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LABORATORY MEASUREMENT OF PULLOUT RESISTANCE OF GEOTEXTILES AGAINST COHESIVE SOILS

by

Paul A. Gilbert, Jessie C. Oldham, L. Rodgers Coffing, Jr.

Geotechnical Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

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13. ABSTRACT *(Maximum 200 words)*
This report focuses on laboratory tests performed by the US Army Engineer Waterways Experiment Station (WES) to evaluate the performance of three geotextiles with four soils from the Bonnet Carre Spillway area. The purpose of this evaluation was to produce methodologies to assess the feasibility and value of using geotextiles under conditions that exist in the US Army Engineer District, New Orleans. Laboratory-measured parameters of the geotextile/soil systems in question are compared with prototype field tests and used in the analysis of full-size soil structures to evaluate configurations for strength, economy, and effectiveness. This report focuses on the equipment, performance, and results of the WES laboratory study.

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PREFACE

This report, prepared by the Geotechnical Laboratory (GL) of the US Army Engineer Waterways Experiment Station (WES), describes field and laboratory pullout tests performed to investigate reinforcement of soil using geotextiles. The work was authorized by the US Army Engineer District, New Orleans, under Intra-Army Order for Reimbursable Services (DA Form 2544) number CELMNED-90-56, dated 30 May 1990. The laboratory tests were performed by Mr. L. Rodgers Coffing under the supervision and direction of Mr. Jessie C. Oldham, Chief, Soils Testing Facility, Soils Research Center (SRC), Soil and Rock Mechanics Division (S&RMD). The report was written by Mr. Paul A. Gilbert of the Soils Research Facility (SRF), SRC, S&RMD, Mr. Oldham, and Mr. Coffing, under the supervision of Mr. Gene P. Hale, Chief, SRC; Dr. Don C. Banks, Chief, S&RMD; and Dr. William F. Marcuson III, Director, GL.

At the time of the publication of this report, Dr. Robert W. Whalin was Director of WES. Col Leonard G. Hassell, EN, was Commander and Deputy Director.

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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
gallons (US liquid)	3.785412	cubic decimetres metres
horsepower (550 fast-pounds (force) per second per ton (force)	83.82	watts per kilonewton
inches	2.54	centimetres
ounces (mass) per square yard	33.90575	grams per square metre
pounds (force)	4.448222	newtons
pounds (force) per square inch	6.894757	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (mass) per square foot	4.882428	kilograms per square metre
square feet	0.09290304	square metres
square yards	0.8361274	square metres
tons (2,000 pounds, mass)	907.1847	kilograms

LABORATORY MEASUREMENT OF PULLOUT RESISTANCE OF GEOTEXTILES AGAINST COHESIVE SOILS

PART I: INTRODUCTION

1. Geotextiles are usually permeable synthetic fabrics (polyesters) used with soil, rock, or other earth materials in geotechnical engineering structures to enhance performance and economy while reducing maintenance requirements. The fabrics may be woven or nonwoven. Woven fabrics consist of fibers plaited together in a systematic pattern and generally possess higher tensile resistance and stiffness than nonwoven fabrics that consist of matted and randomly tangled fibers pressed into sheets of varying thickness and density. Geotextiles perform their function by providing some combination of filtration, drainage, and reinforcement to the structures in which they are installed.

2. One of the principal advantages gained by the use of geotextiles, and the one on which the present investigation will be focused, is that of improved strength as the result of soil reinforcement. Geotextile reinforcement of a soil structure is accomplished through a combination of the high tensile strength of the fabric along with contact friction and adhesion of fabric against the soil to provide anchorage. For economic justification as well as structural analysis, it is important for designers to know how much the strength of a system is enhanced by a particular geotextile. Strength enhancement can be positively quantified only if the soil/fabric resistance is known, and this must be determined by performing resistance tests on the soil/geotextile system under consideration and using the results obtained to analyze the structure in question. Soil is, obviously, an integral part of a soil/fabric system, and the difficulties involved in determining the strength of soil as a single component are well known; measurement of soil properties is influenced by many factors, among which are specimen size, rate of loading, drainage conditions, water content, degree of saturation, density, and soil mineralogy, to name a few. When soil is combined with a geotextile, the result is an indeterminant composite structure with extremely complex nonisotropic behavior because the stress-strain and strength characteristics of both components (soil and fabric) are, in general, nonlinear and very different in behavior. Additionally, the response of soils and geotextiles is

stress level and history (in the case of soil) dependent, which further complicates composite system behavior.

Work by Previous Investigators

3. Because of the difficulty that would be involved in obtaining a mathematical solution of the (load-deformation-strength) behavior of soil/fabric composites, information on such systems is most easily determined by direct physical measurement, and in the 10 or so years since geotextiles have come into widespread use, several investigators have performed and reported soil/fabric behavior and resistance measurement studies.

4. Martin, Koerner, and Whitty (1984) reported an investigation in which pullout tests were conducted with several fabrics, the results of which are summarized in Table 1. However, it should be pointed out that all tests reported in the 1984 study were performed with cohesionless soils. The "efficiency" referred to in Table 1 is the ratio of the friction angle between the fabric and sand to the angle of internal friction of the sand expressed as percent. The results of the 1984 study indicate that the efficiency of a sand/fabric system is typically less than 100 percent.

5. Myles (1982) performed soil-to-fabric friction tests in a square shear box with an area of 0.1 m^2 and concluded (from his 1982 study and 10 previous years of stated practical experience) that a lower limit for the efficiency of soil/fabric systems and one which will yield conservative designs in geotextile reinforced soil structures is 75 percent. However, it must be pointed out that the 1982 laboratory study was conducted entirely on cohesionless materials and the efficiencies observed and reported from experiments performed during the study ranged from 82 to 98 percent.

6. In a study involving a poorly graded river sand, Miyamori, Iwai, and Makiuchi (1986) performed pullout tests in a large direct shear box designed to test a specimen that was 31.6 by 31.6 cm (0.1 m^2) in plan. They concluded from their investigation that the frictional resistance of sand on a nonwoven fabric is generally smaller than the frictional resistance of the sand on itself as measured in direct shear. Efficiency (as used by Martin, Koerner, and Whitty (1984)) of the Miyamori, Iwai, and Makiuchi sand/fabric systems ranged from 72 to 87 percent for dense sand; however, efficiency increased as density decreased and approached 100 percent in loose sand.

Table 1

Soil-to-Fabric Friction Angles and Efficiencies (in Parentheses) in Cohesionless Soil*

Geotextile Type	Manufacturer's Designation	Concrete Sand $\phi = 30°$	Rounded Sand $\phi = 28°$	Sandy Silt $\phi = 26°$
Woven, monofilament	Polyfilter X	26° (87%)	--	--
Woven, slit film	500X	24° (80%)	24° (86%)	23° (88%)
Nonwoven, heat set	3401	26° (87%)	--	--
Nonwoven, needled	CZ600	30° (100%)	26° (93%)	25° (96%)

* After Martin, Koerner, and Whitty (1984).

7. Palmeira and Milligan (1990) performed pullout tests on two geotextiles and three geogrids on (Leighton Buzzard) sand in a cubical direct shear box 1 m on a side. Their study demonstrated that progressive failure occurred in pullout tests on extensible reinforcement systems. They point out the difficulty in predicting the pullout resistance of extensible grids and conclude that tensile strains and load distribution in polymeric reinforcement can be accurately predicted only by having reliable load-time-temperature data on the polymeric reinforcement.

8. Very few soil/geotextile resistance investigations involving cohesive soil have been performed and reported in the literature. Christopher and Berg (1990) report a limited number of pullout tests of a cohesive soil on two geotextiles; one test on each of the geotextiles appears to have been performed in their 1990 investigation. The geotextiles used in their study were a slit-film woven fabric and a needled nonwoven fabric whose thicknesses were 0.76 and 2.8 mm, respectively. Tests were conducted in a direct shear box 0.7 m wide by 0.5 m deep by 1.3 m long on a silt, ML (liquid limit (LL) = 45 percent, plasticity index (PI) = 14 percentage points) which was allowed to consolidate before the fabrics were pulled out at a rate of 1 mm/min, a rate stated by the authors to be rapid enough to maintain an undrained condition during loading. Tests were performed at a soil water content of 18.5 percent

and wet unit weight of 16.7 kN/m^3* (about 106 pcf) under a normal stress of 35 kN/m^2 (about 5 psi). The investigation by Christopher and Berg (1990) focused mainly on pullout testing of geogrids, so the information gathered and conclusions drawn on the pullout resistance of geotextiles in clay were of a limited and nonspecific nature.

9. Garbulewski (1990) performed soil-to-fabric friction tests on a cohesive soil from Druzno Lake near Gdansk, Poland. The soil tested is a highly plastic clay (LL = 90 percent, PI = 56 percentage points), is soft at its natural water content of 54 percent, contains about 13-percent organic matter, and has a wet density of 15 kN/m^3 (about 95.5 pcf). In the investigation, soil was placed in the upper half of a square shear box in which the soil/geotextile contact area was 100 cm^2 and displaced against the fabric at a rate of 0.1 mm/min. No mention was made of time allowed for consolidation. Test results for two soil/fabric combinations investigated in the study are summarized on Figure 1, which also includes the results of direct shear tests on the subject soil without fabric. The figure shows that the frictional resistance of one soil/fabric system is almost identical to that of soil on soil. The other soil-fabric system tested shows an efficiency of 90 percent. The symbol μ on the figure is the coefficient of friction between soil and fabric ($\tan \delta$) and between the soil and itself ($\tan \phi$).

Geogrids

10. Geogrids are used primarily for reinforcement in soil structures and are constructed in gridlike patterns using a variety of materials and configurations. For example, they may be manufactured from sheets of plastic (polymeric material) by punching a regular pattern of holes to form a netlike configuration; the sheets may or may not be prestressed by stretching to add strength and reduce susceptibility to creep, and it must be noted that all polymeric materials are susceptible to creep. If very high strength and low creep response is required in a geogrid, the product may be constructed of

* Note: kN/m^3 is not a consistent set of units for expressing mass density and conversion factors for this set of units does not appear in American Society for Testing and Materials (ASTM) E380-86 (1986). The newton is uniquely associated with force and not mass; therefore mass density cannot be sensibly expressed in terms of (kilo) newtons. However, if one accepts the "poetic license" that some investigators take with the kilonewton, multiply kN/m^3 by 6.366 to get pcf.

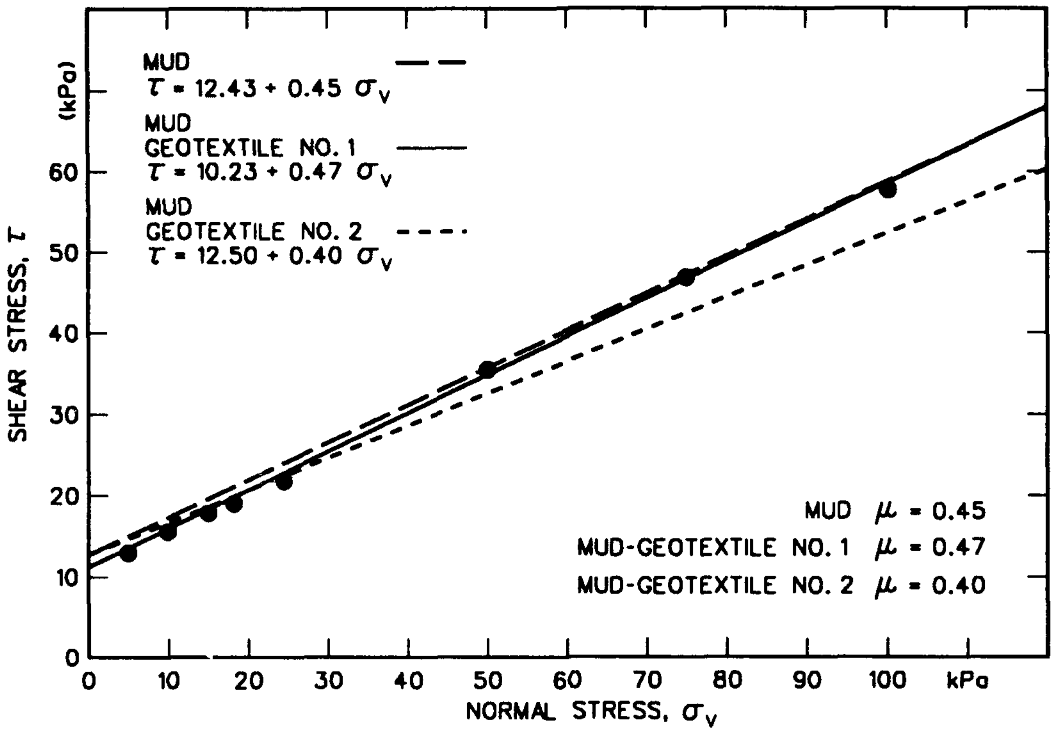

Figure 1. Direct shear test results

metal wire grid or welded metal bars or rods. Geogrids, like geotextiles, reinforce geotechnical structures; however, hole sizes in geogrids are so large (generally 0.5 to 2 in.* or more in size) that geogrids are ineffective for use as filters. However, they may be used in zoned geotechnical structures to separate layers of relatively coarse-grained materials. Geogrids cannot be counted on to provide (in plane) drainage or filtration, and although they have been formed from woven or welded metal lattices to provide very high reinforcement strength in special applications (Fowler et al. 1986), the tensile strength of polymeric geogrids is usually smaller than that of (woven) geotextiles. Geogrids may be used alone for reinforcement or in combination with geomembranes and/or geotextiles to give strength, increased friction, or some other desired property to a composite in which it is incorporated as an element. Laboratory investigations of pullout resistance of geogrids in clay have been performed and reported in the literature. Some of these results will be discussed later.

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* A table of factors to convert non-SI units of measurement to SI (metric) units is presented on page 4.

PART II: BACKGROUND FOR PRESENT INVESTIGATION

11. Plastic clays of high water content and low strength with varying amounts of organic matter occur extensively in the New Orleans District (NOD), and construction on and with soils having these characteristics is generally difficult and expensive. Geotextiles may be used to facilitate construction in such situations and, in fact, often produce the greatest benefit in extreme conditions such as very weak soils and soft foundations. Geotextiles have been used by the NOD for the past several years in construction of levee systems; however, the factors necessary for precise economic evaluation of geotechnical design with geotextiles are elusive. The present laboratory investigation was undertaken, in part, to evaluate the performance of three geotextiles with four soils from the Bonnet Carre Spillway area for the purpose of producing methodology to assess the feasibility and value of using geotextiles under conditions that exist in the NOD. Laboratory-measured parameters of the geotextile/soil systems in question will ultimately be compared with prototype field tests and used in the analysis of full-size soil structures to evaluate configurations for strength, economy, and effectiveness.

12. Full-scale pullout tests have been conducted at the Bonnet Carre Spillway and at Belle Chasse, LA, to determine prototype soil/geotextile resistance. Companion laboratory tests were performed at the US Army Engineer Waterways Experiment Station (WES) in this investigation to duplicate in situ conditions at the field test sites. This report will focus on the equipment, performance, and results of the WES laboratory study and, where appropriate, comparison of field and laboratory results. However, analyses of levee sections in the NOD and detailed consideration of procedures for levee analysis using the results of laboratory and/or field tests are beyond the scope of this report.

PART III: LABORATORY EQUIPMENT

Direct Shear Test

13. Since a modified version of the direct shear test will be used for the laboratory investigation of this study, a brief discussion of the test is considered to be appropriate. In a typical direct shear test, a thin rectangular parallelepiped soil specimen is placed in a rigid split box, a normal pressure is applied to the specimen, and the soil is allowed to consolidate and come to equilibrium under that pressure. When consolidation is complete, the soil is loaded to failure by displacing one of the halves of the box relative to the other while measuring the force required to shear through the soil specimen inside the box. This loading is usually conducted at a very slow rate to allow any pore pressures to dissipate. This procedure is repeated on identical soil specimens under several normal pressures to define a failure envelope for the soil in question.

14. The principal use of the direct shear test in soil and foundation work is to determine the maximum consolidated-drained shear strength and angle of internal friction for use in strength and stability analysis of soil structures. The test has several advantages; notably, the specimen is thin so that drainage occurs quickly and drained strength is easily determined by use of the test. In addition to being fast (because of short drainage path and therefore rapid consolidation time), the test is relatively simple to perform and economical. In fact, direct shear tests on many samples from a given area have been found to furnish a much better picture of the distribution of the drained shear strength of soil than can (for the same cost) be obtained from triaxial compression tests (American Society for Testing and Materials (ASTM) STP 131 (1952)). The disadvantages of the direct shear test are that it cannot produce accurate constitutive properties, the stress and strain distribution within the specimen is highly nonuniform, and drainage is difficult to control. However, the direct shear test is ideally suited for the investigation at hand because pullout resistance is a function of maximum shear strength and constitutive properties are not required.

Description of Equipment

15. Laboratory tests in this investigation were performed in a large direct shear apparatus (Figure 2). The apparatus is similar to and has all the essential features of small direct shear boxes used in routine laboratory testing; it essentially consists of a box formed by two rigid frames that stack on top of each other. The frames have an open square space in the center in which the soil specimen is contained. Normal load is applied to the specimen through a piston; then one frame is displaced relative to the other to force a (theoretically horizontal) shear plane through the soil specimen contained in the system. The test specimen is 24- by 24-in. in plan and is surrounded by 2.5-in.-thick structural steel frames which are sufficiently stiff that they do not expand from internal pressure applied by the specimen. Consequently, no lateral (horizontal) specimen strain occurs during either consolidation or pullout. The height of each (upper and lower) half of the shear box is 6 in., so the maximum specimen thickness that the apparatus can accommodate is 12 in.

16. The shear box is fastened to a base structure consisting of three parallel 4-in.-high I-beams. Normal (vertical) load is applied to the soil specimen through a piston that consists of a 1-in.-thick stiffened aluminum plate which is 23.870 by 23.870 in. in plan; if this piston sits squarely on the soil specimen, a clearance of about 1/16 in. will be left between the outer periphery of the piston and the inside edge of the shear box. Four steel tension rods 1.50 in. in diameter mounted around the shear box furnish reaction for the applied normal load. The tension rods pull against the frame of H-beams at the base and against a built-up head structure of welded steel plates and H-beams at the top. The piston assembly, which consists of a built-in screw for height adjustment and the bearing plate that comprises the actual piston, reacts against the head structure that is fastened in place on the four steel tension rods with nuts.

17. Normal load is applied using a 14-in.-diam flatjack with an effective travel of 1 in. The flatjack is placed in line with the piston assembly, and internal pressure is applied to the device using a bleeding, self-relieving regulator supplied by the (220-psi maximum pressure) laboratory-compressed air system. Normal force is monitored with an electronic load cell that has a capacity of 20,000 lb. However, it should be mentioned that the apparatus is designed for a maximum normal load of 200,000 lb.

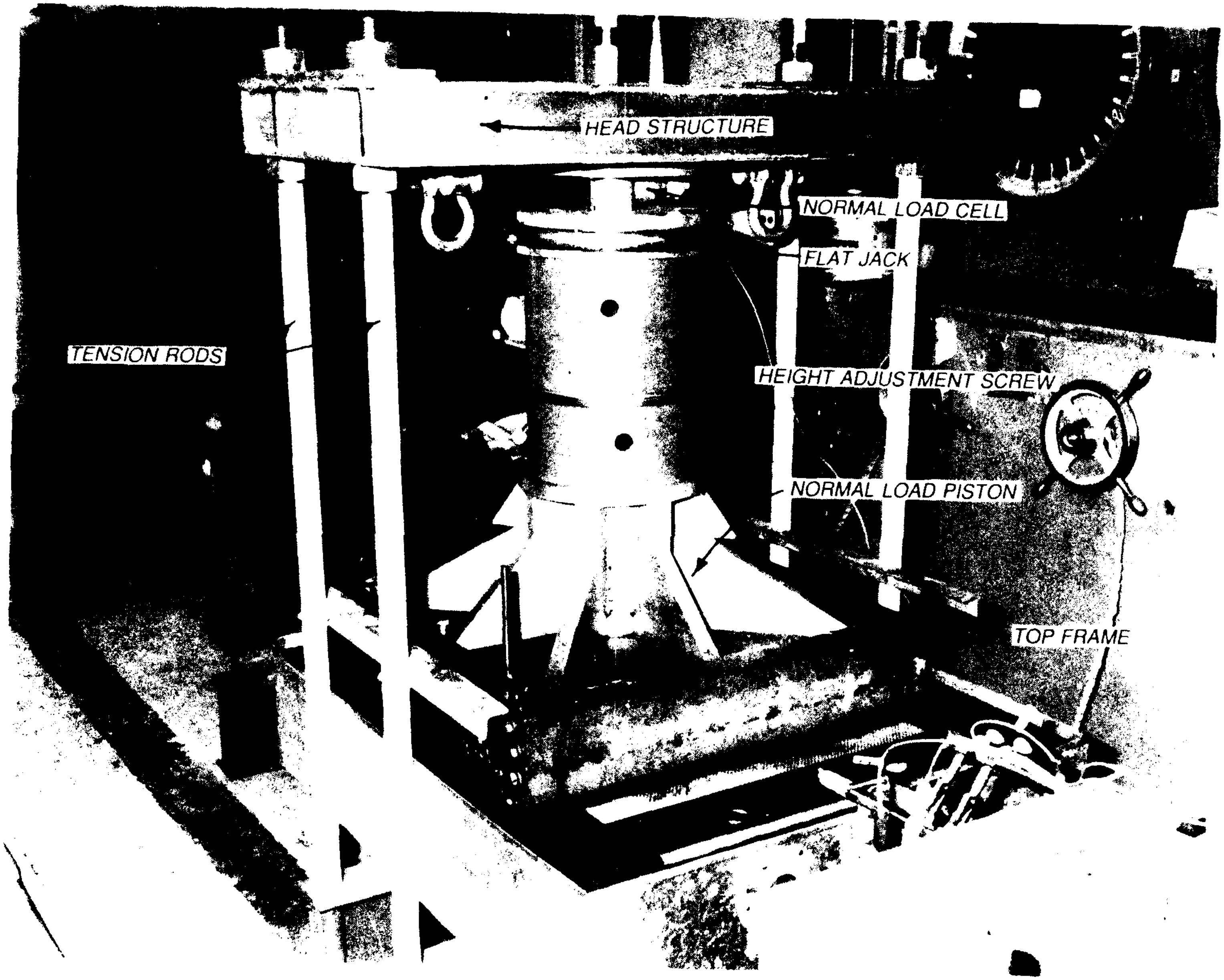


Figure 2. Pullout test configuration

18. Shear force is applied by two worm gear actuators whose points of force application are separated by a distance of 11.5 in. in the horizontal plane. The fact that the system is devised with two actuators separated by a distance allows a more uniform load to be applied to the fabric edge. The actuators have a load capacity of 30 tons each, are fitted with hardened Acme threaded rods, and are driven by a 3/4-hp variable speed direct current motor acting through a transmission. The motor and drive system are capable of applying rates of deformation from 0.001 to 0.250 in./min. Shear force is measured with two 50,000-lb-capacity load cells that are electronically summed to indicate total force applied by the shear force system. Horizontal displacement during the test is measured with a 5-in. linear variable differential transformer (LVDT); horizontal displacement and shear force are recorded on an X-Y analog continuous line recorder.

19. It must be stated that the tests performed in this laboratory study are slightly different from usual direct shear tests and required a different testing configuration. The objective of the test program was, essentially, to measure the coefficient of the friction or maximum pullout resistance between a fabric (geotextile) and soil. The objective was accomplished by applying several different normal loads to a layered soil/fabric system placed in the direct shear box and measuring the force required to move the fabric against the soil under different normal loads. To enable the fabric to be pulled in a manner considered to be predictable and repeatable (from test to test), one edge of the fabric was wrapped/looped between three 1/4-in. steel plates which were then clamped very tightly against the fabric to prevent slippage (see Figure 3). The plates were then attached to the screw actuators with clevis pins. The system was devised such that the actuators applied their pull to the fabric in the plane where the two halves of the shear box came together. Therefore, no component of the pull could cause eccentric loading about the plane separating the two halves of the shear box to misalign the testing apparatus and corrupt measurement of soil/fabric resistance. Several soil water contents were investigated in the study to characterize variation in pullout resistance of clay with water content.

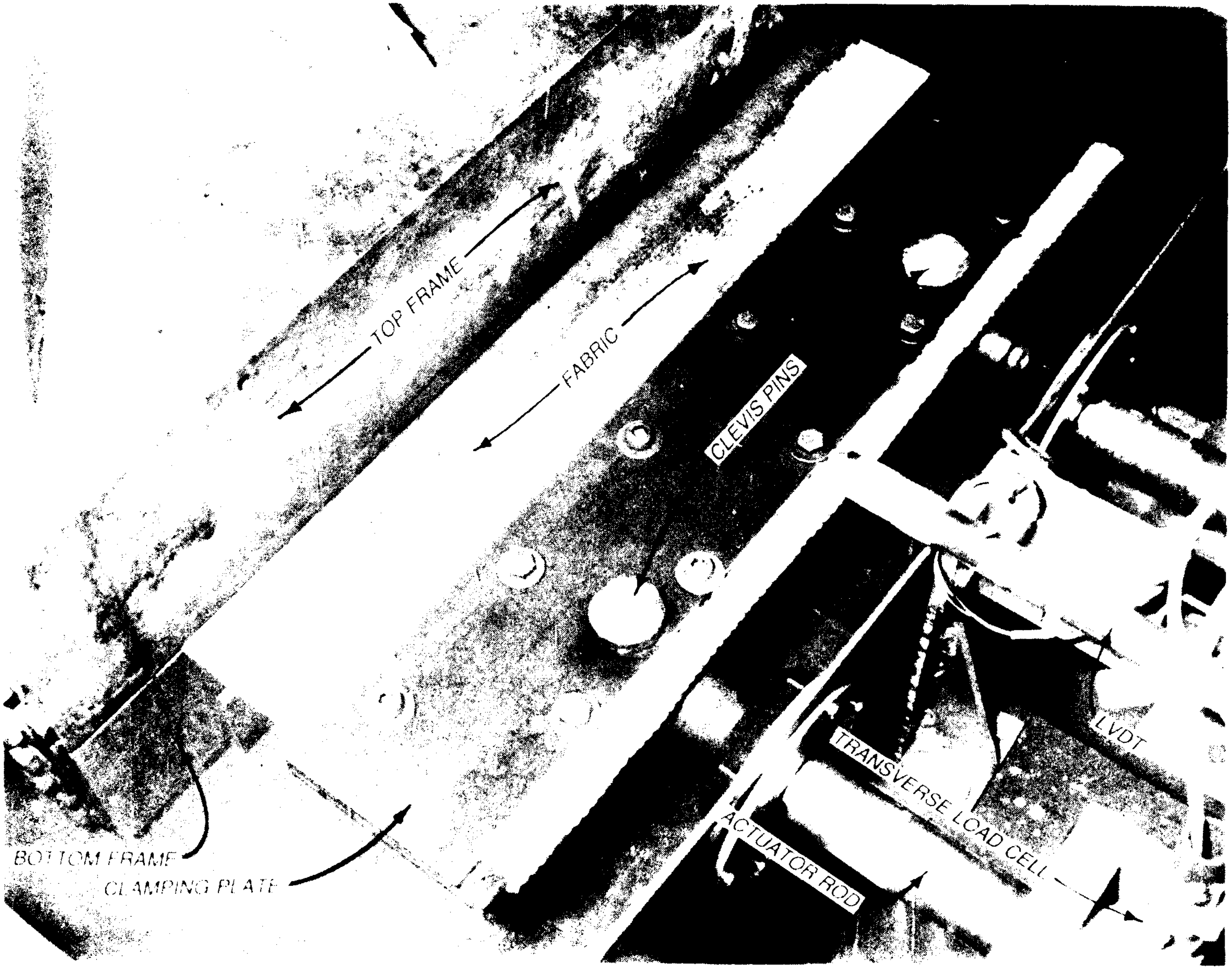


Figure 3. Geotextile clamp system

PART IV: TEST TYPES

Anchorage Test

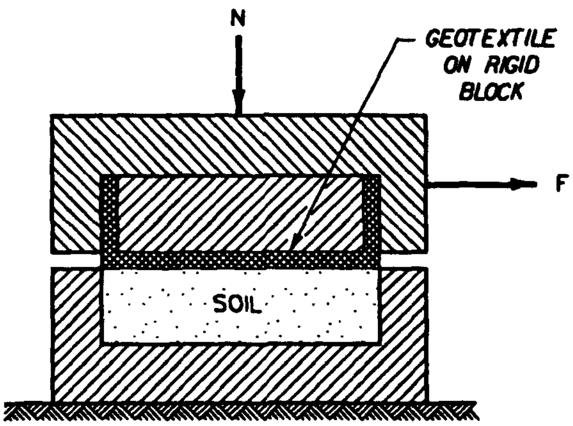
20. Two types of tests were performed in this investigation: the pullout test (sometimes called the anchorage test) and the fixed shear test (which is also known as the soil-to-fabric friction test) (Martin, Koerner, and Whitty 1984). In the pullout test, the geotextile is sandwiched (embedded) between two layers of soil, a normal stress is applied to the system, and after a period of consolidation, the fabric is pulled out of the soil mass. In this type of test, the soil on each side of the fabric can be identical, or there is the option to place a different soil type on one side of the fabric than is on the other.

Soil-to-Fabric Friction Test

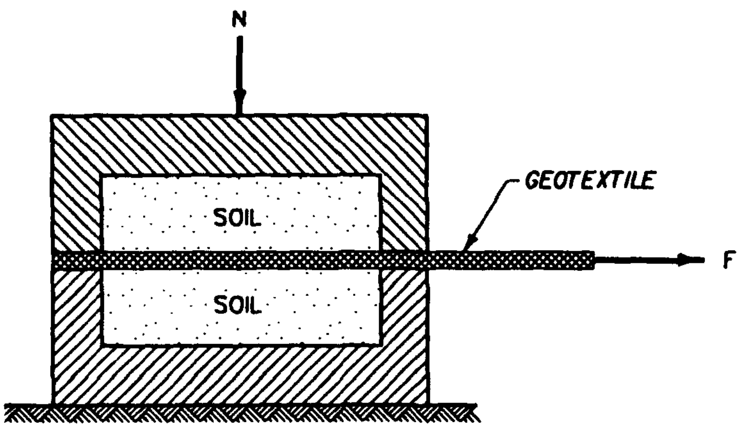
21. In the soil-to-fabric friction tests of this investigation, the upper half of the shear box is filled with soil, and the fabric is supported by the (planar) surface of a rigid mass placed in the lower half of the shear box; normal pressure is applied to the system, and after consolidation, the soil in the top half of the shear box is pulled across the stationary fabric. An obvious variation of the soil-to-fabric friction test just described is that the fabric and its support block may be fixed in the top half of the shear box and the soil contained in the bottom half.

Discussion

22. The anchorage and soil-to-fabric friction tests are shown schematically in Figure 4, which was taken from Koerner (1986). Figure 5 shows typical graphical presentation of pullout resistance test results; the shear box represented has effective area, A; normal force, N, is applied to the specimen, and maximum parallel force, F_{max}, is measured. δ and C_a are the apparent friction angle and adhesion, respectively, between the soil and geotextile. The force measured in the pullout test is ideally twice as great as the corresponding force measured in the soil-to-fabric friction test because the fabric surface area in the pullout test is twice as much as that in the soil-to-fabric test.



a. Soil-to-fabric friction test



b. Fabric pullout (anchorage) test

Figure 4. Schematic diagrams of test setups for friction and pullout evaluation of geotextiles in soils (modified from Koerner (1986))

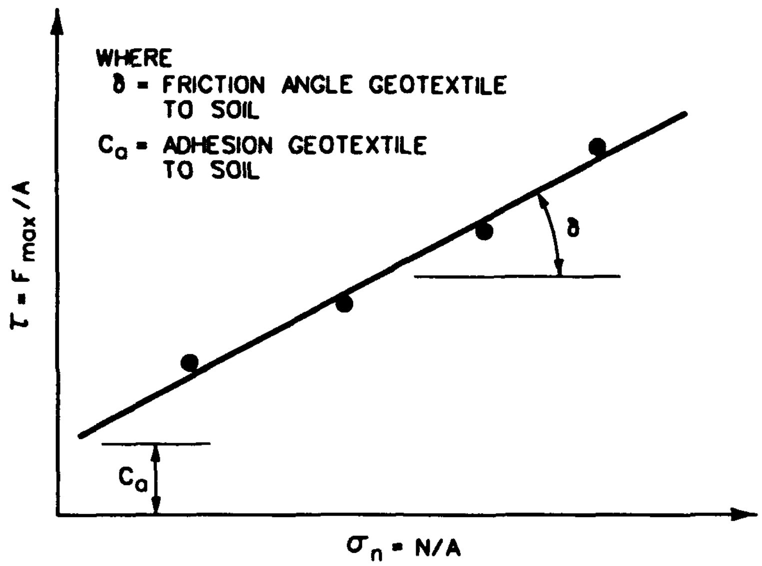


Figure 5. Resistance test representation

23. An advantage of the pullout test configuration in the laboratory is that dissimilar soils may be placed on each side of the fabric to model situations in construction where different materials occur on each side of the fabric; a disadvantage of this configuration is that twice the amount of soil must be processed and placed in the test apparatus.

PART V: LABORATORY TEST PROCEDURES

Background

24. One of the objectives of the investigation was to compare laboratory and field pullout results for the purpose of determining pullout resistance for the soils and fabrics in question. It was recognized that soil/fabric pullout resistance will be influenced by (a) water content and compaction process applied to the fill soil, (b) normal pressure between soil and fabric, (c) rate of applied deformation/loading, and (d) whether the soil/fabric system is inundated (submerged) or in the dry. The laboratory investigation that was designed after the field study was devised to address these issues and resolve questions not fully answered by the field study. The laboratory study was performed in three phases that were modified as work progressed to make the most effective use of available resources by taking advantage of information gained and lessons learned during the study. The initial research plan called for testing a combination of four different soils with three geosynthetics (two geotextiles and a geogrid) and two normal loads. However, as geogrids were not being used in the reinforced levee systems in the NOD, they were eliminated from the laboratory program. Similarly, it was initially planned to perform a complete suite of tests on three different fabrics (geotextiles), but after performing a set of comparative tests on the fabrics, it was determined that the pullout/frictional response of all the fabrics was similar enough to be considered identical, and as a result it would be necessary to perform a complete suite of tests on only one fabric.

25. Basically, as the result of lessons learned during preliminary stages of the laboratory investigation combined with field experience and expertise, the study evolved into a program where it was determined to test one fabric and one soil at three water contents under three normal loads. Three lots of material were received from a borrow pit at the Bonnet Carre Spillway for the laboratory test program. Although the materials came from essentially the same location in the borrow pit, the Atterberg limits varied slightly. The material for phase 1 was classified CH with a liquid limit of 53 percent and a plasticity index of 34 percentage points. After laboratory processing and preparation for testing, phase 2 material was classified CL, was seen to possess different compaction characteristics than the phase 1 material, and was of a notably different color and texture. The liquid limit

of phase 2 material was 46 percent, and the plasticity index, 28 percentage points. Phase 3 material was slightly more plastic than phase 1 material with a liquid limit of 57 percent and a plasticity index of 38 percentage points. Since the borrow area soils at the Bonnet Carre Spillway are generally very plastic and since highly plastic soils generally present a more difficult field construction problem, a full suite of tests was not performed on the phase 2 soil since it was not very characteristic of the site, nor did it represent the most difficult condition.

Soil Processing

26. Soil used in the program was received in 55-gal barrels in a moist condition. The material was processed by drying it in a 60° C oven, then pulverizing the larger chunks into particles that pass a No. 4 sieve. The material was then placed into a commercial food processor, an amount of water was added that was sufficient to bring the soil to the desired test water content, and the soil and water were thoroughly mixed. The mixture was then placed into airtight 5-gal steel cans and allowed to cure for a minimum of 24 hr before placement in the test apparatus.

Soil Compaction

27. Soil compaction in the field investigation was achieved by four passes of a low ground pressure (4.7 psi) bulldozer on lifts with a loose thickness of about 15 in. and a compacted thickness, roughly, of about 12 in. It was determined that 15 blow compaction (60 percent of standard compactive effort, ASTM D 698) duplicates the density achieved in the field at the water content selected for testing. Therefore 15 blow compaction curves were developed for each of the four materials tested in this program. The specimens were compacted in three layers using a 5.5-lb rammer falling through a distance of 12 in. The compaction curves are shown in Figures 6, 7, 8, and 9. The materials are: (Figure 6) a silty sand (SM) that was used in phase 1 as the second soil in pullout tests in which different materials were placed on each side of the fabric, (Figure 7) a clay (CH) that was the predominant soil of phase 1, (Figure 8) a clay (CL) that was tested in phase 2, and (Figure 9) a clay (CH) that was used in phase 3 and is considered to be very similar to the clay of phase 1.

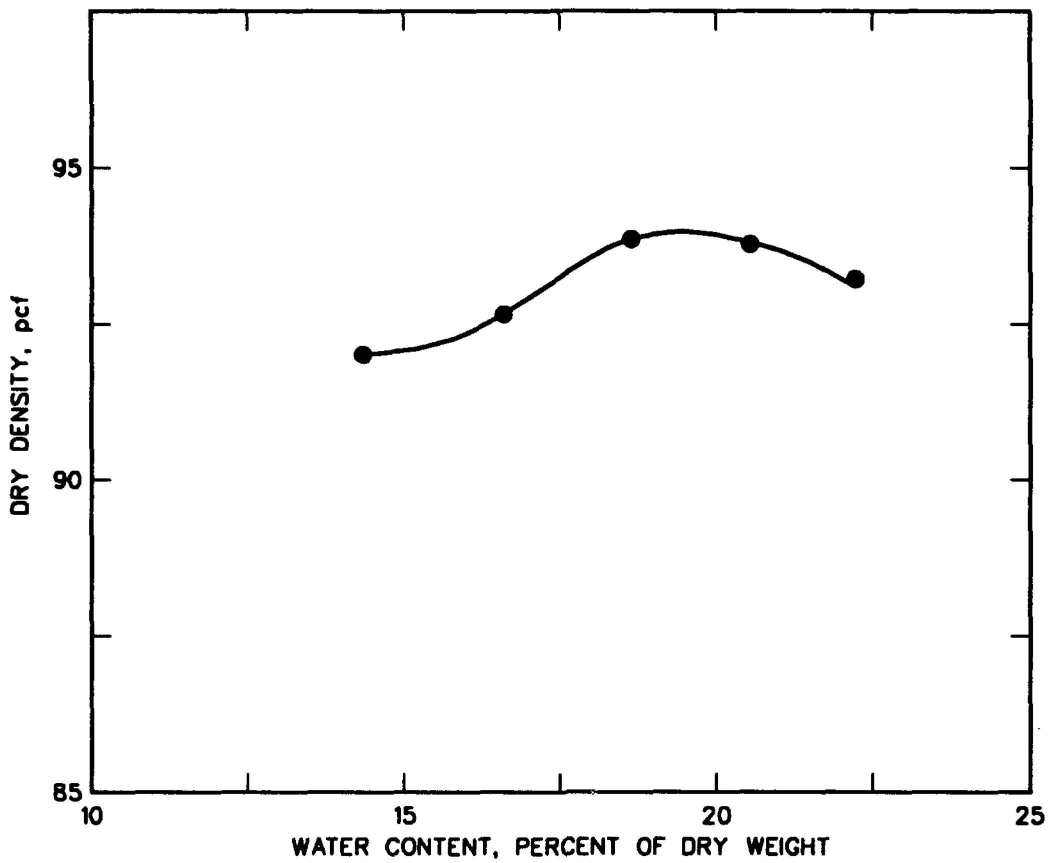


Figure 6. Compaction of silty sand of phase 1

Specimen Molding/Placement

28. Desired specimen average density was obtained by placing a known mass (weight) of moist material into the shear box and compacting it with a tamper into a predetermined volume (see Figure 10). Since the plan area of the shear box was fixed, it was sufficient to compact the appropriate mass of material to a predetermined height in the box. The pullout specimen was compacted in four layers, two in the bottom half of the shear box and two in the top half. When the bottom half of the specimen had been properly prepared, the test fabric was placed on that soil surface and connected to the actuator force assembly through the geotextile clamp mechanism. The upper portion of the shear box was then set in place and the top half of the specimen prepared in the same manner as the bottom.

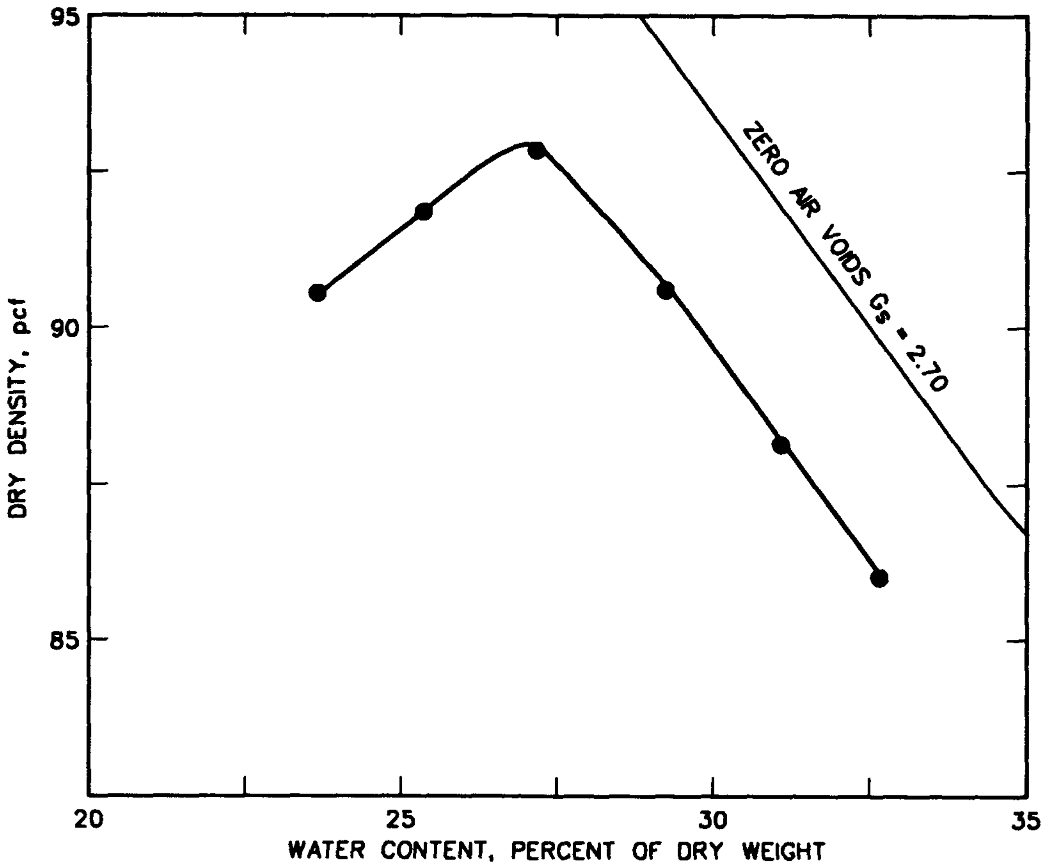


Figure 7. Compaction of clay (CH) of phase 1

Consolidation

29. After compaction, the piston and head assembly were placed on the test specimen, normal load was applied with the flatjack, and the specimen was allowed to consolidate for 24 hr. In tests performed in the inundated state, water was placed in the shear box reservoir up to a level about 2 in. above the soil/fabric plane immediately after consolidation load was applied. Time versus settlement readings were not usually observed during consolidation since it was discovered early in the investigation that 24 hr was sufficient for the completion of primary consolidation. (In fact it was learned that the time required for completion of primary consolidation was slightly less than 4 hr.) However, before the initiation of pullout during a test, it was confirmed (by monitoring with a dial gage for about 1 hr) that settlement had, in fact, ceased. An advantage of the direct shear test is that specimens are

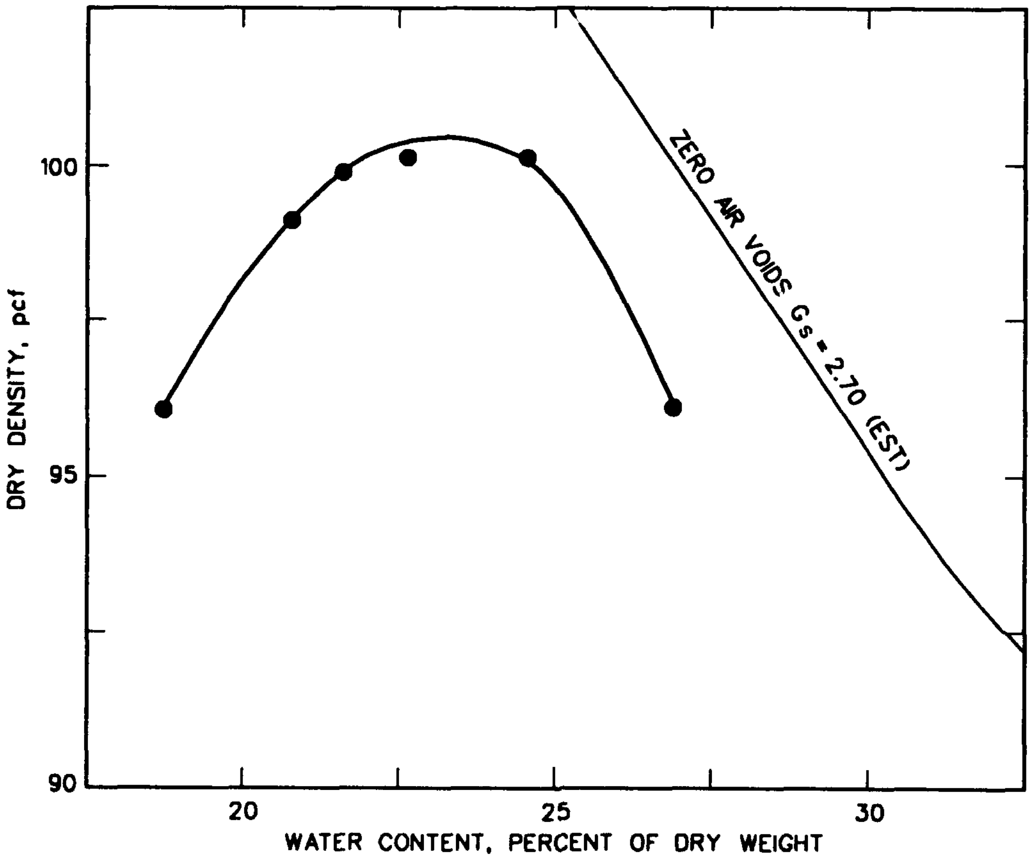


Figure 8. Compaction of clay (CL) phase 2

relatively thin for the amount of volume involved, and therefore consolidation occurs quickly. The presence of a geotextile in the test specimen enhances this advantage to an even greater extent. Although total specimen thickness was 8 in. in this investigation, drainage was facilitated by the fabric, which literally cut the soil thickness in half and, as a result, the longest drainage path in the specimen was 2 in. Consolidation time was further reduced because the specimens were generally unsaturated and pore air pressure was able to escape very quickly from the soil pore space.

Resistance Determination

30. Two procedures were used for resistance determination: direct pullout and fixed shear. In performing the direct pullout procedure, a 3/8-in. gap was set between the upper and lower halves of the shear box and the fabric pulled using the screw actuators. The gap was set and maintained

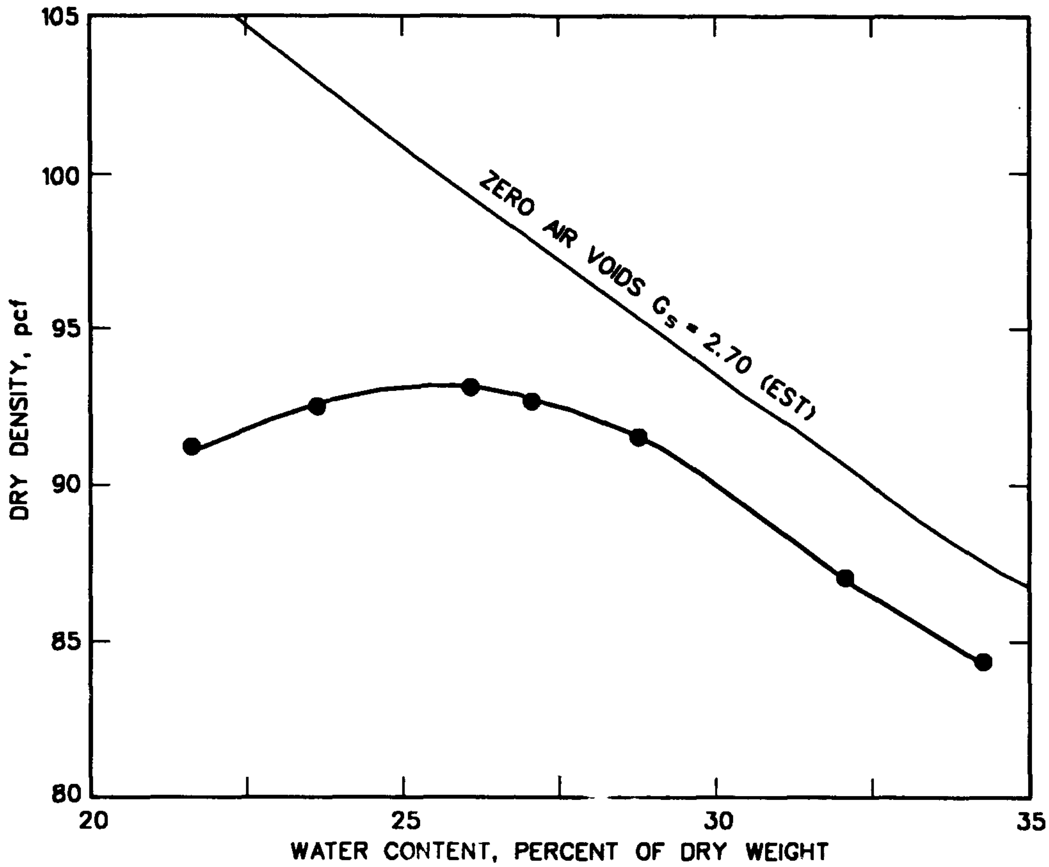


Figure 9. Compaction of clay (CH) of phase 3

during pullout to ensure that the fabric was pulled without coming into contact with the box. Operating in this manner allows the measurement of pullout resistance that is not corrupted by incidental boundary friction. During direct pullout, force and the corresponding deformation were recorded electronically with a continuous line recorder. The test was stopped when 4-in. of fabric had been pulled out of the shear box by the two actuators.

31. In the fixed shear procedure, the shear box was initially gapped and Teflon strips were placed between the top and bottom halves of the box to minimize friction that would result if the two halves of the box came into contact during the test. The soil that was in contact with the fixed fabric was then pulled along with the top half of the shear box and the resisting force measured and recorded (electronically) along with the corresponding

Figure 10. Technician preparing pullout specimen

deformation. The test was continued until the top half of the shear box had been displaced 4 in. by the actuators.

32. Two rates of displacement/pullout were used in this investigation to study rate effects; they were 0.25 in./min (0.635 cm/min) and 0.0016 in./min (0.004 cm/min). The rates were somewhat dictated by the capacity of the testing apparatus in that the two rates selected represent, basically, the upper and lower speed limits of the displacement control system. Tests performed at the faster rate will be referred to as "quick" tests, and those at the slower rate will be called "slow" tests.

33. A typical force-displacement relationship for a geotextile pullout test is shown in Figure 11. The geotextile is initially in an unstressed/ unstretched state. The fabric is drawn taut and begins to experience tensile stress (and strain) as edge pullout force is applied. As pullout force is increased, the embedded length of fabric subjected to tensile stress begins to extend farther into the soil mass. With continuously applied edge displacement, the entire embedded length of the geotextile will become stresses (in tension), and the back edge of the fabric will begin to move (this position is identified on the force-deformation curve of Figure 11). At this position, the maximum available stress/strength available from the soil/geotextile system is mobilized, as demonstrated on Figure 11. Obviously, if the depth of embedment of a geotextile is sufficiently great, a pullout force can be developed that exceeds the strength of the fabric, causing it to fail in tension. Geotextile embedment depth greater than that which produces pullout force exceeding the strength of the fabric is unproductive. Laboratory and/or field tests along with analysis will allow designers to determine practical and productive embedment depths for geotextiles under site-specific conditions.

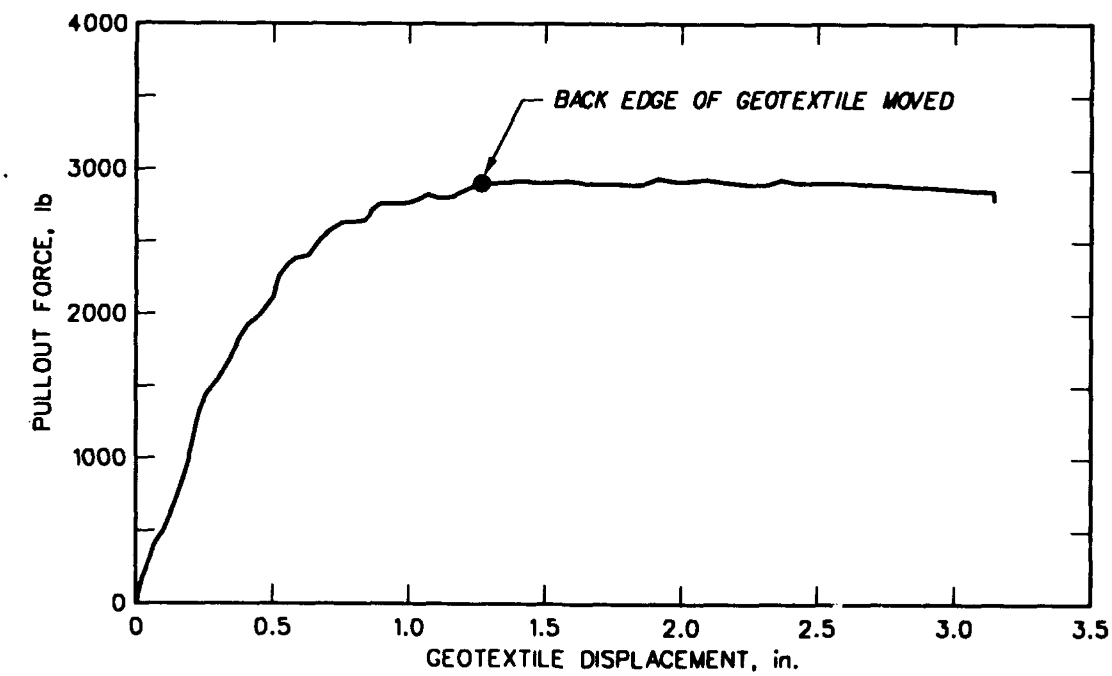


Figure 11. Geotextile pullout force versus movement

PART VI: MATERIALS TESTED

Soils Tested

34. Four soils were tested in this investigation, two very similar clays (CH), a silty clay (CL) and a silty sand (SM). The materials were all taken from borrow pits near the Bonnet Carre Spillway. The silty sand is a uniform material all of which is finer than the No. 50 sieve. Hydrometer analyses of the phase 1 and 2 clays and a grain-size distribution curve for the sand (of phase 1) are shown on Figures 12, 13, and 14, respectively. A hydrometer analysis of the phase 3 soil was not performed because of its similarity to the phase 1 soil.

35. Typical Atterberg limits of phase 1 clay are LL = 53 percent, PI = 34 percent; the material has a specific gravity of 2.70 and is visually identified as brownish gray clay (CH) with a trace of sand. A gray silty nonplastic sand (SM) with a specific gravity of 2.67 was also tested in phase 1. The phase 2 soil is visually classified as a brown clay (CL) with Atterberg limits, LL = 46 percent and PI = 28 percent. The phase 2 soil has a specific gravity of 2.70.

Geotextiles

36. Three high-strength polyester woven geotextile fabrics were tested in this investigation; the trade names and/or companies associated with the fabrics are Nicolon, Wellman Quline, and Exxon. The Wellman Quline and Exxon geotextiles are layered, needle-punched polyester fabrics with stitch lines approximately 0.25-in. apart throughout the fabric. The Nicolon is a heavy single woven fabric. Manufacturers' technical information on the fabrics tested, if available, is given in Table 2. Only a small number of the tests in this investigation were conducted on the Wellman Quline and Exxon fabrics; the vast majority of tests were performed with the Nicolon fabric since laboratory tests revealed that pullout resistance is essentially identical for all three fabrics, as is shown on Figure 15. The fabric comparison tests were conducted on the phase 1 clay (CH) at a water content of 32 percent and at the "quick" rate of displacement of 0.25 in./min under inundated conditions. A least squares analysis of the pullout data shows that the coefficient of determination for all the fabric/soil data taken together is 0.96, which

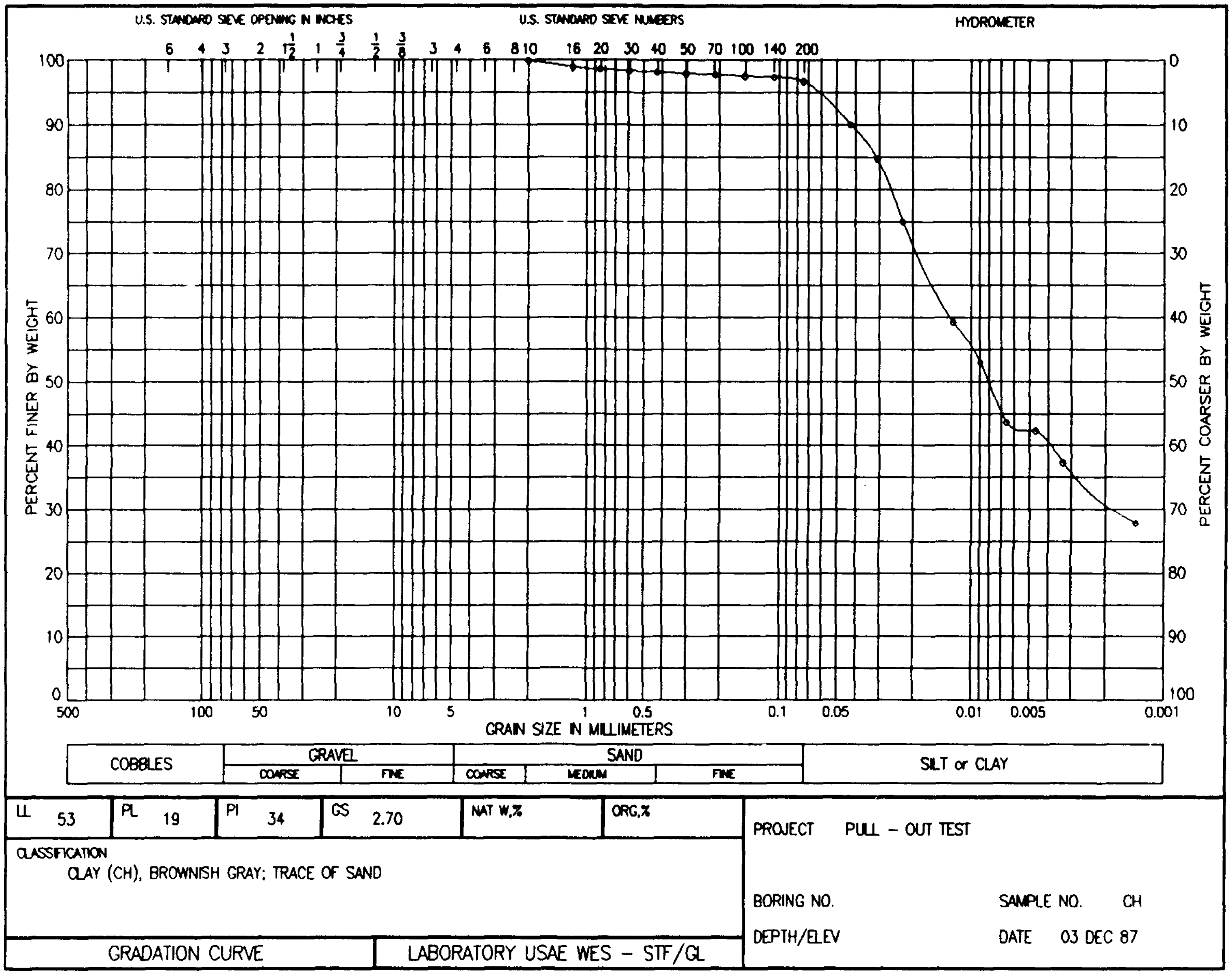


Figure 12. Hydrometer analysis of phase 1 soil

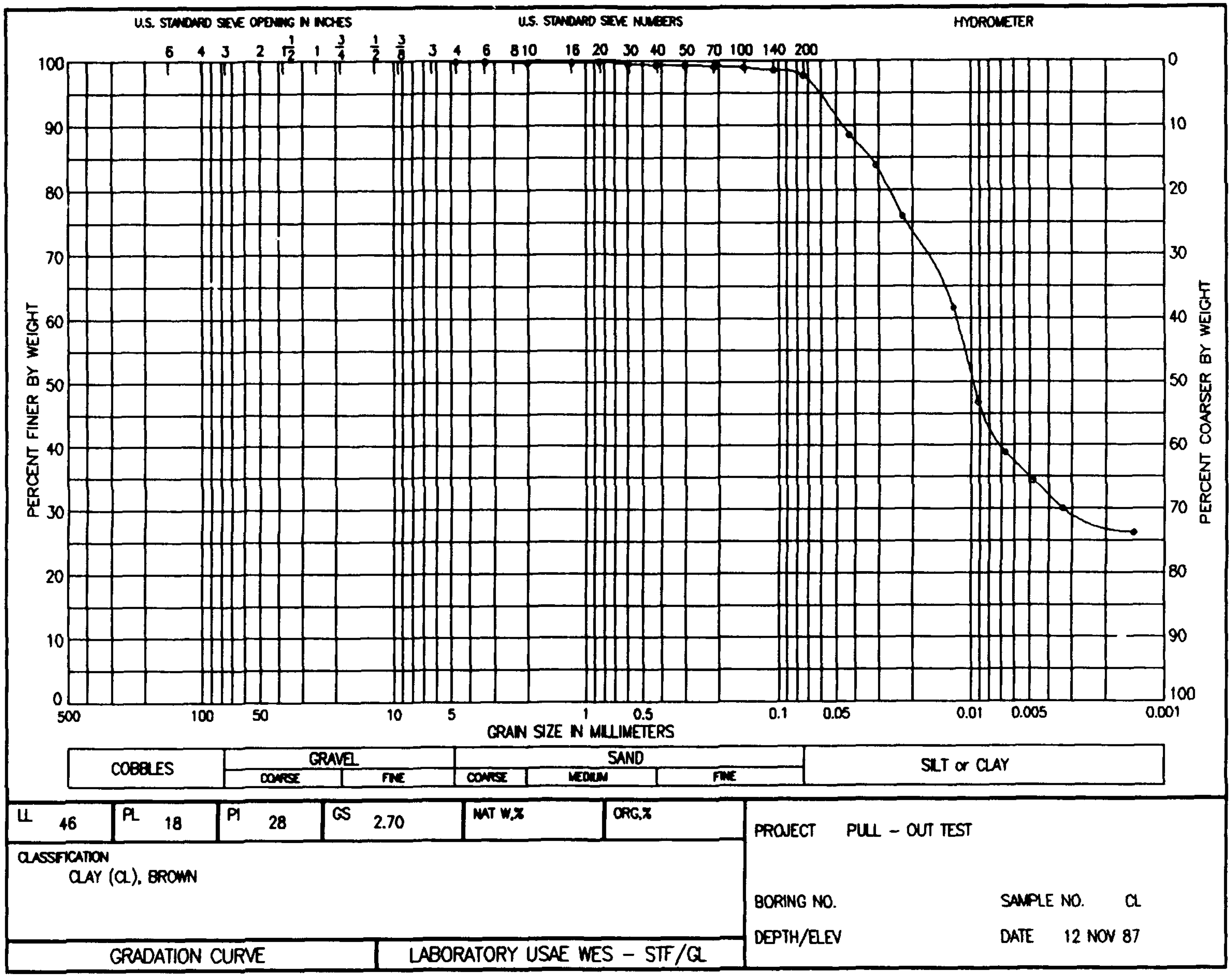
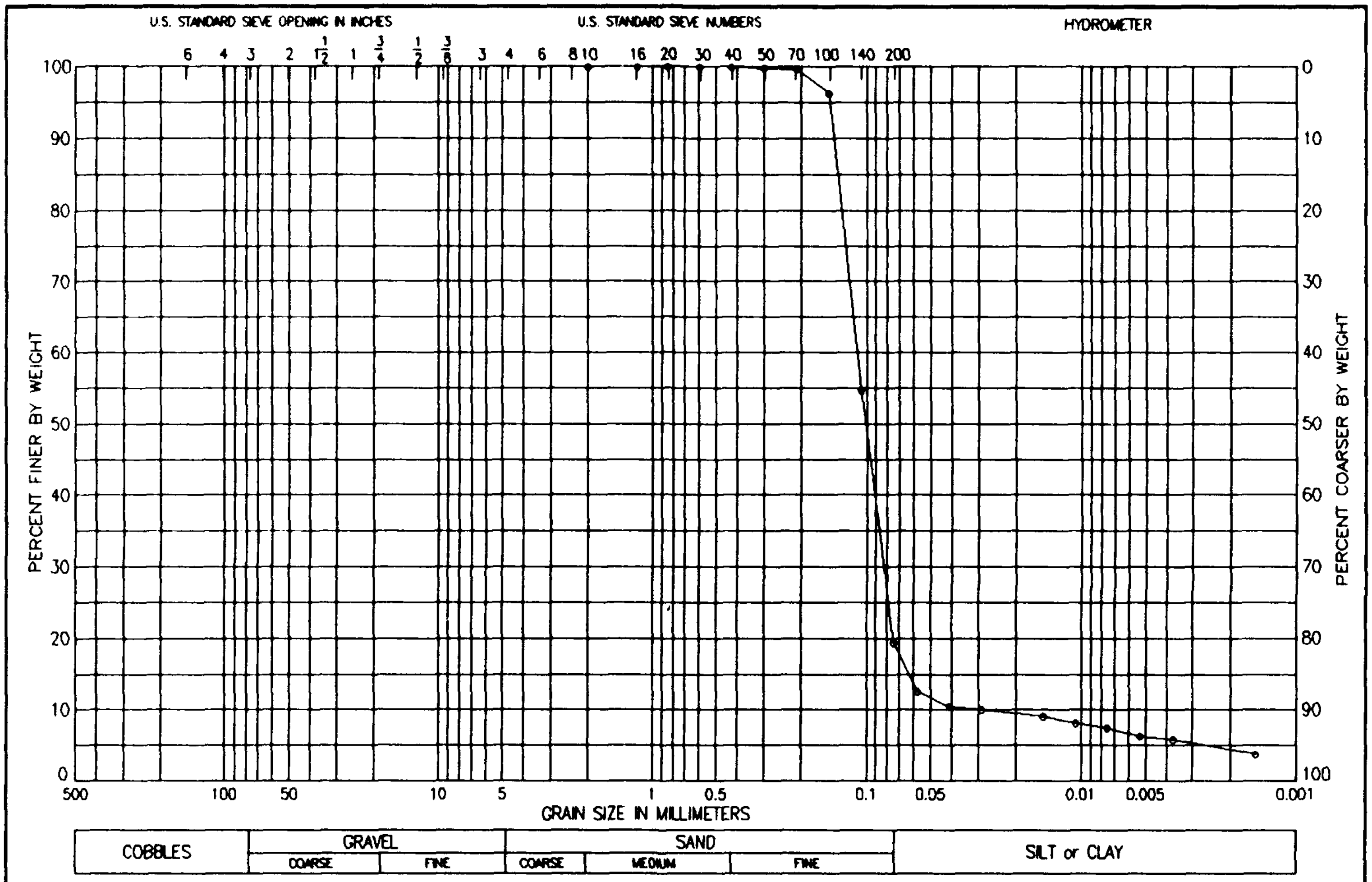


Figure 13. Hydrometer analysis of phase 2 soil



LL NP	PL	PI	GS 2.67	NAT W,%	ORG,%

CLASSIFICATION
SILTY SAND (SM), GRAY

PROJECT PULL - OUT TEST	
BORING NO.	SAMPLE NO. SM
DEPTH/ELEV	DATE 10 FEB 88

GRADATION CURVE | LABORATORY USAE WES - STF/GL

Figure 14. Grain-size distribution of phase 1 silty sand

Table 2

Typical Manufacturer Stated Physical Properties

of Geotextiles Tested*

Property	Wellman Quline	(Single Layer)* Exxon	Nicolon
Weight (oz/yd^2)	40	16	60.7
Thickness (mils)	320	70	134
Grab strength (lb) (ASTM D-4632)	850	--	--
Grab elongation (%) (ASTM D-4632)	80	--	--
Puncture strength (lb)	410	360	1160
Wide width strength (lb/in.) (Warp Direction x Fill Direction)	--	1000 x 850	2775 x 1725
Wide width elongation (%)	--	11	11.3
Mullen burst strength (psi)	1100	>1200	>1500
Apparent opening size (Sieve Size)	140 - 230	40 - 100	100
Permittivity (gal/min/ft^2)	80	--	6
Coefficient of permeability (cm/sec)	--	0.01	0.028

* Fabric tests consisted of three single layers stitched together.

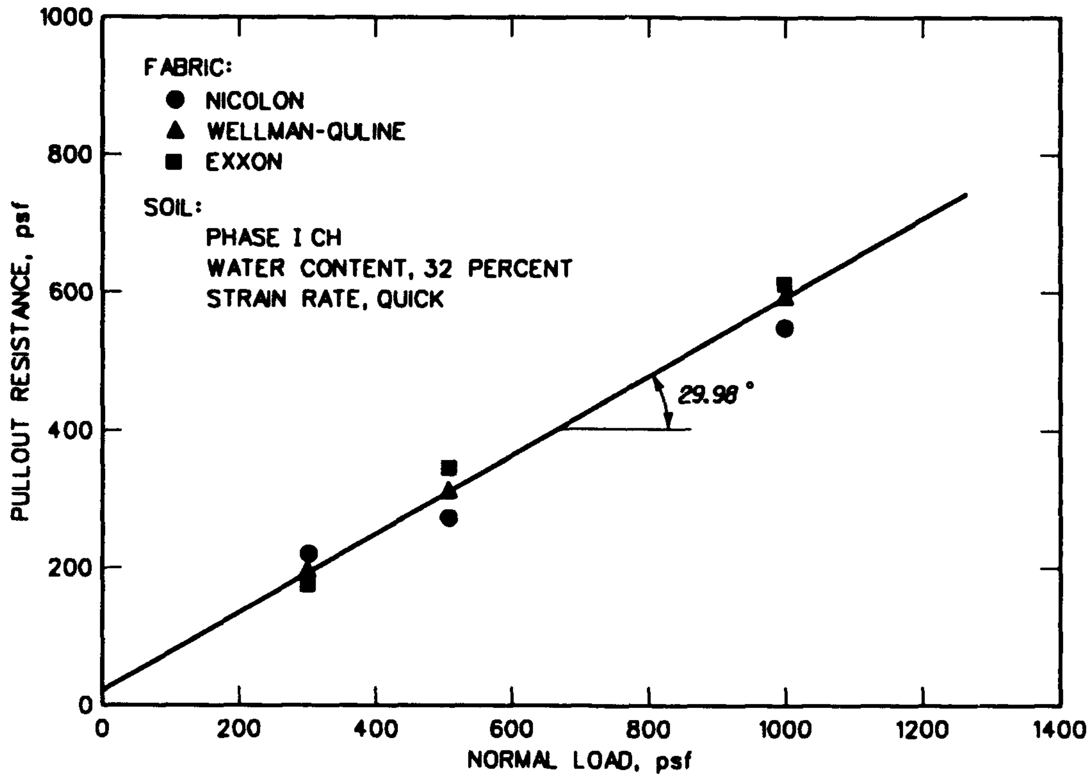


Figure 15. Pullout comparison tests conducted on Nicolan, Wellman Quline, and Exxon geotextiles

indicates very good linear correlation for these data and demonstrates that there is no identifiable difference between fabrics (as far as pullout resistance is concerned). The friction angle is computed to be 29.98 deg for these data, and the cohesion/adhesion intercept is about 29 psf.

37. Stress-strain characteristics and the tensile strength of the Nicolon fabric were determined by direct tension tests performed by the NOD and by the Nicolon Corporation, who furnished stress-strain data for their fabric; the results are summarized on Figure 16. The two tests show very similar material behavior; however, the test procedures used were very similar in each case. In each test, the fabric was pulled in the warp direction at a deformation rate of 0.4 in./min and began from a preload condition which was 60 lb for the NOD and 100 lb for Nicolon Corporation. Each test specimen was approximately 4 in. wide by 4 in. long. Figure 16 shows that the load level in each test was about 1,500 lb/in. at 5-percent strain, and fibers began to break in both specimens between 9- and 10-percent strain. After partial recovery, modulus values diminished somewhat up to the maximum load, which was 3,775 lb/in. at 10.7-percent strain in the NOD test and 4,120 lb/in. at

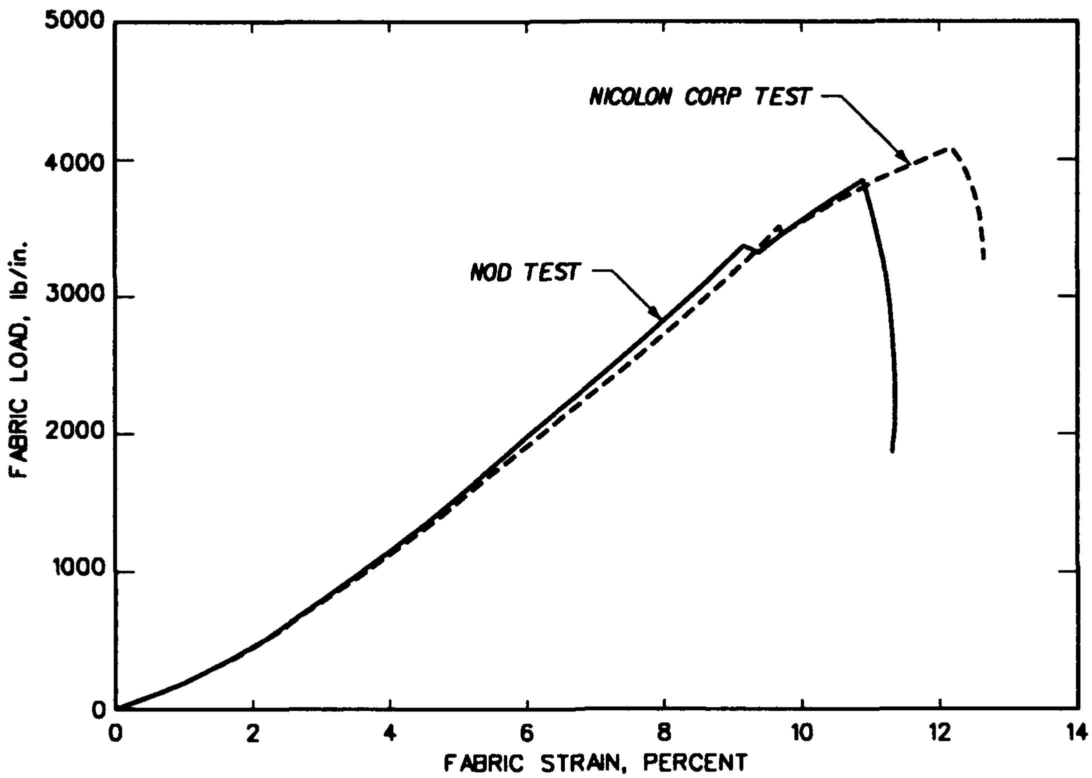


Figure 16. Stress-strain characteristics of Nicolon fabric

12-percent strain in the test by the Nicolon Corporation. After the maximum load occurred, load carrying capacity of the fabric decreased dramatically with the application of additional deformation up to what must be considered rupture. Specifications for the project on which the Nicolon fabric was used, given by the NOD (1989a and b), show a load level of 1,250 lb/in. at 5-percent strain and an ultimate load of 2,500 lb/in., both of which are lower than the results measured in the tests and shown on Figure 16.

PART VII: FULL-SCALE TESTS BY NOD

38. Two full-size pullout studies were performed by the NOD, one at the Bonnet Carre Spillway and one at the Belle Chasse, LA, landfill. These studies will be described briefly because laboratory and full-scale test results will be compared later. The geotextile used in the field studies was a heavy (60 oz/yd^2) polyester fabric manufactured specifically for the NOD by the Nicolon Corporation. In both field studies, the size of the fabric tested was 24 ft long by 6.75 ft wide; massive pieces of construction equipment were used to provide reaction for the pullout force, which was applied by a hydraulic winch acting through a pulley system. Pullout force was measured with dynamometers, and normal stress was applied (and quantified) in terms of the height of fill. Tensile force was transferred/applied to the fabric through a stiff reinforced steel pipe which slipped through a sleeve sewn in the fabric perpendicular to the warp direction. The fill in each study was retained by a mat consisting of heavy timbers laid against vertically driven, timber support piles, 12 in. in diameter. A slot/gap was provided in the mat through which the fabric was pulled.

39. The Bonnet Carre Spillway tests were performed under fill heights of 3, 4.5, and 6 ft in a plastic clay at three water contents, approximately 26, 35, and 40 percent. The fill was placed and spread with a low ground pressure bulldozer which applied 4.7 psi underneath its tracks; four passes of the bulldozer provided the compaction required. The average wet density of the clays tested was 117 pcf. Water content was measured with a nuclear gage and checked in the NOD Laboratory in the conventional oven. It should be noted that the differences between the conventional oven water contents and the nuclear gage water contents was considerable in some instances. The standard deviation of the difference between the conventional water content and the nuclear gage water content is 3.9 percent.

40. Tensile pullout force was measured with an MSI dynamometer rated at 300,000 lb and graduated in 100-lb increments. High-strength piano wires were attached to the fabric in seven locations within the fill section to determine the load (and corresponding amount of applied deformation) to cause each location to move during the test. The piano wires were extended out of the back of the test fill, guided over a system of pulleys, and stretched taut by hanging a small weight on the end of the wire. When the fabric had been strained by the applied force to the extent that a particular point began to

move, the hanging weight attached to that point by wire outside the fill also began to move. With this simple but effective system, internal movement of the fabric could be tracked during load application. A general schematic of the test configuration is shown in Figure 17.

41. The Belle Chasse tests were performed using techniques very similar to the Bonnet Carre tests under fill heights of 3, 4, and 6 ft in a heterogeneous soil mix consisting of plastic clay, silt and fine sand; however, when the soil components were mixed together in a homogeneous mass, the resulting material was classified as lean clay (CL). The mixed fill (CL) tested had an average wet density of 103.4 pcf and a water content of 38 percent. A Dillon dynamometer with a load capacity of 100,000 lb graduated in 500-lb increments was used for force measurement. In these tests, high-strength piano wire was attached to the fabric at three locations within the fill section to determine the load and deformation in each section of the fabric during the test.

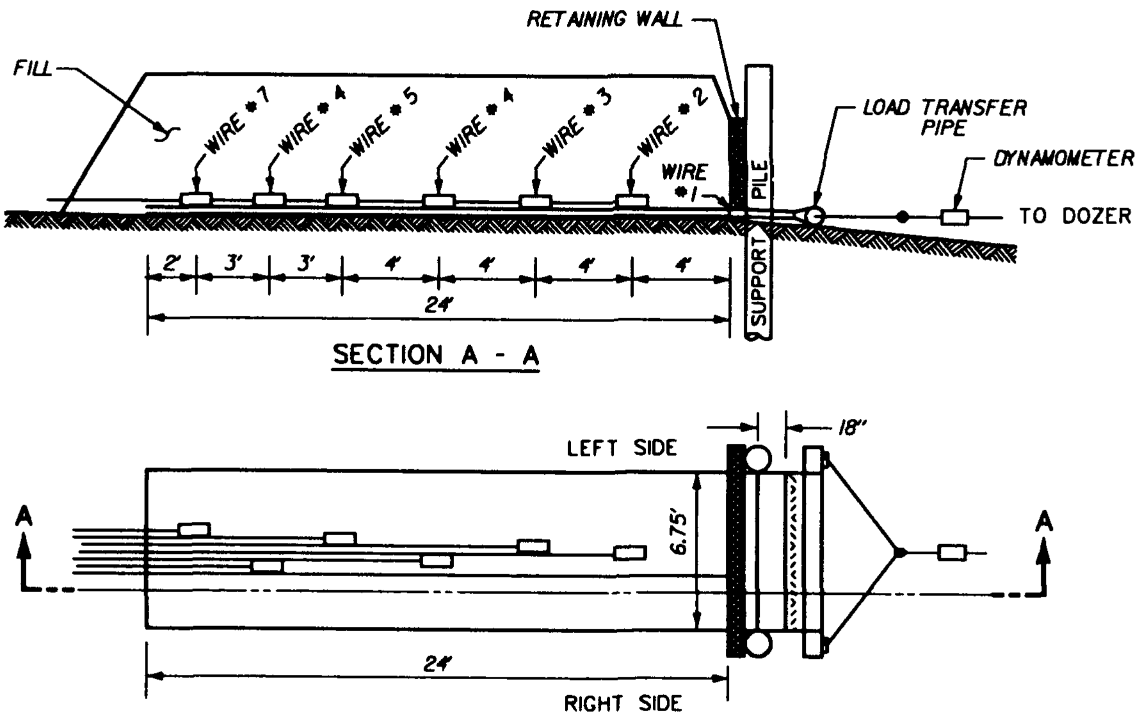


Figure 17. Schematic of test configuration at Bonne Carre Spillway

PART VIII: LABORATORY TEST RESULTS

42. The investigation was conducted in three phases, as stated earlier. Test results will be presented in the three phases in which they were conducted. Phase 1 tests are summarized on Table 3; phases 2 and 3 are summarized on Tables 4 and 5, respectively. Pullout force-deformation relationships for all the tests conducted in phases 1, 2, and 3 are included in Appendices A, B, and C, respectively.

43. As can be seen from the tables, the purpose of phase 1 was to determine the difference in resistance offered by the different geotextiles. Two soils were used in phase 1, a clay (CH) and a silty sand (SM). Phase 2 was used to evaluate the influence of normal stress. The specimens were usually not inundated and were (with two exceptions) tested at the quick rate of displacement.

44. In phase 3, the effect of water content variation was investigated. A normal load was selected for the study which represented a typical condition in the field. Twenty-two tests were performed in phase 3; six were consolidated under a normal stress of 350 psf, 12 under 550 psf and the remaining four under 1,000 psf. Five-hundred-fifty psf represents an embankment height of 5 ft in a clay with a wet density of 110 pcf, and this stress was considered to be typical for some geotextile reinforced structures in the NOD.

Table 3

Data Summary for Phase 1 Tests

Test No.	Max. Pullout Force, lb	Normal Load psf	Max. Soil/Geotextile Interface Stress psf	Fabric	Soil Types	Type Test	Horizontal Displacement at Failure in.	Comments
1	1690	500	220	Nicolon	CH/CH	Pull-out	0.634	
2	3750	1000	489	Nicolon	CH/CH	Pull-out	2.292	
3	1250	500	163	Wellman	CH/CH	Pull-out	0.650	Slipping observed
4	1640	300	214	Nicolon	CH/CH	Pull-out	0.796	
5	1410	300	184	Wellman	CH/CH	Pull-out	1.729	
6	2490	500	324	Wellman	CH/CH	Pull-out	1.332	
7	1350	300	176	Exxon	CH/CH	Pull-out	0.732	
8	2640	500	344	Exxon	CH/CH	Pull-out	1.585	
9	4740	1000	617	Exxon	CH/CH	Pull-out	2.179	
10	4160	1000	542	Wellman	CH/CH	Pull-out	1.759	
11	1450	300	189	Nicolon	CH/SM	Pull-out	1.000	
12	2210	500	288	Nicolon	CH/CH	Pull-out	0.954	Possibly slipped
13	4410	1000	574	Nicolon	CH/CH	Pull-out	1.423	
14	2900	500	378	Exxon	CH/SM	Pull-out	1.687	
15	5160	1000	671	Exxon	CH/SM	Pull-out	3.569	

(Continued)

Notes:

1. All soil specimens inundated in phase 1.
2. Strain rates were quick (0.25 in./min) except as indicated.
3. Water content was 32 percent, dry density 86.7 pcf.
4. Atterberg limits: CH - LL = 53 percent, PI = 34 percentage points, SM - nonplastic.
5. Compactive effort was 60 percent of standard effort: optimum water content 27 percent, max. dry density, 93 pcf.

Table 3 (Concluded)

Test No.	Max. Pullout Force, lb	Normal Load psf	Max. Soil/Geotextile Interface Stress psf	Fabric	Soil Types	Type Test	Horizontal Displacement at Failure in.	Comments
16	5170	1000	673	Nicolon	CH/SM	Pull-out	3.100	
17	900	300	117	Nicolon	CH	Fixed	0.565	
18	1800	500	235	Nicolon	CH	Fixed	2.383	
19	2000	1000	261	Nicolon	CH	Fixed	1.480	
20	1350	500	176	Nicolon	CH	Fixed	0.909	
21	710	300	93	Nicolon	CH/CH	Pull-out	1.153	Strain 0.0016 in./min
22	4840	1000	630	Nicolon	CH/CH	Pull-out	2.162	Strain 0.0016 in./min
23	1520	500	198	Nicolon	CH/CH	Pull-out	1.467	Strain 0.0016 in./min
24	3240	1000	422	Nicolon	CH/CH	Pull-out	3.900	Strain 0.0016 in./min (Rear of shear box lifted up)
25	2080	700	271	Nicolon	CH	Fixed	2.514	Strain 0.0016 in./min
26	1840	700	240	Nicolon	CH	Fixed	2.627	Strain 0.0016 in./min
27	1800	300	235	Nicolon	CH/CH	Pull-out	1.471	

Table 4

Data Summary for Phase 2 Tests

Test No.	Max. Pullout Force lb	Horiz. Displ. at Failure in.	Max. Soil/Geotextile Interface Stress psf	Max. Pullout Resistance psf	Displ. in Fabric* for Full Mobilization in.	Water Content %	Dry Density pcf	Rate of Pullout in./min
1	2090	1.067	300	272	0.324	27.0	92.0	0.25
2	4340	2.213	500	565	0.783	27.0	92.0	0.25
3	8150	3.739	1000	1061	0.581	27.0	92.0	0.25
4	2030	1.075	300	265	0.648	27.0	92.0	0.25
5	2800	4.024**	300	365	0.445	27.0	92.0	0.25
6†	3270	4.004**	300	426	0.432	27.0	92.0	0.25
7	--	--	500	--	0.351	27.0	92.0	0.25
8	4430	2.926	500	577	0.527	27.0	92.0	0.25
9††	3770	1.612	1000	491	0.432	27.0	92.0	0.25
10	2760	1.063	500	360	0.459	27.0	92.0	0.25
11	5680	2.134	1000	740	0.635	27.0	92.0	0.25
12	2260	0.779	500	295	--	27.0	92.0	0.0016
13	3460	0.885	500	451	--	27.0	92.0	0.0016
14	2360	2.139	500	308	0.446	32.0	86.7	0.25

Note: Atterberg limits: CL - LL = 46 percent, PI = 28 percentage points. 60-percent standard effort compaction: optimum water content, 23 percent, maximum dry density, 98.5 pcf.

* Displacement when back edge of fabric began to move.

** Pullout force still increasing at 4-in. displacement.

† Test stopped because shear box tilted.

†† Specimen inundated.

Table 5

Data Summary for Phase 3 Tests

Test No.	Max. Pullout Force lb	Horiz. Displ. at Failure in.	Max. Soil/Geotextile Interface Stress psf	Max. Pullout Resistance psf	Displ. in Fabric* for Full Mobilization in.	Water Content %	Dry Density pcf	Rate of Pullout in./min
1	1420	0.897	350	185	0.70	27.1	92.4	0.25
1A	3210	1.858	350	418	0.38	26.8	92.6	0.25
2	4880	3.952	550	636	0.57	27.0	92.5	0.25
2A	3990	2.955	550	520	0.62	26.4	92.9	0.25
3	2780	3.983	350	362	0.32	33.9	84.7	0.25
4	3630	3.877	550	473	0.38	34.0	84.6	0.25
4B	3290	2.685	550	429	0.45	33.1	85.2	0.25
5	2070	4.000	350	270	0.41	39.7	83.4	0.25
6	1390	1.339	550	181	0.43	38.8	83.9	0.25
6A	1980	3.748	550	258	0.46	38.7	84.0	0.25
7	1910	2.625	350	249	--	33.8	84.7	0.0016
8	1020	1.944	550	133	--	33.4	85.0	0.0016
8A	850	1.234	550	111	--	35.1	83.9	0.0016
8B	1230	1.268	550	160	--	33.2	85.1	0.0016
9**	1700	0.528	550	222	0.38	25.8	93.3	0.25
10**	1700	0.808	550	222	0.43	32.4	85.6	0.25

(Continued)

Note: Atterberg limits: CH - LL = 57 percent, PI = 38 percentage points.
60 percent of standard effort compaction: optimum water content, 26 percent, maximum dry density, 93.2 pcf.

* Displacement when back edge of fabric began to move.

** Specimen inundated.

Table 5 (Concluded)

Test No.	Max. Pullout Force lb	Horiz. Displ. at Failure in.	Max. Soil/Geotextile Interface stress psf	Max. Pullout Resistance psf	Displ. in Fabric* for Full Mobilization m	Water Content %	Dry Density pcf	Rate of Pullout in./min
11	1640	1.067	350	214	--	34.0	84.6	0.0016
12	7410	2.944	1000	965	0.788	26.0	93.2	0.25
13	5220	2.292	1000	680	0.639	34.0	84.6	0.25
14	2660	1.215	1000	347	0.378	40.0	83.2	0.25
15	3640	2.106	1000	474	--	34.0	84.6	0.0016
16	1830	1.080	550	239	--	34.0	84.6	0.0016

PART IX: RESULTS

Effect of Rate of Deformation

45. The effect of increasing the rate of pullout deformation is to increase the apparent strength of a cohesive soil, as can be seen from Figure 18. Clay soils exhibit liquid, plastic, or brittle behavior under the action of a shear stress, depending on the water content of the clays under shear. At water contents greater than the liquid limit, clays behave as liquids in that they cannot support static shear stress; when they are subjected to shear stress, they deform continuously in the manner of Newtonian fluids. At water contents less than the plastic limit, clays tend to exhibit brittle behavior and take on the characteristics of Hookean solids where time and rate of deformation (ideally) have no effect on strength. However, at water contents between the liquid and plastic limits (which is the condition under which most natural clays exist and the condition of the soils under study in this investigation), clays behave as viscous materials, in that shear stress generated in response to a load is a function of the rate of strain/loading to which they are subjected. Clays may deform continuously after a certain threshold yield stress has been exceeded in the manner of a Bingham material, but behavior (of clay) is not that of the classical Bingham material because the viscosity of clays is not constant. According to Hvorslev (1969), the coefficient of viscosity of clays depends on maximum velocity gradient and the time elapsed after the maximum velocity has been attained. The tendency for clays to exhibit time dependent behavior also depends on plasticity with highly plastic materials showing a greater tendency for time dependent behavior.

46. In a study by Al-Hussaini and Gilbert (1974), shear stress was measured between clay and neoprene rubber at different rates of displacement in a rotational shear apparatus. The test equipment in this 1974 study was essentially a direct shear device with an annular specimen geometry. Because the shear box is annular in shape and circles around to meet itself, it is effectively of infinite length in that large deformations may be applied to a test specimen in one direction without reversal and without change in specimen cross section. Soil used in the study was Vicksburg Buckshot Clay (VBC), which is a medium plasticity clay with a liquid limit and plasticity index of 56 percent and 34 percentage points, respectively; specific gravity of VBC is

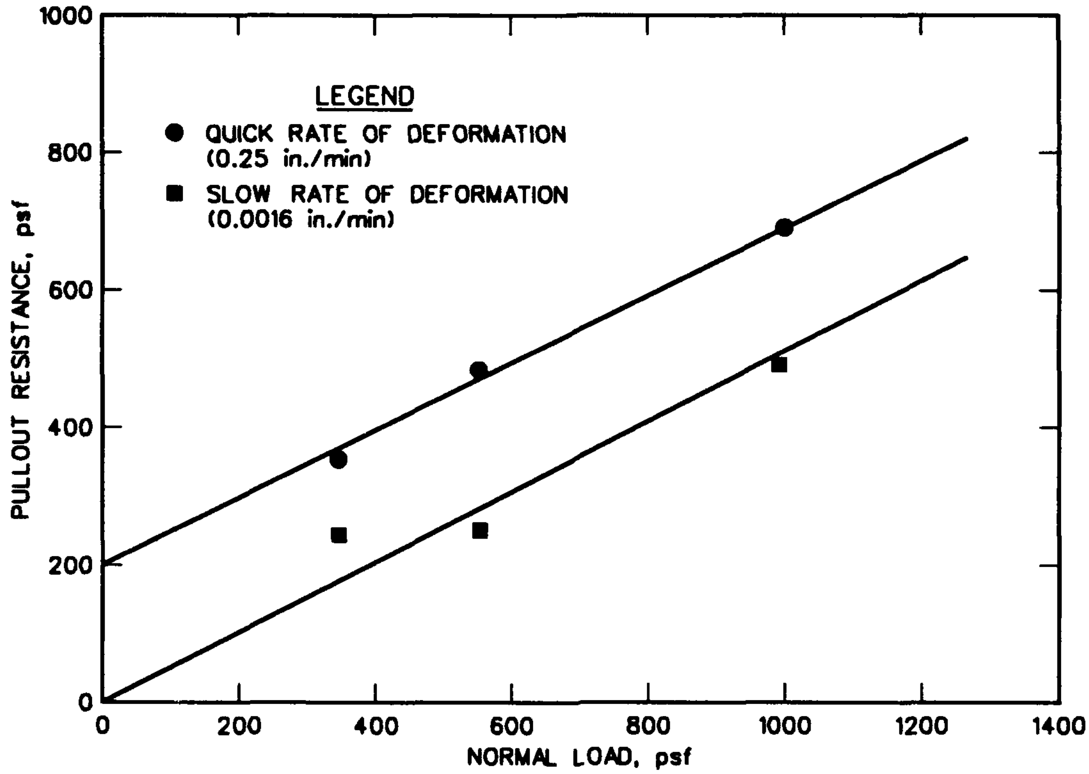


Figure 18. Pullout resistance of influenced by rate of deformation

2.68, and the test water content and dry density are 26 percent and 95 pcf, respectively. Vicksburg Buckshot Clay is a material with similar properties and is molded to conditions similar to those of the clays of phases 1 and 3.

47. The results of the rate of deformation versus shear stress relationship determined in the 1974 study are shown on Figure 19. Shear stress was measured between soil and neoprene rubber under three normal stresses and at three rates of deformation, 0.002, 0.2, and 2 in./min.

48. Pullout resistance in the present study was measured at rates of deformation of 0.0016 and 0.25 in./min. The strength envelopes for the two rates are shown on Figure 18, from which it is seen that the difference between the two envelopes is about 200 psf. The difference between the peak shear stress at 0.002 in./min and that at 0.31 in./min as measured between rubber and soil in the 1974 study is about 270 psf on the 5 psi (720 psf) normal stress curve; 0.31 in./min was chosen as the upper limit because the ratio of 0.25:0.0016 and 0.31:0.002 are approximately equal and there were no data below 0.002 in./min in the 1974 study.

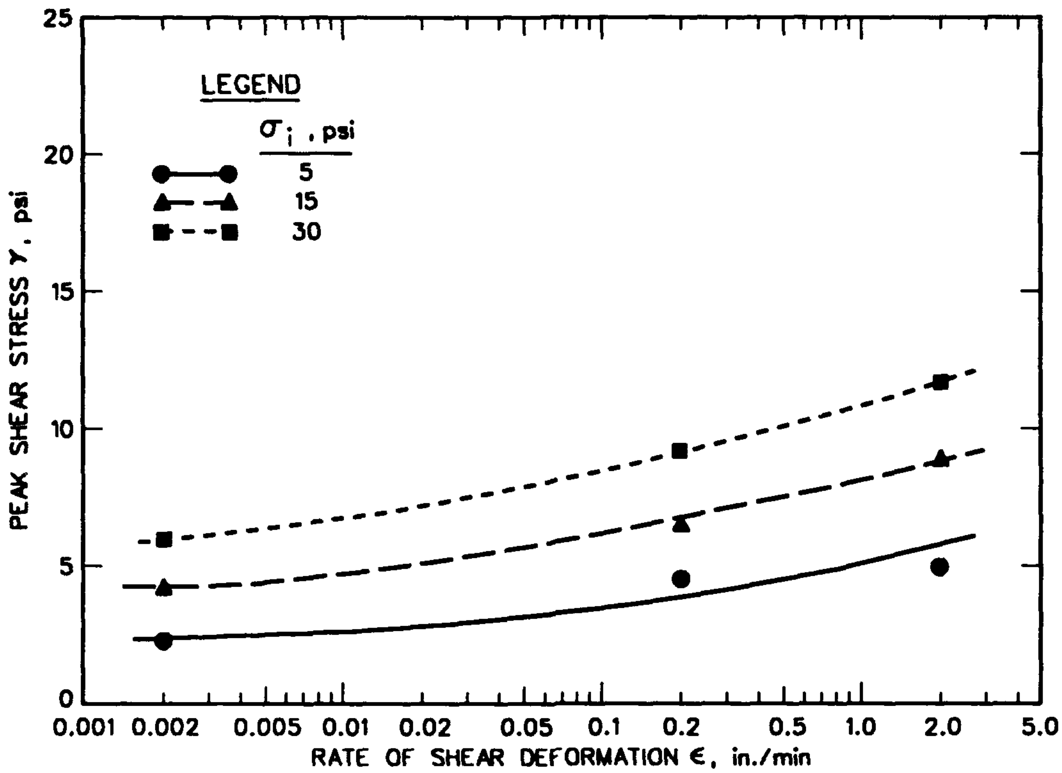


Figure 19. Relationship between the shear stress τ and the rate of shear deformation at interface of soil and rubber

49. Even though slightly different geosynthetics were used, the soils and molding conditions were similar in the 1974 study and the present study. The difference between approximately 200 psf for the additional stress resulting from pullout rate effects in the present study and 270 psf of additional stress in the early study of rotational shear seems reasonable and comparable. These effects of rate of displacement are likely due, in both cases, to soil viscosity as described by Hvorslev (1969).

50. A component of shear stress from rapidly applied rate of pullout or displacement is real, can be readily and easily observed and measured in the laboratory, and can be of significant magnitude. However, this component of shear stress or strength comes from the mobilization of viscous forces that occur as the result of a forced test condition which does not occur in the general case. Hence, additional strength from high rates of displacement cannot be expected or depended upon in nature; therefore, it would be inappropriate and dangerous to consider and include these viscous components of strength in the analysis of any geotechnical structure.

Effect of Submergence

51. The effect of submergence, as confirmed by Figure 20, is to lower the apparent pullout resistance. Submergence appears to simply shift the envelope determined from the soil/fabric system tested dry vertically downward without significantly affecting the measured friction angle. This result is not surprising considering that the effect of capillary tension in unsaturated soils is to give the soils apparent cohesion (Lambe 1965). According to Lambe, moist soils should not be tested in direct shear because of the component of cohesion that would be added to the strength envelope. Lambe suggests that only "saturated" soils should be tested in direct shear and soil specimens should be saturated by placing the direct shear box containing moist/unsaturated soil in a pan of water.

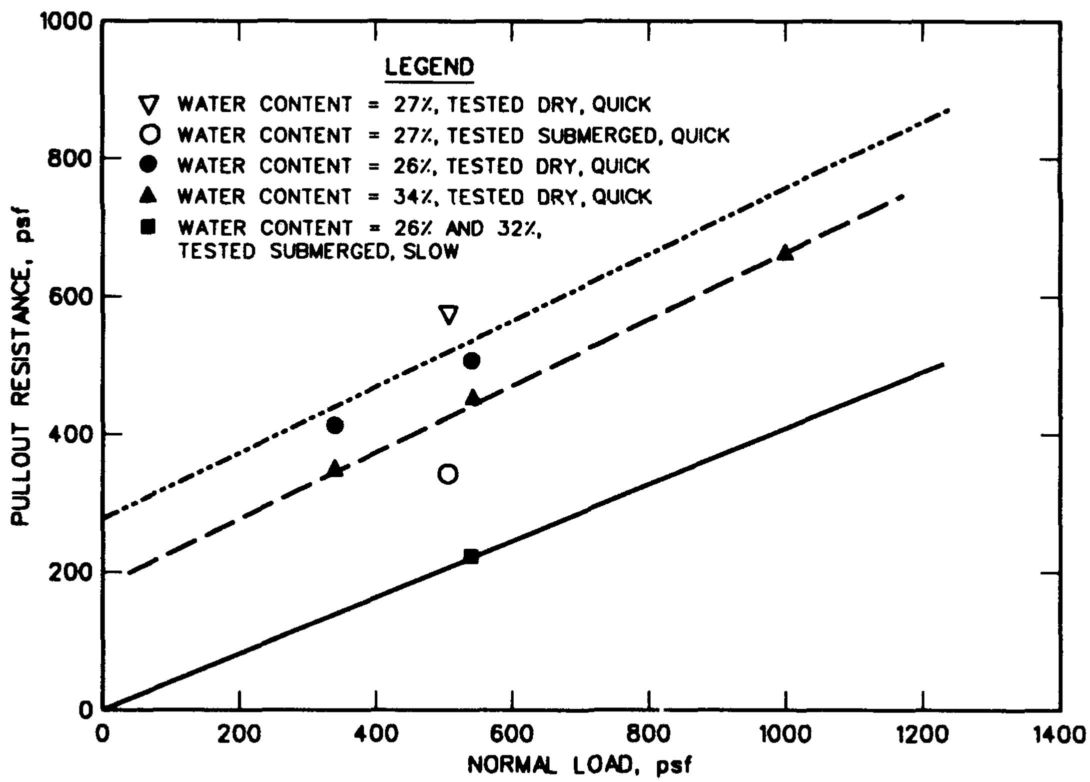


Figure 20. Effect of submergence on pullout resistance

52. Placing the direct shear box in a pan of water may not produce a true saturated condition in the soil specimen, but will essentially eliminate capillary tension and the associated component of capillary cohesion.

Apparent or capillary cohesion should not be relied upon in strength analysis of soil structures because its value is related to water content of the associated soil; capillary cohesion will virtually disappear in both sands and clays if the soil becomes exposed to water and is allowed to absorb freely. The effective friction angle is also affected by exposure to water; inundation generally lowers the value by 1 to 2 deg (Wu 1967). This range is small enough to be within the band of random experimental variation for the (relatively small number of) tests performed in the present investigation, so change in effective friction angle due to submergence cannot be quantified.

Stress and Strain Conditions in Test Specimen

53. Stress and strain conditions in a direct shear specimen are inherently nonuniform (Lambe 1965). The specimen is relatively thin so that drainage is easily achieved; however, one of the disadvantages of a thin specimen in a test like the direct shear test is that it is difficult to maintain an undrained condition during the test. Additionally, because a direct shear specimen is thin and because normal stress is applied through rigid plates at the top and bottom of the specimen, spatial density inhomogeneities from molding will further degrade an already nonuniform stress state.

54. The basic configuration of the present investigation is that of a direct shear device, and the tests were performed in slightly modified direct shear equipment. Therefore, the experiments of this study will suffer all the nonuniform stress conditions inherent in direct shear tests. Specimens for this investigation were molded directly in the shear box by placing an amount of material in the box, then tamping it to the height required to obtain a target density. Operating in this manner ensures that a correct average density will be achieved, but does not ensure density uniformity. Because the molding of each specimen is slightly different, there will be variation in the spatial density distribution and the resulting stress state of each specimen. Because the specimen is thin, the effect will be to aggravate specimen stress nonuniformity because according to Saint-Venant's principle (Wright, Gilbert, and Saada 1978), there is not sufficient distance (in the thickness of the specimen) to allow the stresses to become more uniform. Therefore, because of highly nonuniform and variable stress conditions (from specimen to specimen), some scatter in measured pullout resistance must be expected as a natural consequence of the specimen preparation technique.

55. Hvorslev (1969) studied deformation patterns in direct shear specimens by coloring thin vertical zones within direct shear specimens before shear and observing the distortions that occurred as the result of shear. In this way, highly nonuniform deformation conditions were confirmed in direct shear test specimens as well as the tendency for progressive failure to occur.

56. In a similar attempt to understand and define deformation patterns in the specimens of the present investigation, small shafts 0.25 in. in diameter were drilled vertically through some of the specimens (of phase 1) and filled with sand before pullout; after the tests, the shafts were exposed so that posttest deformation patterns could be observed. One such posttest sand shaft pattern is shown in Figure 21. Sand shafts shown in the figure were placed at longitudinal quarter points and laterally in the center of the specimen. Since it is known that the fabric started from an unstressed (unstretched) state and had to be drawn taut before soil shear stress could be mobilized, the sand shaft pattern of Figure 21 offers certain information about how internal deformation occurred in this test specimen. For ease of explanation, the pattern of Figure 21 is shown schematically on Figure 22. As tension was applied and the fabric was pulled taut up to sand shaft 1, soil in the entire bottom half of the specimen was mobilized in that the entire sand shaft 1B rotated uniformly in the direction in which the fabric was being pulled. Only a very small thickness of the top half of the specimen was mobilized in that shaft 1T remained essentially vertical and undeformed except very near the fabric where all of the movement apparently took place. As the fabric was pulled taut up to point 2, it is seen that there is less rotation of shaft 2B and therefore less material mobilized in the bottom half of the specimen at point 2 than at point 1; in fact, a portion of shaft 2B did not rotate. In the top of the specimen, soil deformation was still very localized and close to the fabric. However, when the fabric had been pulled taut at position 3, the bottom of the specimen was undisturbed except possibly very near the fabric; that is to say, shaft 3B experienced essentially no rotation. However, rotation over about one-half of the length of shaft 3T indicated that material in the top half of the specimen was being mobilized at this time, although most of the displacement was still occurring fairly close to the fabric. Obviously, the internal behavior of this specimen is very complex (as demonstrated by the sand shafts) and is probably related to the initial state of stress and density uniformity. The preliminary use of sand shafts in this

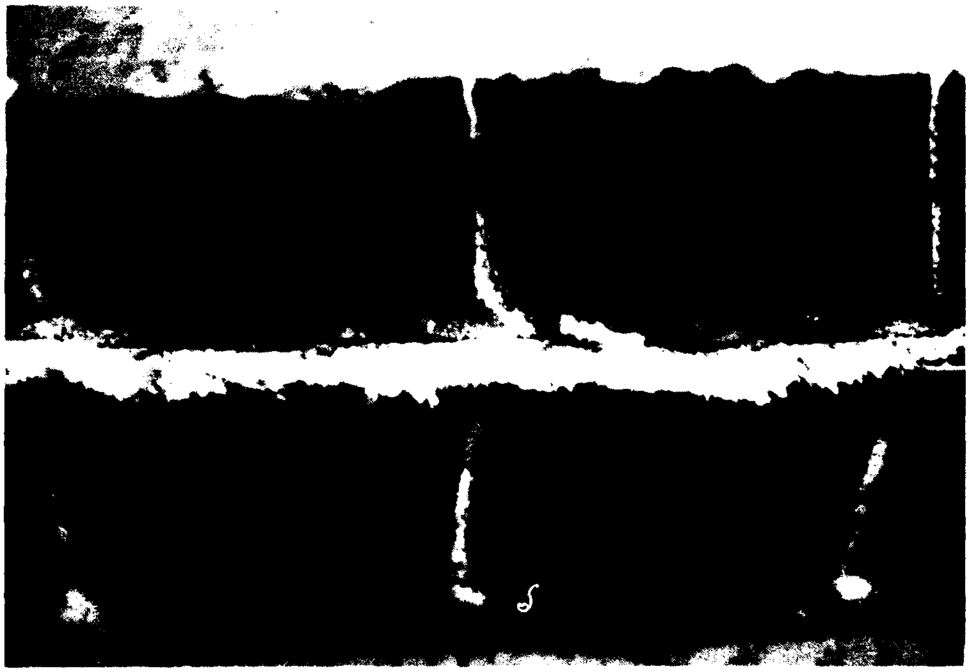


Figure 21. Vertical sand shifts showing internal
of formation pattern

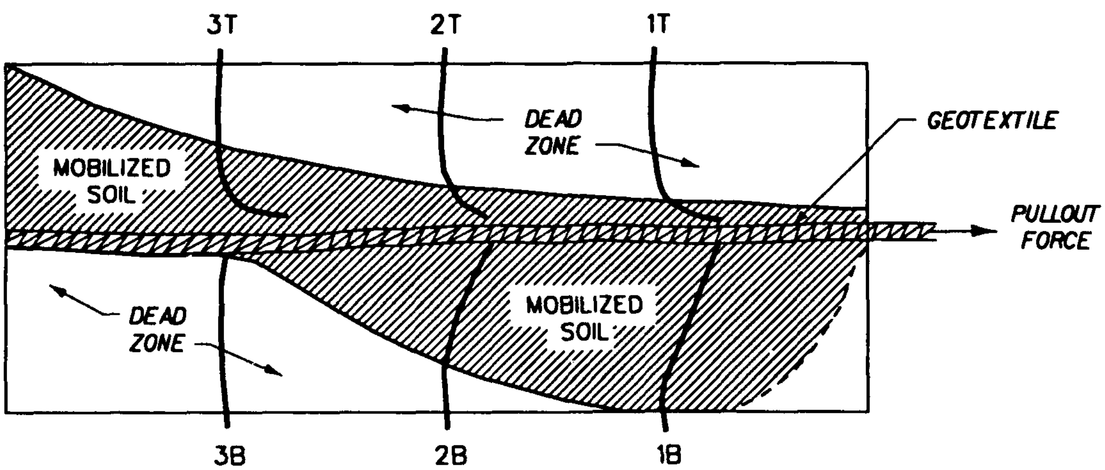


Figure 22. Schematic of sand shafts shown in Figure 21

study suggests that they could prove useful in the laboratory as well as in the field for investigating and defining internal deformation patterns if the shafts do not disrupt stress and/or pore water flow continuity within the test specimen.

57. Most fabrics are very permeable relative to soil with which they may be used, and woven fabrics typically have rough-textured surfaces. During consolidation, normal pressure will likely force a plastic clay soil with a water content between the liquid and plastic limit into solid contact with the fabric and even into the grain and texture of the fabric. If there is a tendency for the soil to develop excess pore pressure from shear strains as pullout is initiated, drainage occurs easily near the soil/fabric interface because of close proximity to the fabric and the associated short drainage path of soil near the fabric. Consequently, a thin layer or film of material with a higher density and lower water content will form at the soil/fabric interface. Because of the surface texture of the fabric and the layer of denser and stronger material immediately adjacent to it, a failure zone or slip surface will probably not occur at the fabric/soil interface. Instead, the adhesion between boundary soil and fabric is greater than the cohesion (and/or friction) between the soil and itself and forces shear to occur in the softer and less dense soil a short distance away from the soil/fabric interface (as confirmed in Figure 22). Additionally, field observation of full-size fabric pullout tests support that the failure surface was soil on soil; as the fabric was pulled through the slot in the soil retention mat during field tests as described, a thickness of clay adhered to both the top and bottom surfaces with no evidence of soil-to-fabric slippage (see Figure 23). Similar evidence of clay adherence to fabric during pullout tests in the laboratory may be seen Figure 3. The suggestion is that, for clays, the upper limit of efficiency (as defined by Martin, Koerner, and Whitty 1984) is 100 percent for ideal conditions of stress and density uniformity within the soil/fabric system, and some researchers do indeed achieve 100-percent efficiency. For example, the soil/fabric system described in the investigation by Garbulewski (1990) on soft cohesive soil showed essentially identical friction angles for soil/soil tests and soil/fabric tests, indicating 100-percent efficiency. Investigators who primarily tested cohesionless soil/fabric systems (Martin, Koerner, and Whitty 1984) typically measured efficiencies less than 100 percent except in very loose sand (Miyamori, Iwai, and Makiuchi 1986). However, the upper limit of efficiency is not always achieved. Appendix D provides a table of pullout

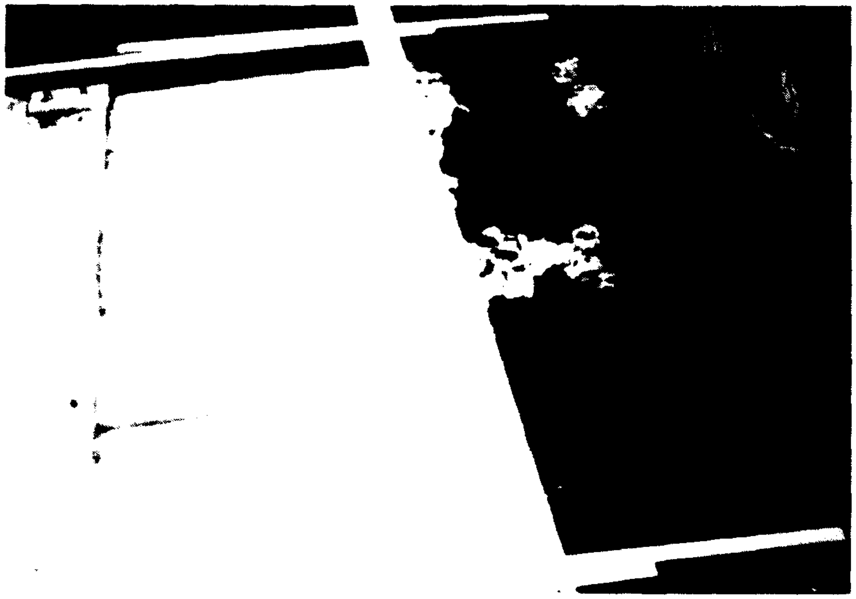


Figure 23. Clay adhering to geotextile in field pullout test

resistances compiled by Duncan, Sehn, and Bosco* and shows that efficiencies may be as low as 60 percent for geotextiles. After thorough analysis of the data in the table, Duncan, Sehn, and Bosco recommend that if it is not possible to perform pullout tests, the assumption that the tangent of the friction angle between soil and fabric is two-thirds of the tangent of the friction angle of soil on itself will yield conservative results. It should be noted that the standard procedure is to inundate (submerge) direct shear specimens during consolidation and shear; for this reason, the pullout tests presented on Figure 24 are performed on submerged specimens. The direct shear soil-on-soil shear tests were performed on 3-in. square specimens that were 0.554 in. thick. The rate of displacement used was 0.00018 in./min. As can be seen from the figure, soil/fabric pullout resistance gives a friction angle of about 20 deg, and the soil/soil friction angle is about 30 deg. This suggests good agreement with the recommendations made by Duncan, Sehn, and Bosco.

58. The conclusions are that high efficiencies may be achieved in normally consolidated <u>cohesive</u> soil/fabric systems, but allowances must be made for unexpected internal conditions and the friction angle/strength between

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* J. M. Duncan, A. L. Sehn, and G. Bosco, 1988,"Stability of Reinforced Soil Walls, Anchored Walls, Reinforced Slopes and Reinforced Embankments," unpublished draft report prepared under Contract No. DACA39-87-C-0055, for US Army Engineer Waterways Experiment Station, Vicksburg, MS.

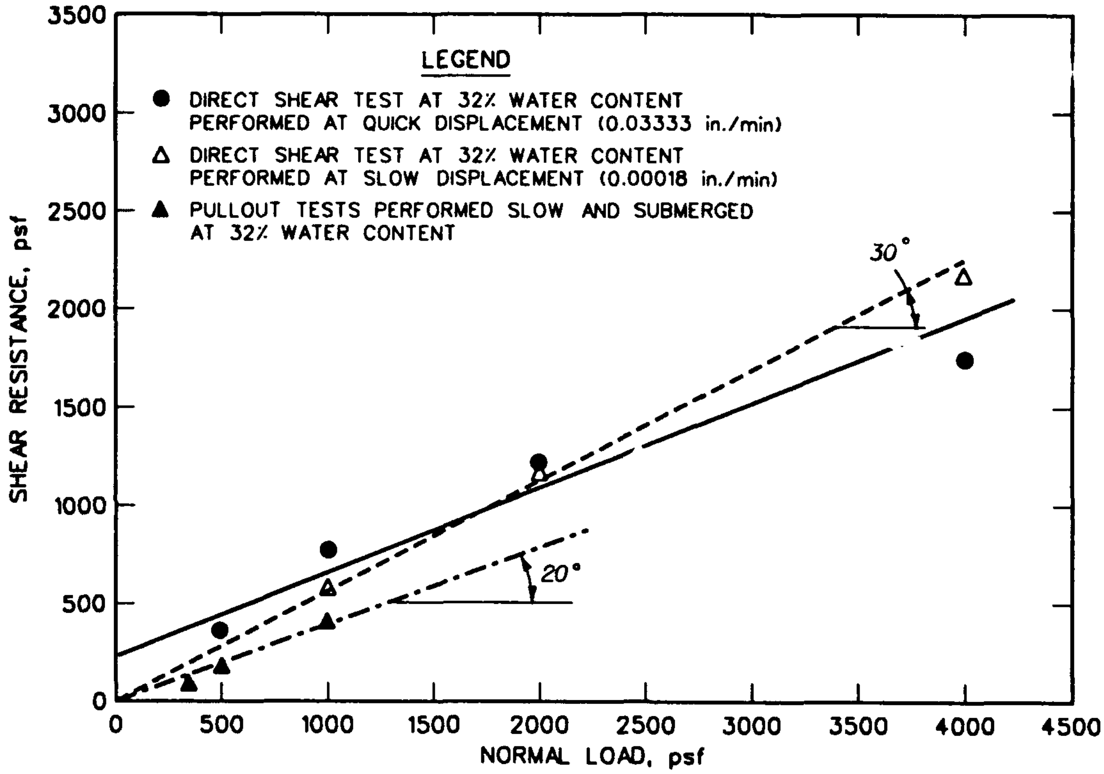

Figure 24. Comparison of pullout resistance with the shear strength of clay

soil on soil should be scaled down by one-third to achieve conservative results. Results of tests on geogrids are also presented in the table of Appendix D, and as a result, Duncan, Sehn, and Bosco* recommend that the friction angle of a soil/geogrid system be taken as 90 percent of the friction angle for soil on soil. This recommendation agrees with a result given by Schmertmann et al. (1987). It should be mentioned, additionally, that cohesive soils at high water contents are subject to creep, that is, undergo shear strain at constant/sustained shear stress. The magnitude of creep is a function of sustained shear stress level, and it should be realized that creep failure may occur if sustained shear stress is sufficiently high in materials susceptible to creep. However, the treatment of creep is beyond the scope of this investigation.

* Op. cit.

Effect of Normal Stress

59. In all tests performed both in the laboratory and in the field, the effect of increasing the normal stress is to increase pullout resistance. This is a very expected result, and the effect is seen in tests that were conducted at fast and slow rates of displacement as well as in those tests that were conducted submerged as well as in the dry. The effect of normal stress was somewhat diminished in tests conducted at very high water contents (approximately 40 percent); those tests are discussed in the following section.

Effect of Molding Conditions

60. Water content and density have a very great influence on the stress-strain and strength characteristics of a molded soil. Degree of saturation will be dictated by the molding conditions and is also an important variable in determining the load response of a soil. For example, as water content and degree of saturation increase beyond the optimum water content, density and strength typically decrease. Figure 25 demonstrates this trend convincingly by showing how unconfined compressive strength decreases in soil specimens (of phase 3 soil) at water contents and densities achieved by 15 blow count compaction wet of the optimum water content. The trend of Figure 25 is exactly the same as that demonstrated by Seed and Chan (1959), who presented data showing how strength decreases and the tendency for more plastic stress-strain behavior increases in clay specimens compacted wet of the optimum water content. Figure 25 also shows a comparison of 15 blow count laboratory compaction with field compaction of Phase III clay as obtained by four passes of a low ground pressure (4.7 psi) bulldozer. The field compaction data presented on Figure 25 are taken from the NOD report, "Geotextile Prototype Pullout Tests, Bonnet Carre Spillway, October-November, 1989" (NOD 1989b). Field wet densities were determined in the New Orleans Laboratory from tube samples; water contents were determined in the conventional oven in the laboratory. There is more scatter in the field compaction data than in the laboratory data, as can be seen from Figure 25; obviously, it is more difficult to control water content and compactive effort in the field than in the laboratory. Although the laboratory curve is located close to the bottom of the band defined by the field data, there is reasonably good agreement between

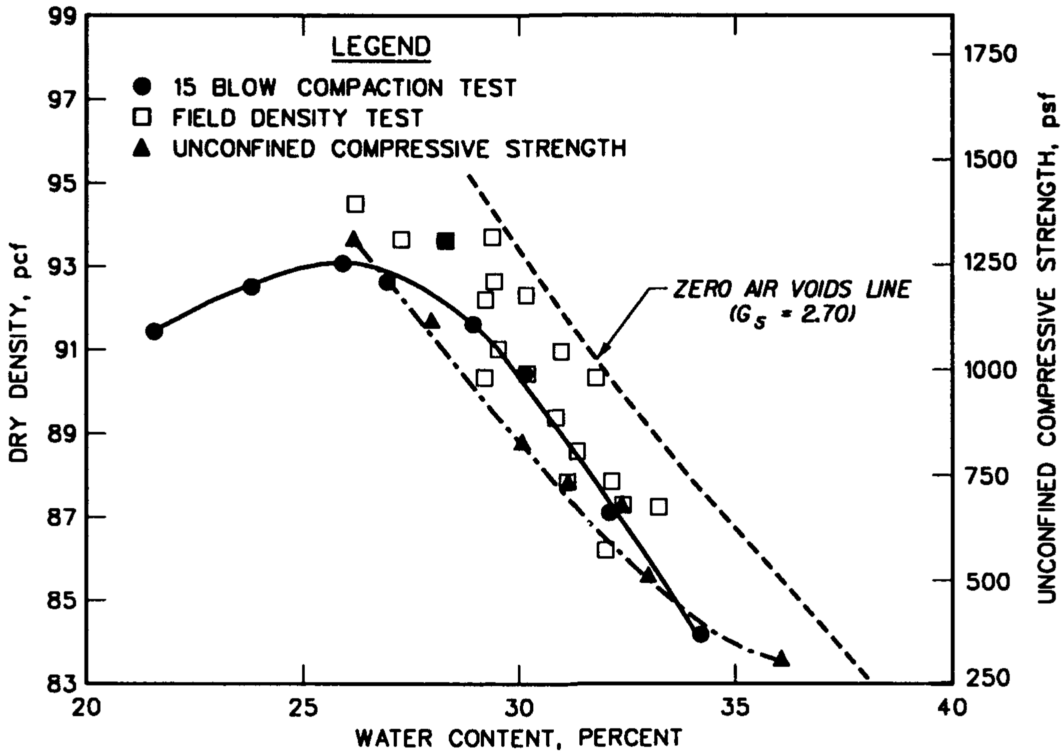

Figure 25. 15 Blow compaction and unconfined strength variation with water content for phase 3 soil

laboratory and field compaction data. For any given water content, the difference between the laboratory and (top of the scatter band) field density is about 1.5 pcf. This suggests that higher densities were produced by field compaction; however, Figure 25 shows that lower densities (than those of the laboratory compaction curve) were also produced in the field by the compaction. The zero air voids line shown on Figure 25 demonstrates that if the observed field compaction data are statistically representative of the field soil mass, a higher degree of saturation exists in the field compacted material. It must be realized that laboratory compaction was conducted in a much more controlled environment than that in the field. The field material had to be wetted from a dryer state in this instance; as a result, there was more variation in the water content and density in the field than in the laboratory.

61. Figure 26 shows how pullout resistance decreases with increasing water content and appears to reach a limiting value at about 40 percent for all normal loads. It is believed that an essentially saturated undrained condition is achieved in specimens with a target water content of 40 percent

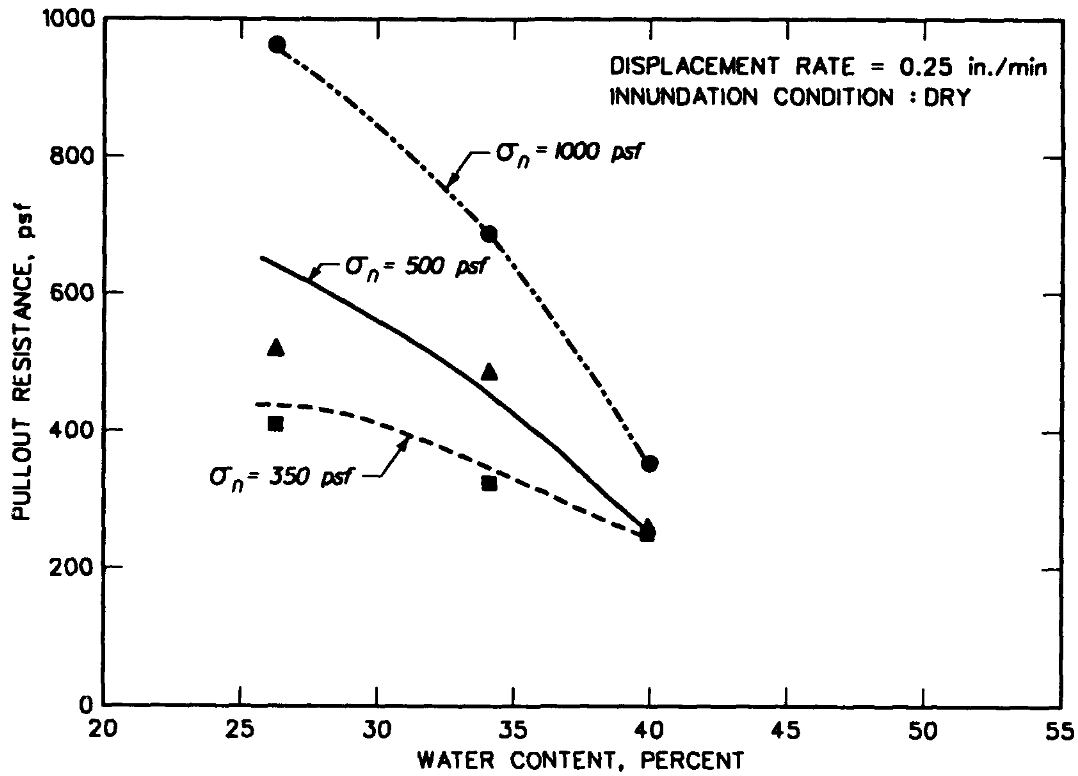

Figure 26. Effect of water content on pullout resistance

tested at the quick (0.25 in./min) rate of deformation used in laboratory testing. It can be shown that 100-percent saturation occurs in specimens of soil with a specific gravity of 2.70, a dry density of 83.2 pcf, and a water content of 38 percent, which is close to the actual molding conditions achieved in laboratory specimens tested at the condition of high water content. Because pore pressure develops in such highly saturated soil specimens and requires an extended time period for dissipation (because of low permeability in the plastic clay), applied normal stress becomes ineffective in generating grain-to-grain friction in the soil, and the strength/resistance of the soil/fabric becomes a function of cohesion/adhesion alone. This effect may be demonstrated on Figures 27, 28, and 29, which show that for the lower water contents (26 and 34 percent), resistance increases with normal stress in the laboratory as well as in the field because the high compressibility of air present in the soil voids at these water contents permits effective grain-to-grain contact. However, at the nominal 40 percent water content, strength appears to be essentially constant and unaffected by normal stress because, with shear, pressure develops in voids filled with water and reduces grain-to-grain contact pressure from which frictional resistance is derived.

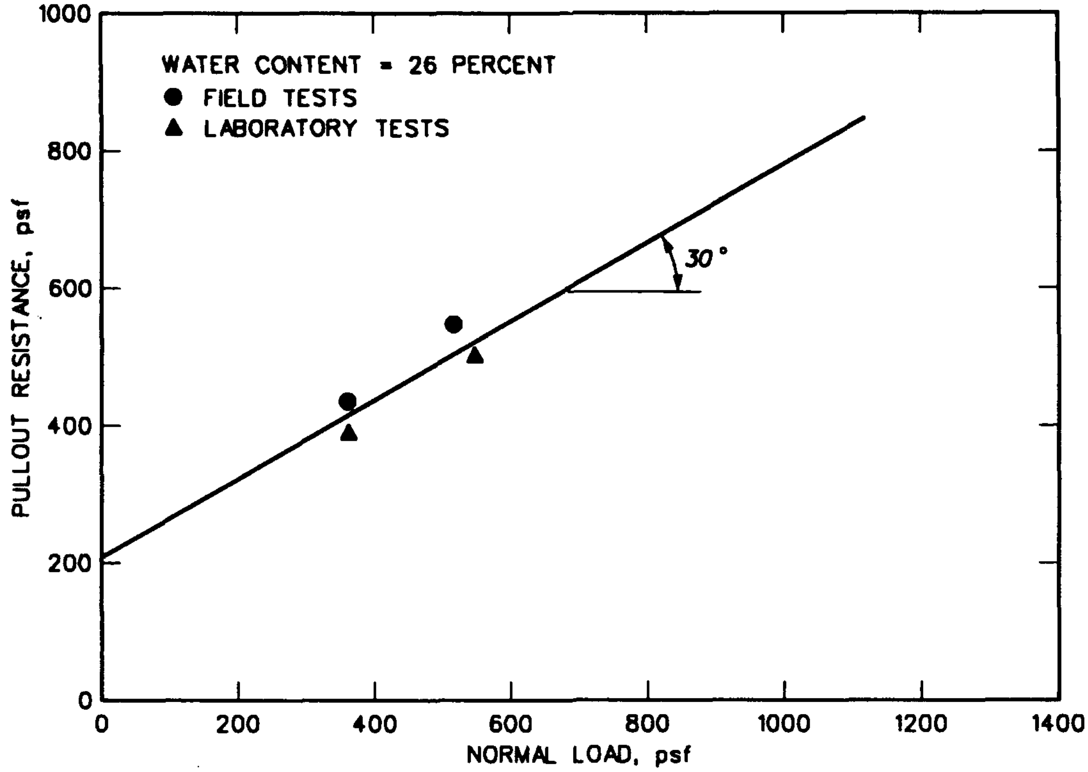

Figure 27. Pullout resistance versus normal load in clay specimens molded at 26-percent water content

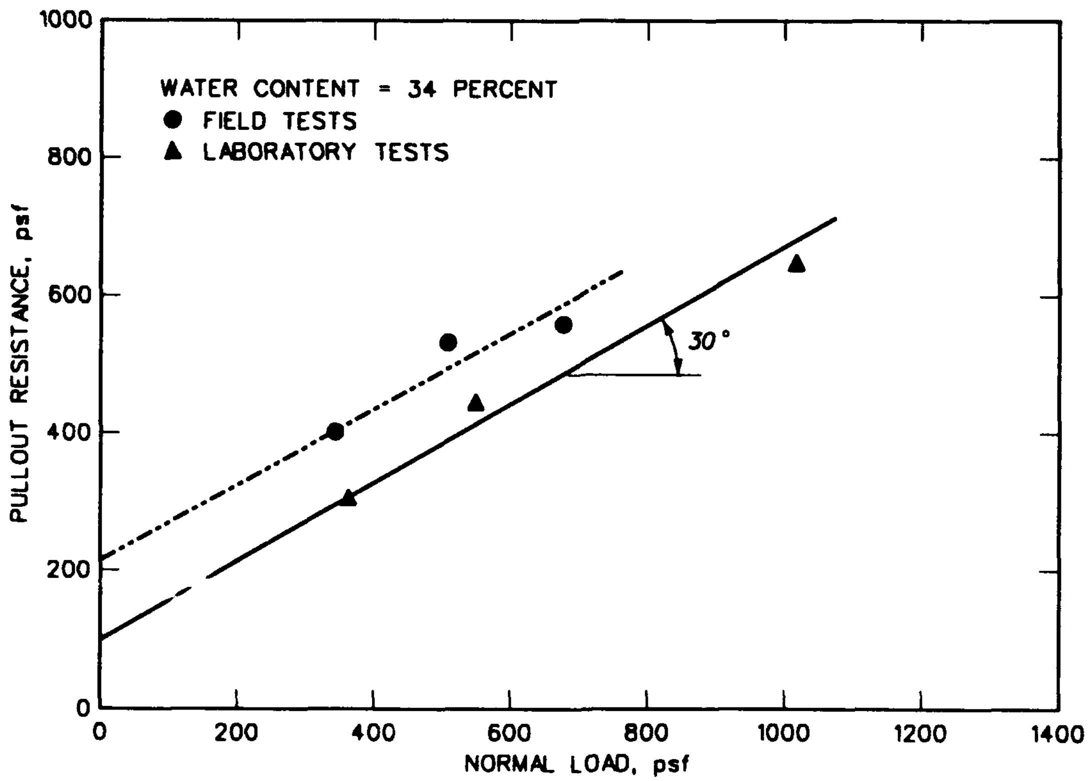

Figure 28. Pullout resistance versus normal load in clay specimens molded at 34-percent water content

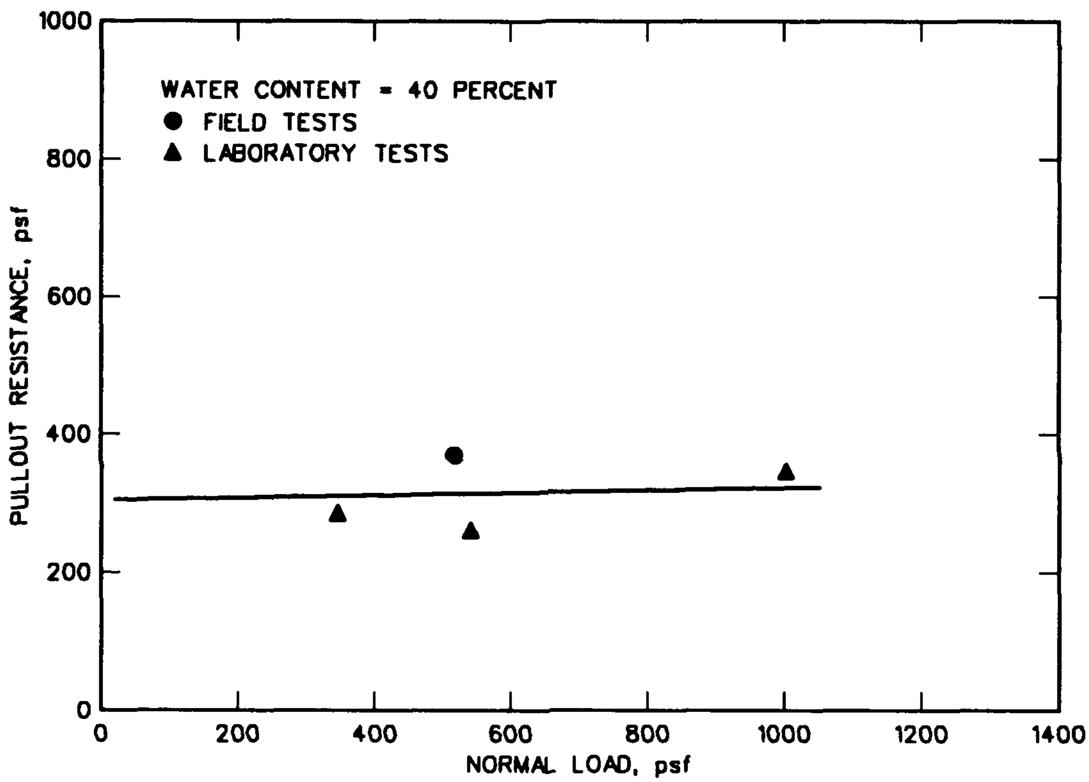

Figure 29. Pullout resistance versus normal load in clay specimens molded at 40-percent water content

PART X: EFFICIENCY OF SOIL/GEOTEXTILE SYSTEM

62. Efficiency of the soil/geotextile system is the ratio of the friction angle between fabric and soil to the angle of internal friction of the soil in question expressed as percent. Various investigators have discussed efficiencies of soil on geotextile systems, notably, Miyamori, Iwai, and Makiuchi (1986); Myles (1982); and Martin, Koerner, and Whitty (1984). These investigators concluded that the efficiency of a geotextile on sand system is approximately 100 percent for loose sand, but the efficiency decreased as density increased. A possible explanation for this behavior may be the dilatant/contractive behavior of sands. Sand particles small enough to be embedded into the texture of a fabric interlock with and "stick" to the fabric; if the fabric is then displaced, sand along the boundary layer tends to be dragged along with it; that is, no slippage occurs between the fabric and sand, and this forces shear strains to occur within the sand mass. When subjected to a shear strain, loose sands are contractive; therefore, consolidation will occur, increasing the soil density and strength in the vicinity of the interface. Therefore, with continued displacement, a failure/sliding surface will be forced away from the interface and into the sand mass where there is lower density (and strength). Therefore, the resultant pullout force (shear strength between sand and fabric) will be that associated with the lower density of the surrounding sand. The failure stress or strength will be that of sand-on-sand at the surrounding (lower) density which will indicate an efficiency of 100 percent.

63. However, if sand surrounding a geotextile is dense, dilation will occur upon application of a pullout displacement, creating a zone of lower density in the vicinity of the fabric. Failure will occur in this zone of lower density, and pullout force/failure stress will be that associated, not with the shear strength of sand at the original high density, but with a shear strength of sand at a lower density produced by dilation. Therefore, a lower "efficiency" will be determined relative to the strength of sand at the higher undisturbed density.

64. In the case of a clay/geotextile system, because of the small particles and deformable nature of clay, there will likely be good contact and adherence between soil and the texture of the fabric as the result of placement and consolidation. Therefore, there is a very great likelihood that the soil will "stick" to the fabric. Because of the easy access to drainage at

and near the soil/fabric interface, logic would suggest that soil density is slightly higher there than at points deeper within the soil mass. Therefore, when displacement is applied to the fabric, clay in contact with the fabric is displaced by the same amount with resulting shear strains occurring deeper in the clay mass (that is, away from the zone of higher density at the interface). If the water content of the soil is high enough that there is a tendency for pore pressure to develop, then because of easy access to drainage for soil in close proximity to the fabric, drainage will occur on the failure surface as deformation is applied. The result is that failure stress (or measured pullout resistance) will be the drained soil-on-soil strength with a maximum possible efficiency of 100 percent. However, the determination of this study and the experience suggested by previous investigations suggest that an efficiency of 100 percent cannot be relied on. Because of uncertainties and factors that cannot be confidently controlled or quantified, only two-thirds of the soil-on-soil strength should be used for soil-on-fabric strength.

PART XI: COMPONENTS OF PULLOUT RESISTANCE

65. The laboratory tests performed in this investigation determined the pullout resistance of geotextiles against soil, and it was shown that this resistance consisted of several components, namely friction, a viscous component, and a component due to capillarity. It was also shown that the viscous component and the capillarity component were circumstantially determined and could not be relied on in the general case or in an uncontrolled environment. The frictional component of strength in soil/geotextile systems is the one that can be considered constant and reliable and the only one that can be trusted in the evaluation of long-term strength and stability.

66. The simplest mathematical representation of friction between two bodies is a linear equation of the form,

$$F = \mu * N \tag{1}$$

where

F = force developed as result of friction

μ = coefficient of proportionality, usually called the coefficient of friction

N = normal pressure between the bodies

67. The simplest representation of the basic concept of friction as it relates to soil mechanics is an almost identical expression,

$$\tau = \sigma \tan \phi \tag{2}$$

where

τ = shear stress in a soil element

σ = normal stress acting on a soil element

ϕ = angle of internal friction of the soil

68. A direct correspondence exists between the coefficient of friction (in Equation 1) and $\tan \phi$ in Equation 2. $\tan \phi$ is therefore the coefficient of proportionality between normal stress on an element of soil and the maximum shear stress which can be developed in that element as a result.

69. Because of the difficulties and uncertainties involved in laboratory soil testing, it is necessary to apply factors of safety to parameters determined in the laboratory for conservative design of soil structures to allow for adverse conditions which cannot be identified in the laboratory. Experience gained from the consideration of "efficiency" as defined and discussed previously sheds some light on the selection of an appropriate factor of safety for the analysis of soil/geotextile systems. Efficiency may approach 100 percent for cohesive soils, or loose cohesionless soils. However, factors that adversely influence the overall behavior of a soil/geotextile system make it untenable to depend on 100-percent efficiency for long-term stability. For example, cohesive soils (especially those of high plasticity) are subject to creep and to strength loss because of induced pore water pressure. Myles (1982) suggests that assuming an efficiency of 75 percent will lead to conservative designs in cohesionless soil/geotextile structures. Note that this suggestion by Myles means that a factor of 0.75 applied to the term, $\tan \phi$, in Equation 2 would serve as the "coefficient of friction" between the soil and the geotextile.

70. Using identical reasoning but basing their analysis on an in-depth survey of the available literature, Duncan, Sehn, and Bosco* propose the equation

$$\tan \delta = 2/3 * \tan \phi \tag{3}$$

This is a more conservative version of the equation offered by Myles. Duncan, Sehn, and Bosco* provide strong support that Equation 3 gives a lower limit of soil/geotextile interface strength, so design resistance computed using the equation is "almost always" (as stated by the authors) conservative. For example, it has been demonstrated by a number of investigators that the coefficient of friction between fabric and soil is essentially the same as that between soil and soil for cohesive soils. Equation 3 reduces the probable coefficient of friction (and thus the allowable shear stress) between fabric and soil by one-third. If the data generated in this investigation are checked against Equation 3, the friction angle of soil against fabric, or soil against soil, is about 30 deg (see Figures 24, 27, and 28). Now if the slow,

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* Op. cit.

submerged laboratory test results are compared against Equation 3, the angle of friction between soil and fabric should be (according to the equation) about 20 deg. Figure 30 shows that the friction angle, δ , determined from laboratory tests between clay specimens and fabric performed under slow, submerged conditions, is about 20 deg, which determines a strength envelope that includes only mobilized friction (the influence of viscosity and capillarity have been removed).

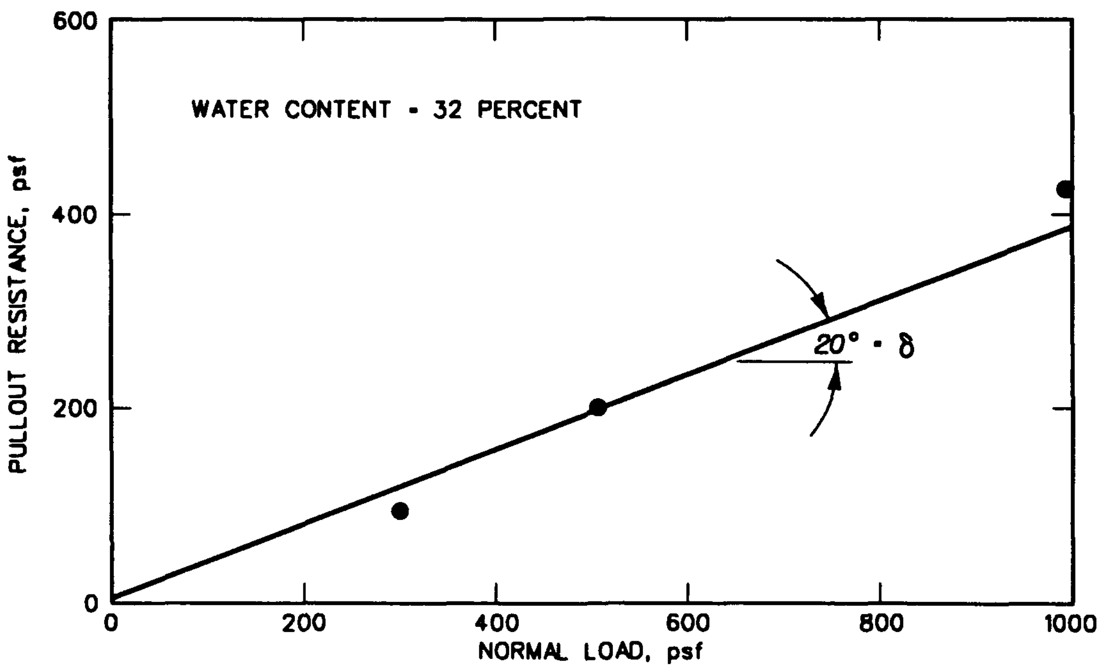


Figure 30. Pullout resistance for slow, submerged tests on phase 1 clay performed in the laboratory

PART XII: CONCLUSIONS

71. Conclusions that are believed warranted from laboratory tests conducted in this study as well as the consideration and analysis of other data available from the open literature are as follows:

a. The coefficient of friction between medium plasticity clays and woven geotextiles available from several (different) commercial vendors did not vary substantially. Slight differences in surface texture and roughness between brands did not result in significant variations in pullout resistance.

b. The effect of increasing the rate of pullout deformation in a clay/geotextile system is to increase the apparent pullout resistance of the system. However, the observed increase in strength is due to a mechanism much like viscosity; its magnitude is entirely dependent on displacement rate; therefore, this strength component should not and must not be relied on in strength analysis.

c. The effect of water submergence on a clay/geotextile system is to decrease the apparent pullout resistance (as the result of the loss of capillary tension/suction). Since a prototype clay/geotextile structure cannot be protected from submergence or exposure to water (e.g., as the result of rainwater runoff), the component of pullout resistance due to capillary tension cannot necessarily be depended upon.

d. Slippage does not occur at the clay/fabric interface of a geotextile reinforced soil structure; indications from this laboratory study are that the failure/slip surface which develops does so within the soil mass.

e. The theoretical efficiency of clay/geotextile systems approaches 100 percent; however, efficiency of clay/geotextile systems is usually less than 100 percent, and only about two-thirds of the clay/clay strength can be relied on in soil/geotextile systems. Strength efficiency of a loose sand/geotextile system approaches 100 percent, but decreases as sand density increases.

f. Tests performed at laboratory scale as well as full-size prototype field tests confirm that the effect of increasing normal stress/overburden pressure within a soil mass is to increase pullout resistance in soils where the water content is small enough that complete saturation is not produced.

g. Tests performed at laboratory and prototype scale show that pullout resistance of a soil/geotextile system may decrease substantially as the result of induced pore water pressure. In these tests, the soil had been placed at a water content (about 40 percent) large enough to produce essentially complete water saturation.

h. When induced pore water pressure reduces pullout resistance in a system where the soil is essentially saturated, the component of strength produced by normal stress/overburden pressure is lost, and the lower limit of pullout strength becomes that of the cohesion of the clay or adhesion between geotextile and clay.

i. Good agreement was observed between pullout resistance observed in laboratory tests and those observed during full-size field tests performed at the Bonnet Carre Spillway.

PART XIII: RECOMMENDATIONS

72. Based on observation and knowledge gained during this investigation, additional research is needed in geotextile pullout research, especially in the area of cohesionless soil/geotextile systems. The following recommendations are made for additional research:

- a. Laboratory test specimens in future pullout studies should be placed more uniformly (with respect to density) by pneumatic compaction rather than kneading compaction to attain greater stress and strain uniformity within the test specimen. This will afford less scatter in laboratory strength test results.
- b. Laboratory apparatus should be modified to apply normal stress pneumatically instead of with a rigid plate. This practice will also substantially increase uniformity conditions in the specimen.
- c. Laboratory apparatus should be modified to eliminate shifting and/or tilting of components during the application of geotextile displacement. This will also improve certainty and diminish scatter in laboratory test results.
- d. Effort should be spent to saturate some soil specimens and measure pore water pressure in the vicinity of the soil/fabric interface to characterize and study the failure mechanism.
- e. Sand cylinders (as used briefly in this study) should be used to define internal deformation patterns in the test specimen (of cohesionless soil).
- f. If laboratory or field pullout resistance information is unavailable for analysis of geotextile-reinforced soil structures and a conservative design is desired, Equation 3 should be used to estimate pullout resistance.
- g. Creep behavior of clay may significantly degrade the performance of a geotextile reinforced soil structures, especially if the clay is highly plastic (as are many of the clays in the NOD). A laboratory investigation should be performed to study and evaluate the influence of creep in such soils.

REFERENCES

American Society for Testing and Materials. 1986. "Standard for Metric Practice," Designation: E380-86, Philadelphia, PA.

American Society for Testing and Materials, 1952. Symposium on Direct Shear Testing of Soils," Special Publication No. 131, Philadelphia, PA.

Al-Hussaini, M. M., and Gilbert, P. A. 1974. "Stresses and Shearing Resistance Beneath a Rigid Wheel," Technical Report S-74-7, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Brand, S. R., and Duffy, D. M. 1987. "Strength and Pullout Testing of Geogrids," Proceedings Geosynthetics '87 Conference, New Orleans, La., February 24-25, Industrial Fabrics Association International, St. Paul, MN, pp 226-236.

Christopher, B. R., and Berg, R. R. 1990. "Pullout Evaluation of Geosynthetics in Cohesive Soils," Proceedings 4th International Conference on Geotextiles Geomembranes and Related Products, The Hague, The Netherlands, May 28-June 1, Vol 2, pp 731-736.

Formazin, J., and Batereau. 1985. "The Shear Strength Behavior of Certain Materials on the Surface of Geotextiles," Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, CA., pp 1773-1775.

Fowler, Jack, Leach, R. E., Peters, J. P., and Horz, R. C. 1986. "Mohicanville Reinforced Dike No. 2 Design Memorandum," Miscellaneous Paper GL-86-25, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Garbulewski, K. 1990. "Direct Shear and Pullout Frictional Resistance at the Geotextile-Mud Interface," Proceedings 4th International Conference on Geotextiles Geomembranes and Related Products, The Hague, The Netherlands, May 28-June 1, Vol 2, pp 737-742.

Haliburton, T. A., Anglin, C. C., and Lawmaster, J. D. 1978. "Testing of Geotechnical Fabric for Use as Reinforcement," Geotechnical Testing Journal, GTJODJ, Vol 1, pp 203-212.

Holtz, R. D. 1977. "Laboratory Studies of Reinforced Earth Using a Woven Polyester Fabric," International Conference on the Use of Fabrics in Geotechnics, Paris, France, April 20-22, Vol 3, pp 149-154.

Hvorslev, M. J. 1969. "Physical Properties of Remolded Cohesive Soils," Translation No. 69-5, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Ingold, T. S. 1982a. "An Analytical Study of Geotextile Reinforced Embankments," Second International Conference on Geotextiles, Las Vegas, NV, August 1-6, Vol 3, pp 683-688.

__________. 1982b. Reinforced Earth, Thomas Telford Ltd., London.

__________. 1982c. "Some Observations on the Laboratory Measurement of Soil-Geotextile Bond," Geotechnical Testing Journal, American Society for Testing and Materials, Vol 5, No. 3/4, September/December, pp 57-67.

__________. 1983. "Laboratory Pullout Testing of Grid Reinforcement in Sand," Geotechnical Testing Journal, GTJODJ, American Society for Testing and Materials, Vol 6, No. 3, September, pp 101-110.

Jarrett, P. M., and Bathurst, R. J. 1987. "Stran Development in Anchored Zones," Proceedings Geosynthetics '87 Conference, New Orleans, LA, February 24-25, Industrial Fabrics Association International, St. Paul, MN, pp 169-180.

Koerner, R. M. 1986. Designing with Geosynthetics, Prentice-Hall, Englewood Cliffs, NJ.

Lafleur, J., Sall, M. S., and Ducharme, A. 1987. "Frictional Characteristics of Geotextiles with Compacted Lateritic Gravels and Clays," Proceedings Geosynthetics '87 Conference, New Orleans, LA, February 24-25, Industrial Fabrics Association International, St. Paul, MN, pp 205-215.

Lambe, T. William. 1965. "Soil Testing for Engineers," John Wiley and Sons, New York.

Leshchinsky, D., and Field, D. A. 1987. "In-Soil Load Elongation, Tensile Strength and Interface Friction of Non-Woven Geotextiles," Proceedings Geosynthetics '87 Conference, New Orleans, LA, February 24-25, Industrial Fabrics Association International, St. Paul, MN, pp 238-249.

Martin, J. P., Koerner, R. M., and Whitty, J. E. 1984. "Experimental Friction Evaluation and Slippage Between Geomembranes, Geotextiles and Soils," Proceedings International Conference on Geomembranes, Denver, CO., June 20-24, Vol 1, pp 191-196.

Miyamori, T., Iwai, S., and Makiuchi, C. 1986. "Frictional Characteristics of Non-Woven Fabrics," Proceedings Third International Conference on Geotextiles, Vienna, Austria, April 7-11, Vol 3, pp 701-705.

Myles, B. 1982. "Assessment of Soil Fabric Friction by Means of Shear," Proceedings Second International Conference on Geotextiles, Las Vegas, NV, August 1-6, Vol 3, pp 787-791.

New Orleans District, Corps of Engineers. 1988. "Geotextile Prototype Pullout Tests, Belle Chasse, Louisiana, May 1988," New Orleans, LA.

__________. 1989a. "Bonnet Carre Spillway Geotextile Reinforced Test Section and Pullout Tests," New Orleans, LA.

__________. 1989b. "Geotextile Prototype Pullout tests, Bonnet Carre Spillway, October - November, 1989," New Orleans LA.

Palmeira, E. M., and Milligan, G. W. E. 1990. "Large-Scale Pullout Tests on Geotextiles and Geogrids," Proceedings 4th International Conference on Geotextiles Geomembranes and Related Products, The Hague, The Netherlands, May 28-June 1, Vol 2, pp 743-746.

Paulson, N. J., and Langston, M. C. 1987. "Stitchbonded Composites," Proceedings Geosynthetics '87 Conference, New Orleans, LA, February 24-25, Industrial Fabrics Association International, St. Paul, MN, pp 193-204.

Rowe, R. K., Ho, S. K., and Fisher, D. G. 1985. "Determination of Soil-Geotextile Interface Strength Properties," Second Canadian Symposium on Geotextiles and Geomembranes, Edmonton, Alberta, pp 25-34.

Saxena, S. K., and Budiman, J. S. 1985. "Interface Response of Geotextiles," Proceedings, Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, CA, August, pp 1801-1804.

Schmertmann, G. R., Chouerty-Curtis, V. E., Johnson, R. D., and Bonaparte, R. 1987. "Design Charts for Geogrid-Reinforced Soil Slopes," Proceedings Geosynthetics '87 Conference, New Orleans, LA, February 24-25, Industrial Fabrics Association International, St. Paul, MN, pp 108-120.

Seed, H. B., and Chan, C. K. 1959. "Structure and Strength Characteristics of Compacted Clay," Journal of Soil Mechanics and Foundation Division, American Society of Civil Engineers, Vol 85, No. SM 5.

Shen, C. K., Mitchell, J. K., DeNatale, J. S., and Romstad, K. M. 1979. "Laboratory Testing and Model Studies of Friction in Reinforced Earth," International Conference on Soil Reinforcement: Reinforced Earth and Other Techniques, Paris, France, March 21-22, Vol 1, pp 169-174.

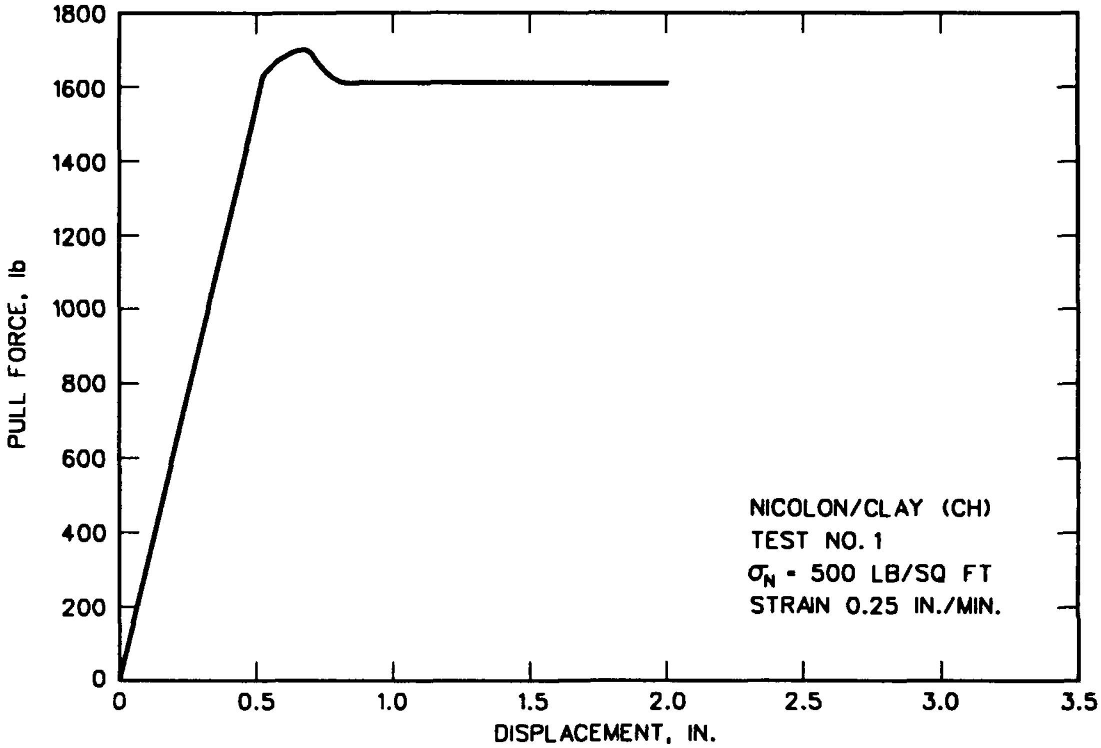
Williams, N. D., and Houlihan, M. F. 1987. "Evaluation of Interface Friction Properties Between Geosynthetics and Soils," Proceedings Geosynthetics '87 Conference, New Orleans, LA, February 24-25, Industrial Fabrics Association International, St. Paul, MN, pp 616-627.

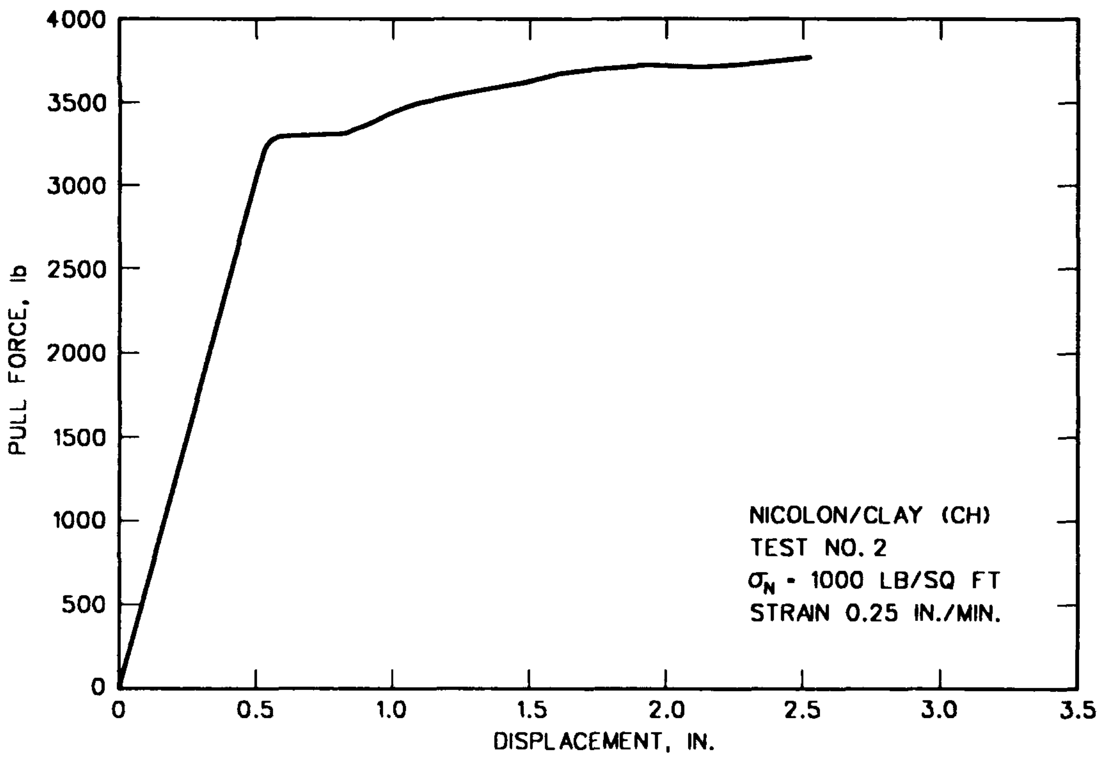
Wright, D. K., Gilbert, P. A., and Saada, A. S. 1978. "Shear Devices for Determining Dynamic Soil Properties," Proceedings of the 1978 G. T. Specialty Conference, American Society of Civil Engineers, Pasadena, CA.

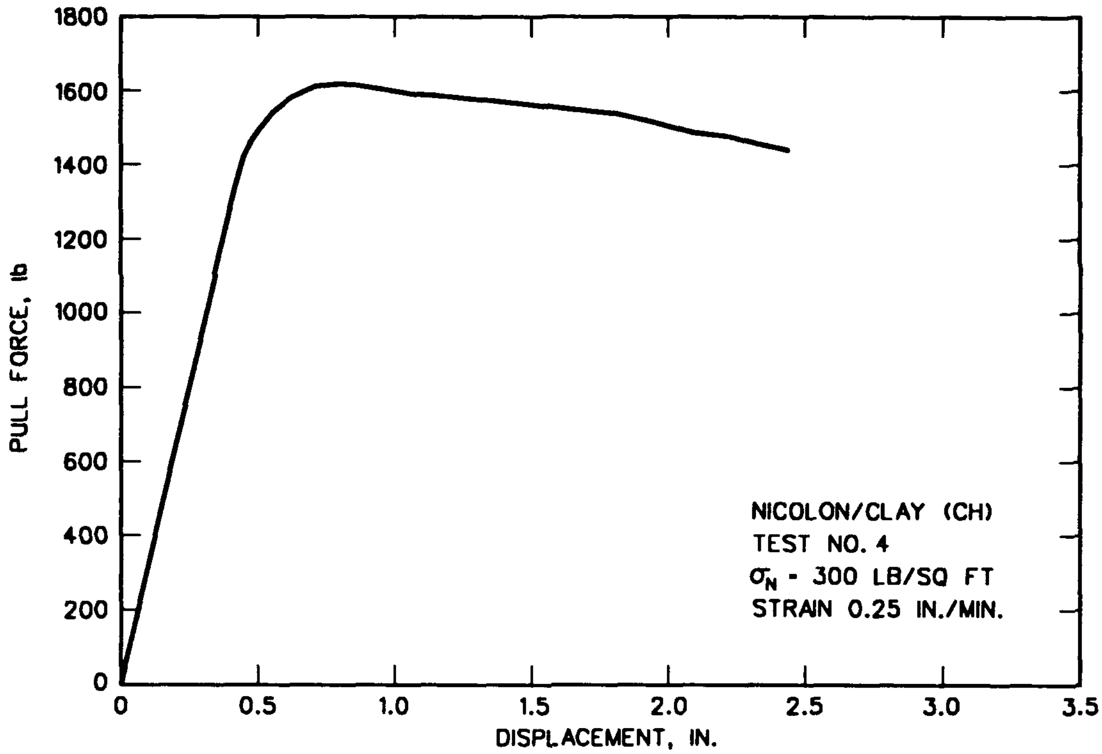
Wu, T. H. 1967. Soil Mechanics, Allyn and Bacon, Inc., Boston, MA.

APPENDIX A:

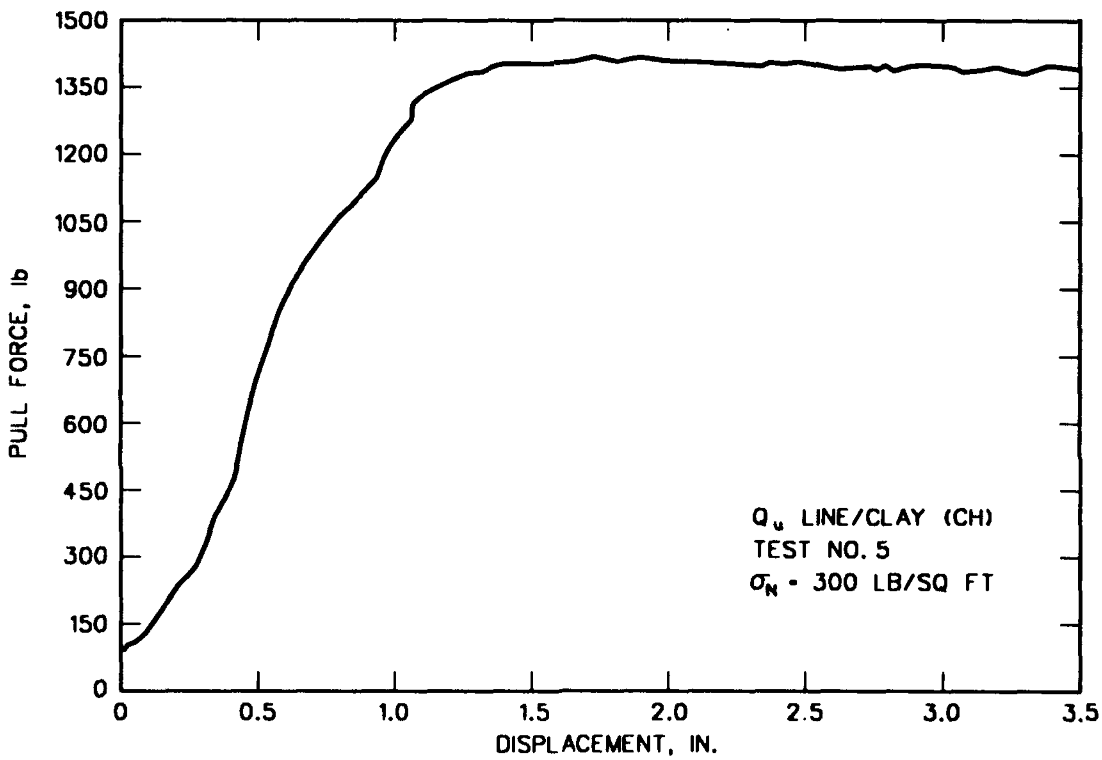
FORCE-DISPLACEMENT RELATIONSHIPS FOR PHASE 1 TESTS


1800
1600
1400
1200
1000
800
600
400
200
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 1
σ_N = 500 LB/SQ FT
STRAIN 0.25 IN./MIN.

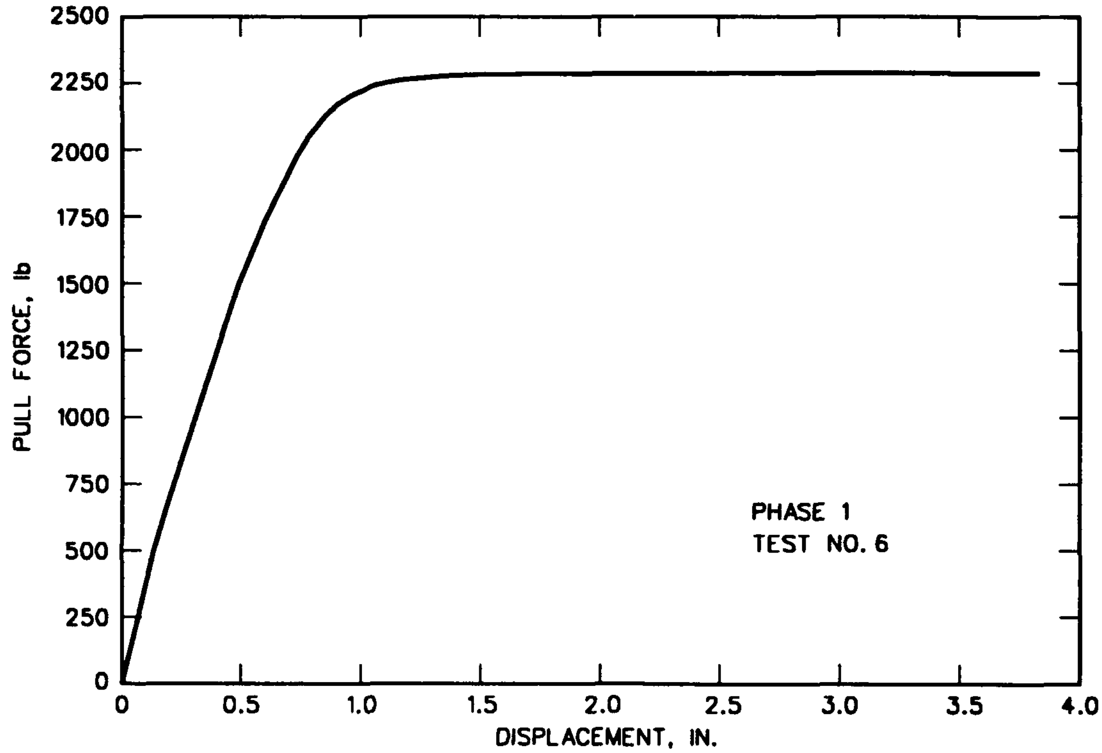

4000
3500
3000
2500
2000
1500
1000
500
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 2
σ_N = 1000 LB/SQ FT
STRAIN 0.25 IN./MIN.

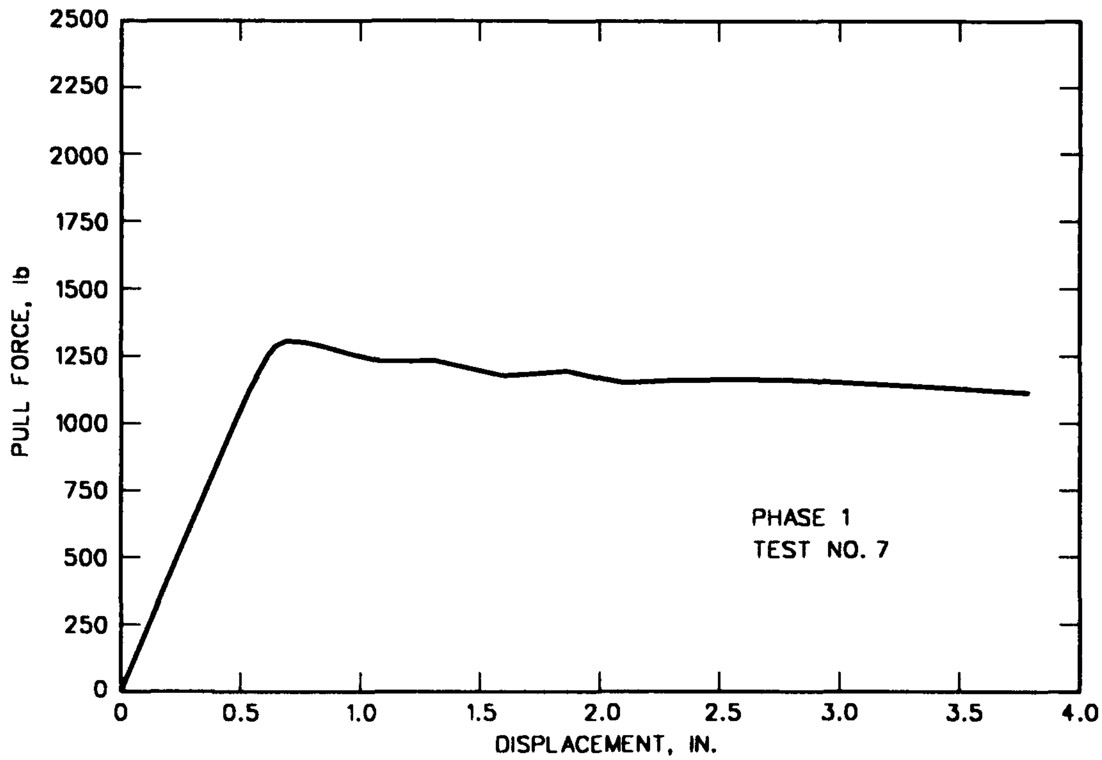


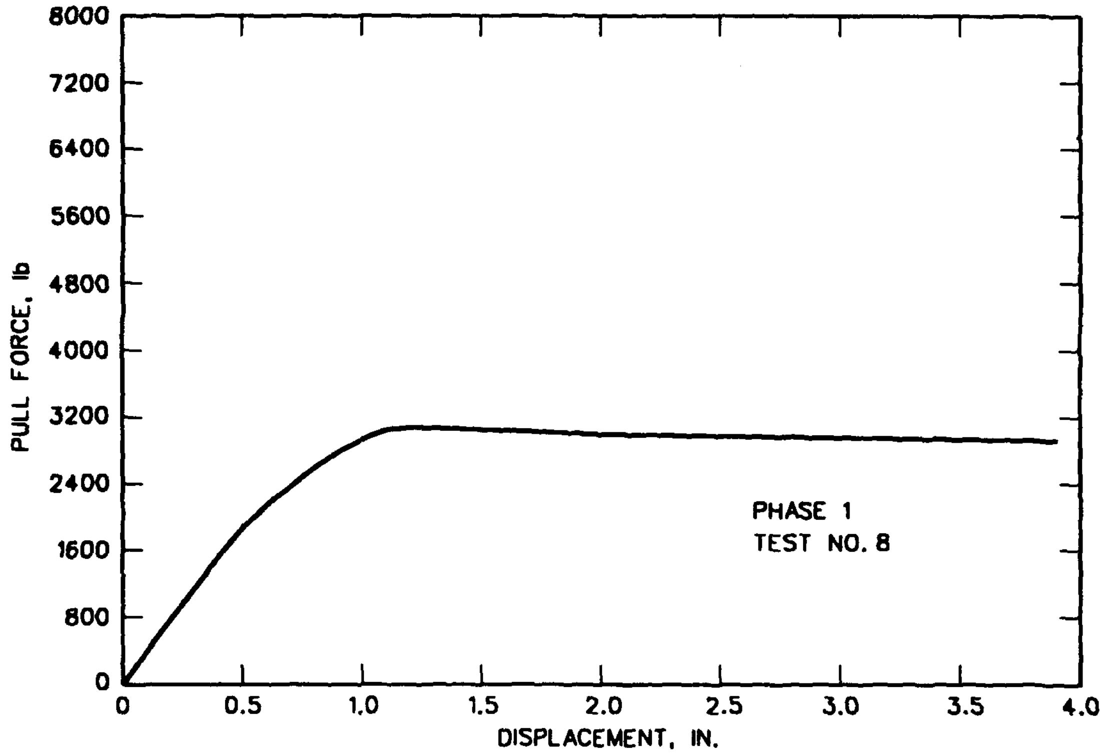
1800
1600
1400
1200
1000
800
600
400
200
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 4
σ_N - 300 LB/SQ FT
STRAIN 0.25 IN./MIN.

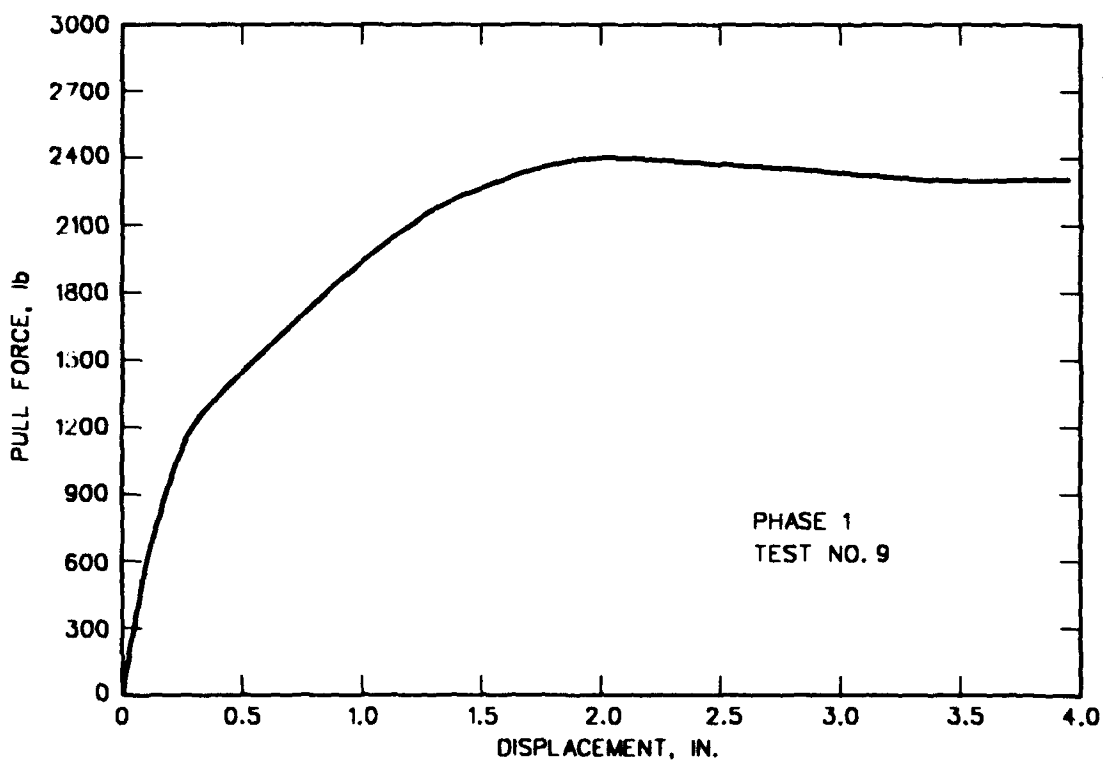


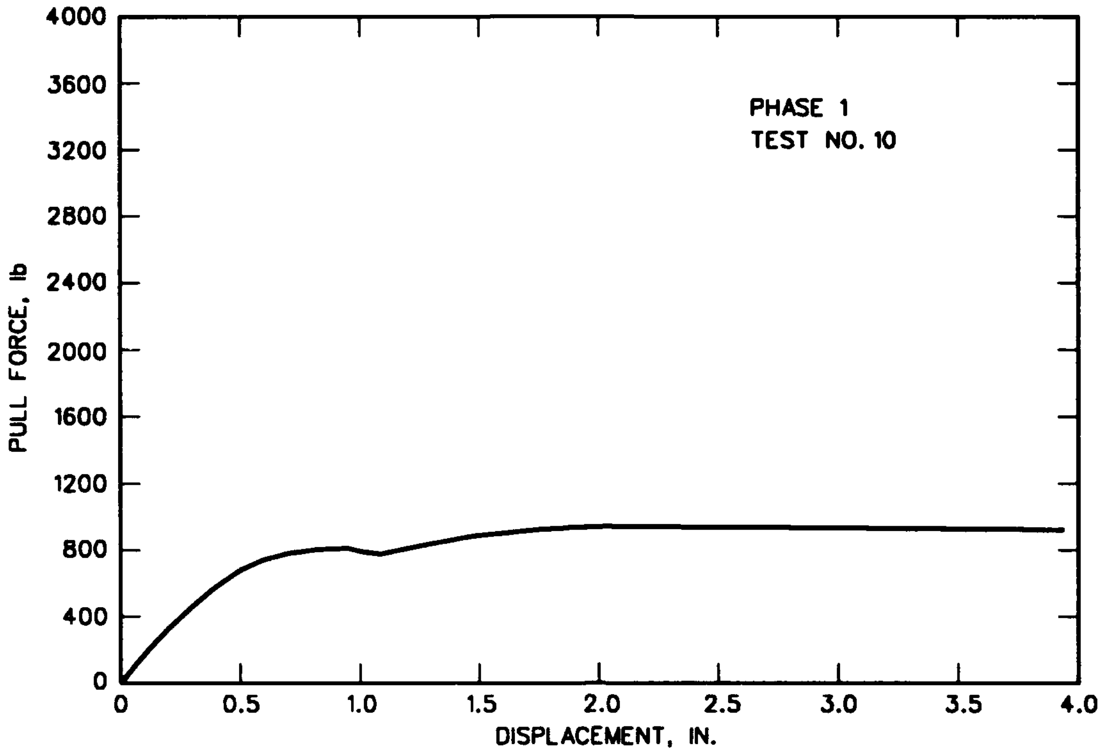
1500
1350
1200
1050
900
750
600
450
300
150
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
DISPLACEMENT, IN.
Q_u LINE/CLAY (CH)
TEST NO. 5
σ_N - 300 LB/SQ FT

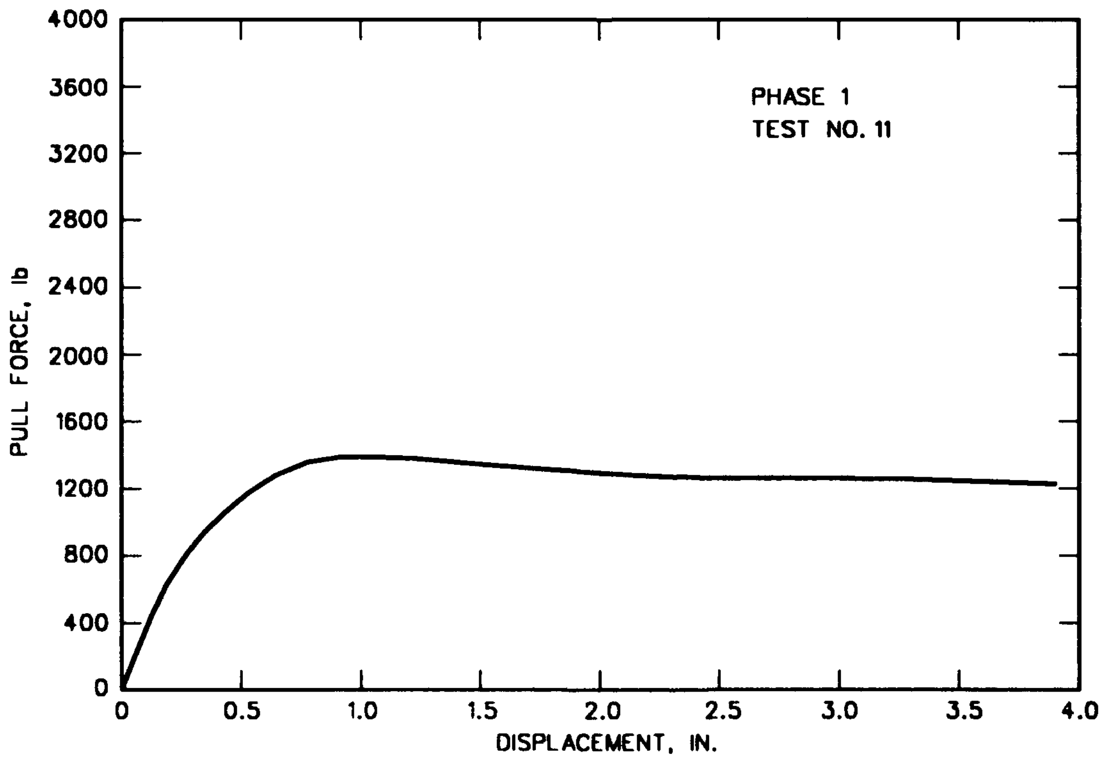

2500
2250
2000
1750
1500
1250
1000
750
500
250
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.
PHASE 1
TEST NO. 6

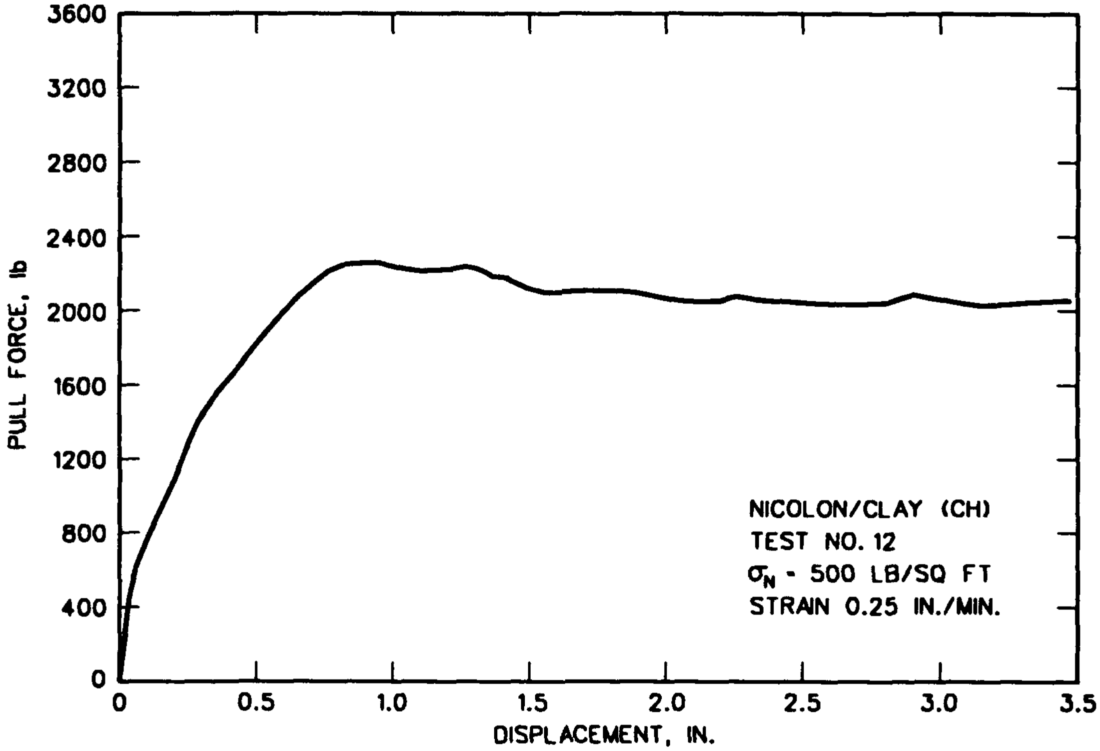

2500
2250
2000
1750
1500
1250
1000
750
500
250
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.
PHASE 1
TEST NO. 7

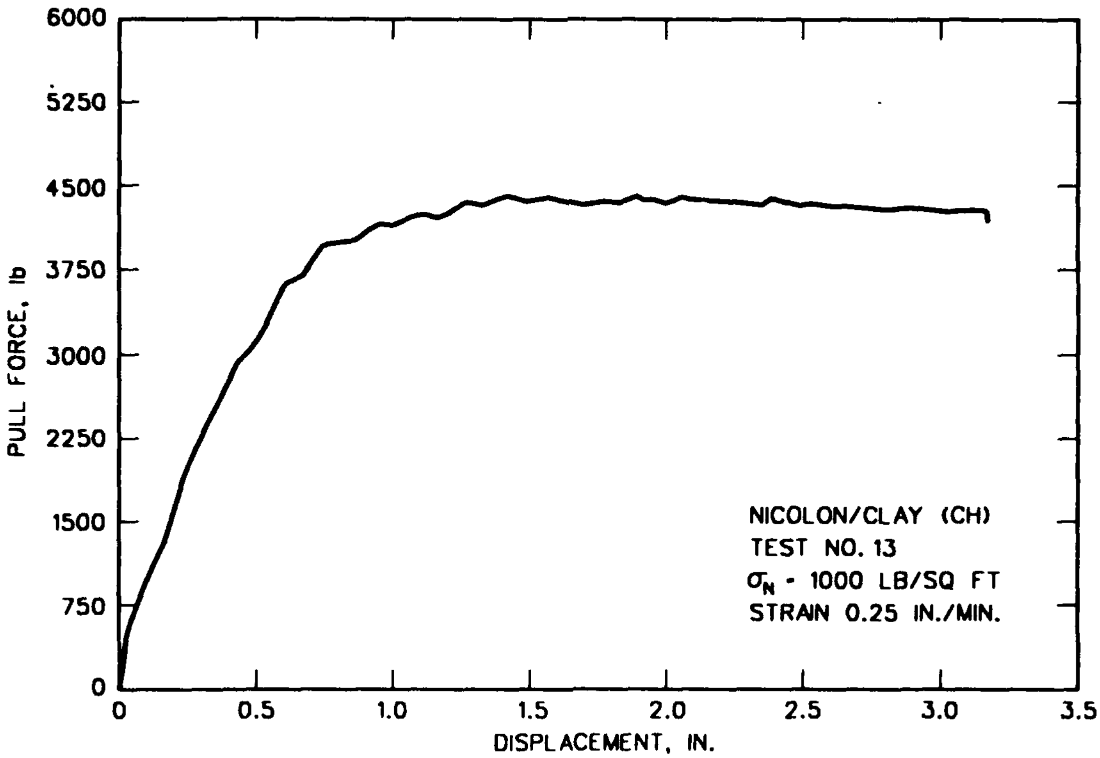

8000
7200
6400
5600
4800
4000
3200
2400
1600
800
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.
PHASE 1
TEST NO. 8

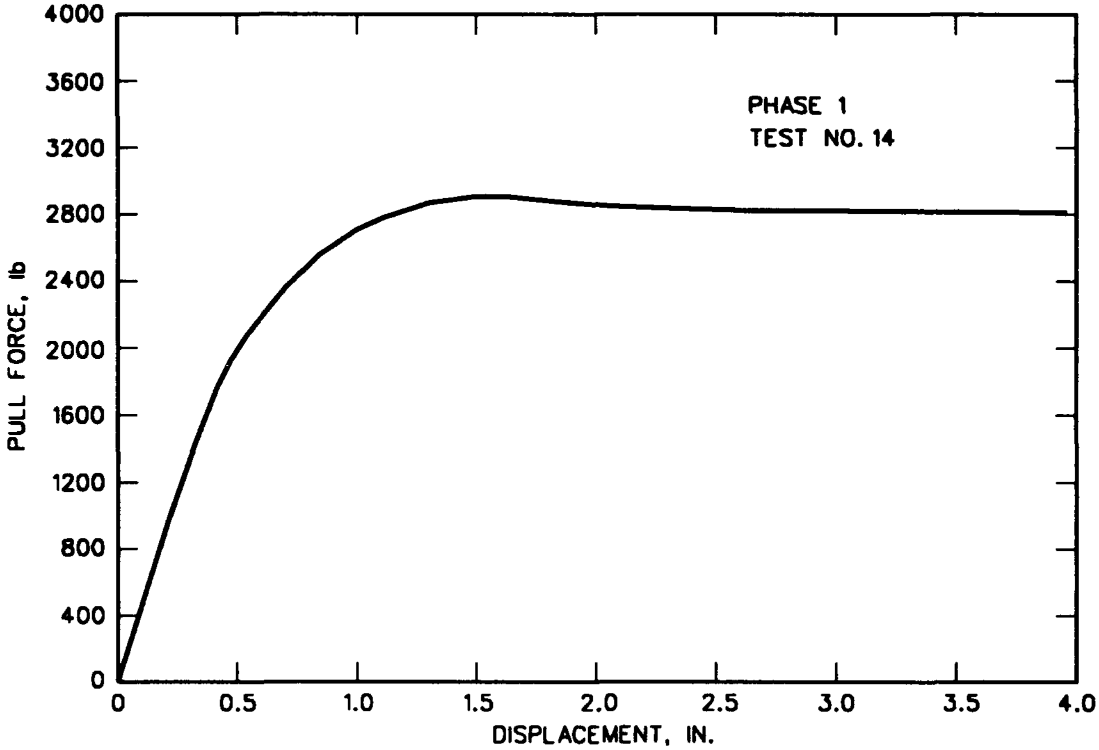

3000
2700
2400
2100
1800
1500
1200
900
600
300
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.
PHASE 1
TEST NO. 9

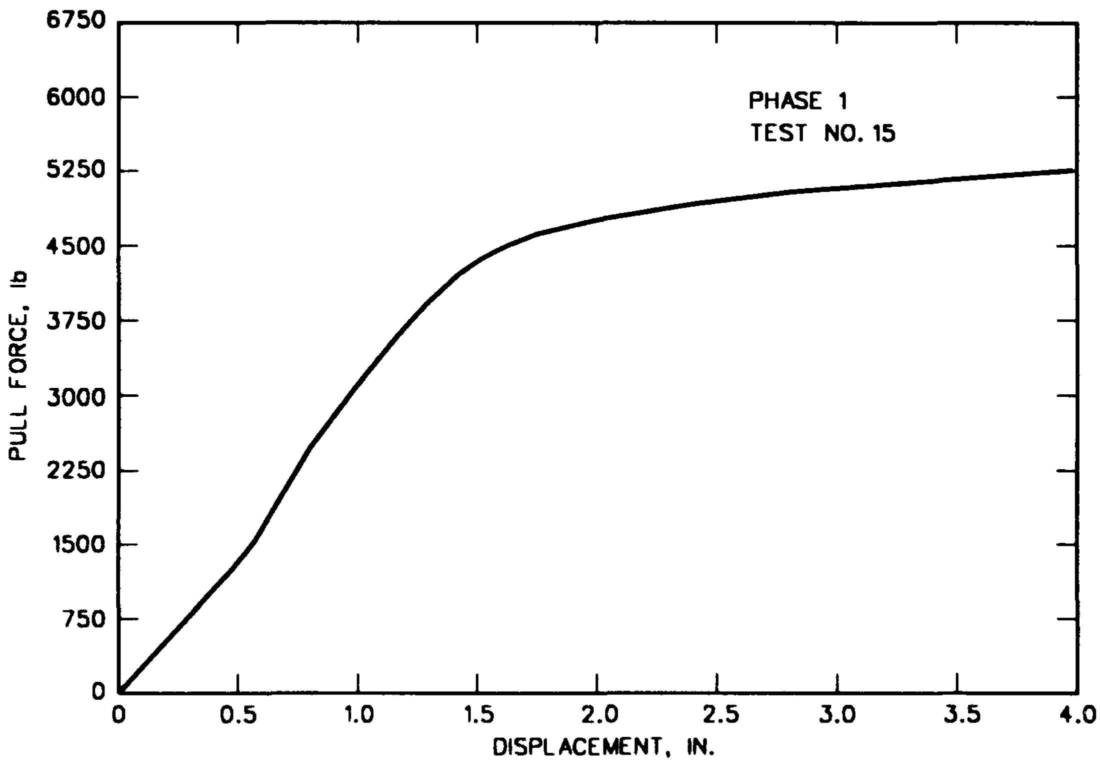

PHASE 1
TEST NO. 10
PULL FORCE, lb
DISPLACEMENT, IN.
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

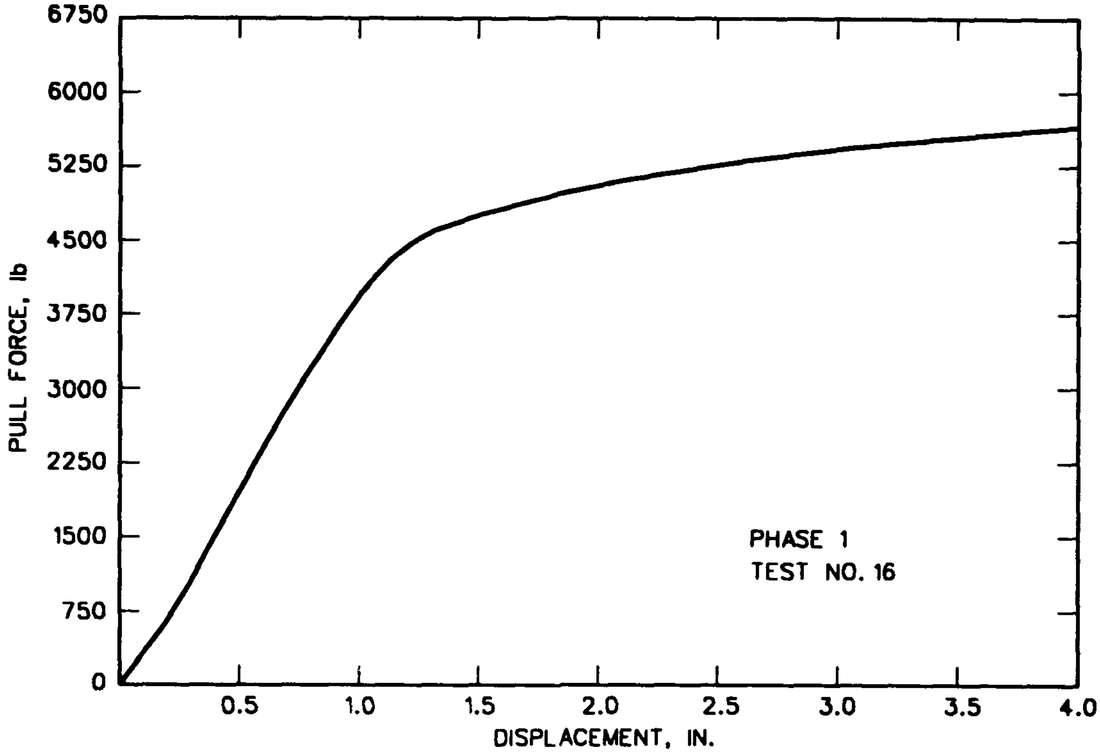

PHASE 1
TEST NO. 11
PULL FORCE, lb
DISPLACEMENT, IN.
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0


3600
3200
2800
2400
2000
1600
1200
800
400
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 12
σ_N - 500 LB/SQ FT
STRAIN 0.25 IN./MIN.

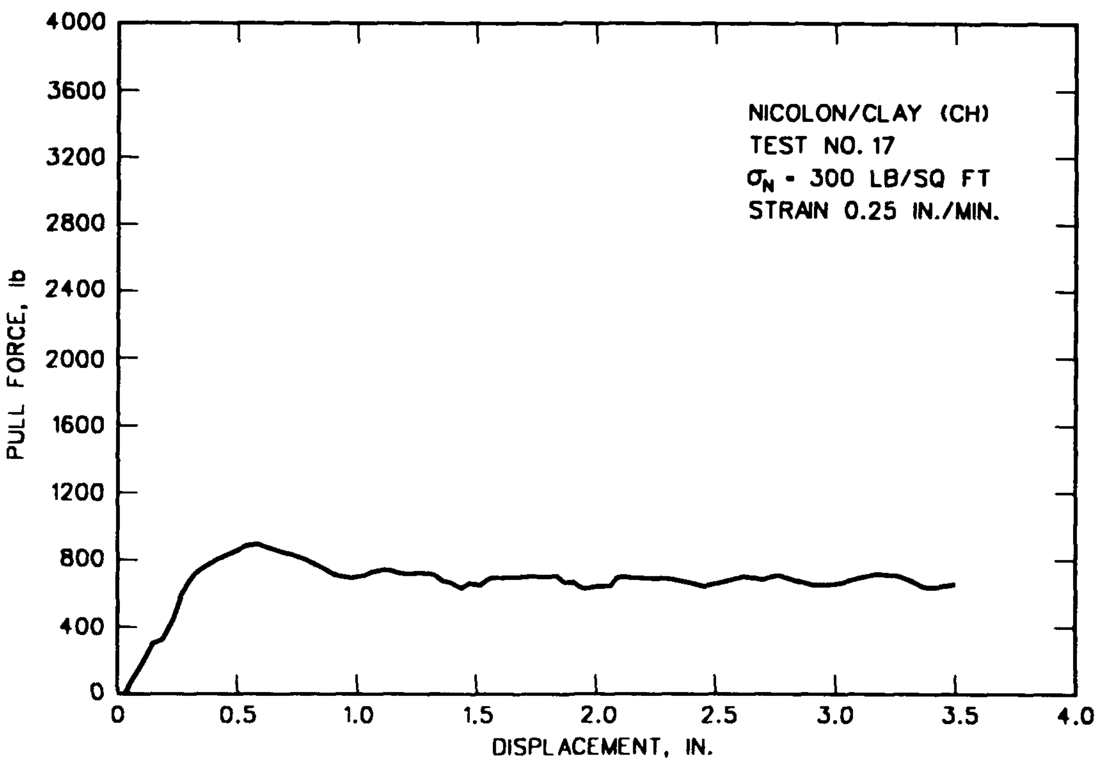

6000
5250
4500
3750
3000
2250
1500
750
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 13
σ_N - 1000 LB/SQ FT
STRAIN 0.25 IN./MIN.


PHASE 1
TEST NO. 14
PULL FORCE, lb
DISPLACEMENT, IN.
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

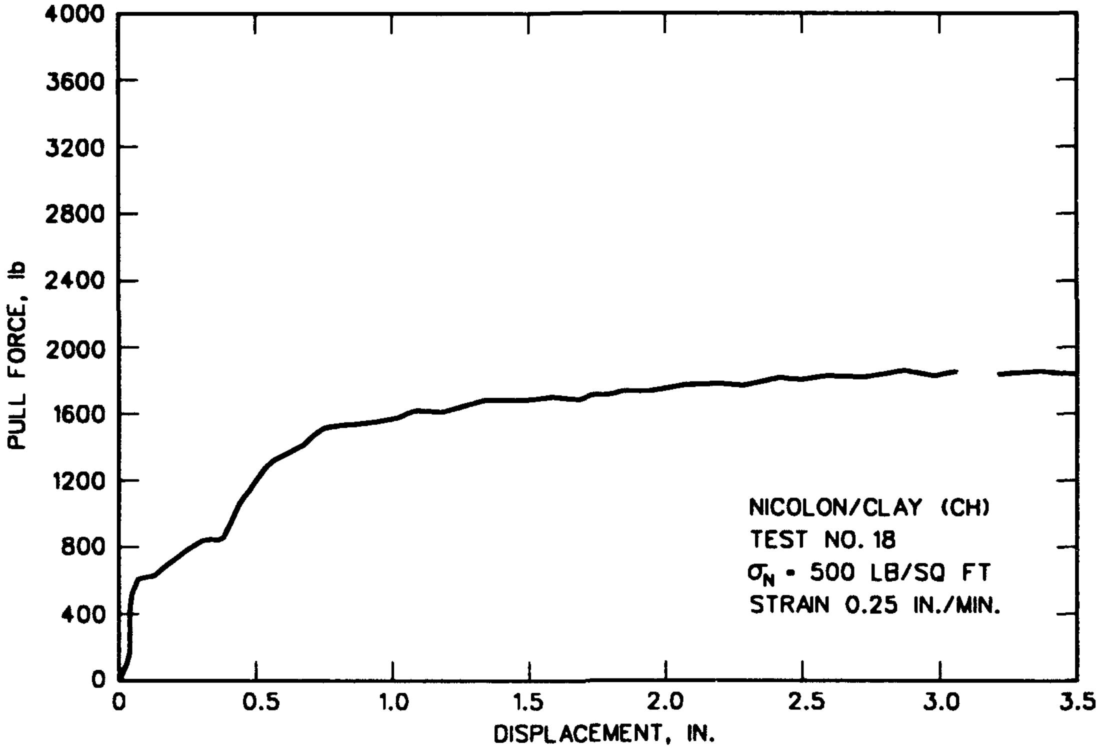

PHASE 1
TEST NO. 15
PULL FORCE, lb
DISPLACEMENT, IN.
0
750
1500
2250
3000
3750
4500
5250
6000
6750
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0



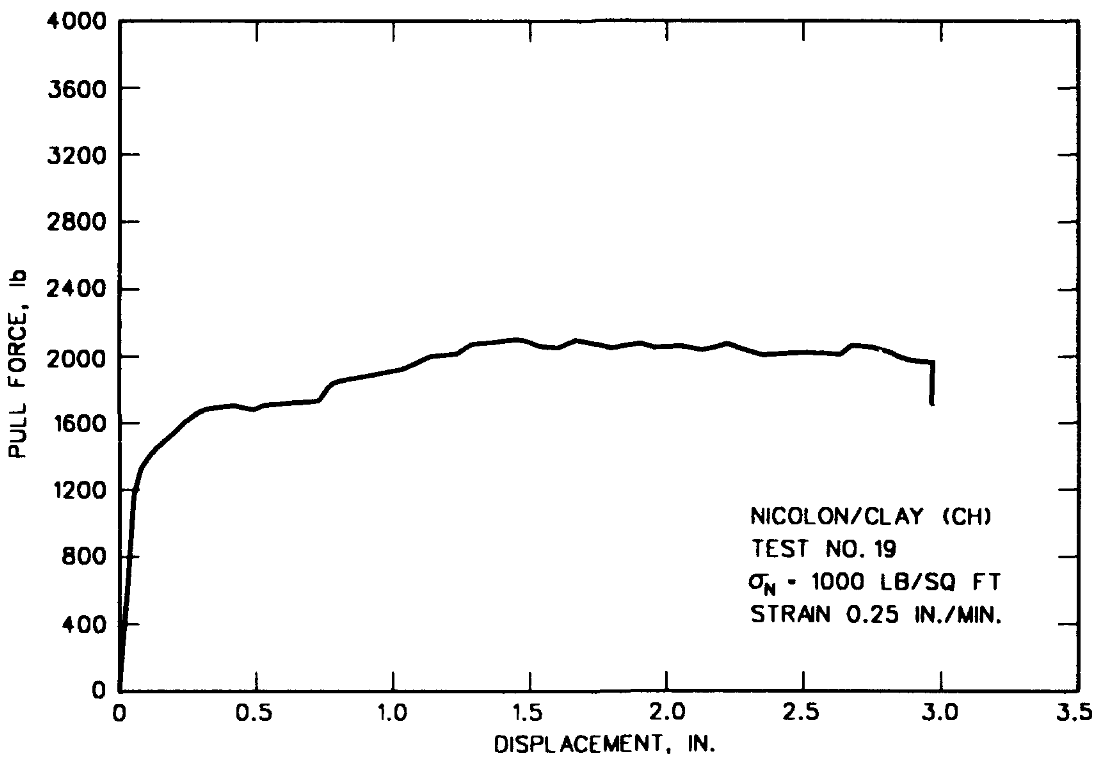
6750
6000
5250
4500
3750
3000
2250
1500
750
0
PULL FORCE, lb
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.
PHASE 1
TEST NO. 16



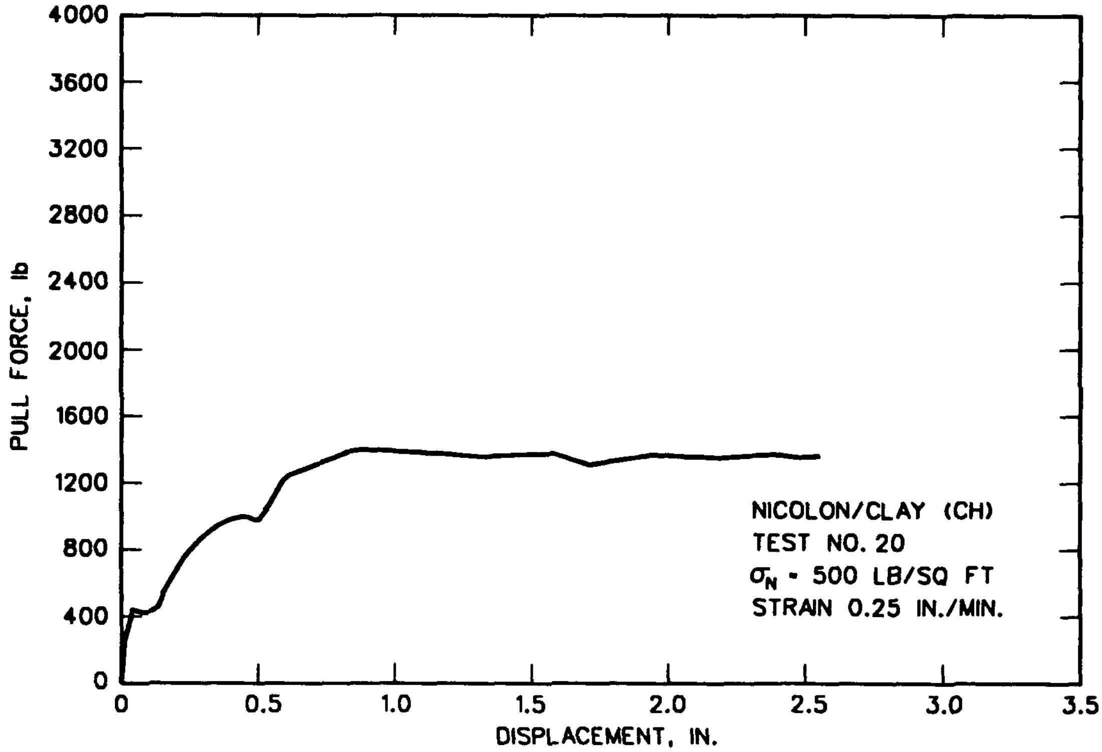
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 17
σ_N - 300 LB/SQ FT
STRAIN 0.25 IN./MIN.



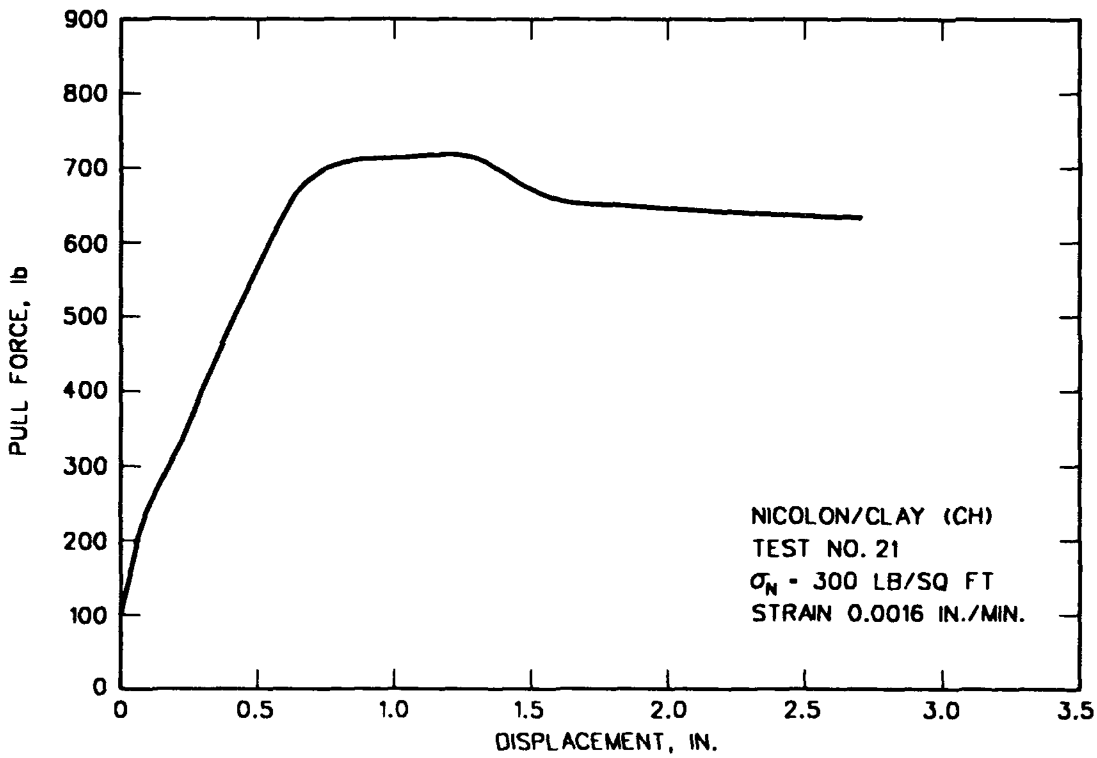
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 18
σ_N = 500 LB/SQ FT
STRAIN 0.25 IN./MIN.



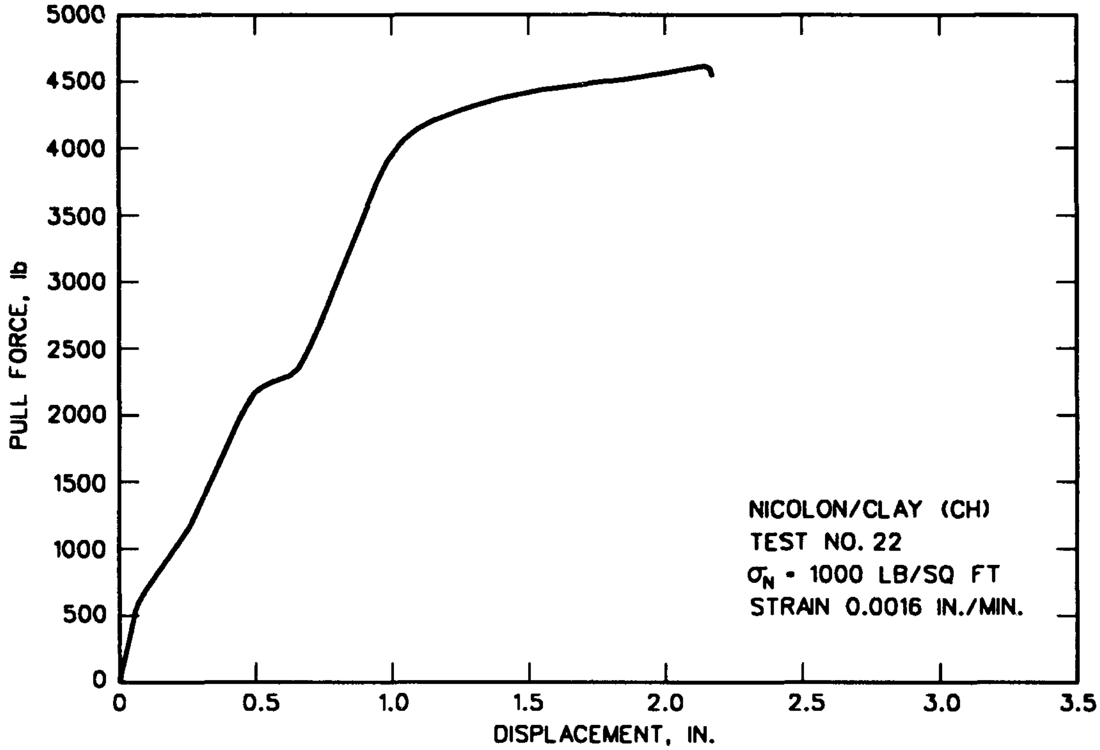
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 19
σ_N = 1000 LB/SQ FT
STRAIN 0.25 IN./MIN.

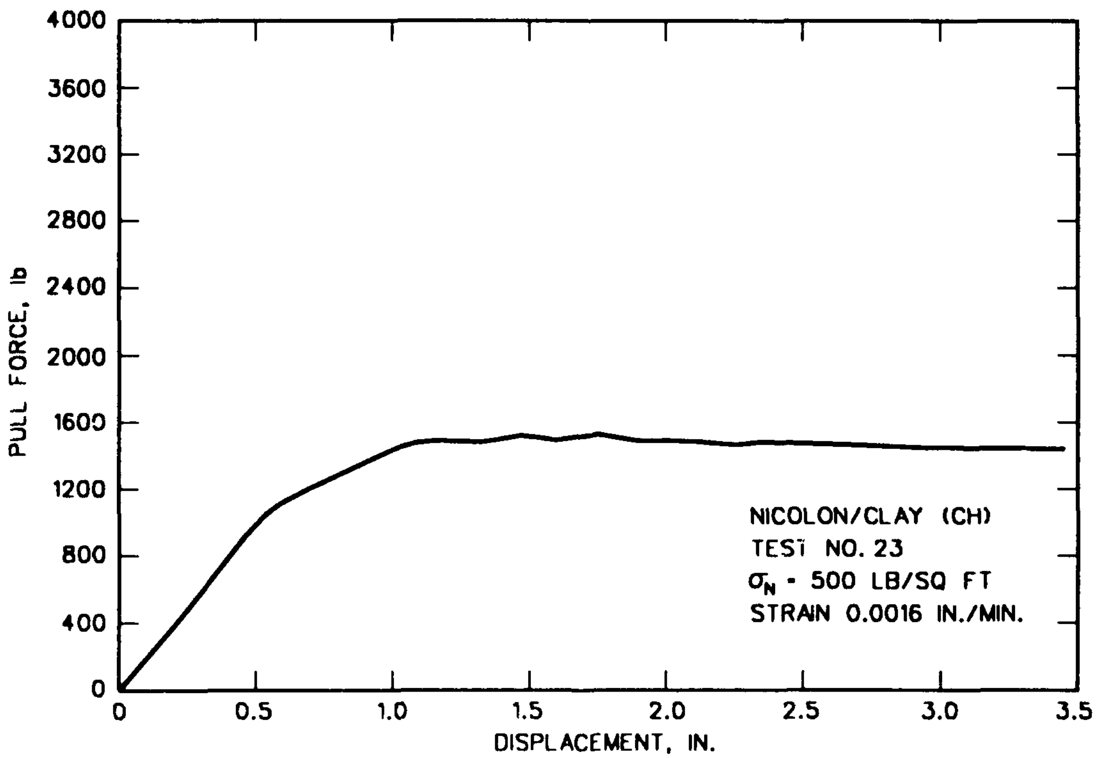


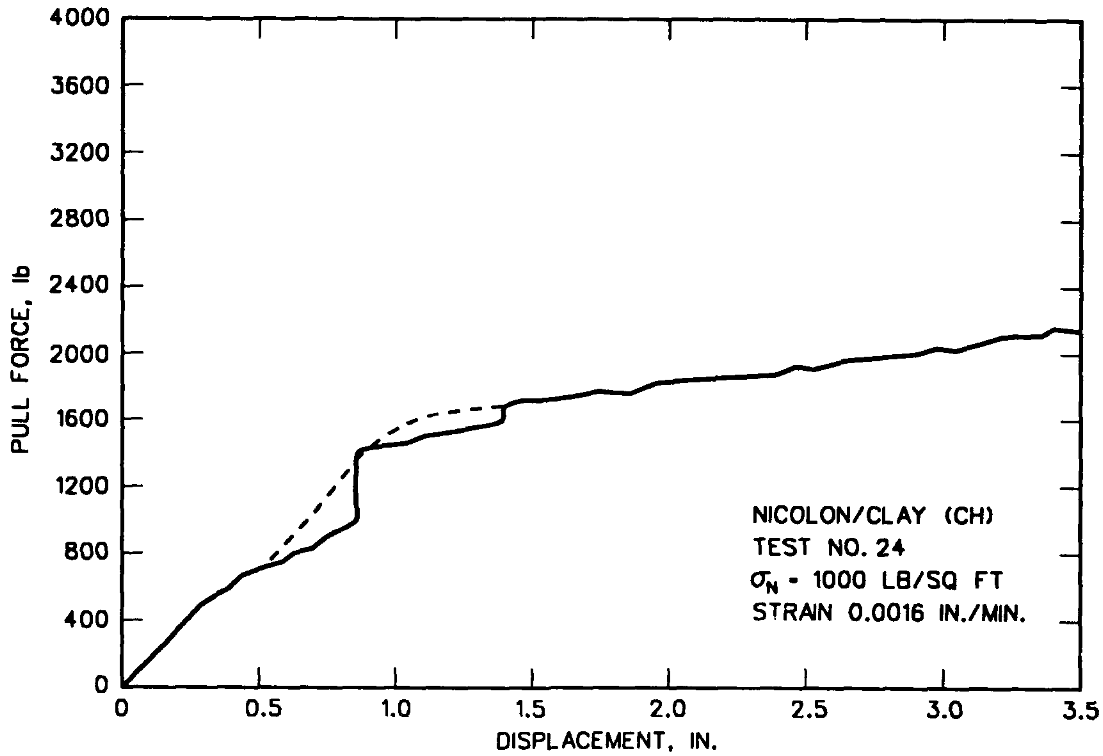
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 20
σ_N = 500 LB/SQ FT
STRAIN 0.25 IN./MIN.

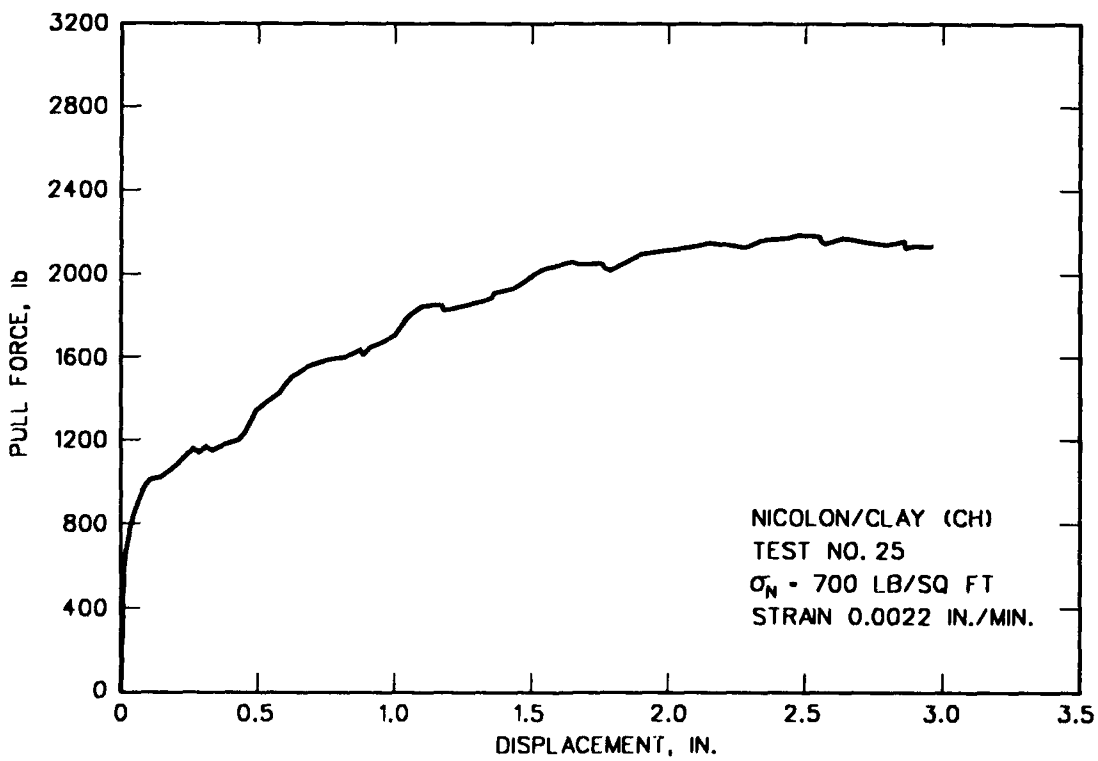


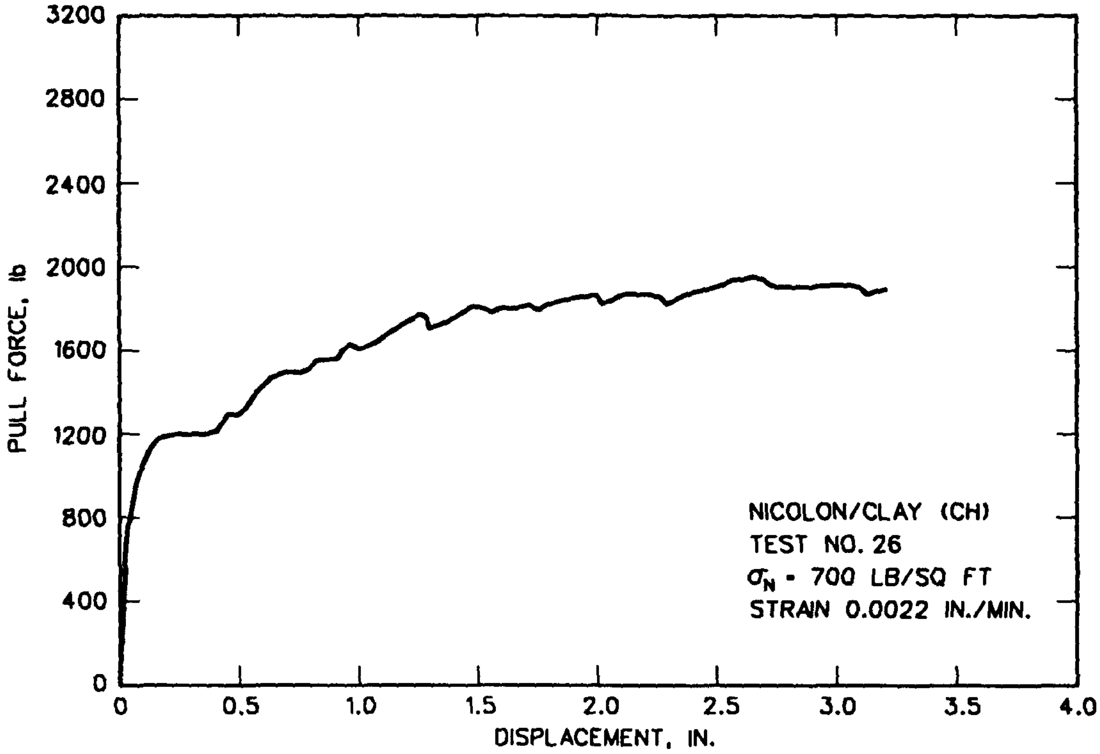
900
800
700
600
500
400
300
200
100
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 21
σ_N = 300 LB/SQ FT
STRAIN 0.0016 IN./MIN.

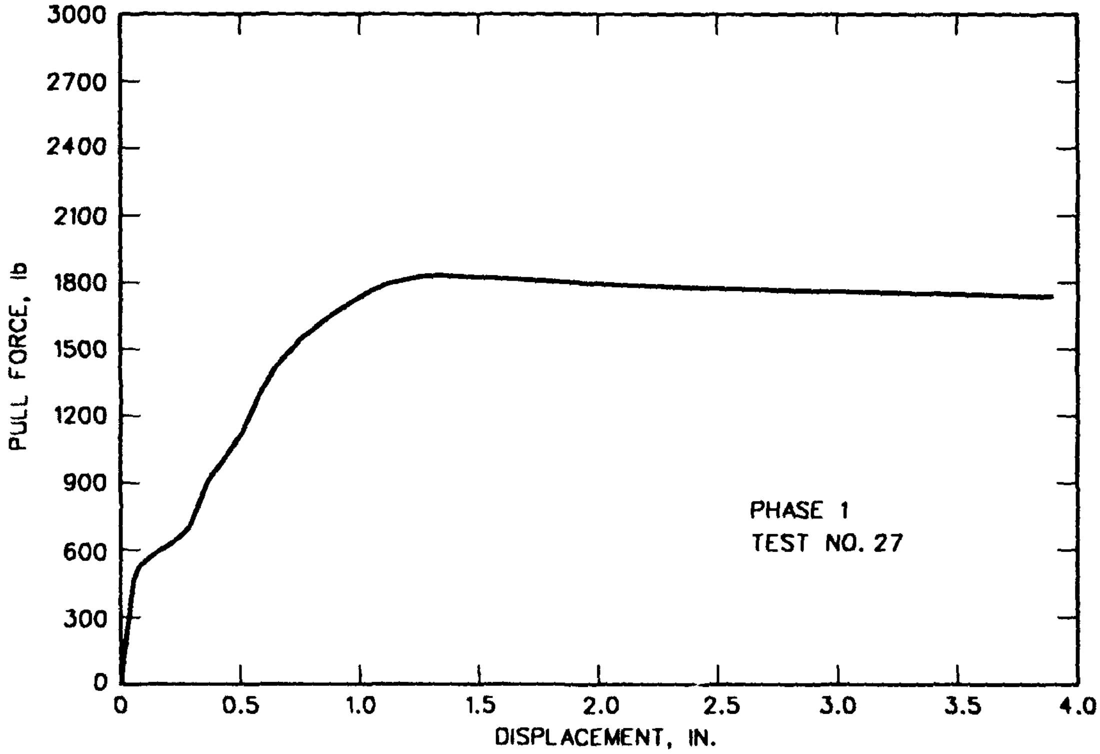

PULL FORCE, lb
5000
4500
4000
3500
3000
2500
2000
1500
1000
500
0
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 22
σ_N = 1000 LB/SQ FT
STRAIN 0.0016 IN./MIN.


PULL FORCE, lb
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 23
σ_N = 500 LB/SQ FT
STRAIN 0.0016 IN./MIN.


4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 24
σ_N = 1000 LB/SQ FT
STRAIN 0.0016 IN./MIN.

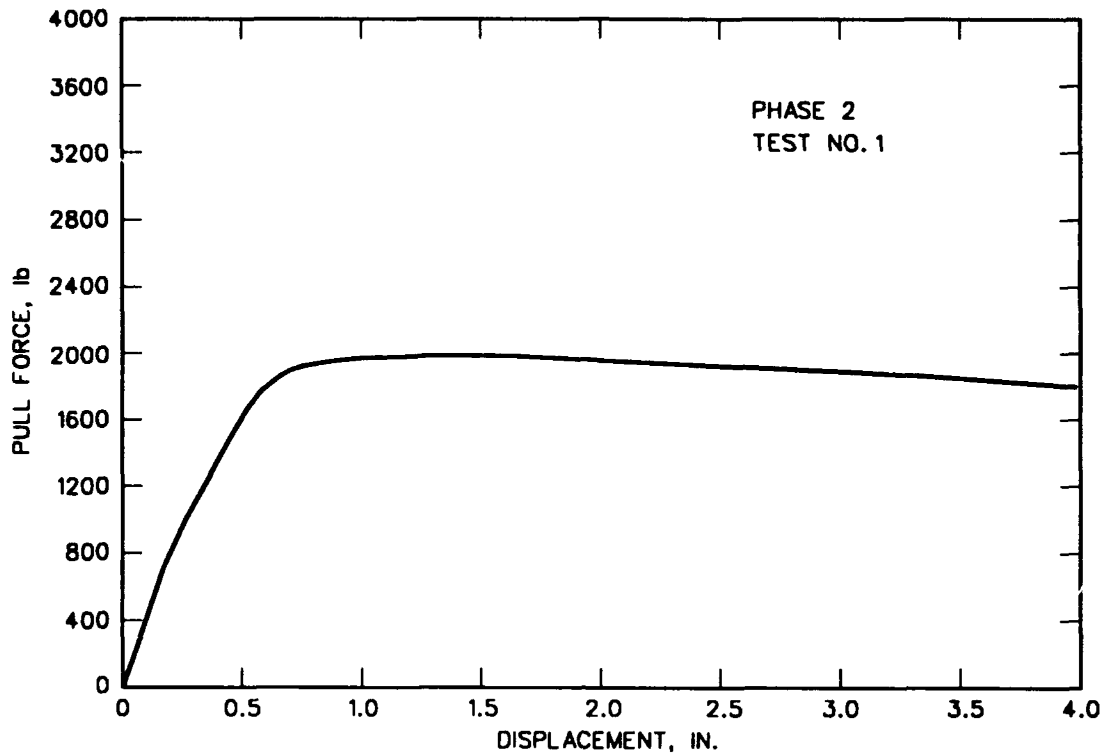

3200
2800
2400
2000
1600
1200
800
400
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 25
σ_N = 700 LB/SQ FT
STRAIN 0.0022 IN./MIN.

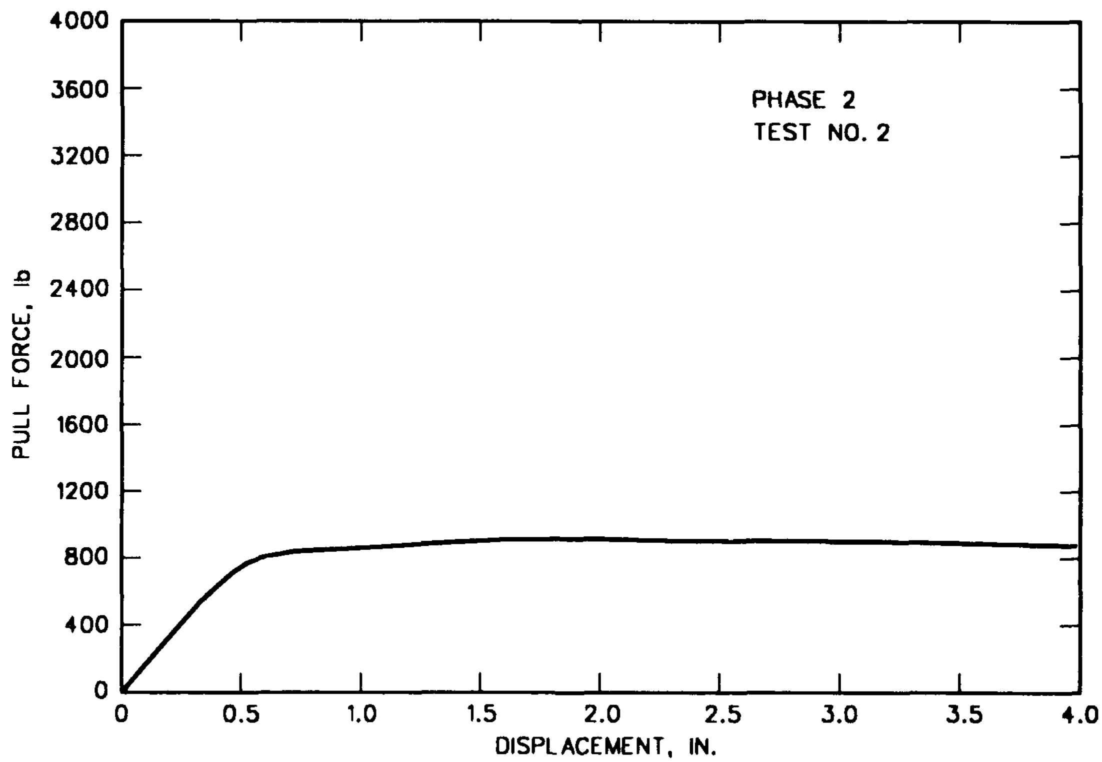

3200
2800
2400
2000
1600
1200
800
400
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.
NICOLON/CLAY (CH)
TEST NO. 26
σ_N = 700 LB/SQ FT
STRAIN 0.0022 IN./MIN.

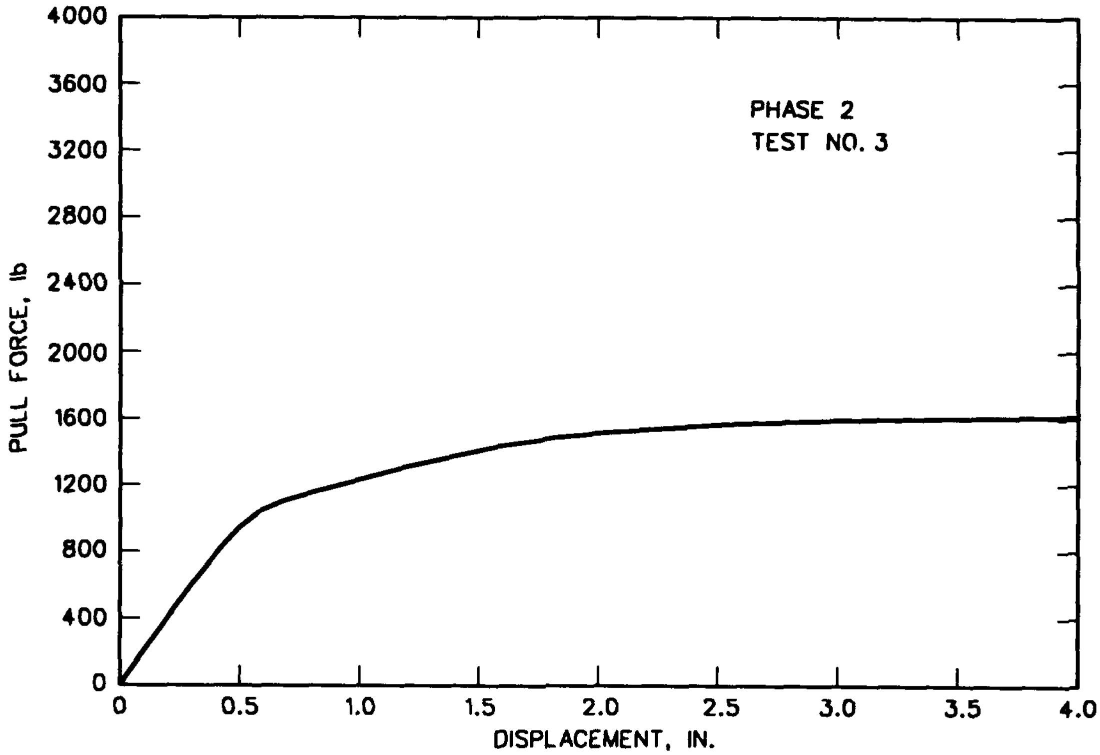

3000
2700
2400
2100
1800
1500
1200
900
600
300
0
PULL FORCE, lb
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.
PHASE 1
TEST NO. 27

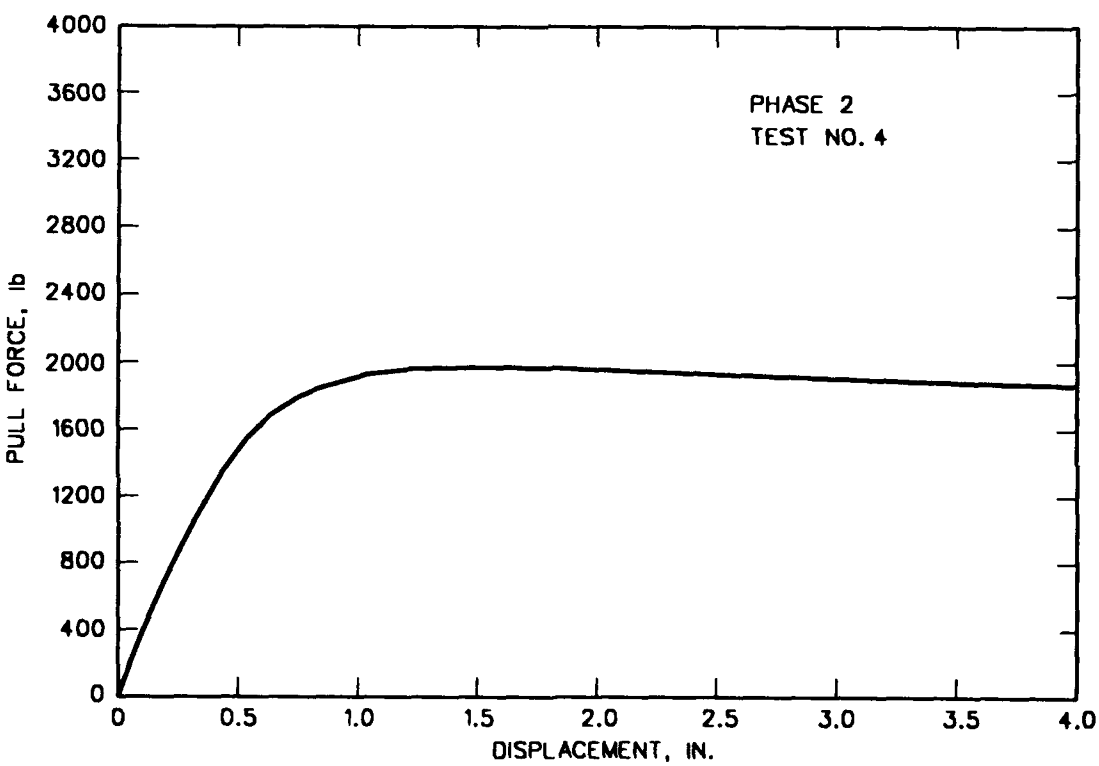
APPENDIX B:

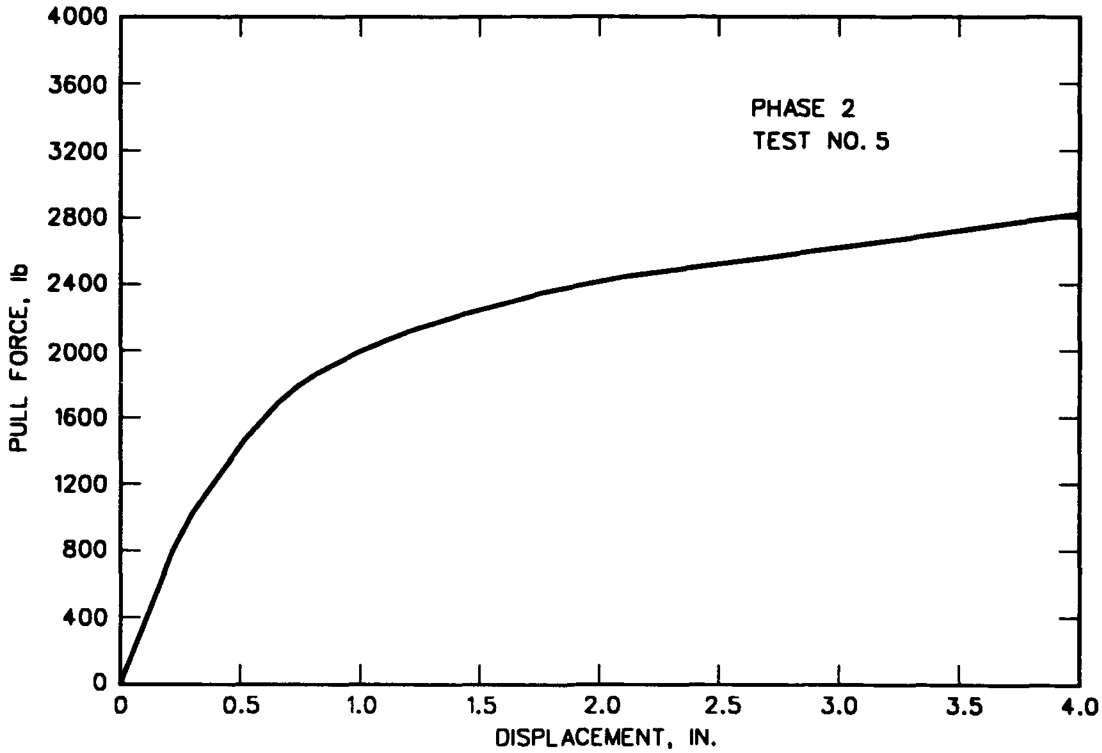
FORCE-DISPLACEMENT RELATIONSHIPS FOR PHASE 2 TESTS

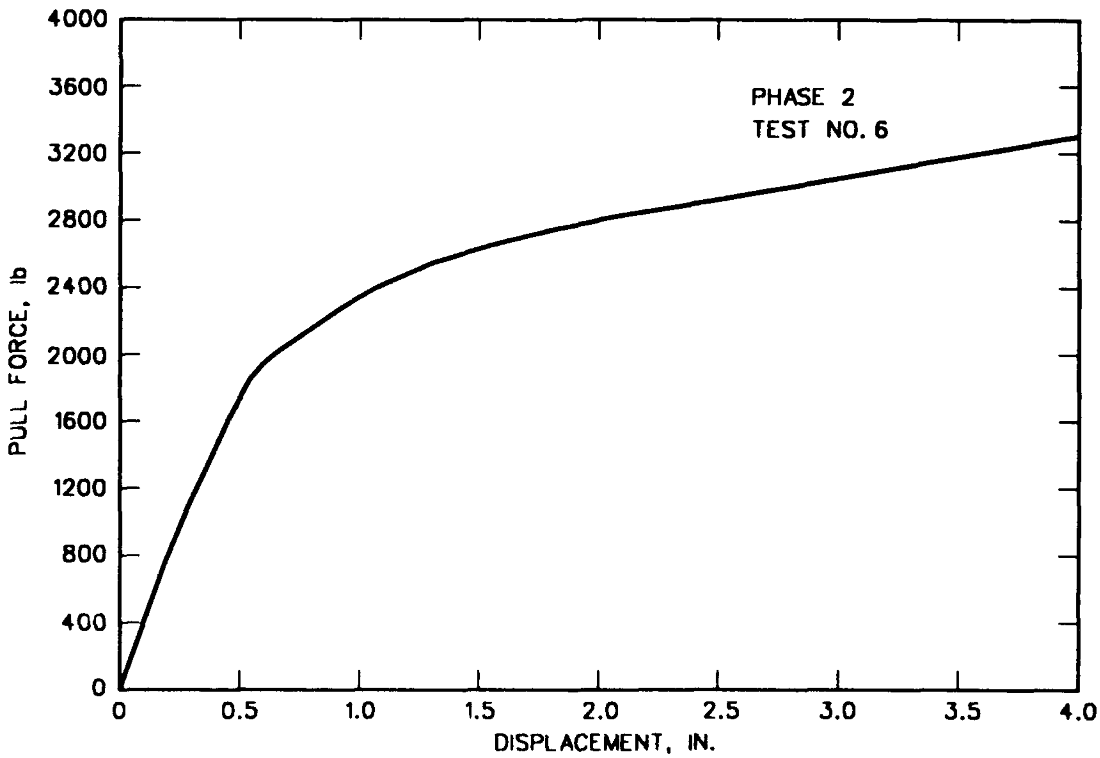

PHASE 2
TEST NO. 1
PULL FORCE, lb
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.

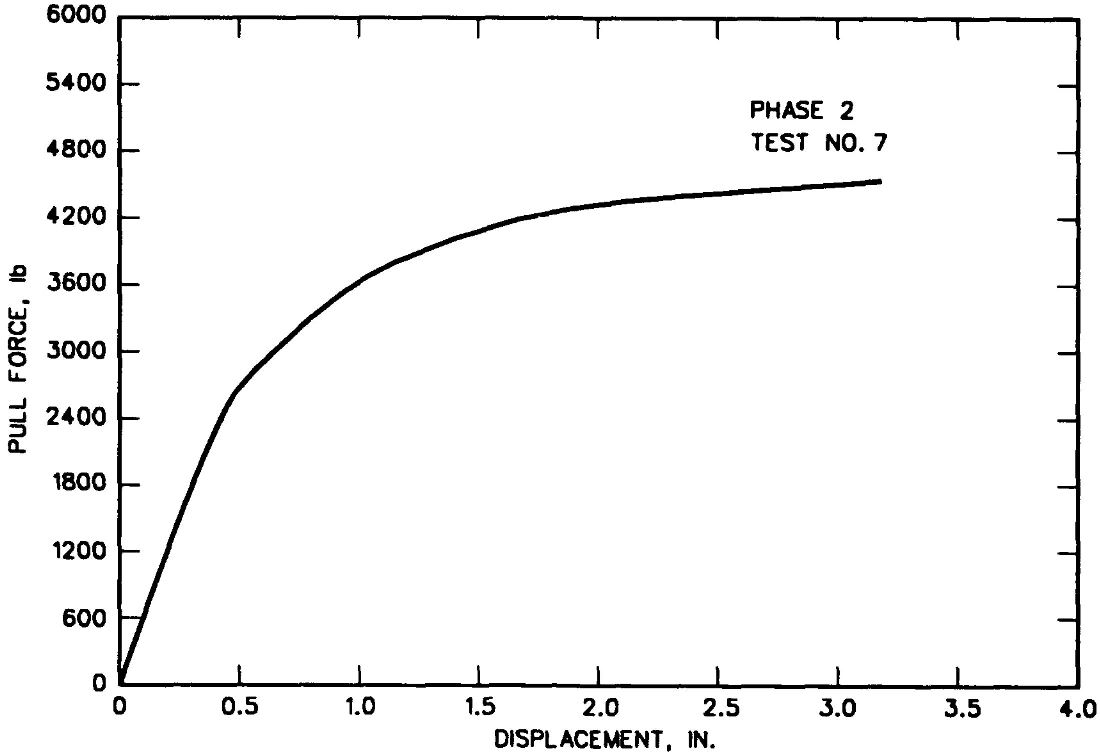

PHASE 2
TEST NO. 2
PULL FORCE, lb
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.


PHASE 2
TEST NO. 3
PULL FORCE, lb
DISPLACEMENT, IN.
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

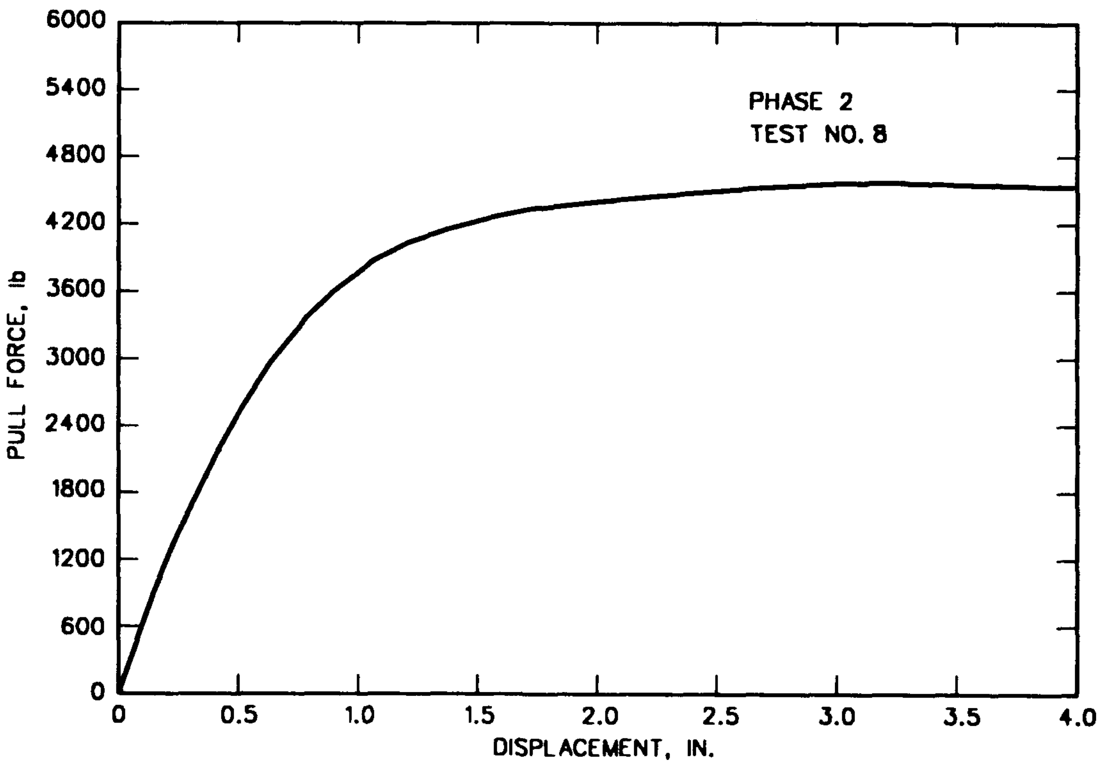

PHASE 2
TEST NO. 4
PULL FORCE, lb
DISPLACEMENT, IN.
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0


PHASE 2
TEST NO. 5
PULL FORCE, lb
DISPLACEMENT, IN.
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

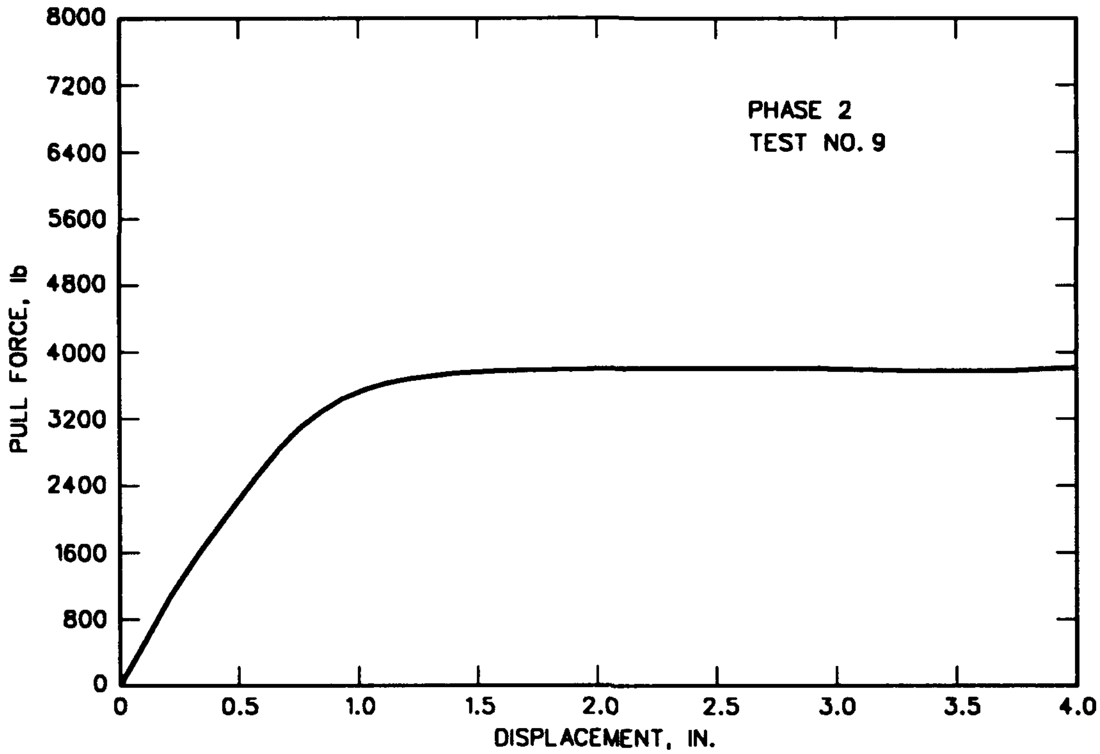

PHASE 2
TEST NO. 6
PULL FORCE, lb
DISPLACEMENT, IN.
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0



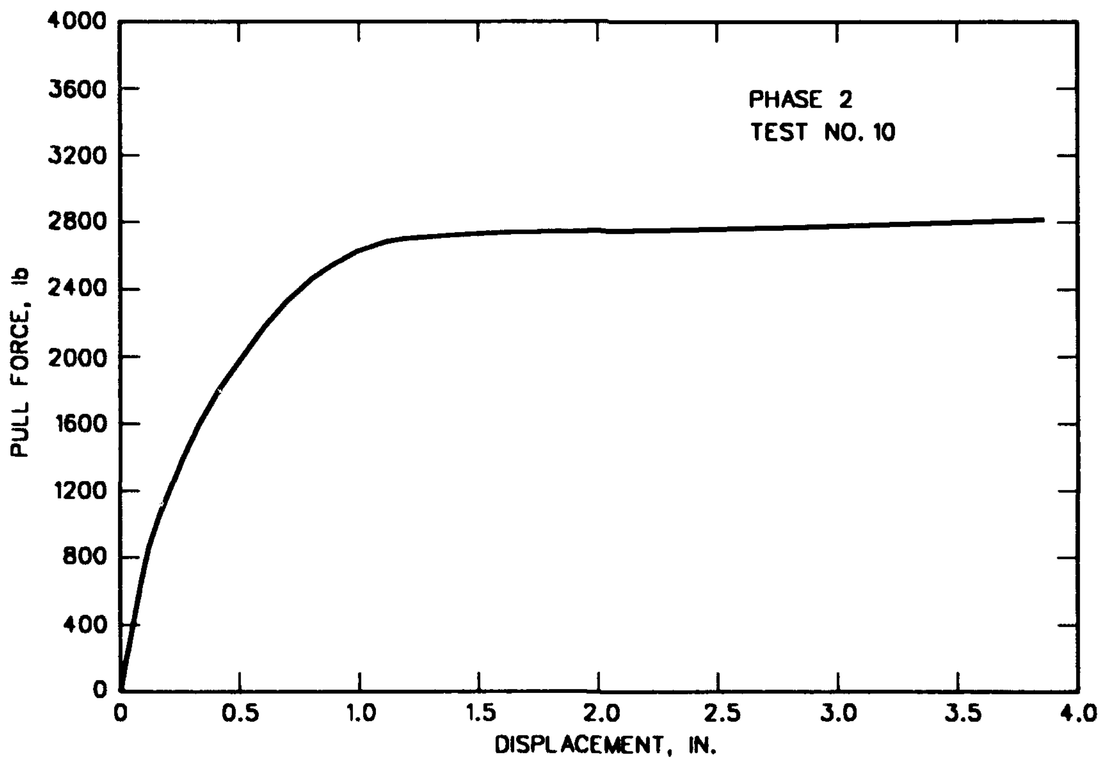
PHASE 2
TEST NO. 7
PULL FORCE, lb
DISPLACEMENT, IN.
0
600
1200
1800
2400
3000
3600
4200
4800
5400
6000
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0



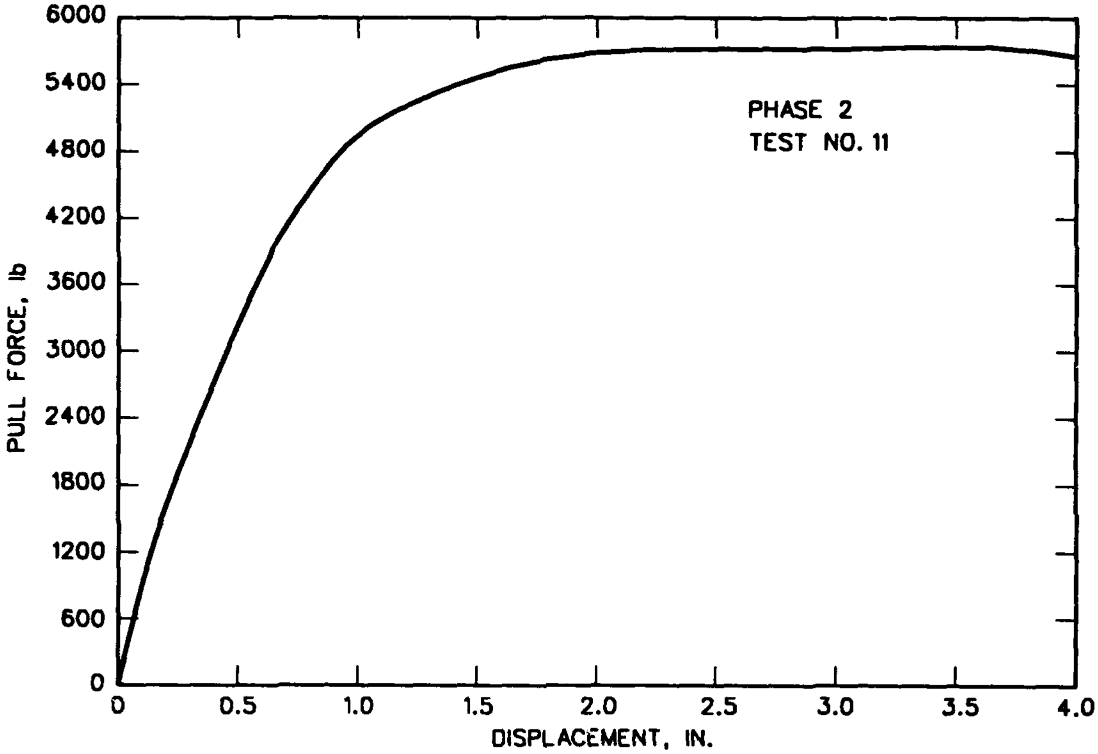
PHASE 2
TEST NO. 8
PULL FORCE, lb
DISPLACEMENT, IN.
0
600
1200
1800
2400
3000
3600
4200
4800
5400
6000
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0



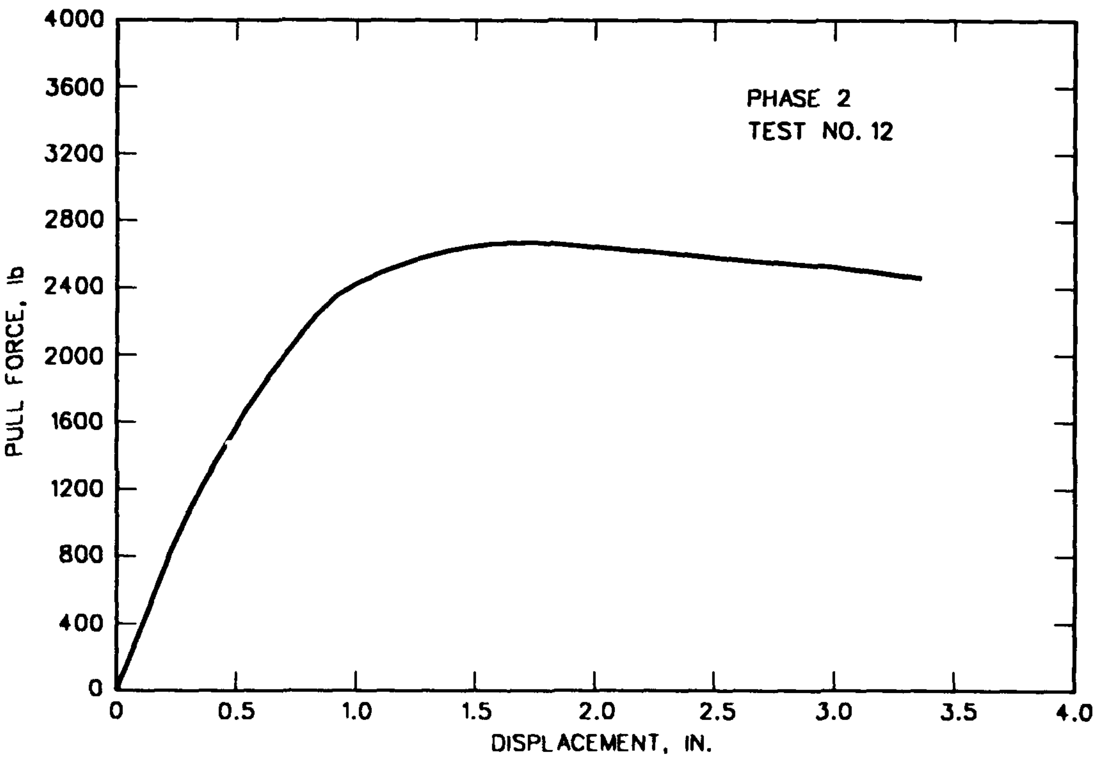
PHASE 2
TEST NO. 9
PULL FORCE, lb
8000
7200
6400
5600
4800
4000
3200
2400
1600
800
0
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.



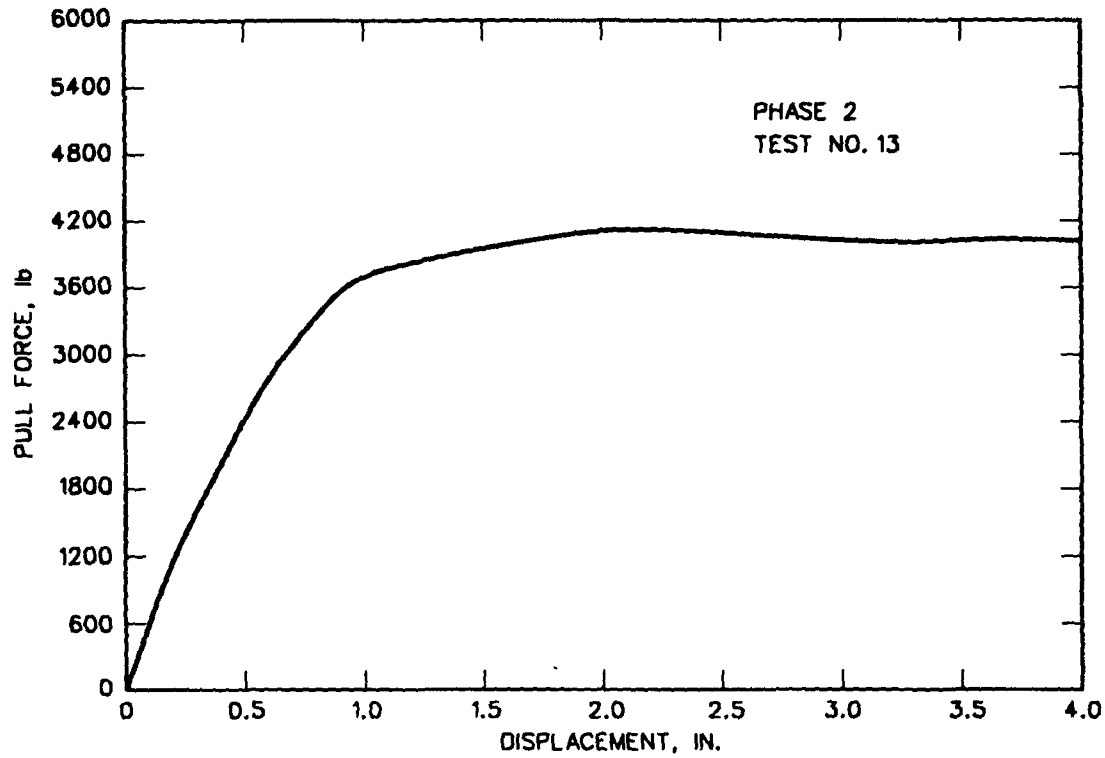
PHASE 2
TEST NO. 10
PULL FORCE, lb
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.

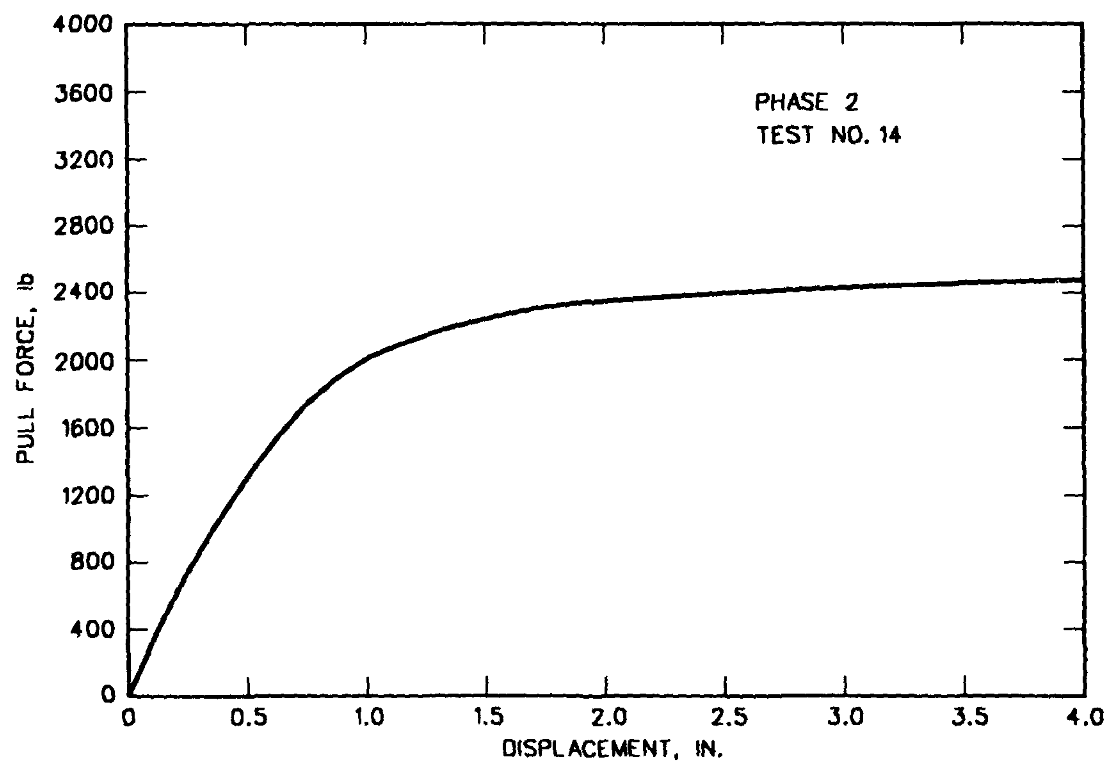


6000
5400
4800
4200
3600
3000
2400
1800
1200
600
0
PULL FORCE, lb
PHASE 2
TEST NO. 11
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.



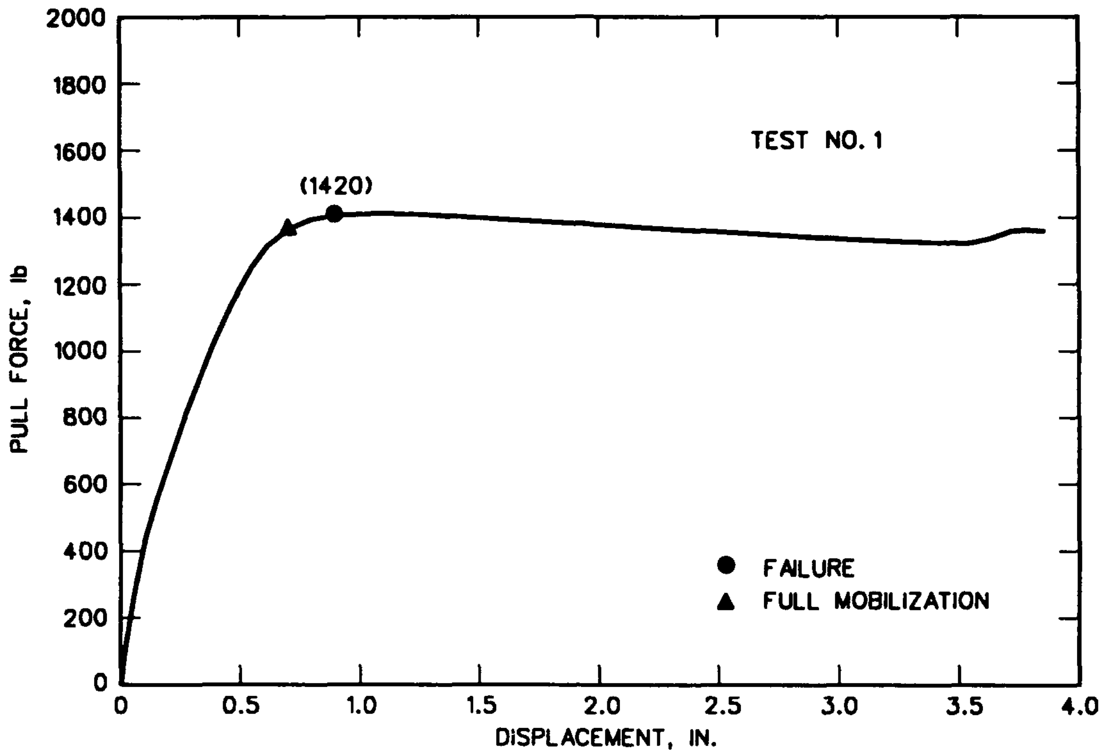
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
PULL FORCE, lb
PHASE 2
TEST NO. 12
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.

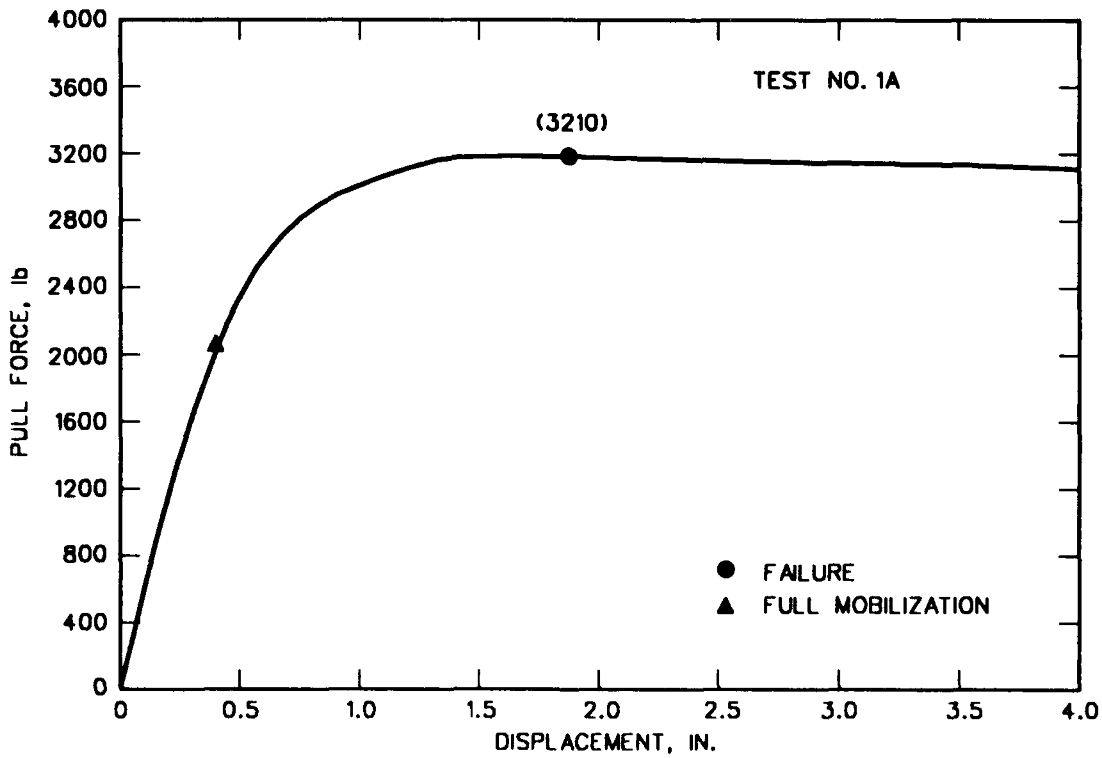

PHASE 2
TEST NO. 13
PULL FORCE, lb
DISPLACEMENT, IN.
6000
5400
4800
4200
3600
3000
2400
1800
1200
600
0
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

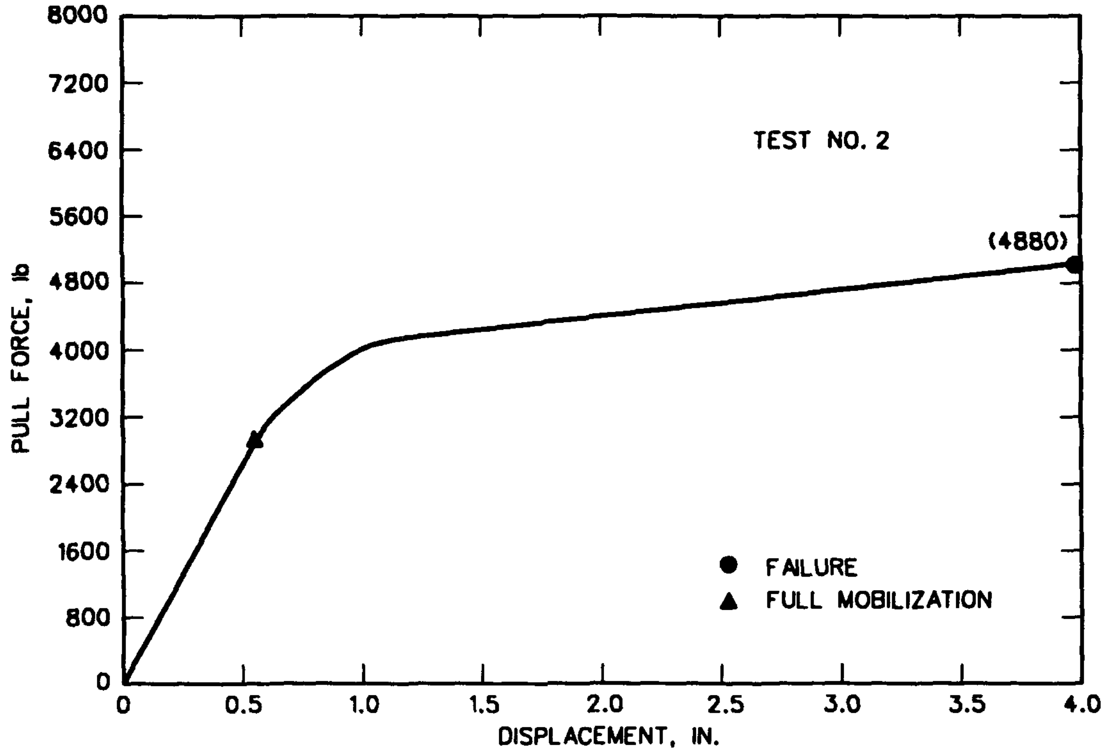

PHASE 2
TEST NO. 14
PULL FORCE, lb
DISPLACEMENT, IN.
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

APPENDIX C:

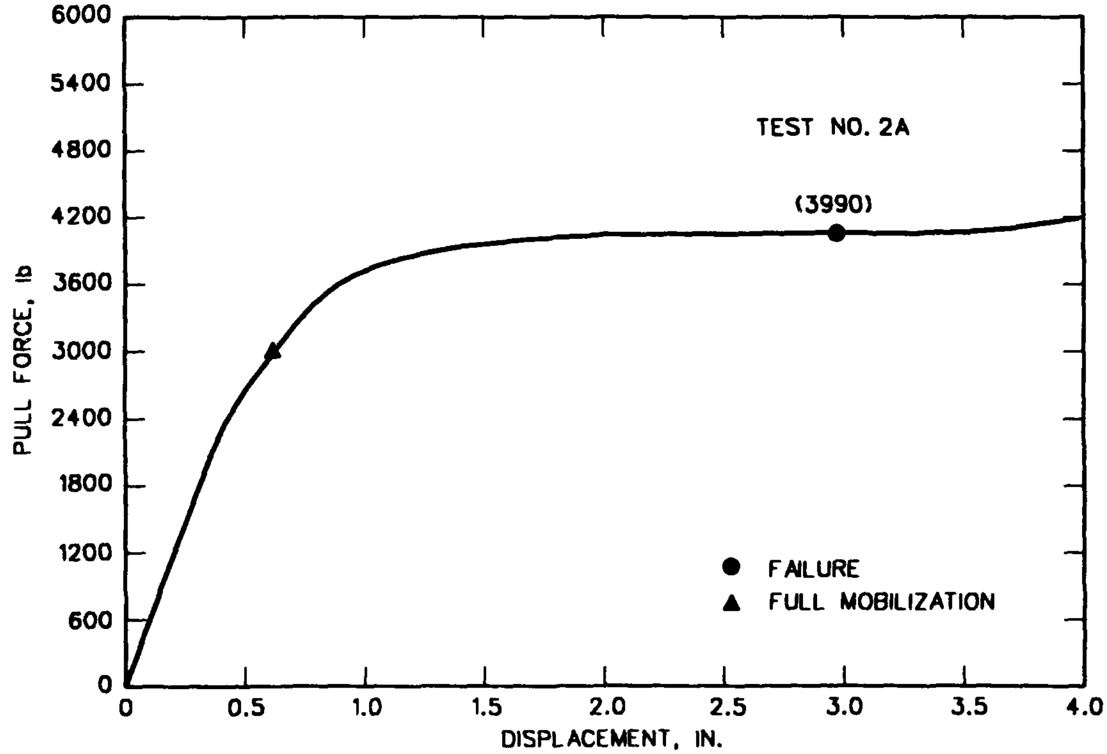
FORCE-DISPLACEMENT RELATIONSHIPS FOR PHASE 3 TESTS


TEST NO. 1
(1420)
FAILURE
FULL MOBILIZATION
PULL FORCE, lb
DISPLACEMENT, IN.
0
200
400
600
800
1000
1200
1400
1600
1800
2000
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

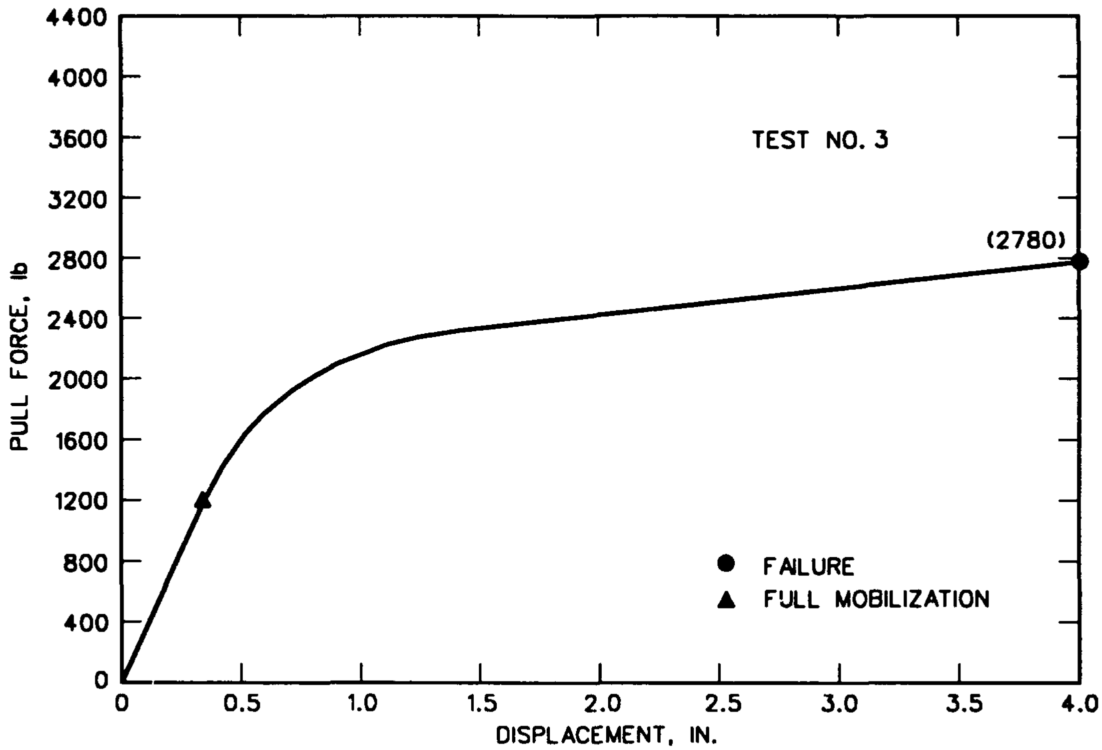

TEST NO. 1A
(3210)
FAILURE
FULL MOBILIZATION
PULL FORCE, lb
DISPLACEMENT, IN.
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0



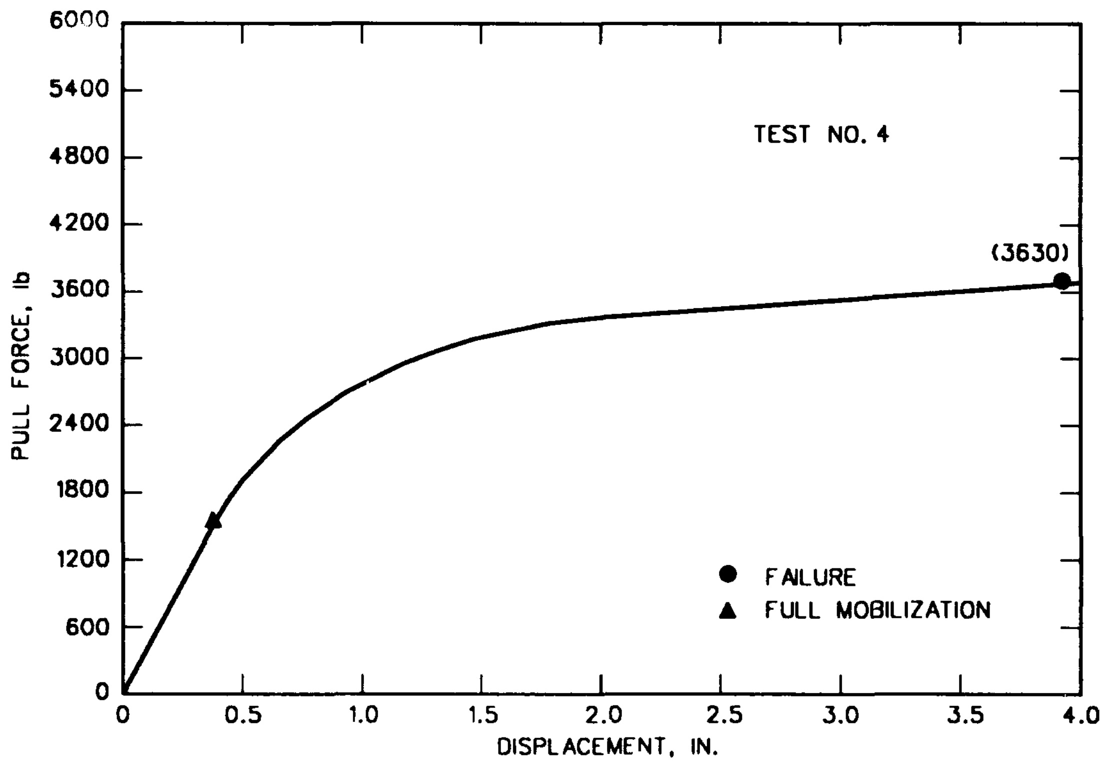
TEST NO. 2
(4880)
FAILURE
FULL MOBILIZATION
PULL FORCE, lb
DISPLACEMENT, IN.



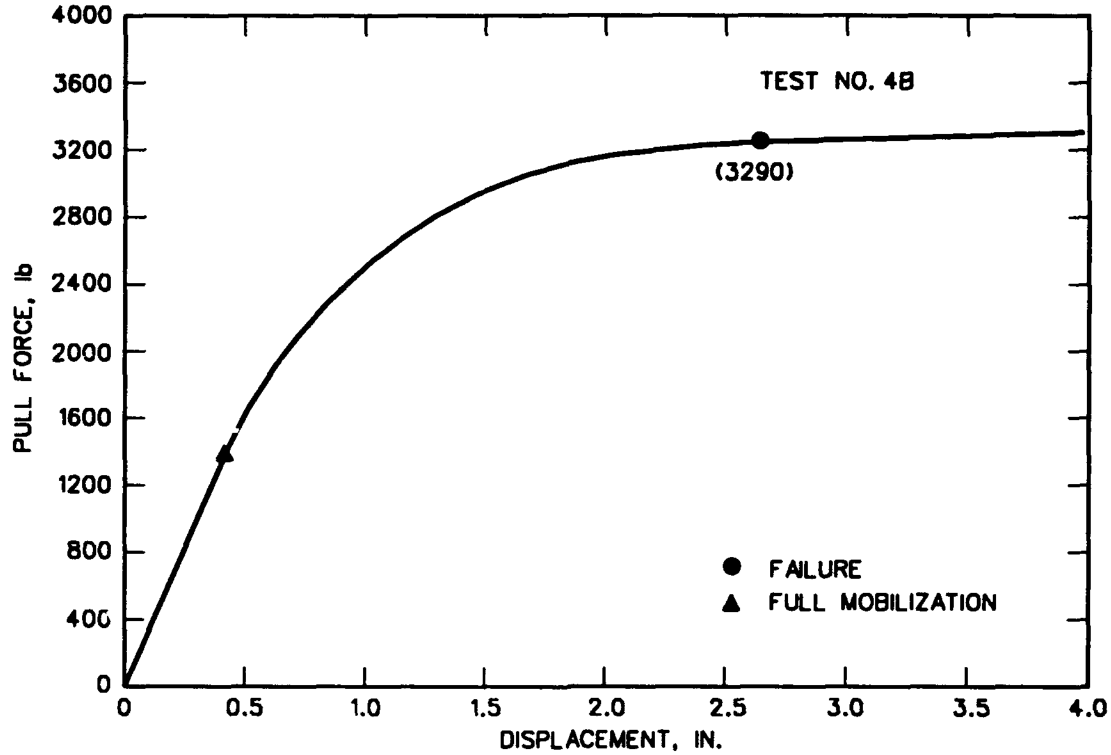
TEST NO. 2A
(3990)
FAILURE
FULL MOBILIZATION
PULL FORCE, lb
DISPLACEMENT, IN.

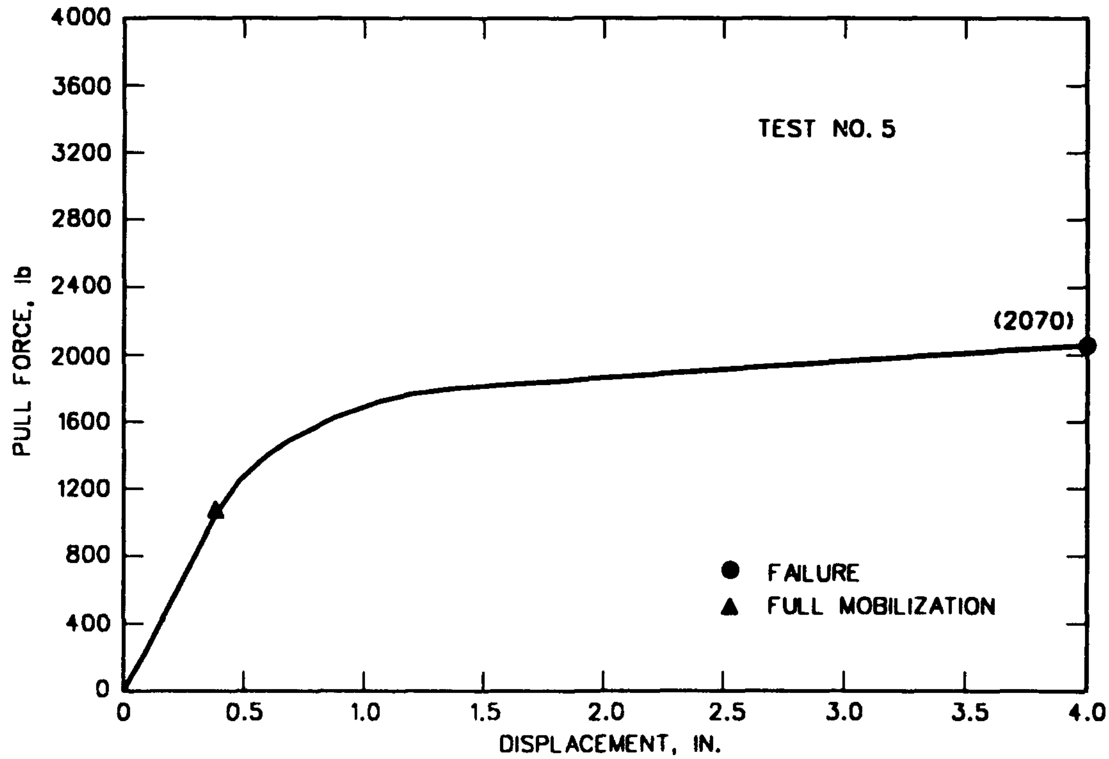


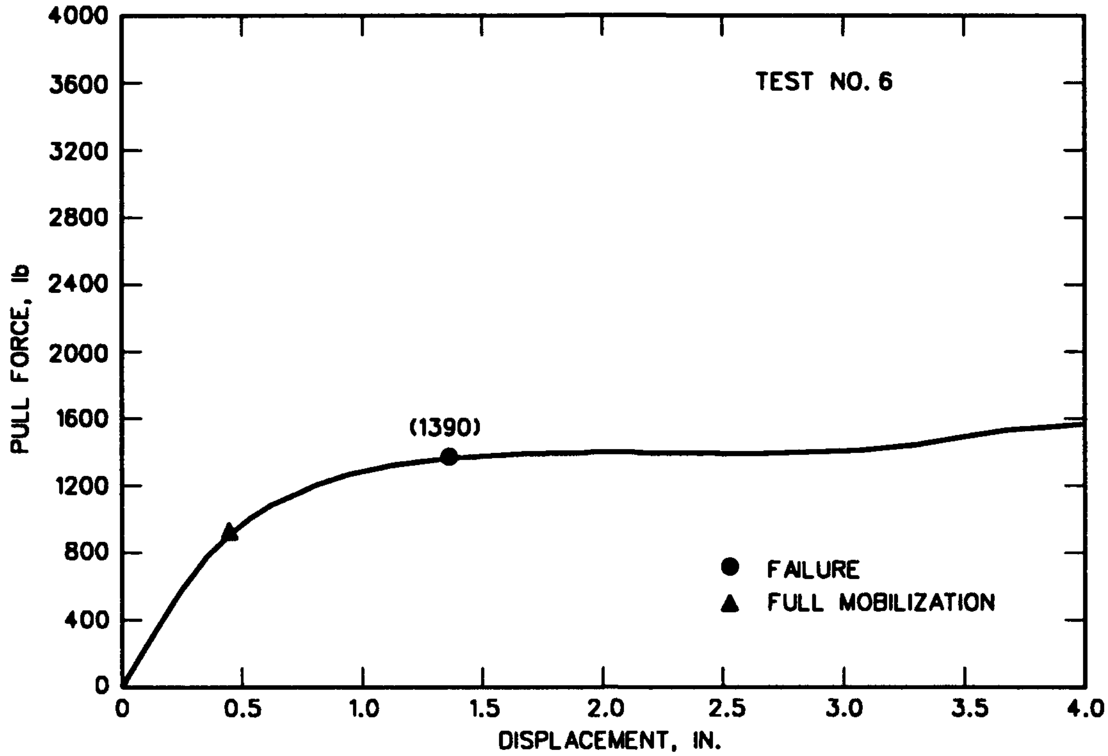
TEST NO. 3
(2780)
FAILURE
FULL MOBILIZATION
PULL FORCE, lb
DISPLACEMENT, IN.
4400
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

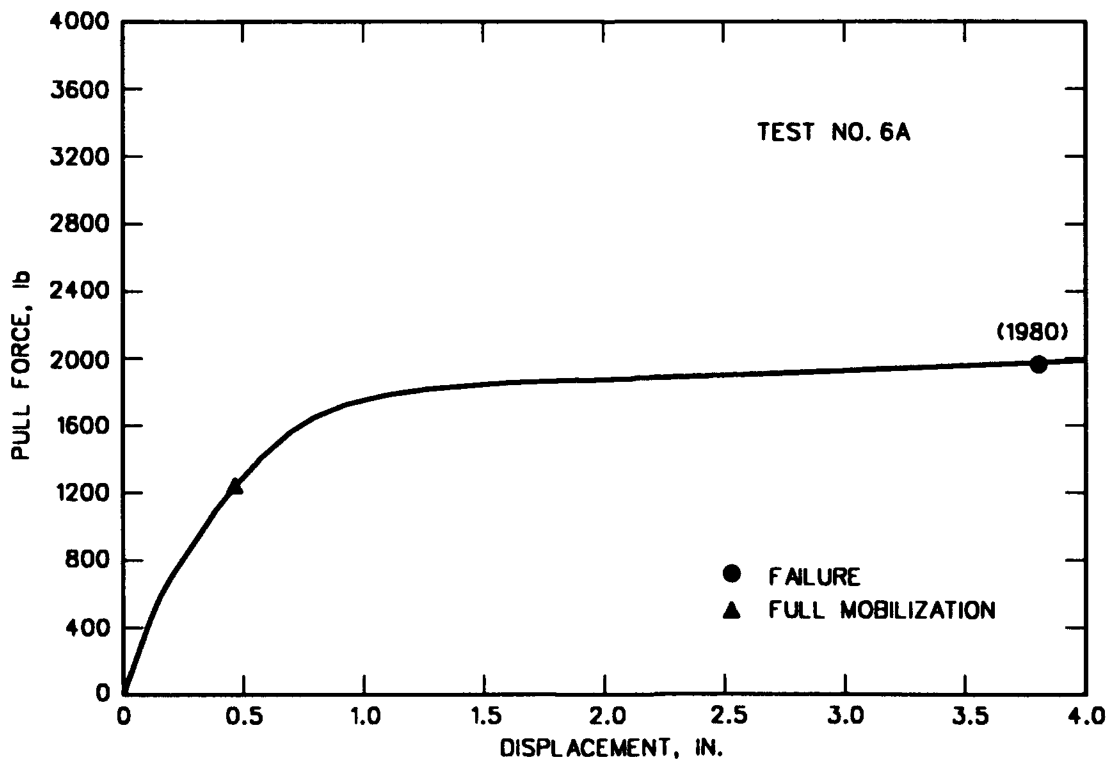


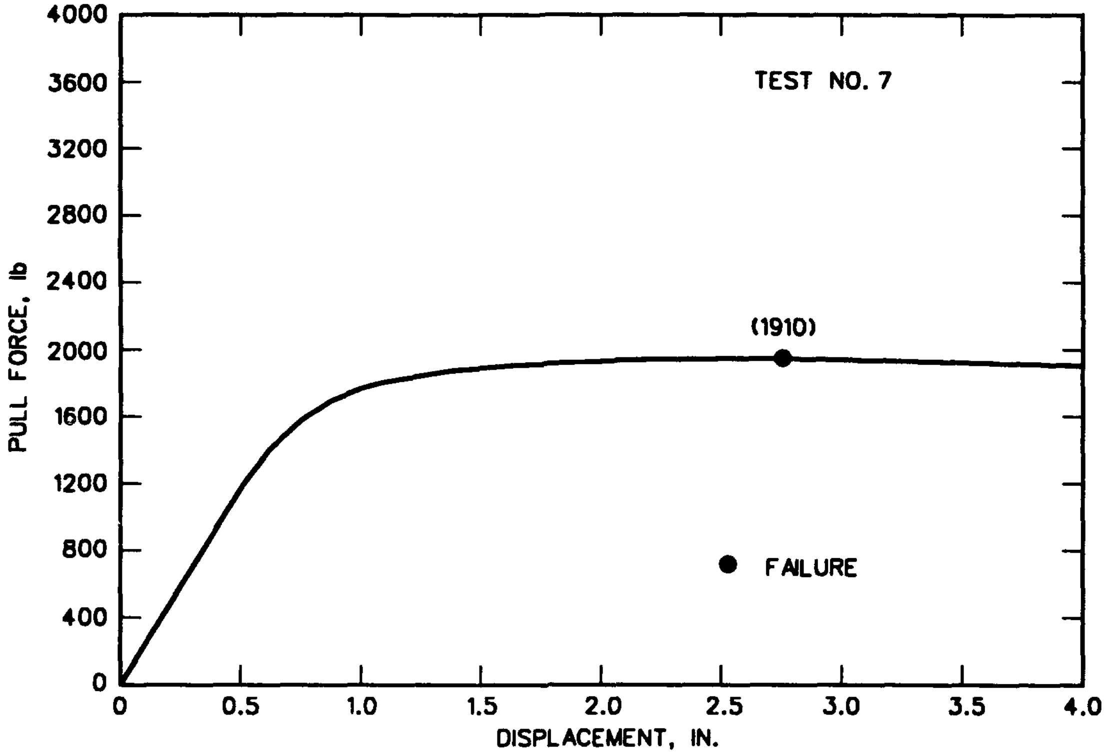
TEST NO. 4
(3630)
FAILURE
FULL MOBILIZATION
PULL FORCE, lb
DISPLACEMENT, IN.
6000
5400
4800
4200
3600
3000
2400
1800
1200
600
0
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0


TEST NO. 4B
(3290)
FAILURE
FULL MOBILIZATION
PULL FORCE, lb
DISPLACEMENT, IN.
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

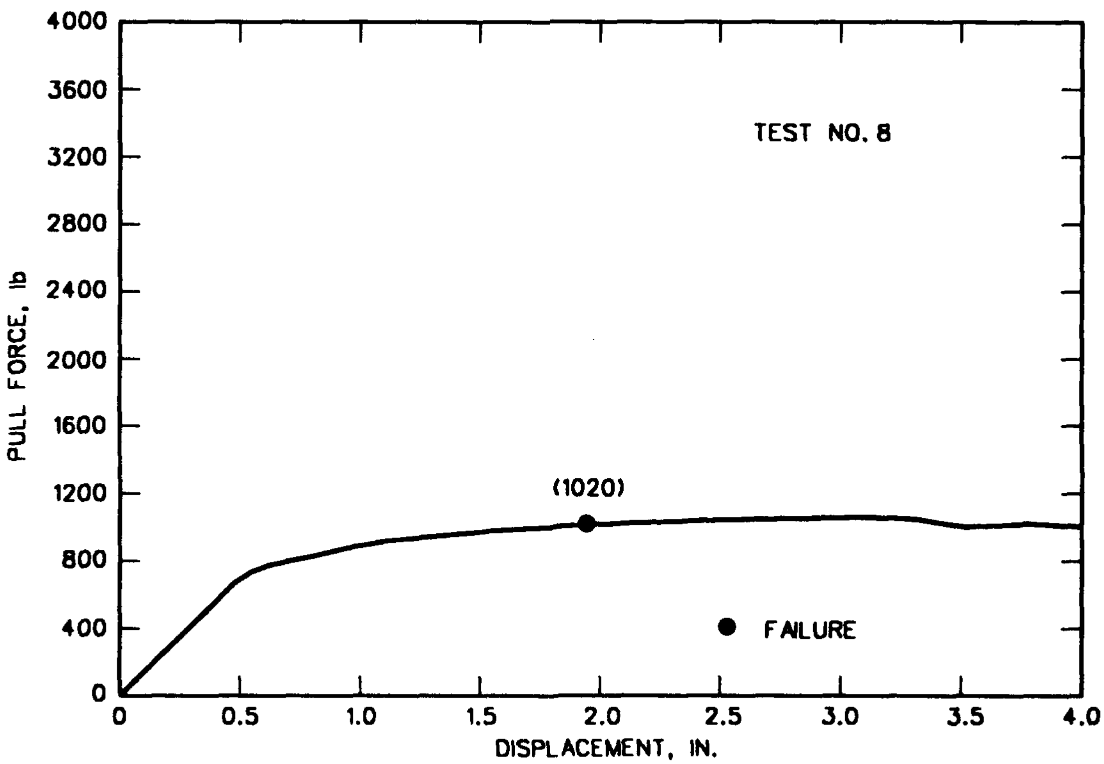

TEST NO. 5
(2070)
FAILURE
FULL MOBILIZATION
PULL FORCE, lb
DISPLACEMENT, IN.
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0


4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
PULL FORCE, lb
TEST NO. 6
(1390)
FAILURE
FULL MOBILIZATION
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.

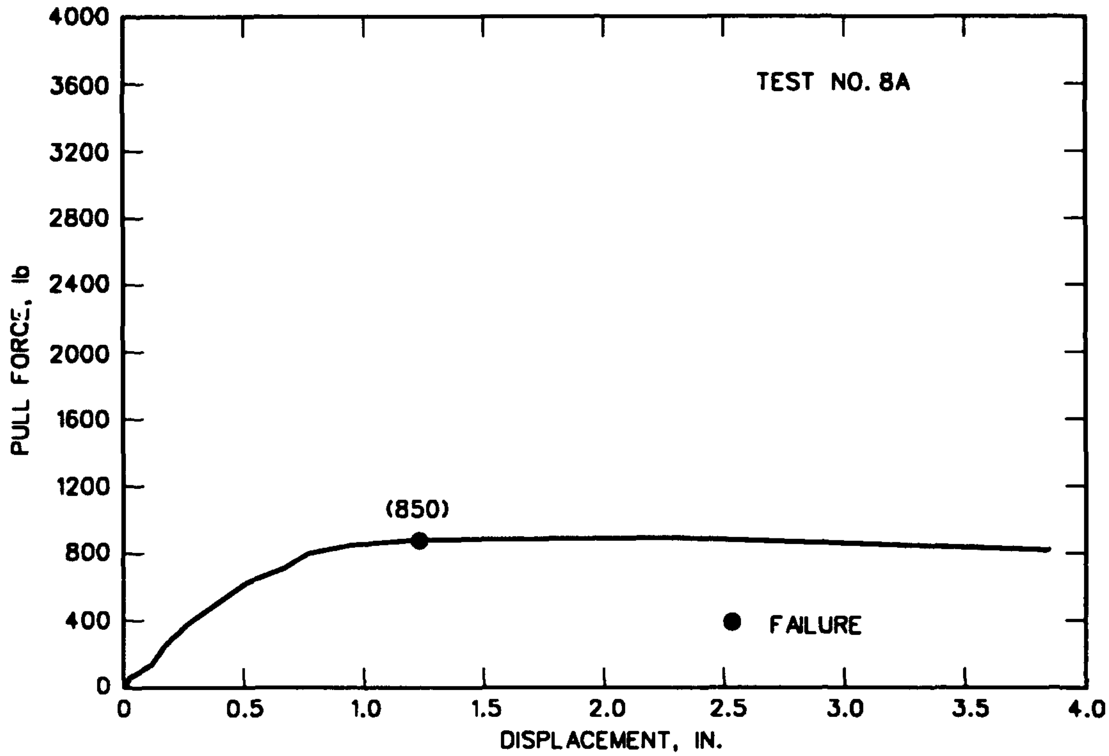

4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
PULL FORCE, lb
TEST NO. 6A
(1980)
FAILURE
FULL MOBILIZATION
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.

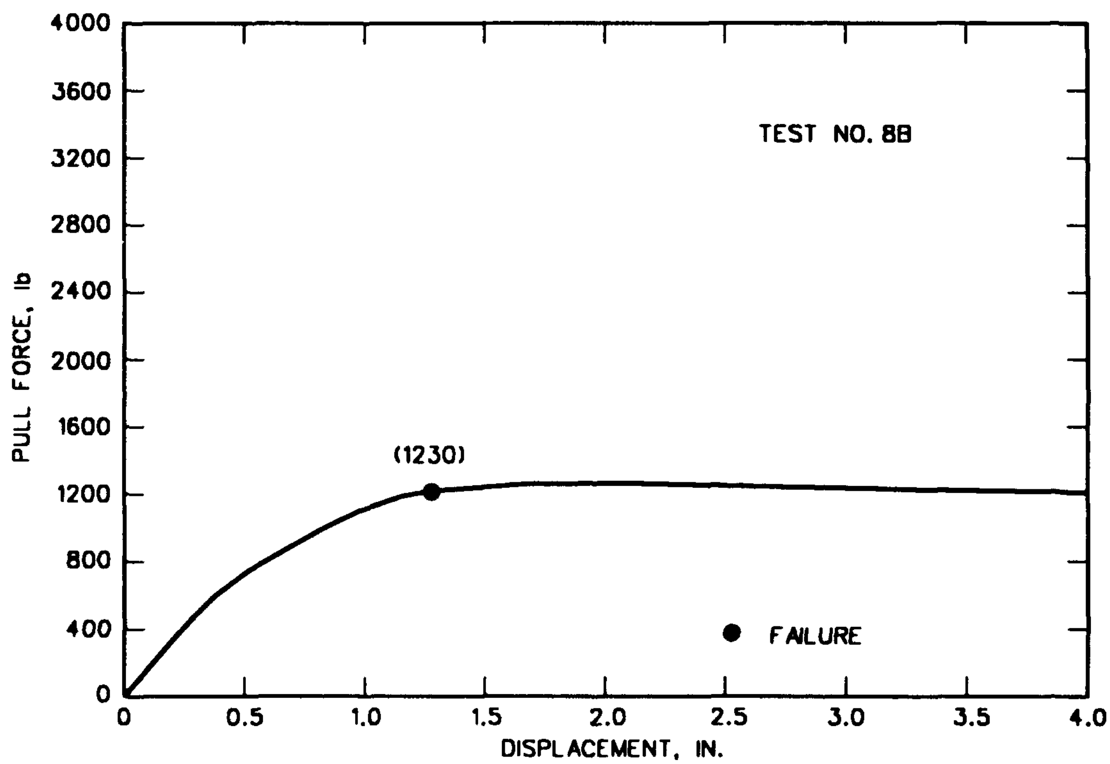


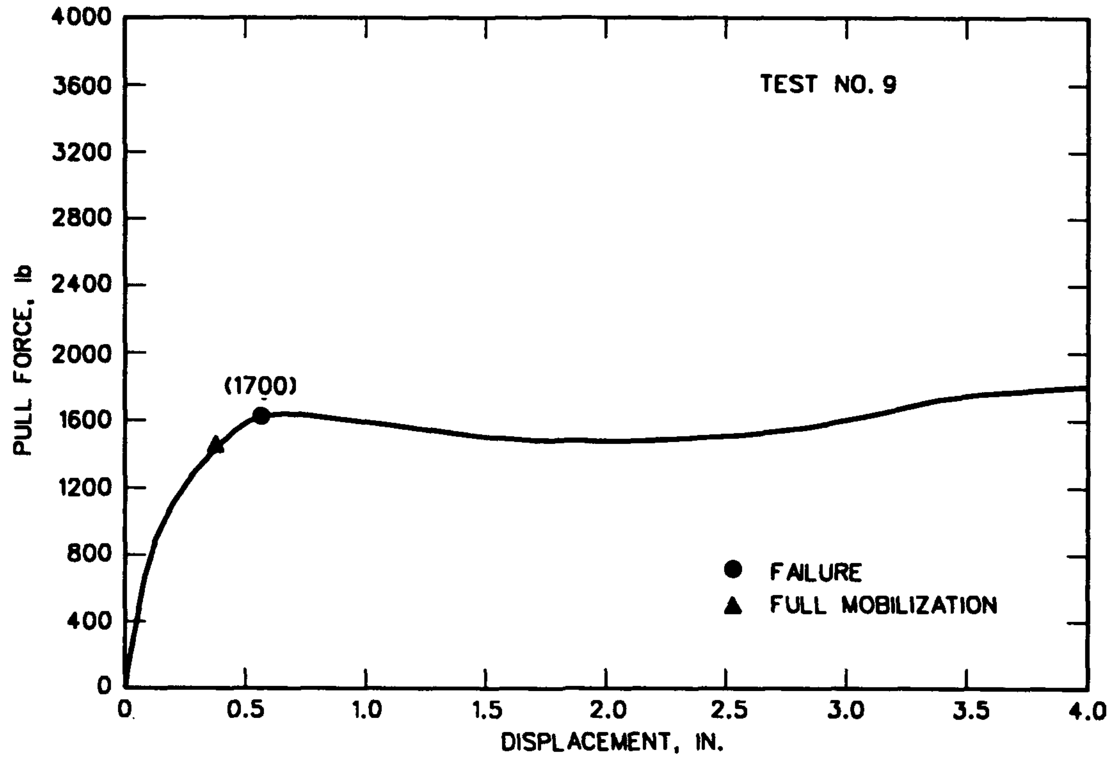
TEST NO. 7
(1910)
FAILURE
PULL FORCE, lb
DISPLACEMENT, IN.
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

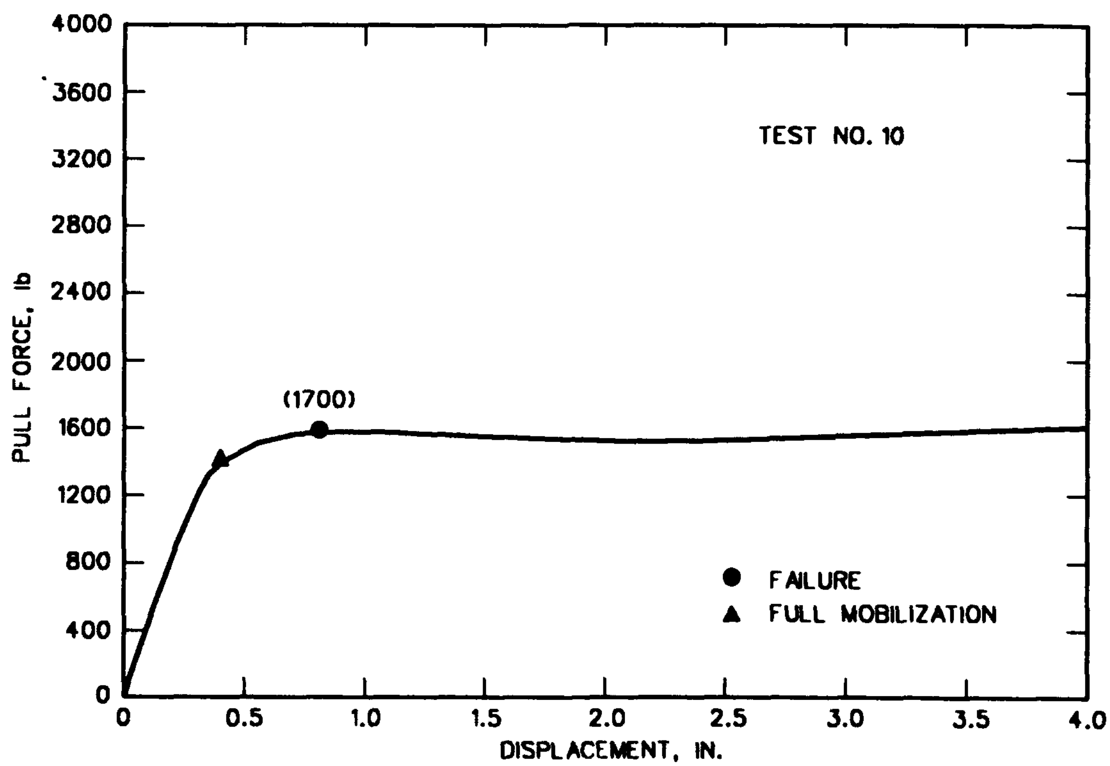


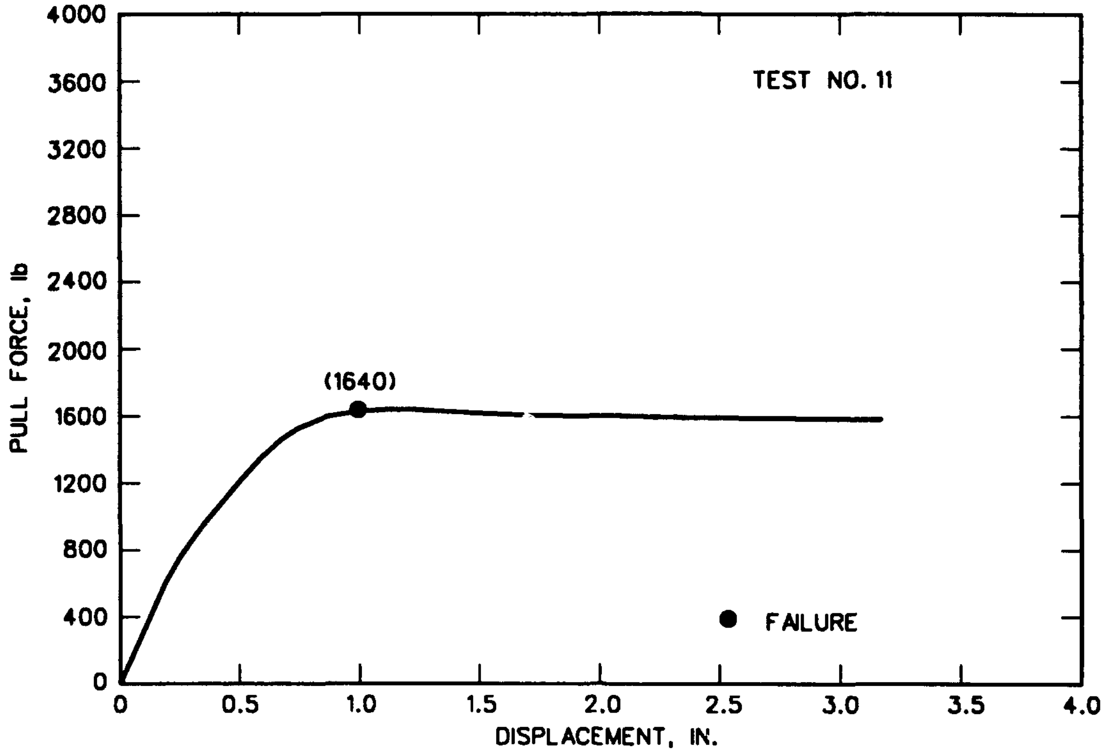
TEST NO. 8
(1020)
FAILURE
PULL FORCE, lb
DISPLACEMENT, IN.
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0


TEST NO. 8A
PULL FORCE, lb
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000
(850)
FAILURE
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.

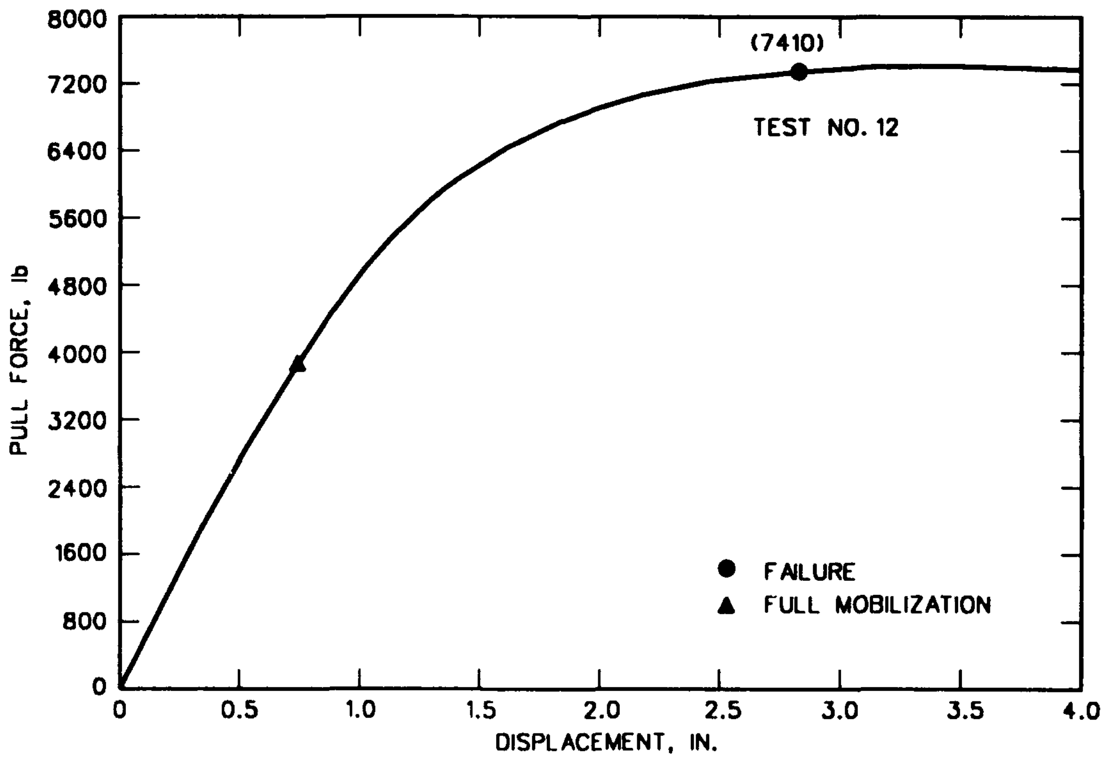

TEST NO. 8B
PULL FORCE, lb
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000
(1230)
FAILURE
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.


TEST NO. 9
(1700)
FAILURE
FULL MOBILIZATION
PULL FORCE, lb
DISPLACEMENT, IN.
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

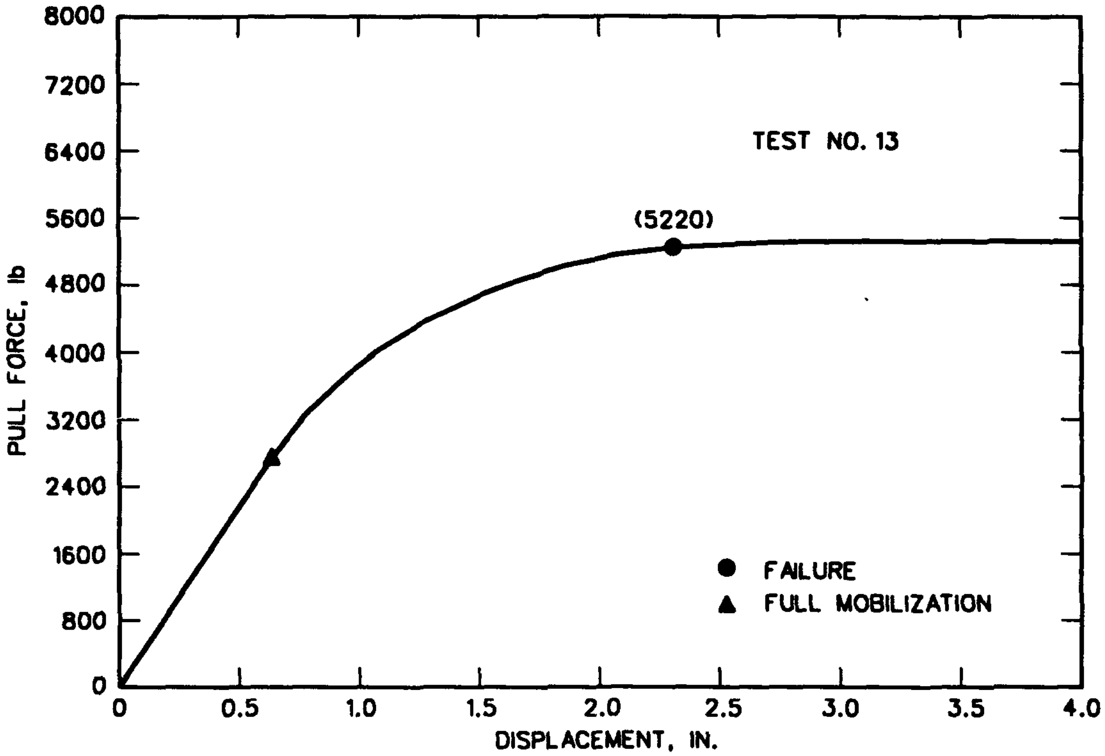

TEST NO. 10
(1700)
FAILURE
FULL MOBILIZATION
PULL FORCE, lb
DISPLACEMENT, IN.
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0



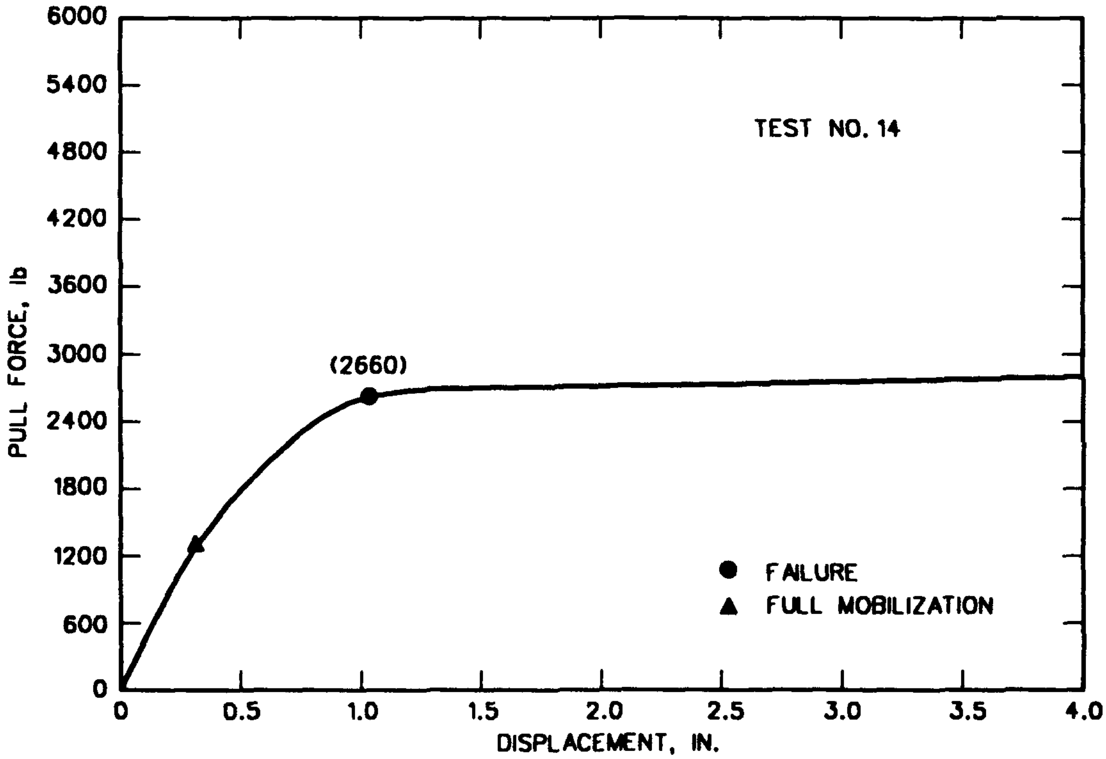
TEST NO. 11
(1640)
FAILURE
PULL FORCE, lb
DISPLACEMENT, IN.



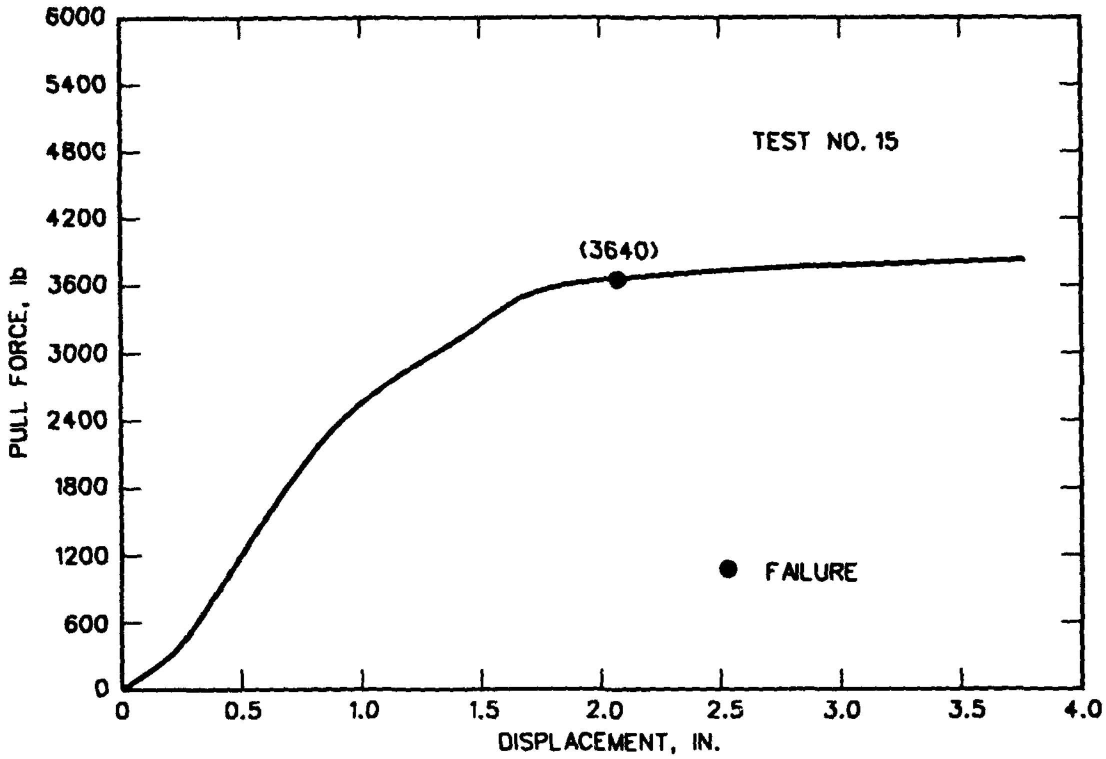
(7410)
TEST NO. 12
FAILURE
FULL MOBILIZATION
PULL FORCE, lb
DISPLACEMENT, IN.

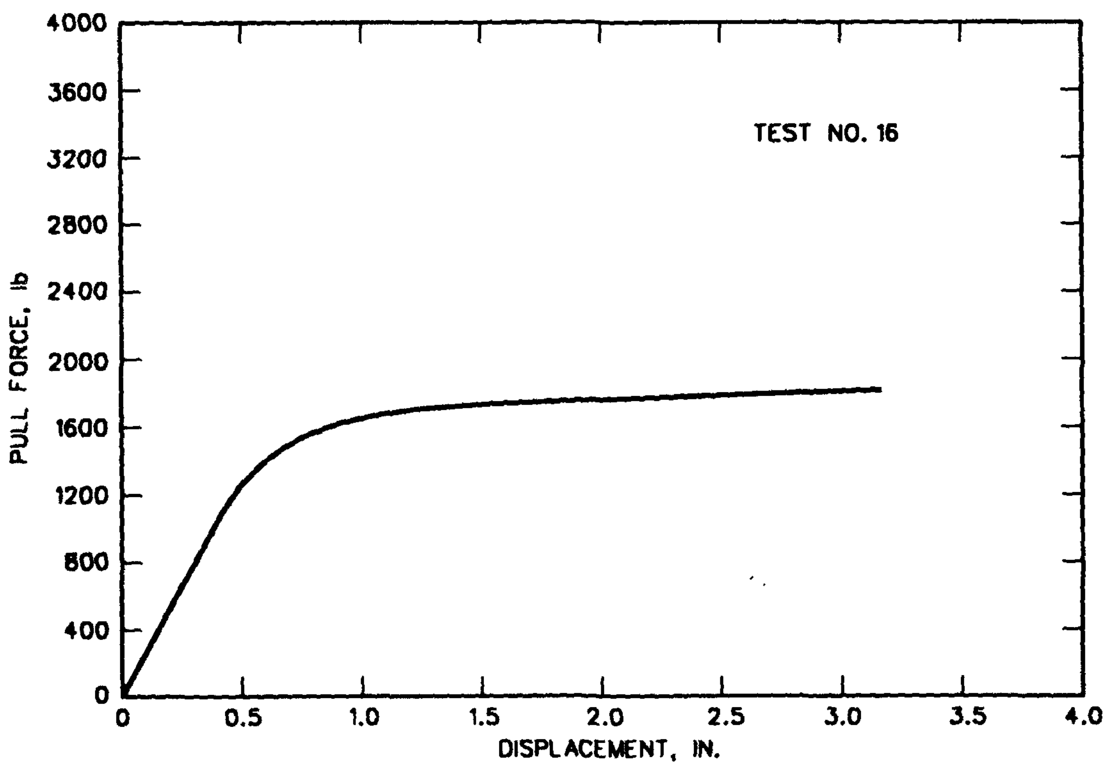


TEST NO. 13
(5220)
FAILURE
FULL MOBILIZATION
PULL FORCE, lb
DISPLACEMENT, IN.
0
800
1600
2400
3200
4000
4800
5600
6400
7200
8000
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0



TEST NO. 14
(2660)
FAILURE
FULL MOBILIZATION
PULL FORCE, lb
DISPLACEMENT, IN.
0
600
1200
1800
2400
3000
3600
4200
4800
5400
6000
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0


6000
5400
4800
4200
3600
3000
2400
1800
1200
600
0
PULL FORCE, lb
TEST NO. 15
(3640)
FAILURE
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.


4000
3600
3200
2800
2400
2000
1600
1200
800
400
0
PULL FORCE, lb
TEST NO. 16
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
DISPLACEMENT, IN.

APPENDIX D:

SUMMARY OF SOIL-REINFORCEMENT INTERFACE STRENGTH

Table D1. Summary of Soil-Reinforcement Interface Strength

Reference	Test Method	Reinforcement	Soil	Normal Stress psi	Friction Angle ϕ	Friction Angle δ	$\tan\delta / \tan\phi$
Formazin & Batereau (1985)*	Shear (water saturated)	Synthetic Bonded Fiber Fabric	Sand				
			d_{50} = 0.20-2.0	--	--	35°	--
			d_{50} = 2.0-5.0	--	--	30	--
			d_{50} = 5.0-10.0	--	--	25	--
		Woven & Knitted Fabric	Sand				
			d_{50} = 0.2-2.0	--	--	30°	--
			d_{50} = 2.0-5.0	--	--	25	--
			d_{50} = 5.0-10.0	--	--	22	--
Ingold (1983)	Pullout	Netlon 1168 (g)	Sand	30-450	35°	37-60°	1.1-2.5
		Netlon FBM 5 (g)		30-4200	35	41-80	1.2-8.1
		Steel Grid BRC-B503		30-2600	35	54-82	2.0-10.2
Ingold (1982)	Pullout	Geogrid	Sand	210-4200	34°	40-71°	1.2-4.3
	Shear	Geogrid		500-4200	34	35-65	1.0-3.2
	Pullout	Geotextile (nw)**		210-1700	34	20-38	0.5-1.2
	Shear	Geotextile (nw)		500-4200	34	25-32	0.7-0.9
Holtz (1977)	Pullout	Teknisk vav No. 600 Woven Polyester	Tullinge Sand	212	45°	38-45°	0.8-1.0
				425	45	42-46	0.9-1.0
				637	45	39-46	0.8-1.0
			G-12 Sand	425	35°	35°	1.0
				637	35	34	1.0
				850	35	32	0.9
				1275	35	28	0.8

(Continued)

* See References at the end of main text.
** NW = nonwoven; W = woven.

(Sheet 1 of 6)

Table D1 (Continued)

Reference	Test Method	Reinforcement	Soil	Normal Stress psi	Friction Angle ϕ	Friction Angle δ	$\frac{\tan\delta}{\tan\phi}$
Jarrett & Bathurst (1987)	Pullout	Geogrids	Peat below and Sand or Gravel above	40-190	--	≈35°	--
Myles (1982)	Shear	Heat Bonded	Sand	1050-3850	38°	34°	0.9
		Needle Punched			38	35.5	0.9
		Heavy-Wght Woven			38	38	1.0
		Light-Wght Woven			38	32	0.8
		Polished Steel			38	28	0.7
		Heat Bonded (nw)*	Fine Sand 0.1-0.2 mm	--	43°	40°	0.9
		Needle Punched(nw)		--	43	42	1.0
		Monofilament (w)		--	43	42	1.0
		Heat Bonded (nw)	Fuel Ash	--	44°	36°	0.8
		Needle Punched(nw)		--	44	38	0.8
		Monofilament (w)		--	44	40	0.9
		Heat Bonded (nw)	Coarse Sand 0.2-1.0 mm	--	45°	39°	0.8
		Needle Punched(nw)		--	45	44	1.0
		Monofilament (w)		--	45	40	0.8
Miyamori, Iwai, and Makiuchi (1986) (after Williams & Houlihan, 1987)	Shear	Needle Punched Resin Bonded (nw)	River Sand	--	44°	37°	0.8
		Needle Punched(nw)		--	44	41	0.9
		Needle Punched Resin Bonded (nw)	River Sand	--	35°	35°	1.0
		Needle Punched(nw)		--	35	32	0.9
		Needle Punched Resin Bonded (nw)	River Sand	--	32°	33°	1.0
		Needle Punched(nw)		--	32	34	1.1

(Continued)

* NW = nonwoven; W = woven.

(Sheet 2 of 6)

Table D1 (Continued)

Reference	Test Method	Reinforcement	Soil	Normal Stress psi	Friction Angle ϕ	Friction Angle δ	$\frac{\tan\delta}{\tan\phi}$
Paulson & Langston (1987)	Shear	Stitch Bonded composite	Ottawa Sand 20/30	--	36°	23-26°	0.6-0.7
		Woven/Nonwoven composite	Ottawa Sand 20/30	—	36°	26°	0.7
Rowe, Ho, & Fisher (1985)	Pullout	Mirafi P600X(w)*	Loose Silty Sand	Varies between 225 and 1700	32°	32°	1.0
		Mirafi P500(w)			32	33	1.0
		Geolon 1250(w)			32	35	1.1
		Permaliner M1195(w)			32	32	1.0
		Terrafix 370RS(nw)			32	37	1.2
		Terrafix 1200R(nw)			32	36	1.2
		Tensar SR2 (g)			32	18**	0.5
Rowe, Ho. & Fisher (1985)	Shear	Mirafi P600X(w)	Loose Silty Sand	Varies between 225 and 1700	32°	32°	1.0
		Mirafi P500(w)			32	33	1.0
		Geolon 1250(w)			32	35	1.1
		Permaliner M1195(w)			32	32	1.0
		Terrafix 370RS(mw)			32	36	1.2
		Terrafix 1200R(nw)			32	36	1.2
		Tensar SR2(g)			32	30**	0.9
Saxena & Budiman (1985)	Shear	Celanese 600X(w)	Sandy Clay	1440	12.5*	14*	1.1
				2880	12.5	15	1.2
				4320	12.5	15	1.2
			Ballast	1440	40°	41°	1.0
				2880	40	38	0.9
				4320	40	32	0.7
		Monsanto C34(nw)	Sandy Clay	1440	12.5°	24°	2.0
				2880	12.5	22	1.8
				4320	12.5	18	1.5

(Continued)

* W - woven; NW - nonwoven.
** Machine direction.

(Sheet 3 of 6)

Table D1 (Continued)

Reference	Test Method	Reinforcement	Soil	Normal Stress psi	Friction Angle ϕ	Friction Angle δ	$\frac{\tan\delta}{\tan\phi}$
Saxena & Budiman (1985)	Shear	Monsanto C34 (nw)*	Ballast	1440	40°	37°	0.9
				2880	40	31	0.7
				4320	40	25	0.6
Shen et al. (1979)	Shear	Polished Stl Strp	Fine Sand	440-2300	31°	21°	0.6
	Pullout	Polished Stl Strp		144-2160	31	12-16.5	0.4-0.5
	Pullout	Undulated Stl Strp		144-1050	31	22	0.7
Lafleur, Sall, and Ducharme (1987)	Shear	MP-500 (w)	Lateritic Gravel (Leona Niang)	1040-3100	49°	44°	0.8
		MP-500 (w)	Lateritic Gravel (Dougar)	1040-3100	49°	35°	0.6
		MP-500 (w)	Plastic Clay	1040-3100	29-35°	15-22°	0.5-0.6
		PF-700 (nw)	LL=56	1040-3100	29-35	29-37	1.0-1.1
		TX-7643 (nw)	PI=30	1040-3100	29-35	28-39	1.0-1.2
Brand & Duffy (1987)	Pullout	Geogrids	Bentonite Clay LL=515, PI=470, W=10%	250-1000	--	30-40°	--
Leshchinsky & Field (1987)	Shear	Mirafi 140N (nw)	Ottawa Sand	1080-4300	37°	24-33°	0.6-0.9
Williams & Houlihan (1987)	Shear	HDPE-Smooth-60 mil	Ottawa Sand 20/30	100,250, 500	38°	19°	0.4**
		PVC-smooth-30 mil			38	26	0.6**
		Typar 3401 (nw)			38	25	0.6
		Trevira 1155 (nw)			38	28	0.7
		Nicolon 900-M (w)			38	35	0.9
		HDPE-Smooth-60 mil	Concrete Sand	100,250, 500	36°	27°	0.7**
		PVC-smooth-30 mil			36	33	0.9**

(Continued)

* NW = nonwoven; W = woven.
** Not normally used as reinforcement.

(Sheet 4 of 6)

Table D1 (Continued)

Reference	Test Method	Reinforcement	Soil	Normal Stress psi	Friction Angle ϕ	Friction Angle δ	$\frac{\tan\delta}{\tan\phi}$
Williams & Houlihan (1987)	Shear	Typar 3401 (nw)*	Concrete Sand	100,250, 500	36	27	0.7
		Trevira 1155 (nw)			36	34	0.9
		Nicolon 900-M (w)			36	35	1.0
		HDPE-Smooth-60 mil	Ottawa Sand 20/30 w/5% Bentonite LL=36, PI=20	100,250, 500	36°	14°	0.3**
		PVC-smooth-30 mil			36	19	0.5**
		Typar 3401 (nw)			36	22	0.6
		Trevira 1155 (nw)			36	27	0.7
		Nicolon 900-M (w)			36	31	0.8
		HDPE-Smooth-60 mil	Ottawa Sand 20/30 w/10% Bentonite LL=47, PI=27	100,250, 500	36°	17°	0.4**
		PVC-smooth-30 mil			36	19	0.5**
		Typar 3401 (nw)			36	22	0.6
		Trevira 1155 (nw)			36	27	0.7
		Nicolon 900-M (w)			36	34	0.9
		HDPE-Smooth-60 mil	Saprollite non-plastic	100,250, 500	36°	21°	0.5**
		PVC-smooth-30 mil			36	28	0.7**
		Typar 3401 (nw)			36	29	0.8
		Trevira 1155(nw)			36	30	0.8
		Nicolon 900-M (w)			36	31	0.8
		HDPE-Smooth-60 mil	Gulf Coast Clay LL=42, PI=14	100,250, 500	20°	25°	1.3**
		PVC-smooth-30 mil			20	23	1.2**
		Typar 3401 (nw)			20	39	2.2
		Trevira 1155 (nw)			20	45	2.7
		Nicolon 900-M (w)			20	43	2.6

(Continued)

* NW = nonwoven; W = woven.
** Not normally used as reinforcement.

(Sheet 5 of 6)

Table D1 (Concluded)

Reference	Test Method	Reinforcement	Soil	Normal Stress psi	Friction Angle ϕ	Friction Angle δ	$\frac{\tan\delta}{\tan\phi}$
		HDPE-Smooth-60 mil	Glacial Till	100,250,	36°	22°	0.6**
		PVC-smooth-30 mil	non-plastic	500	36	25	0.6**
		Typar 3401 (nw)	60% Fines		36	33	0.9
		Trevira 1155 (nw)			36	37	1.0
		Nicolon 900-M (w)			36	35	1.0
Haliburton,	Shear	Nicolon 66475 (w)	Mobile Sand	--	30°	32°	1.1
Anglin, &		Polyfilter-X (w)	Loose	--	30	32	1.1
Lawnmaster		Nicolon 66186 (w)		--	30	29	1.0
(1978)		Advance Type 2(w)		--	30	31	1.0
		Bay Mills 196-380-000 (w)		--	30	33	1.1
		Nicolon 66475 (w)	Mobile Sand	--	50°	41°	0.7
		Polyfilter-X (w)	Dense	--	50	40	0.7
		Nicolon 66186 (w)		--	50	46	0.9
		Advance Type 2(w)		--	50	37	0.6
		Bay Mills 196-380-000 (w)		--	50	45	0.8

* W = woven, NW = nonwoven.

(Sheet 6 of 6)

Waterways Experiment Station Cataloging-in-Publication Data

Gilbert, Paul A.
Laboratory measurement of pullout resistance of geotextiles against cohesive soils / by Paul A. Gilbert, Jessie C. Oldham, L. Rodgers Coffing, Jr. ; prepared for U.S. Army Engineer District, New Orleans.
117 p. : ill. ; 28 cm. — (Technical report ; GL-92-6)
Includes bibliographic references.
1. Geotextiles — Testing. 2. Shear strength of soils — Testing. 3. Soil dynamics. 4. Soil mechanics. I. Title. II. Oldham, Jessie C. III. Coffing, L. Rodgers. IV. United States. Army. Corps of Engineers. New Orleans District. V. U.S. Army Engineer Waterways Experiment Station. VI. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; GL-92-6.
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