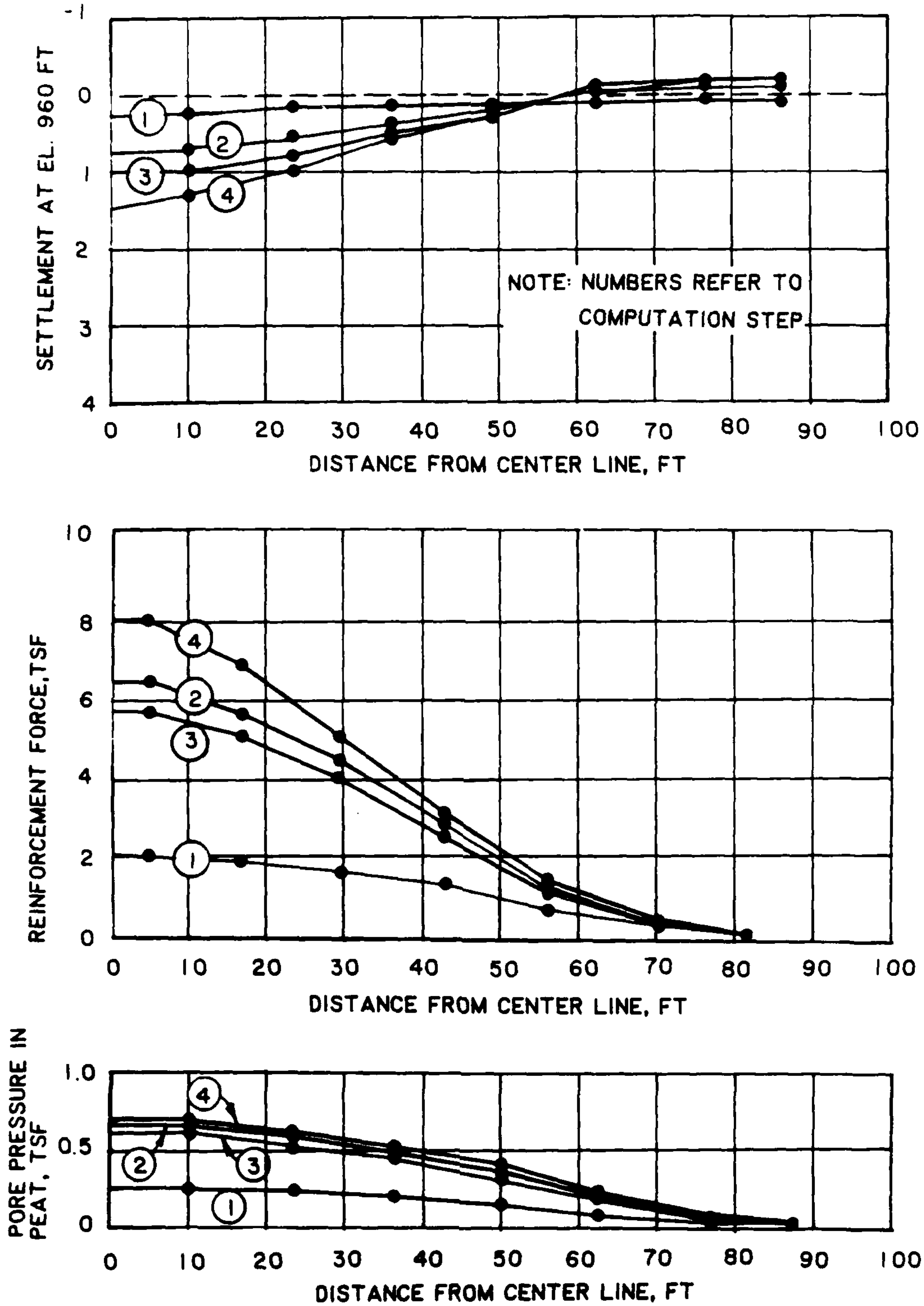


Figure C9c. Settlement, pore pressure, and reinforcement force profiles for K_s = 625 ton/ft

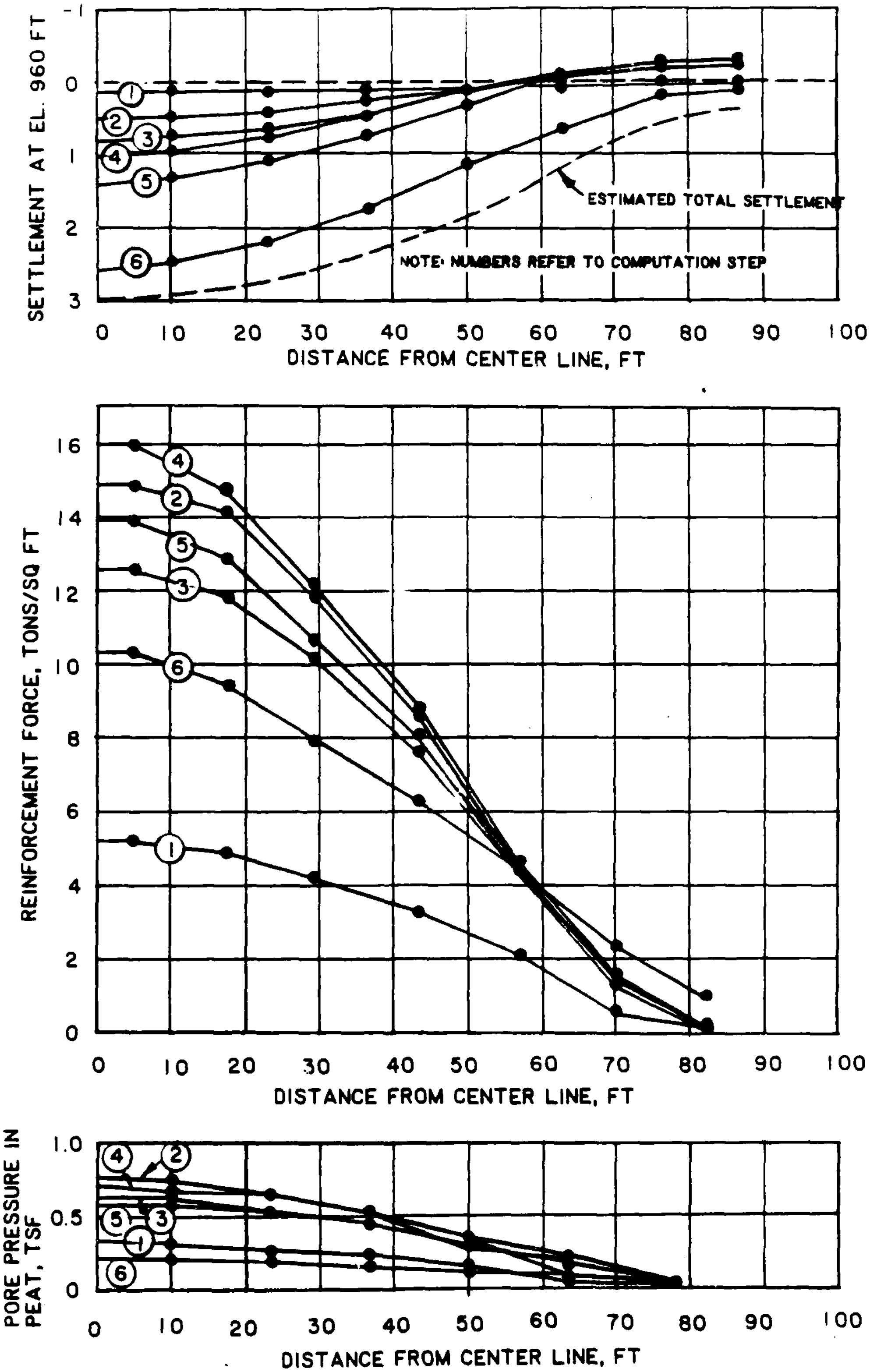


Figure C9d. Settlement, pore pressures, and reinforcement force profiles for K_S = 12,000 ton/ft

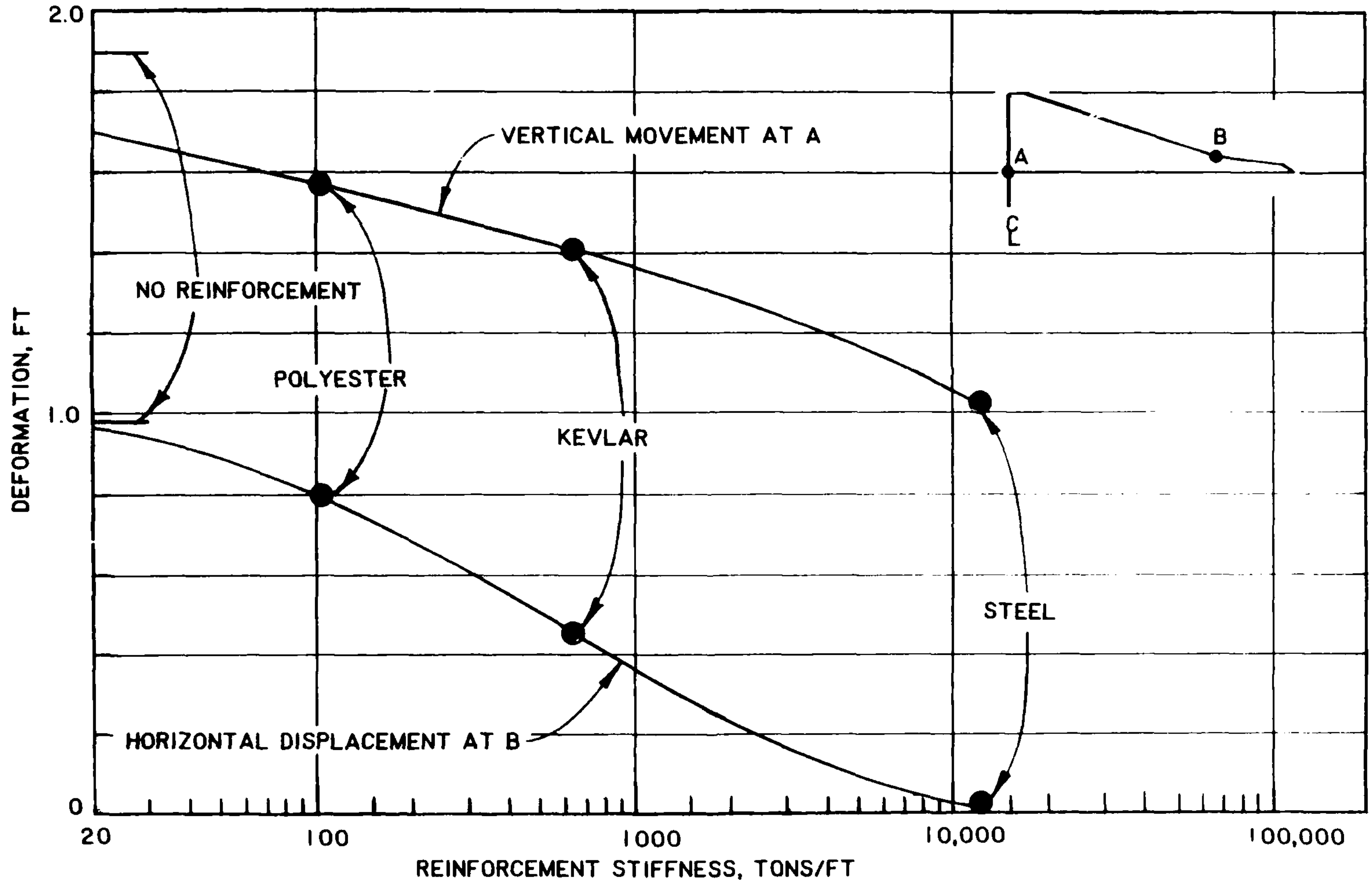


Figure C10. Fabric stiffness versus centerline settlement and lateral spreading

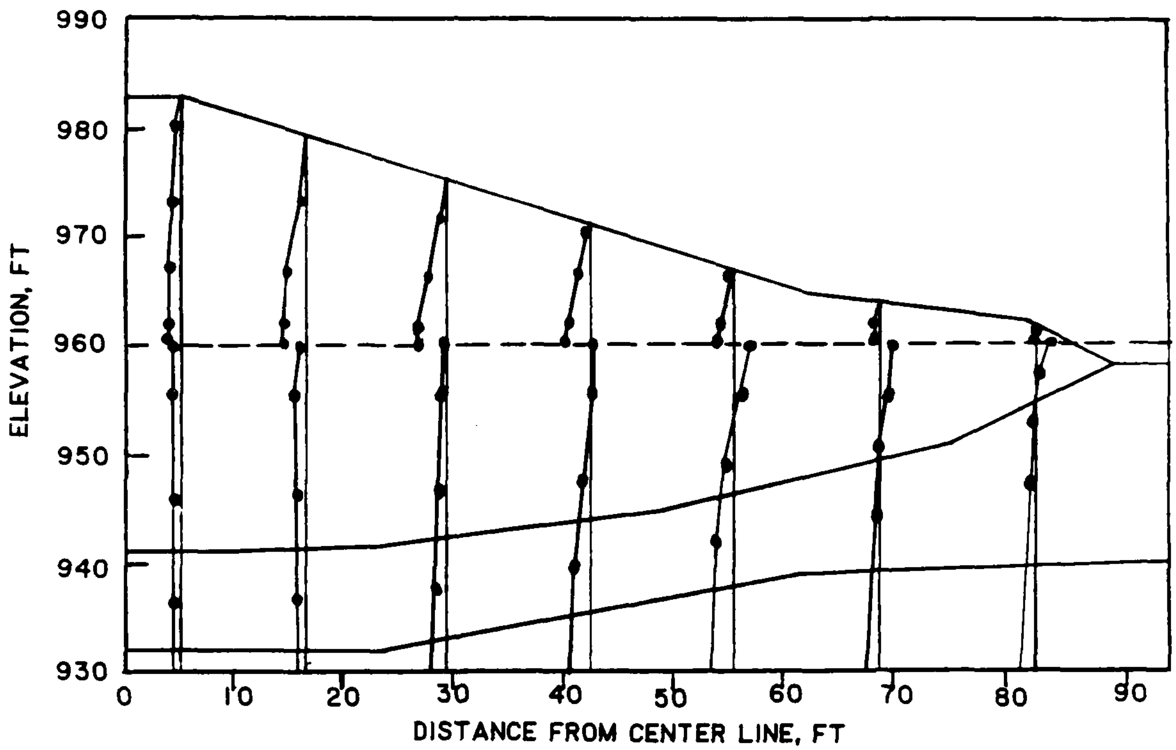


Figure C11. Distribution of shear stress on horizontal plane

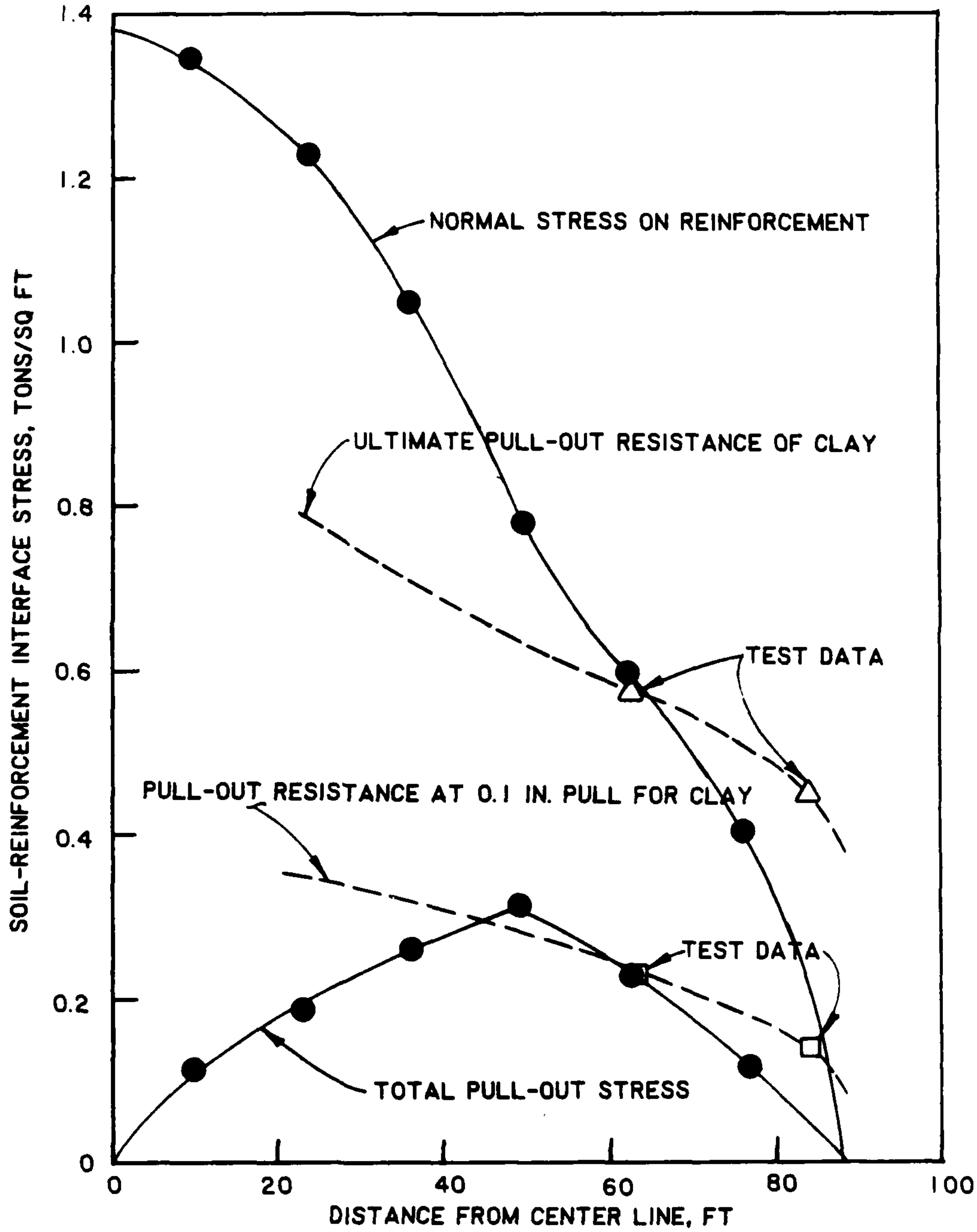


Figure C12. Total pullout force versus distance from centerline for steel reinforcement at end of construction

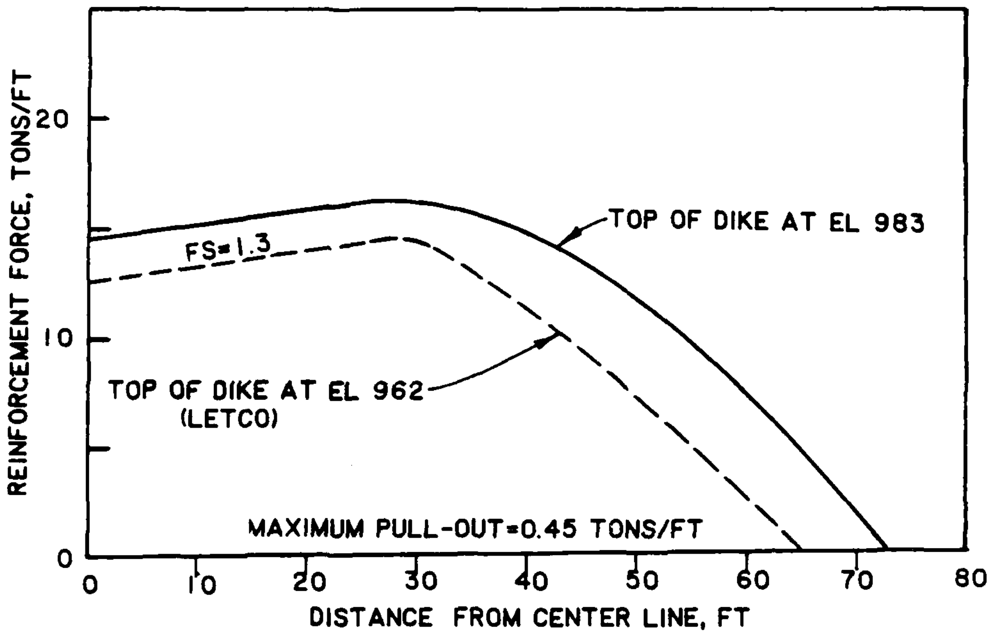


Figure C13. Total pullout force computed from limit equilibrium analyses

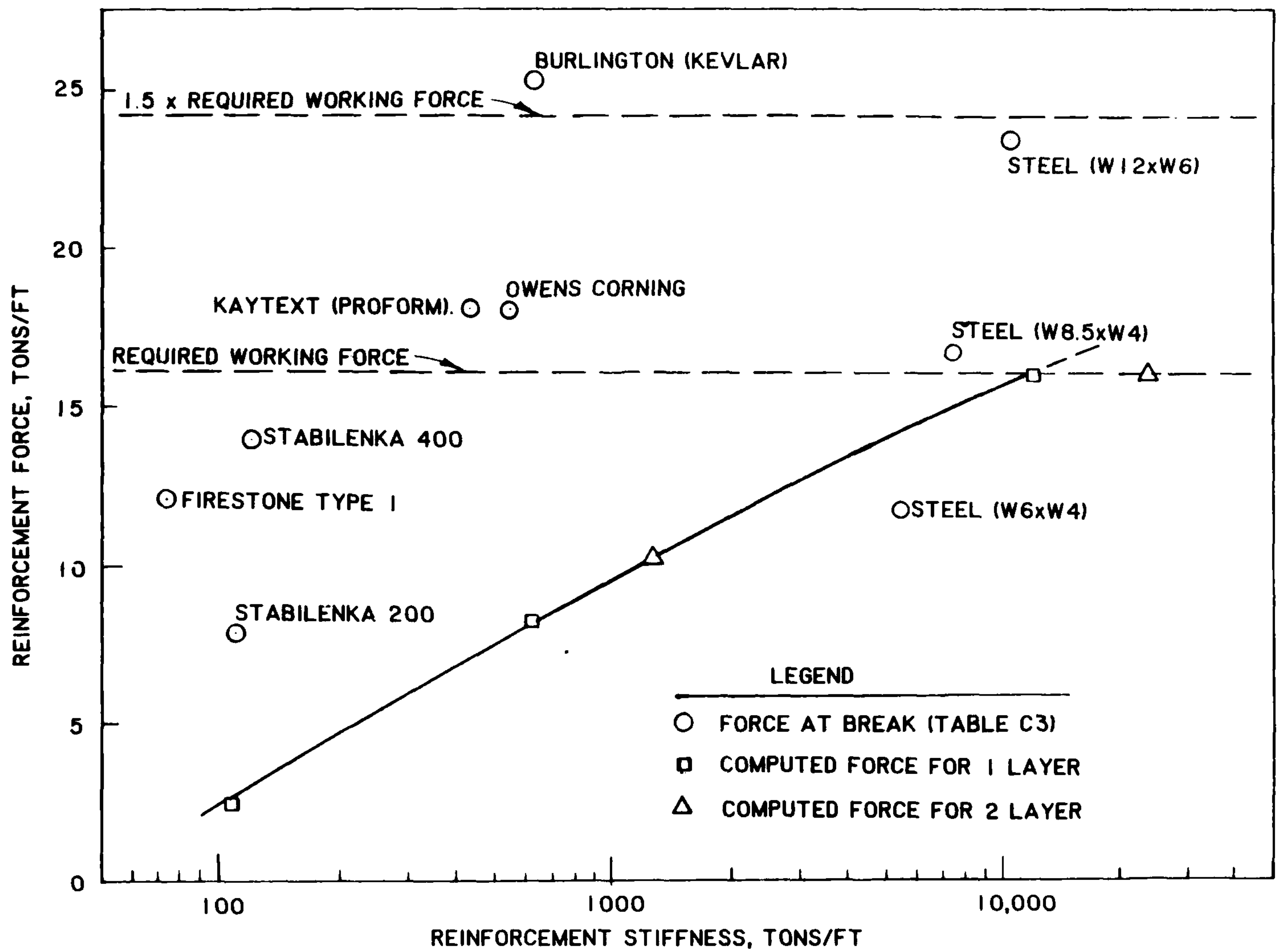


Figure C14. Reinforcement force and strength versus stiffness, K_s

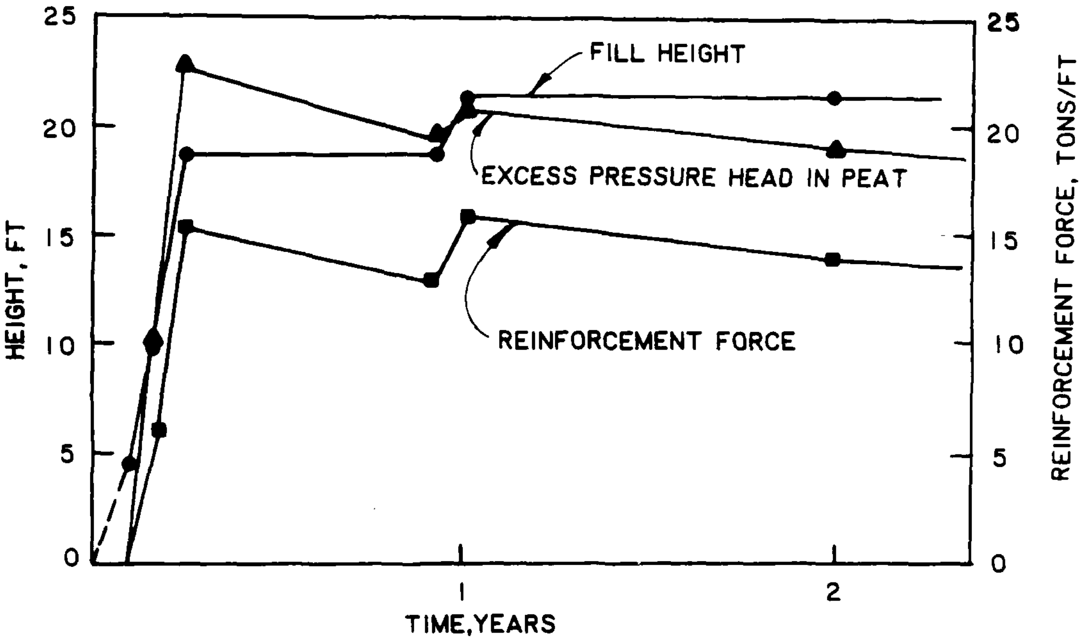


Figure C15. Predicted pore pressure, settlement, and reinforcement force for instrument locations near dike centerline

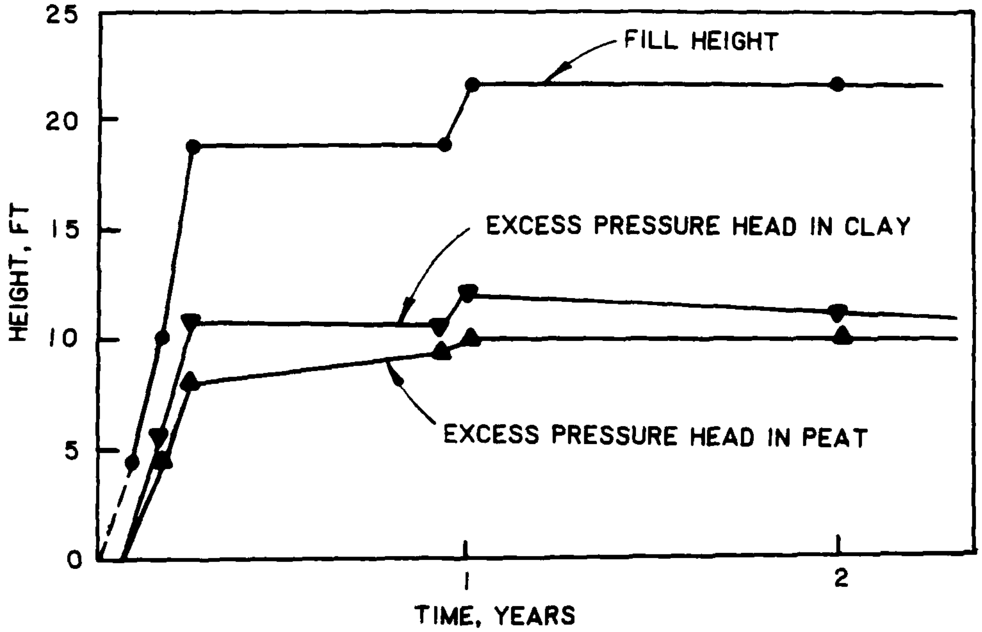


Figure C16. Predicted pore pressure, settlement, and reinforcement force for instrument locations near one-third point

APPENDIX D: CONVENTIONAL ANALYSIS AND BLANKET DESIGN

Conventional Slope Stability Analysis

End of construction case

1. The Fellenius circular arc method of slope stability analysis contained in WES program SA10478 was checked against the LETCO program and each obtained similar results for similar soil properties and physical dimensions. Then using the adopted soil properties and a 1 ft higher embankment a FS of 0.89 was calculated for the present design. The failure arc is shown in Figure 4 of the main text. To obtain a FS of 1.3 as required by EM 1902, the required resisting moment and reinforcement tensile force was determined using an equation recommended by LETCO:

$$\frac{RM}{FS} = AM - T * R$$

or

$$T = \frac{AM - \frac{RM}{FS}}{R}$$

where

- T = required fabric tensile strength
- AM = active moment kip-ft/ft
- FS = factor of safety
- RM = resisting moment of the soil, kip-ft/ft
- R = radius of the critical arc, ft

Using the calculated values shown in Figure 4 (main text), a required fabric tensile strength can be computed.

$$T = \frac{7266 \text{ kip-ft/ft} - \frac{6499}{1.m} \text{ kip-ft/ft}}{70 \text{ ft}}$$

or

$$T = 32.4 \text{ kips/lin ft}$$

or

$$T = 2698 \text{ lb/lin in.}$$

The calculated values of required tensile strength are slightly higher than those determined in the LETCO report due mainly to the increase in embankment height used for the WES analysis. At a meeting of Corp representatives and consultants an FS of 1.5 was adopted for the required reinforcement strength. The required fabric strength, T , for the tensile strength calculated is:

$$FS = 1.5\ T$$

$$= 1.5\ (32.4) \text{ kips/lin ft}$$

$$= 48.6 \text{ kips/lin ft}$$

For a FS of 1.5, steel fabric is the only reinforcement in Table A3 that meets the strength requirements.

Long Term Stability

2. For long term stability two cases from EM 1110-2-1902 were checked: (1) Case II - upstream stability for rapid drawdown from maximum pool; and (2) Case VI - downstream stability for steady seepage at maximum pool. Factors of safety of 1.07 and 1.45 were calculated for Cases II and VI, respectively. These factors of safety are considered adequate because no reinforcement is included in the analysis. The resulting arcs are shown in Figures 5 and 6 of the main text.

Bearing Capacity Analysis

3. A bearing capacity analysis assumes a shear failure from the weight of the dike and that the embankment is a shallow continuous footing of infinite length. Using the theory of plasticity for bearing capacity of $= 0$ materials in the undrained condition, the ultimate undrained bearing capacity, q_d , for a clay loaded as described is somewhere between a

smooth and a rough base and the following equation is recommended (Hammer, D. P., and Blackburn, E. D., "Design and Construction of Retaining Dikes for Containment of Dredged Material," TR D-77-9, 1977):

$$q_d = 5.5C$$

where

c = soil cohesion, psf

The soil pressure at the base of the embankment, q, is given by:

$$q = H$$

where

= unit weight of soil, pcf

H = height of embankment, ft

For the design case where the reinforcement is required to be very stiff, the embankment can be assumed to fail as a rigid body and the soil pressure at the base of the embankment is simply the total weight per ft of embankment divided by the total area per ft of embankment as shown in Figure D1. Since the undrained strengths of the peat and clay varied beneath the embankment, a value of 0.5 ksf was assumed for cohesive strength. For these assumptions a FS of 1.72 was calculated. By use of ultimate bearing capacity, the assumption is made that failure occurs at the embankment base. Its application to deeper strata may therefore be conservative since in doing so the assumption would be made that the full embankment load is transmitted to the deeper strata.

Embankment Lateral Sliding Analysis

4. To prevent lateral sliding of the Mohicanville Dike, the existing active earth pressures must be resisted by the forces developed along the reinforcement. It is assumed that no phreatic surface is present and that construction induced pore pressures will be dissipated near the reinforcement. Lateral earth pressures are maximum at a maximum height of the dike; therefore, the resultant of the active earth pressures, P_A, along the centerline may be calculated per unit length as follows:

$$P_A = 0.5\ {}_{m}H^2 \tan^2 45 - \frac{\ }{2}$$

where:

$_m$	=	moist soil density, pcf
H	=	height of dike, ft
	=	angle of internal friction, deg

For the Mohicanville Dike soil properties described in the laboratory tests (Appendix A:

where:

$_m$	=	130 pcf
H	=	25 ft
	=	30 deg

then:

$$P_A = 0.5(130)(25)^2 \tan^2 45 - \frac{30}{2}$$

or:

$$P_A = 13542\ \#/\text{ft - width}$$

Resisting forces along the reinforcement, P_R, may be calculated as follows:

$$P_R = 0.5\ {}_m \text{ X } H^2 \tan\ {}_{SR}$$

where

$_m$	=	moist soil density, pcf
X	=	dimensionless embankment slope factor (HOR/VER)
H	=	height of dike, ft
SR	=	soil-reinforcement friction angle, deg (determined from labora tory tests)

For a dike slope of 3H:1V and a $_{SR}$ described in the laboratory tests (Appendix A):

where:

$_{m}$ = 130 pcf

X = 3

H = 25 ft

$_{SR}$ = 18 deg

then:

$$P_R = 0.5(130)(3)(25)^2 \tan 18 \text{ deg}$$

or:

$$P_R = 39600 \text{ \#/ft - width}$$

A factor of safety of 2 is recommended by Haliburton et al., (1981) for the resisting forces divided by active forces:

$$FS = \frac{P_R}{P_A}$$

$$= \frac{39600 \text{ \#/ft-width}}{13542 \text{ \#/ft-width}}$$

$$= 2.9$$

5. The case where the slurry trench is breached and the reservoir hydrostatic head is acting at the soil-reinforcement interface was also checked. A phreatic surface was assumed from reservoir level upstream to the original water table (el 958) at a distance 30 ft downstream of the centerline. The uplift force, P_u, created is subtracted from the resisting force, P_R, as follows:

where

$$P_R = (0.5\ {}_{m} * H^2 - P_u) \tan\ {}_{SR}$$

$$P_u = 0.5\ {}_{w}\ {}^{2}$$

$${}_{w} = \text{water density, pcf} = 62.4 \text{ pcf}$$

$$= \text{length of the uplift surface, ft} = 30 \text{ ft}$$

or:

$$P_R = (0.5 * 130 * 3\ \ 25^2 - 0.5 * 62.4 * 30^2) \tan 18 \text{ deg}$$

$$P_R = 30490 \text{ \#/ft - width}$$

For this case the FS is equal to:

$$FS = \frac{P_R}{P_A}$$

$$= \frac{30490 \text{ \#/ft-width}}{13542 \text{ \#/ft-width}}$$

$$= 2.25$$

The FS is greater than 2 and is considered safe. Although the resisting force for the uplift case is somewhat smaller than for the end-of-construction case, the uplift case has only a remote chance of happening and the end-of-construction case will be used for the rest of the analysis. The soil-reinforcement friction angle required to resist embankment sliding may be obtained by combining the active and resisting force equations and solving for ${}_{SR}$. For a FS of 2 the equation is as follows:

$${}_{SR} = \tan^{-1} \frac{4P_A}{{}_{m} * H^2}$$

$${}_{SR} = \tan^{-1} \frac{4(13542 \text{ \#/ft})}{(130 \text{ pcf})\ (3)\ (25 \text{ ft})^2}$$

$${}_{SR} = 12.5 \text{ deg}$$

The soil-reinforcement friction angle required to resist sliding is less than the available soil-reinforcement friction, $_{SR}$ = 18 deg, determined from the laboratory tests shown in Appendix A.

Embankment Splitting Analysis

6. The resultant of the active earth pressure would equal the reinforcement tension when the required soil-reinforcement friction is attained to prevent sliding. The reinforcement tensile strength would resist the lateral earth pressures and should have a FS of 1.5 as recommended by Haliburton et al. Therefore

$$T_R = 1.5\ P_A$$

where

T_R = minimum ultimate reinforcement tensile strength (lb/in. - width)

or

$$T_R = \frac{1.5(1342\ \text{\#/ft-width})}{12\ \text{in./ft}}$$

or

$$T_R = 1690\ \text{\#/in.-width}$$

or

$$T_R = 20.28\ \text{kips/ft-width}$$

This is the required minimum reinforcement tensile strength to prevent embankment splitting. There are numerous reinforcement fabrics listed in Table A-3 that will satisfy this requirement.

Settlement Analysis (LETCO)

7. As discussed in the LETCO report a settlement analysis should be linked to the original settlement that occurred during construction. It was

estimated that 9 ft of compression occurred in the 18 ft of peat. Using recent consolidation tests, Figure 2 (main text), LETCO calculated that primary consolidation accounted for 30 percent of the original consolidation and 41 percent is due to long term secondary consolidation. These values are used in further calculations in the peat. The foundation clay, subjected to shear and flow failures, could not be evaluated in the same manner as the peat but 15 percent more settlement was added to account for secondary consolidation. Using the adjusted values, 38 inches and 28 inches of total consolidation were predicted for the crest and the toe, respectively. The finite element analysis indicates a total settlement of about 3 ft of which 2 ft are the result of consolidation and one foot the result of shear strains in the foundation during construction.

8. Using the time rate parameter, C_v , determined from consolidation tests, LETCO estimated 50 percent consolidation would occur at sta 9+15 in 2.5 years and 90 percent in 11 years. These rates are compatible to those obtained from the finite element analysis (Appendix C). At sta 6+35 where there is a thinner clay layer, it was estimated that 50 percent consolidation would occur in 1 year and 90 percent in 3.5 years. These estimates for consolidation do not include secondary compression or consolidation of the peat layer.

Downstream Toe Filter Design

9. With the dike being founded on a soft foundation, it will be subjected to possible failure by splitting, spreading, sliding, and rotation along an arc. Although the dike is designed against failure there will be horizontal and vertical displacements which could lead to embankment cracking and the formation of possible seepage paths through the dike. To avoid excessive hydrostatic uplift pressures at the downstream toe and possible piping of the embankment material along the seepage paths a filter was designed to be placed under the downstream one-third of the dam. The filter material has to be more permeable than the embankment material and should prevent infiltration of the embankment material into the filter. In EM 1110-2-1913, "Design and Construction of Levees" these criteria are referred to as the "permeability" and "stability" requirements, respectively, and are defined as follows:

Stability

$$\frac{\text{15 percent size of filter material}}{\text{85 percent size of material being drained}} \quad 5 \qquad (1)$$

and

$$\frac{\text{50 percent size of filter material}}{\text{50 percent size of material being drained}} \quad 25 \qquad (2)$$

and

Permeability

$$\frac{\text{15 percent size of filter material}}{\text{15 percent size of material being drained}} \quad 5 \qquad (3)$$

It is stated in EM 1913 that Equation 2 may be disregarded for CL soils without sand or silt partings and that a maximum of 0.4 mm may be used for 15 percent size of filter material for Equation 1. As shown in the gradation curve in Figure 8 for a value of 0.0013 mm (15 percent size of material being drained), a value of 0.0065 mm can be calculated from Equation 3 for the 15 percent size of filter material. The values of 0.4 mm and 0.0065 mm are the upper and lower bounds (15 percent size) of a filter band. The band should be somewhat parallel to the gradation of the material being drained but considerable variation can be used if the 15 percent limits are met and the filter material is not gap or skip graded. Based on locally available material the band shown in Figure D2 as filter 1 (concrete sand, ASTM C33) is recommended for use. The dike material is somewhat gap graded with the coarse material floating in the matrix material rather than deterring the migration of fines; therefore, the filter is designed to stop the infiltration of the matrix material. Details of the filter are shown on construction drawing 7.

10. To avoid the problem of the filter clogging at the downstream exit, a slotted pipe seepage collection system has been added. Criteria for the slotted openings (EM 1913) are as follows:

$$\frac{\text{50 percent size of filter material}}{\text{slot width}} \quad 1.2 \qquad (4)$$

Using this criteria the pipe slot width for filter 1 would be 0.4 mm. To insure flow into the collection pipe a slot width of 4 mm was adopted for design. To avoid piping of the filter material into the pipe, filter cloth will be wrapped around the collection pipe. Details for the collection system are shown on construction drawing 7.

11. The capacity of the drain system was checked as shown in Cedergren 1977:

Q = kih

Q = flow in ft^3/day

k = permeability, ft/day

i = hydraulic gradient, dimensionless

h = head on the drain, ft

If

k = 3 ft/day

i = 2 ft/65 ft

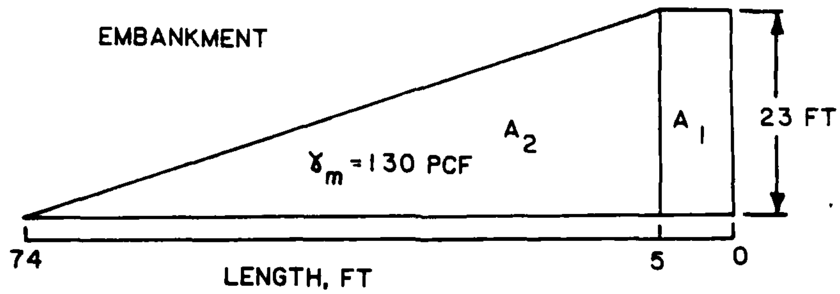
h = 2 ft

then

Q = 0.18 ft^3/day per ft of embankment

or

Q = 0.00096 gpm per ft of embankment



ASSUME A RIGID EMBANKMENT

$$\underline{\text{AREA}}\ A_1 = 5 \times 23 \qquad = 115.0\ \text{ft}^2$$
$$A_2 = 1/2 \times 69 \times 23 = \underline{793.5}\ \text{ft}^2$$
$$\text{Total} \quad 908.5\ \text{ft}^2$$

WEIGHT OF DAM

$$W_T = A_T\ \gamma_m$$
$$= 908.5 \times 130$$
$$= 118100\ \text{P/lin ft}$$
$$W_T = 118.1\ \text{kips/lin ft}$$

PRESSURE AT FOUNDATION

$$q = W_T/\text{EMB. LENGTH}$$
$$= 118.1/74$$
$$q = 1.596\ \text{KSF}$$

FOUNDATION STRENGTH

ASSUME AVERAGE COHESION FOR THE PEAT AND CLAY - c = 0.5 KSF

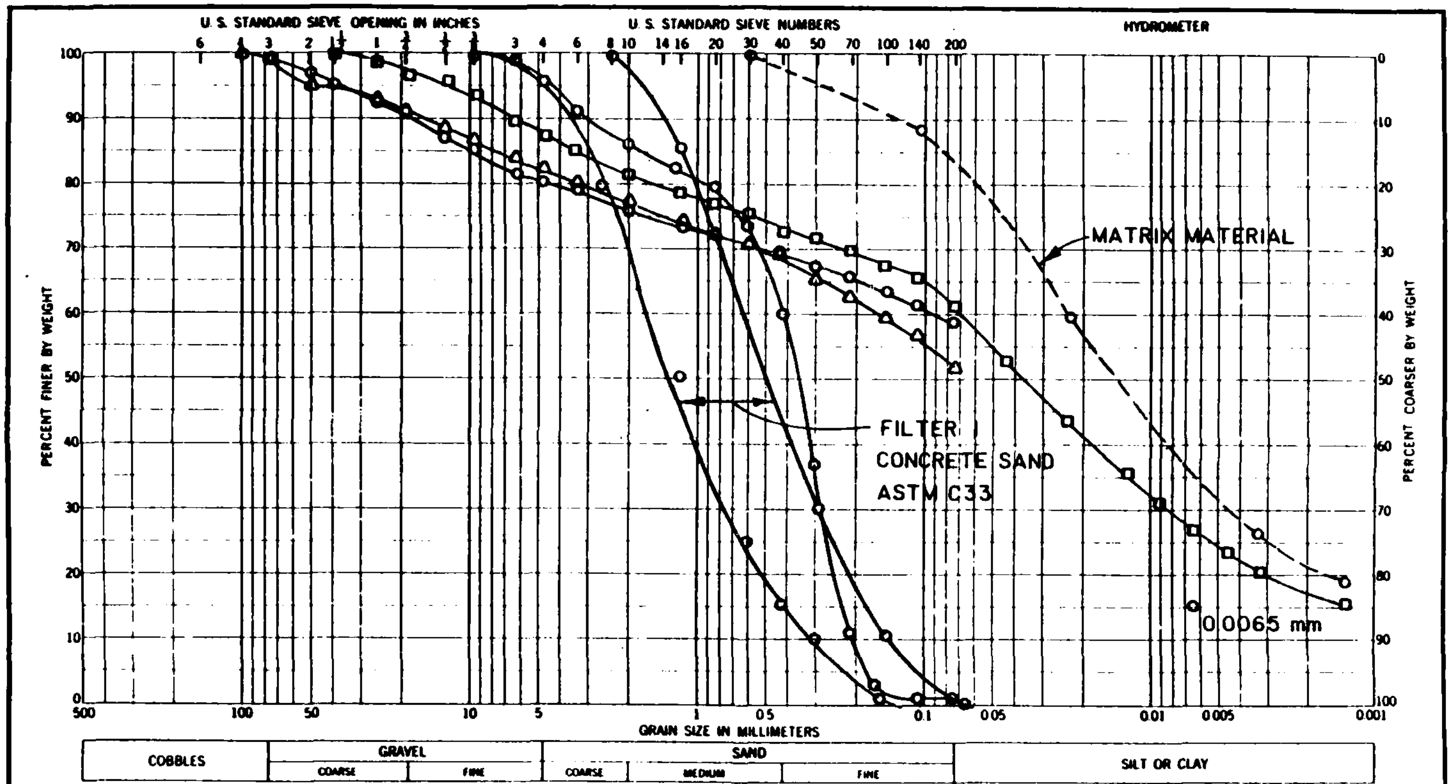
$$q_d = 5.5\ c$$
$$= 5.5\ (0.5)$$
$$= 2.75\ \text{KSF}$$

FACTOR OF SAFETY

PRESSURE BENEATH A RIGID STRUCTURE DECREASES WITH DEPTH, THEREFORE A MINIMUM FS IS CALCULATED USING THE FULL EMBANKMENT PRESSURE

$$FS = q_d/q$$
$$= 2.75/1.596$$
$$= 1.72$$

Figure D1. Bearing capacity analysis



Sample No	Elev or Depth	Classification	Nat w %	LL	PL	PI
Borrow Composite 1 = △		Gravelly Sandy Clay (CL) Brown		32	16	16
Borrow Composite 2 = ○		Gravelly Sandy Clay (CL) Brown		29	16	13
Old Dike Composite = □		Gravelly Sandy Clay (CL) Brown		29	15	14
PULLOUT TEST SAND = ○		Sand (SP) V. Light Tan; trace of Gravel**				
Borrow Composite 2: G_s=2.76, G_A=2.76, G_B=2.35						

Project

Area

Boring No

Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

**RELATIVE DENSITY

γ_D = 108.9 max

γ_D = 100.1 min

Figure D2.

DATE
FILMED
2=8