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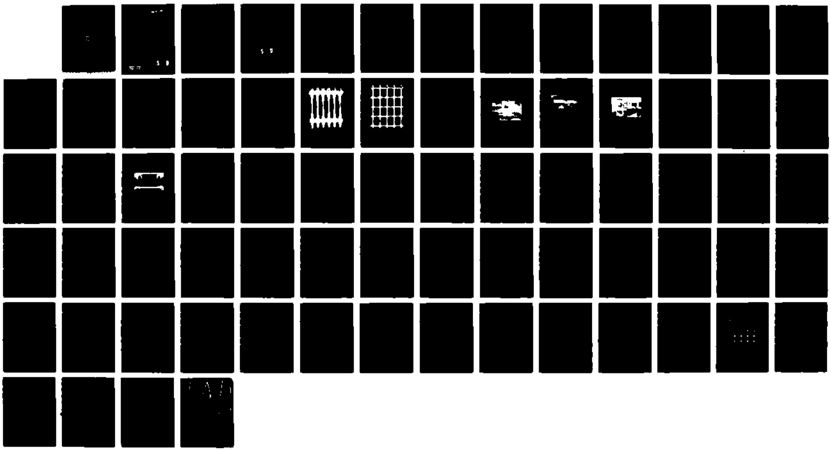

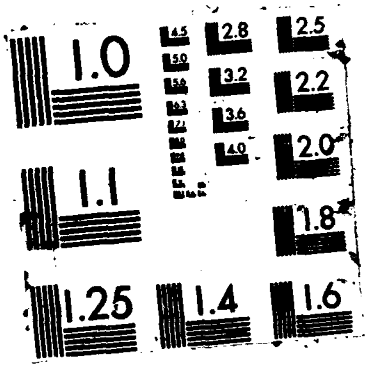
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EVALUATION OF PULLOUT RESISTANCE BETWEEN TENSAR GEOGRIDS AND CONCRETE SAND

CE 551

Submitted in partial fulfillment

of the requirements for the degree of

Master of Science in Civil Engineering

by

Mark S. Buncher

Advisor: Dr. John B. Carney, Jr.

University of New Mexico

November 1986

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Abstract

Tensar geogrids are a relatively new type of material used for soil reinforcement. Their popularity and success in slope stabilization can be attributed to the inherent properties of the material, including the capacity to interlock with soil and aggregate. This interlocking capability, also known as pullout resistance, was measured and analyzed during this research project for the Tensar geogrids SS-1 and SR-2. This report will cover the results of the SS-1 material while Yuen Zehong's report will handle the SR-2 material. Both of us assisted each other in performing the tests. The grids were pulled through concrete sand with varying vertical pressures. A test chamber for determining the pullout resistance, designed, manufactured, and operated at the University of New Mexico, was used.

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Acknowlegements

I wish to thank Dr. John B. Carney not only for his help during this project, but all his help during my entire master's program. I also wish to thank Mark Wittrock for his help while installing the strain gauges and Yuen Zehong for his assistance. Finally, I am grateful to my wife Cathy and children Rachel and Stanley for their support and patience.

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Definition of Terms

ft = feet

Fp = pullout force per unit area of mat embedded in soil

in = inch

lbs = pounds

pcf = pounds per cubic foot

psf = pounds per square foot

Pv = vertical pressure

Introduction to Soil Reinforcement

Soil reinforcement is one of the most significant advances within recent years in the field of geotechnical engineering. Much study and research has been performed or is in progress to better understand the mechanics of reinforced earth. Simply stated, the inclusion of a tension-resistant material within the soil, which has little or no tensile strength, is an effective means of soil reinforcement. The concept adds an element missing in soil, tensile strength, to it's already inherent compressive, shear, and cohesive strengths.

Even though it has not been until recently that engineers have started studying and understanding the concepts of soil reinforcement, the use of inclusions in soil is not new. Straw, sticks, rocks, etc. have been placed into low strength soil for thousands of years to facilitate the building of roads, dams, embankments, homes, etc. Even the planting of trees, shrubs, and other vegetation to help stabilize an embankment is a type of soil reinforcement in which the roots act as the tension-resistant inclusions. There are many more examples of crude soil reinforcement that have been used over the centuries without ever having been studied or analyzed.

A man named Henri Vidal first started studying reinforced soil in the late 1950's and later founded the Reinforced Earth Company. (reference 1) In the last 10 years, more than 1,500 Reinforced Earth structures representing over 1.2 million square meters of wall facing have been completed. Currently, construction of a new Reinforced Earth structure begins every working day. (reference 2)

Many new ideas and designs have been developed since the first

Reinforced Earth structure, some by the Reinforced Earth Company and some by other independent, state, or federal agencies. Materials used as inclusions in soil range from geogrids, geotextiles, galvanized strips, welded wire mesh, and even waste materials such as used automobile tires. Each of these has their own advantages and disadvantages in soil reinforcement. These materials are continually being studied and altered to reflect the latest research performed in this area. The geogrids, namely Tensar geogrids, will be the focus of this study.

Reinforced Earth Walls vs. Reinforced Slopes

Reinforced soil structures can be classified into two catagories for design purposes. First, there are those with wall facing panels which are built upon to form a retaining wall, also known as reinforced earth walls. These provide a vertical separation in grade, the same purpose as a cantilever retaining wall. The other type of reinforced soil structure is one with no wall facing panels, also known as a reinforced slope. The difference is not only in the presence or absence of a retaining wall, but also in the type of performance data utilized in the design.

For a reinforced slope, the design or slope stability procedure utilizes the properties of the soil. Thus, when the soil is reinforced, those properties must be altered to correctly show the increase in strength of the reinforced soil. Of course, the increase in strength depends on the soil type, reinforcement type, number of reinforcements, direction of reinforcement, etc. Most slope-stability methods in use today can be used for analyzing a reinforced soil structure by altering the parameters of the soil to include the soil-reinforcement interaction. Much research has been performed on the selection of these parameters for design. (references 3 and 4) Even the authors of this research admit the conclusions drawn may be oversimplified, but as a better understanding of reinforced soil is developed, so to will a more sophisticated approach.

The second classification of reinforced soil structures consists of those with wall facing elements. These facing elements are usually precast panels that are erected to form a retaining wall. The facing elements are attached to the reinforcing members or inclusions in the soil. The soil fill is placed behind the facing elements and between the reinforcing members. Figure 1 is a simplified drawing of a reinforced earth retaining wall. Reinforced earth walls and most reinforced slopes are built in lifts, laying the reinforcing material on top of compacted lifts. In the walls, however, the reinforcing elements are connected to the facing elements. The advantages of building a reinforced earth wall versus a conventional cantilever wall are numerous with ease of construction, cost, and aesthetics being a few.

The performance data utilized in a reinforced earth wall is different than in a reinforced slope. Rather than altering the soil strength parameters as is done in a reinforced slope stability analysis, a parameter called the pullout resistance is used. This is the resistance or force required to pull the reinforcing member out of the soil. As will be shown, it depends on a number of factors, such as area of embeddment, vertical pressure, type of reinforcing members and type of soil. It is this pullout resistance that provides the basis for this report.

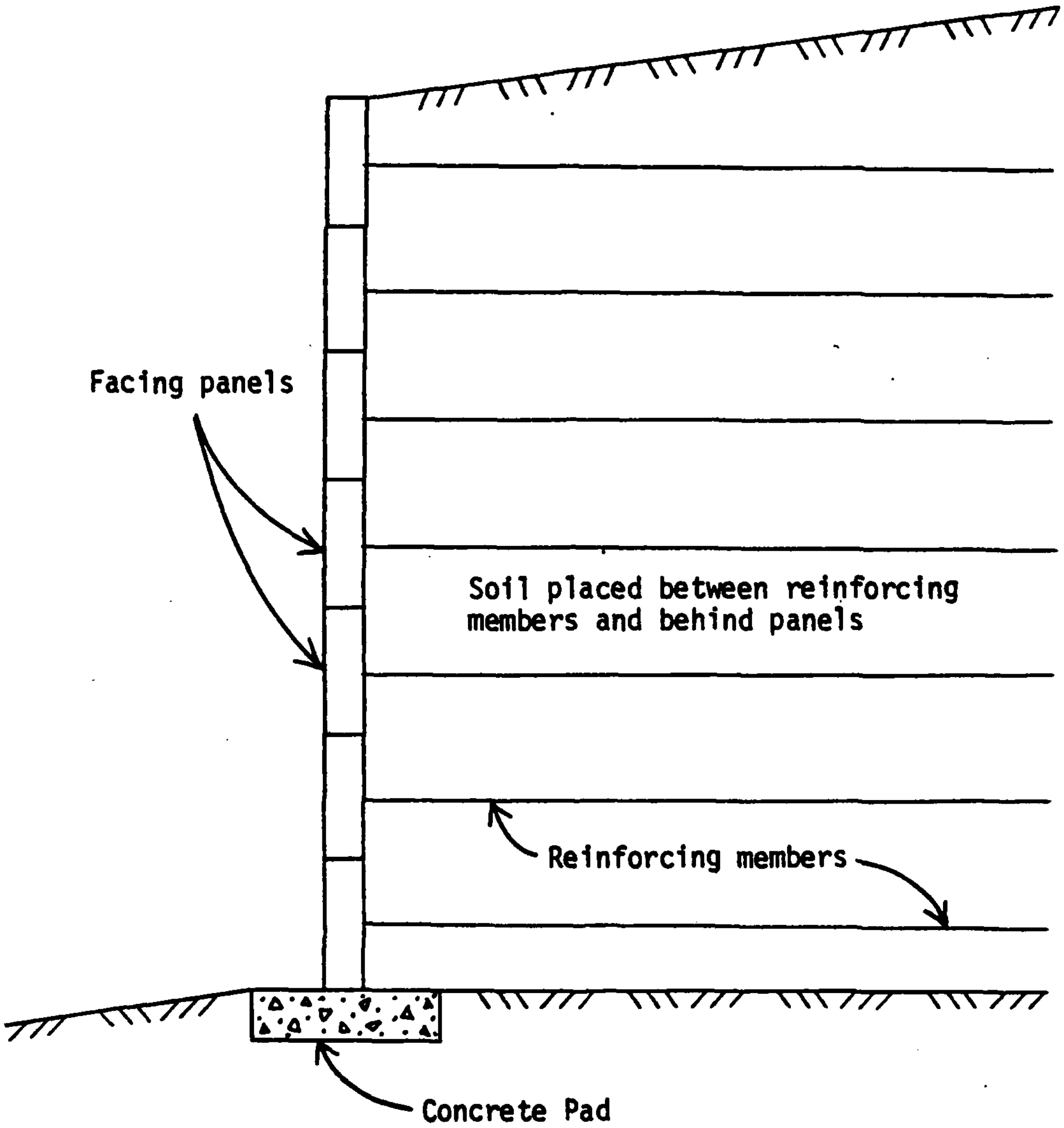


Fig. 1. Reinforced Earth Retaining Wall

Tensar Geogrids

As mentioned earlier, this report will focus on Tensar geogrids as a soil reinforcing material. Tensar geogrids are molecularly oriented polymeric grid structures specifically developed for use as tension-resistant inclusions in soils. They are manufactured by stretching a punched sheet of extruded high quality polymer under controlled conditions. The patented process was developed in the late 1970's and aligns the polymer's long chain hydrocarbon molecules into continuous geometric patterns. These grid structures have tensile strengths comparable to mild steel with no weak links or failure planes. They are manufactured and marketed by the Tensar Corporation whose headquarters is in Atlanta, Georgia. Their use has been rapidly growing in the United States since 1981. (reference 2)

Tensar geogrids come in a wide variety of strengths and configurations. There are the uniaxial grids manufactured from co-polymer grade high density polyethylene and the biaxial grids manufactured from homo-polymer polypropylene. Prior to the production of these grids, carbon black is added to provide ultraviolet protection. The uniaxial grids are stretched in one direction, thus aligning the atomic polymer chains in the direction of draw. This results in a product with high one-directional tensile strength and modulus. The biaxial grids are stretched in two perpendicular directions, resulting in high tensile strength and modulus in both directions of pull.

The uses of Tensar geogrids are widespread and numerous. As a base reinforcement, it is a direct substitute for aggregate fill in applications such as roads, railroads, runways, foundations and commercial or industrial yards.

Design charts are available which show the possible reduction in aggregate thickness if Tensar geogrids are used. (reference 5) Tensar Corporation claims that base layer thicknesses can be reduced as much as 50% and pavement life extended up to 300% when reinforced with Tensar. Interlock and confinement of the base particles by the high strength Tensar with its open grid structure are the key features to base reinforcement.

Other applications of Tensar geogrids are erosion control, fencing and soil reinforcement. It is in soil reinforcement that the designer uses the pullout resistance of the grid. The grids can be cast directly into a concrete facing panel, thereby eliminating any connections. The open grid geometry provides an interlock with soil that resists pullout. This interlock capacity determines the required embeddment lengths of the grids in the design.

Since Tensar geogrids are prestressed during their manufacture, they have a high tensile modulus which immediately takes up load under the smallest strains. This is important since soils develop their peak strength at small strains. When stressed, the grids must sustain the load without rupture and without generating unacceptable large deformations during the lifetime of the structure. Tests have been performed to determine this load-strain-time behavior of various Tensar grids. (reference 6) As can be expected with polymeric materials, the properties vary with strain rates and temperature. Tensar geogrids are resistant to the corrosive environment found in soils and the ultraviolet rays of the sun. This provides for a long service life with allowable design strengths for a 120-year lifetime. (reference 7)

The two Tensar geogrids that were tested during this project were a biaxial grid called SS-1 and a uniaxial grid called SR-2. The actual geometric configuration of each is shown in Figures 2 and 3. These are some of the more popular grids in use and thus easy to obtain. This report will cover the results of the SS-1 pullout tests and Yuen Zehong's report will cover the results of the SR-2 pullout tests. A manufacturer's data sheet and strength values of the SS-1 material are included in the Appendix.

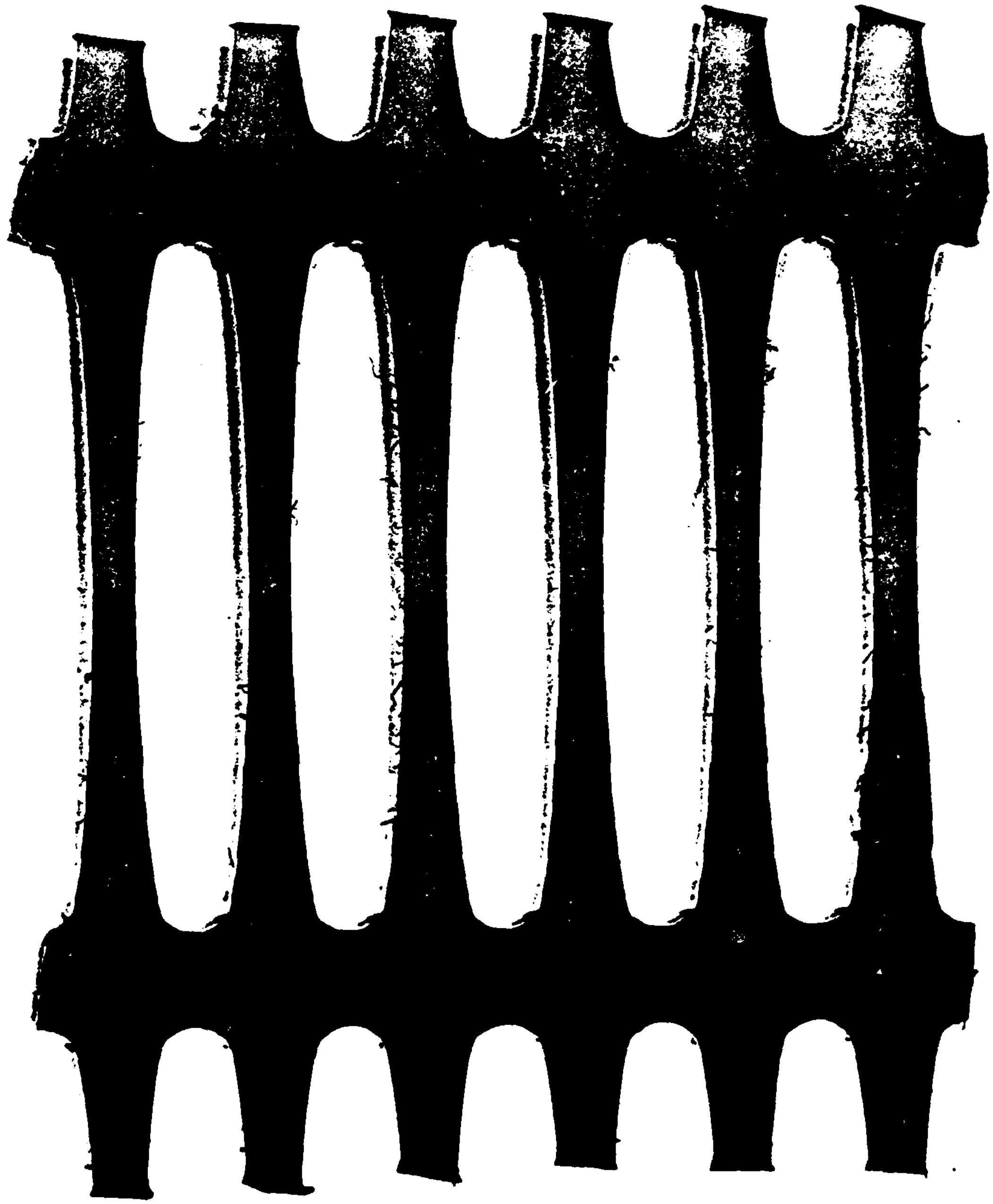


Fig. 2. SR-2 Tensar Geogrid Configuration

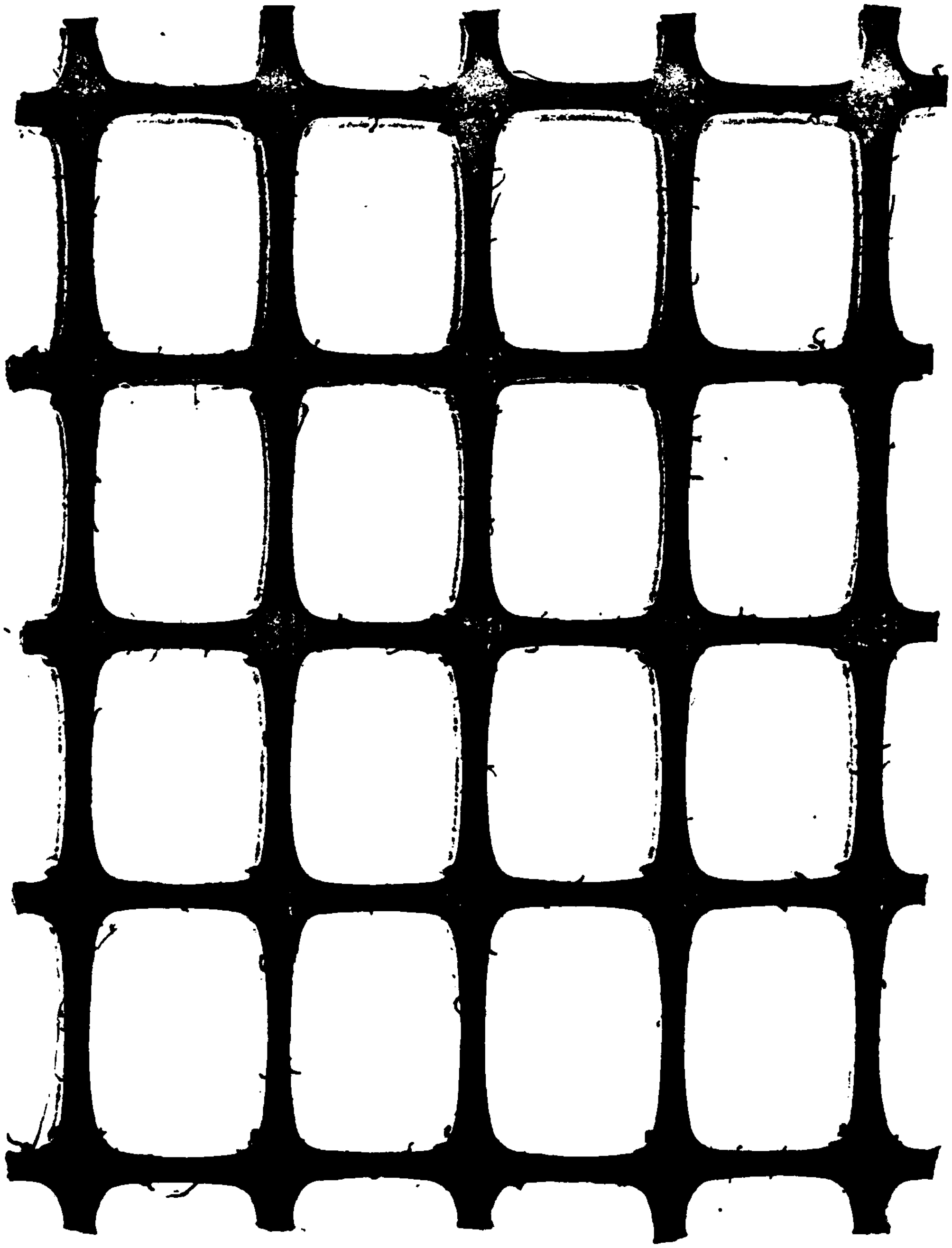


Fig. 3. SS-1 Tensar Geogrid Configuration

Preliminary Work Before Testing

The test chamber for determining the pullout resistance of soil reinforcing members located in the University of New Mexico's Civil Engineering concrete lab was used. The chamber is shown in Figure 4. It is owned by the New Mexico State Highway Department and was designed and manufactured by the University of New Mexico. Many of the same procedures were used as the initial set up and testing performed by Mark A. Wittrock and Dr. John B. Carney when they tested welded wire mesh as a soil reinforcement. (references 8 and 9) The load cell, its operation, and their testing methods are thoroughly covered in the two reports referenced. Therefore, only those items or methods that are different due to the difference in soil reinforcement tested will be discussed in this report.

Two new load cells were manufactured by machine shops at the University of New Mexico. The new load cells were 1/4 inch in diameter compared to 1/2 inch with the older cells. Once installed with strain gauges and calibrated, the new cells were more accurate than the old cells. One of the initial tasks was to install two strain gauges 180 degrees apart on each of the new load cells and wire each up to two dummy gauges to create a full bridge circuit. Figure 5 shows a new load cell with its four strain gauges. The two new load cells (No. 1 and No. 2) were then calibrated in tension in the Mechanics of Materials Lab as shown in Figure 6. Also calibrated was the load cell (No. 5) used to apply the vertical load. All three calibration plots are shown in Figures 7 thru 9. Since the stresses applied are in the elastic range for steel, each plot is linear and a stress-strain modulus constant was used for further calculations.

Concrete sand was dried and used for the soil. By raining the sand into the

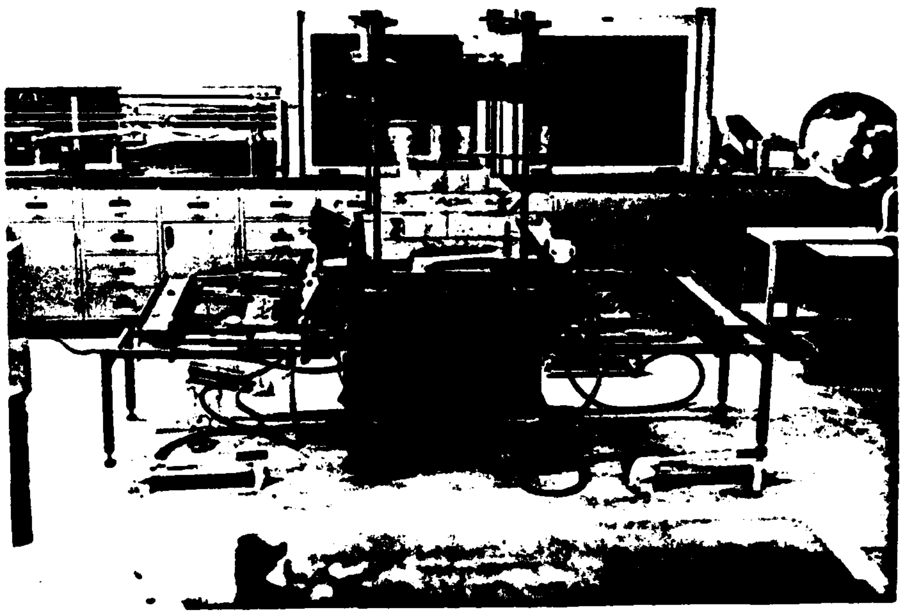


Fig. 4. Test Chamber

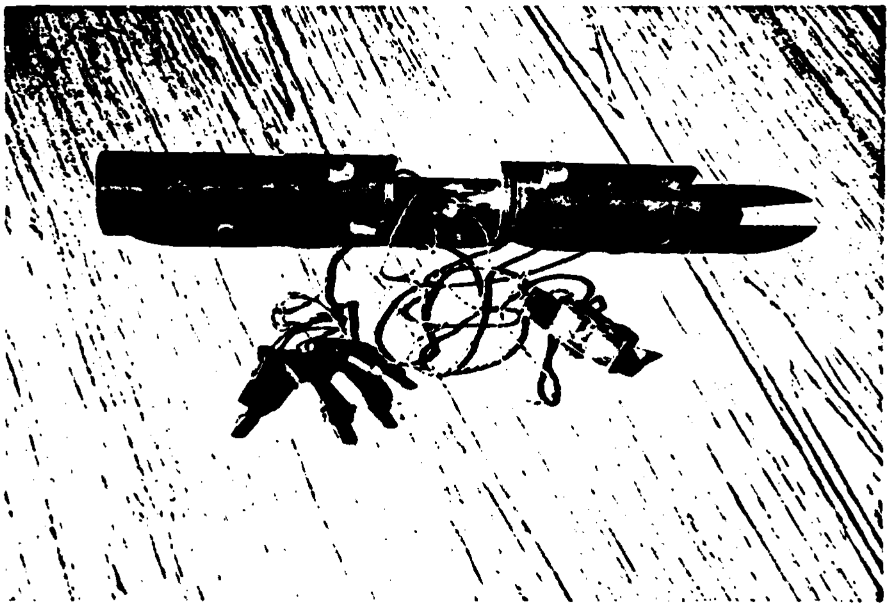


Fig. 5. New Load Cell

Fig. 6. Calibration of New Load Cell

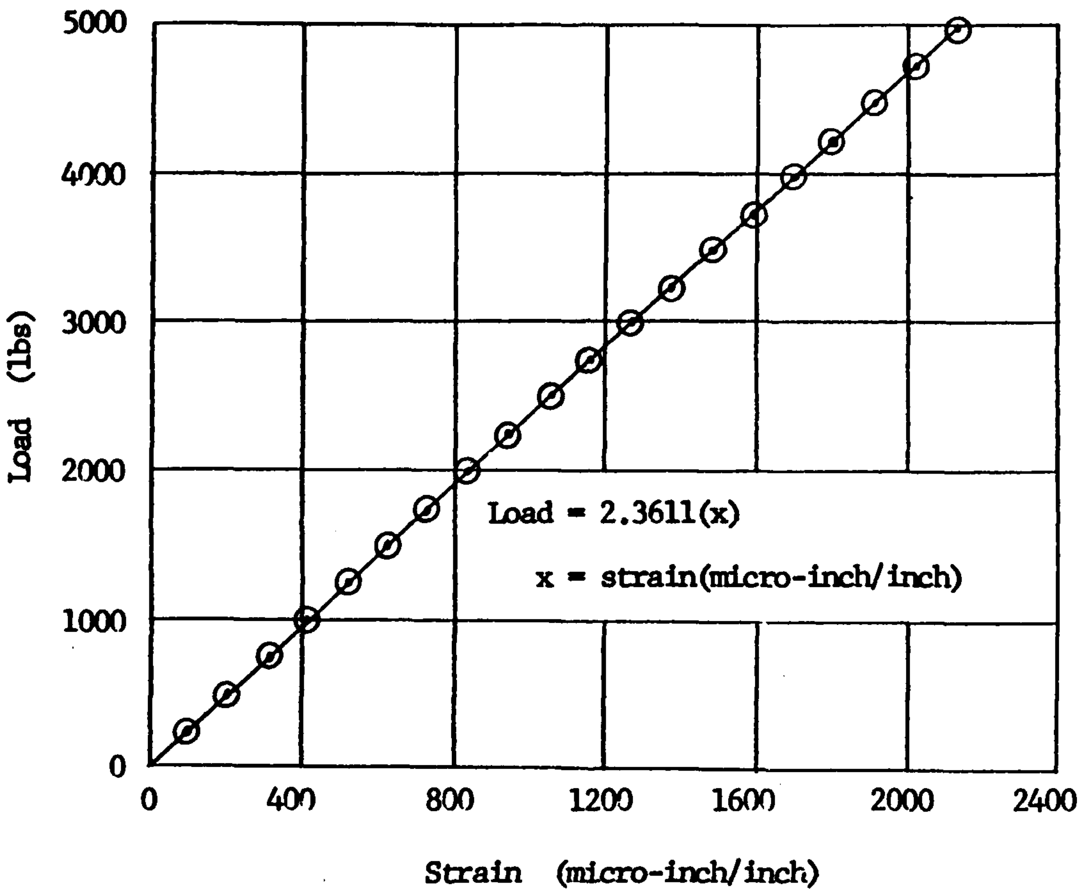


Fig. 7. Joint No. 1 Calibration Plot

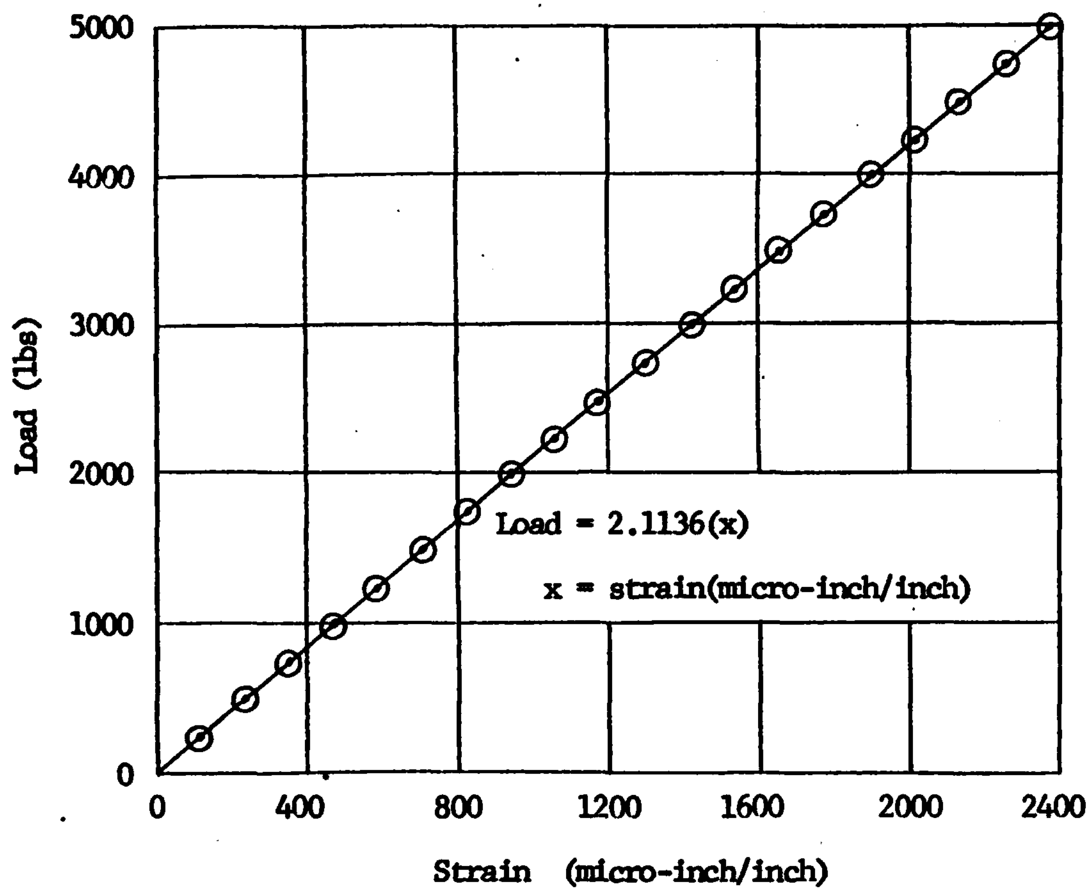


Fig. 8. Joint No. 2 Calibration Plot

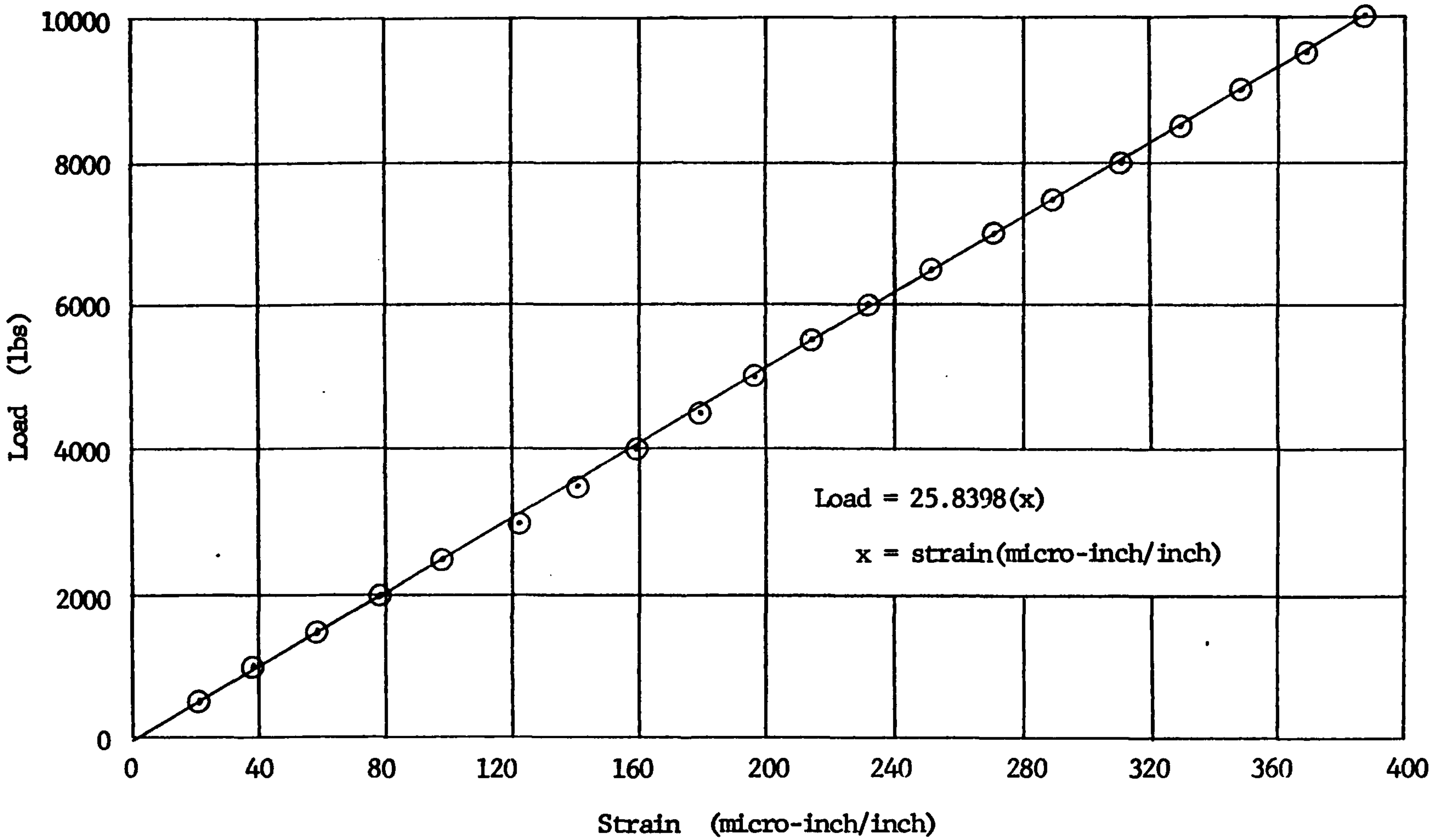


Fig. 9. Joint No. 5 Calibration Plot

chamber at a height of 12 to 16 inches, it was found that the relative density was approximately 70% with a dry unit weight of 111pcf. The grain size distribution curve of the sand is shown in Figure 10. (reference 9) Cardboard was used for templates to keep an excessive amount of sand from pouring out of the box as the Tensar was pulled out. The Tensar was acquired from Armco Inc. in Albuquerque, New Mexico. Samples were cut from the middle of rolls to provide the most representative material. Mat widths were 20 1/2 inches for the SR-2 grid and 20 inches for the SS-1 grid.

New clamps were designed and manufactured for the use of SS-1 and SR-2 geogrids on the pullout machine. A 5/16 inch spacer was placed under the Tensar to ensure no eccentric forces were developed. A 1/2 inch steel bar on top of the Tensar was then clamped thru the Tensar and spacer to the bottom support of the machine. Holes drilled thru the clamps had to be spaced differently for the two types of grids. Figure 11 shows the SS-1 material clamped to the machine.

University of New Mexico
Department of Civil Engineering
University of New Mexico

Grain Size Distribution Curve

Test No. ______

Sample Concrete Sand Tested By Mack Wittcock Party ______ Date 2-17-86

D_{10} 0.09 D_{60} 0.46 U_c 5.11

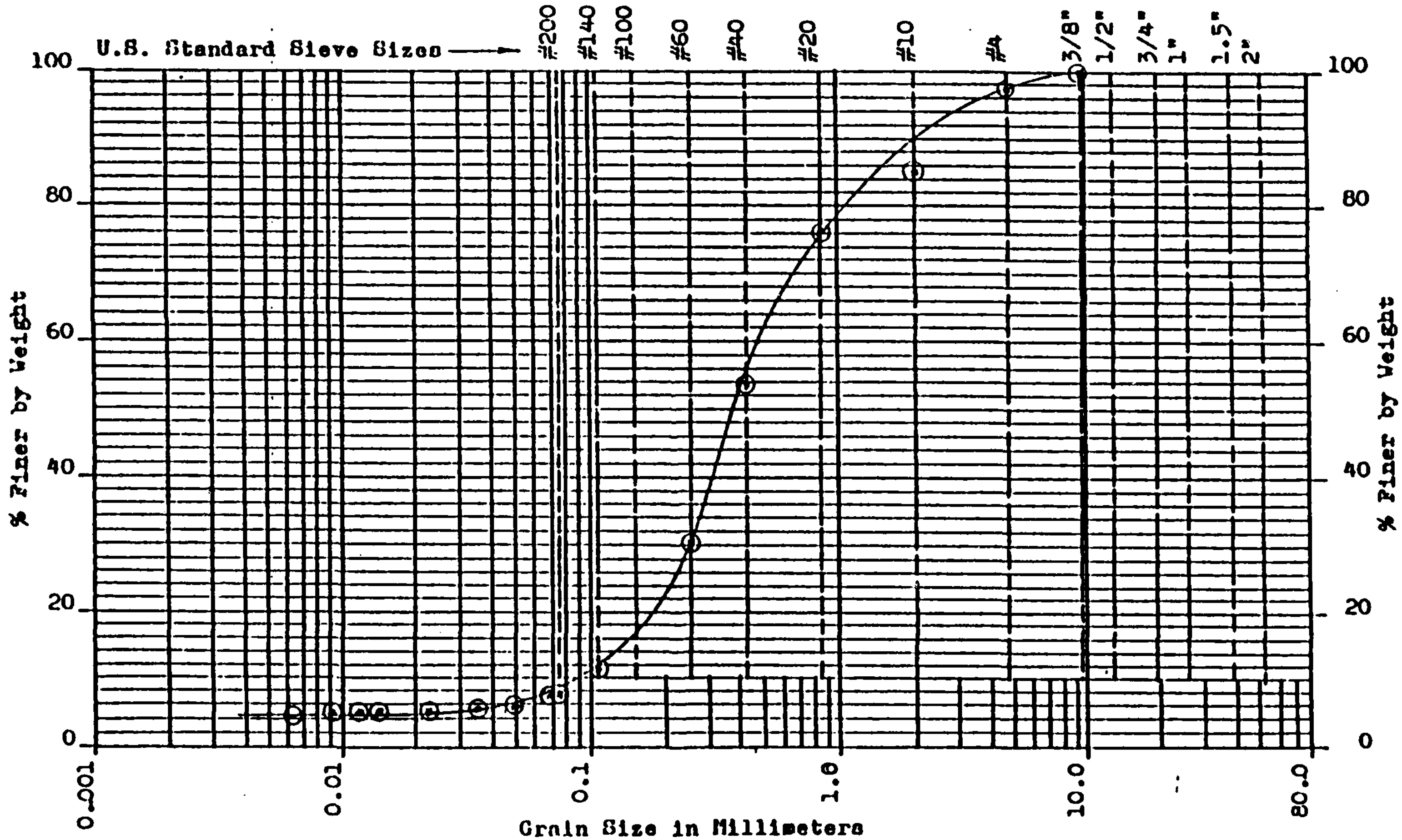


Fig. 10. Grain Size Distribution Curve

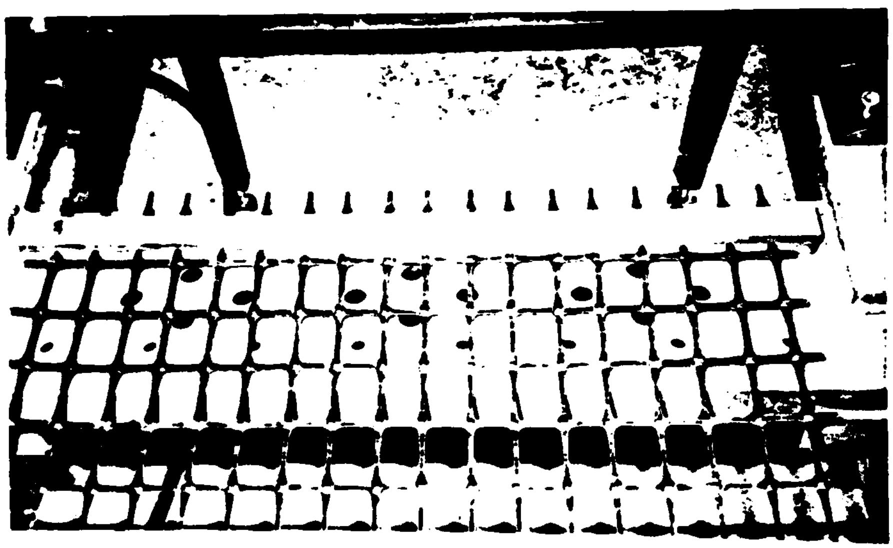


Fig. 11. SS-1 Clamp

Testing

The procedure used to setup and run each test were kept the same to provide a uniform basis for analysis. First, the sand was rained into the chamber until it was one-half full and then leveled. The Tensar mat was then laid into the chamber, clamped, and aligned. The box was then filled to the top with sand. The load cells were attached and wired to their respective strain boxes. Strain dials were installed. The mat was pulled tight and the jacks set so a load would be applied as soon as the jacks started moving. All strain boxes and strain dials were then zeroed before testing began.

A constant vertical pressure (Pv) was maintained throughout a test. The strain box reading required to maintain a certain Pv was calculated for each vertical load used. During testing, a vertical jack was operated to maintain the reading.

The Tensar mat was pulled out at a constant rate of .30 inch/minute. A tape recording calling cadence kept the jack operators in synchronization. The mat was pulled through soil for 6 inches. Since the strain dials only had a stroke of 4 inches, the testing had to be stopped and strain dials reset at 3 inches of pullout. After pulling the Tensar thru 6 inches of undisturbed soil, the load cells, strain boxes, and strain dials were removed and reinstalled to pull the mat back thru 6 inches of disturbed soil in the opposite direction. At every .1 inch of the pullout process, readings from all three strain boxes were taken to determine the load on each cell.

The way each test is titled gives a brief description of the test. The title starts with the type of Tensar grid tested, such as SS-1. This is followed by the amount of feet of sand overburden that corresponds to the Pv maintained during the test. Last, is the letter "a" or "b," "a" meaning the mat was pulled thru undisturbed soil and "b" meaning thru disturbed soil. Thus, the test SS1-1-a means that the Tensar grid SS1 was pulled thru undisturbed soil at an overburden pressure equal to 1 foot of sand. The schedule of tests performed on the SS1 geogrid is shown in Table 1.

Table 1 Schedule of Tests

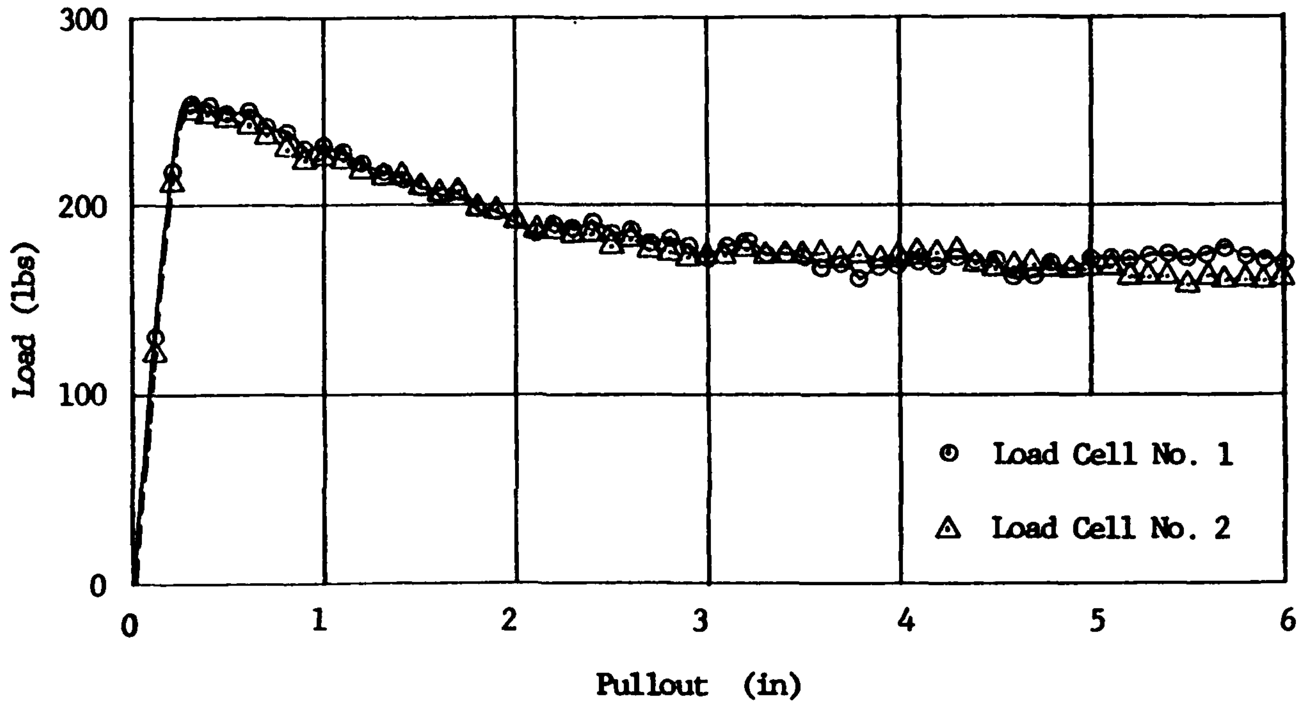
Test No.	Date	Pv (psf)	Comments
SS1-1-a	8JUL86	101.5	
SS1-1-b	8JUL86	101.5	
SS1-5-a	10JUL86	551.7	Tensar ruptured
SS1-2-a	16JUL86	219.5	
SS1-2-b	16JUL86	219.5	
SS1-3-a	17JUL86	330.2	Unable to run b test due to Tensar alignment
SS1-2.5-a	18JUL86	277.1	
SS1-2.5-b	18JUL86	277.1	
SS1-3.8-a	21JUL86	418.8	Ran test at 3.8 ft. of overburden instead of 4 ft. to avoid rupture of Tensar
SS1-3.8-b	21JUL86	418.8	

Test Results

The results of the tests listed in Table 1 are presented in two different formats. First, for each test, the loads carried by load cells #1 and #2 are plotted versus pullout, pullout being the amount of horizontal travel of the Tensar mat. Pullout starts at 0 and increases to 6 inches at the end of each test. These results are shown in Figures 12 thru 21. The difference in load between cells #1 and #2 can be attributed to a number of factors. These could be nonuniformity in the mat or soil, the jacks being operated at slightly varying rates, or one side of the mat having more tension than the other side at the start of the test.

The second format used to present the results of each test are plots of the unit resistance or pullout strength (Fp) versus pullout. Fp is defined as the total load of both cells divided by the mat area embedded in the soil. During our testing of the SS-1 mat, this area remained constant at 4.167 square ft. Tests run at the same Pv thru both disturbed and undisturbed soil are compared on the same plots. These plots are shown in Figures 22 thru 27.

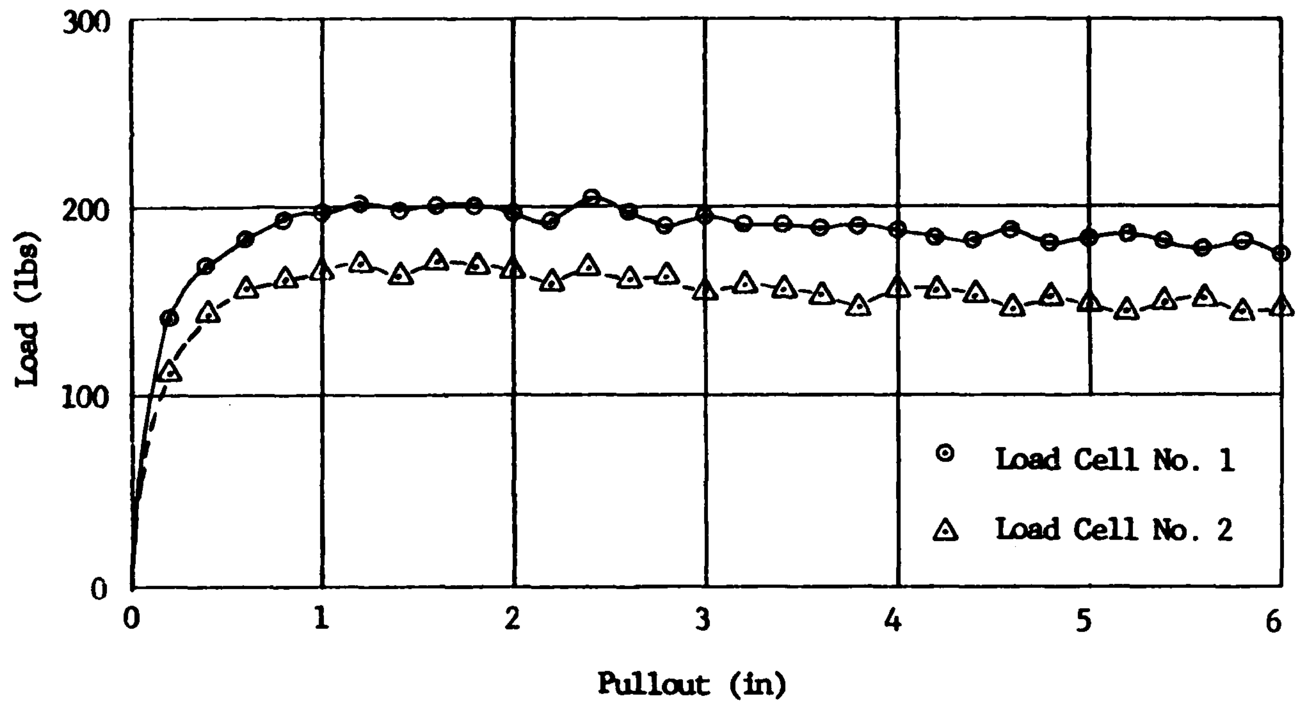
The pullout, as defined above, reflects both the elongation of the Tensar mat that occurs during testing and the movement of the mat through the soil. The elongation of the mat gets proportionately larger as Fp gets larger. The tensile stress distribution in the mat is constant from the point where it is clamped and pulled to the point where the mat leaves the soil. While in the soil, the stress in the Tensar decreases linearly until it reaches zero at the opposite end of the chamber where the mat first enters the soil. The tensile stress remains zero in the portion of mat that is not in soil and at the opposite end of pull.



Average Vertical Pressure = 101.5 psf

Undisturbed Soil

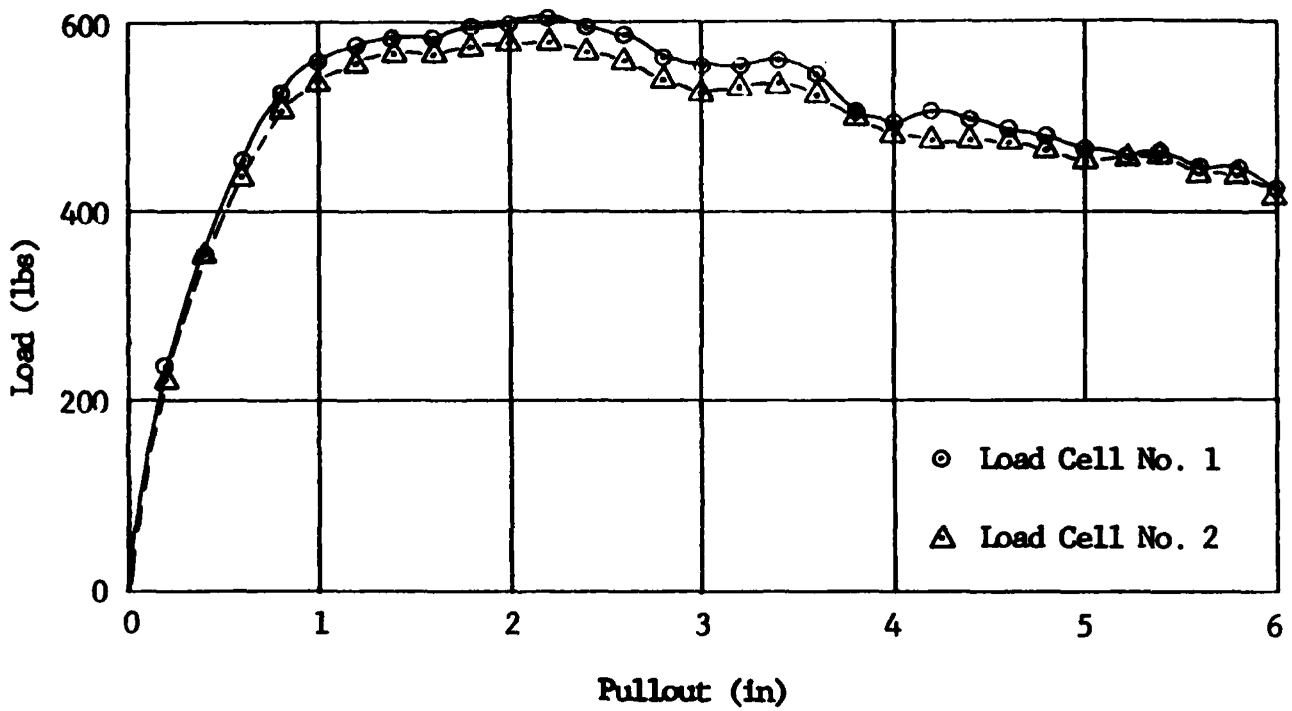
Fig. 12. Load Cells No. 1 and No. 2 vs. Pullout, SS1-1-a



Average Vertical Pressure = 101.5 psf

Disturbed Soil

Fig. 13. Load Cells No.1 and No. 2 vs. Pullout, SS1-1-b



Average Vertical Pressure = 219.5 psf

Undisturbed Soil

Fig. 14. Load Cells No. 1 and No. 2 vs. Pullout, SS1-2-a

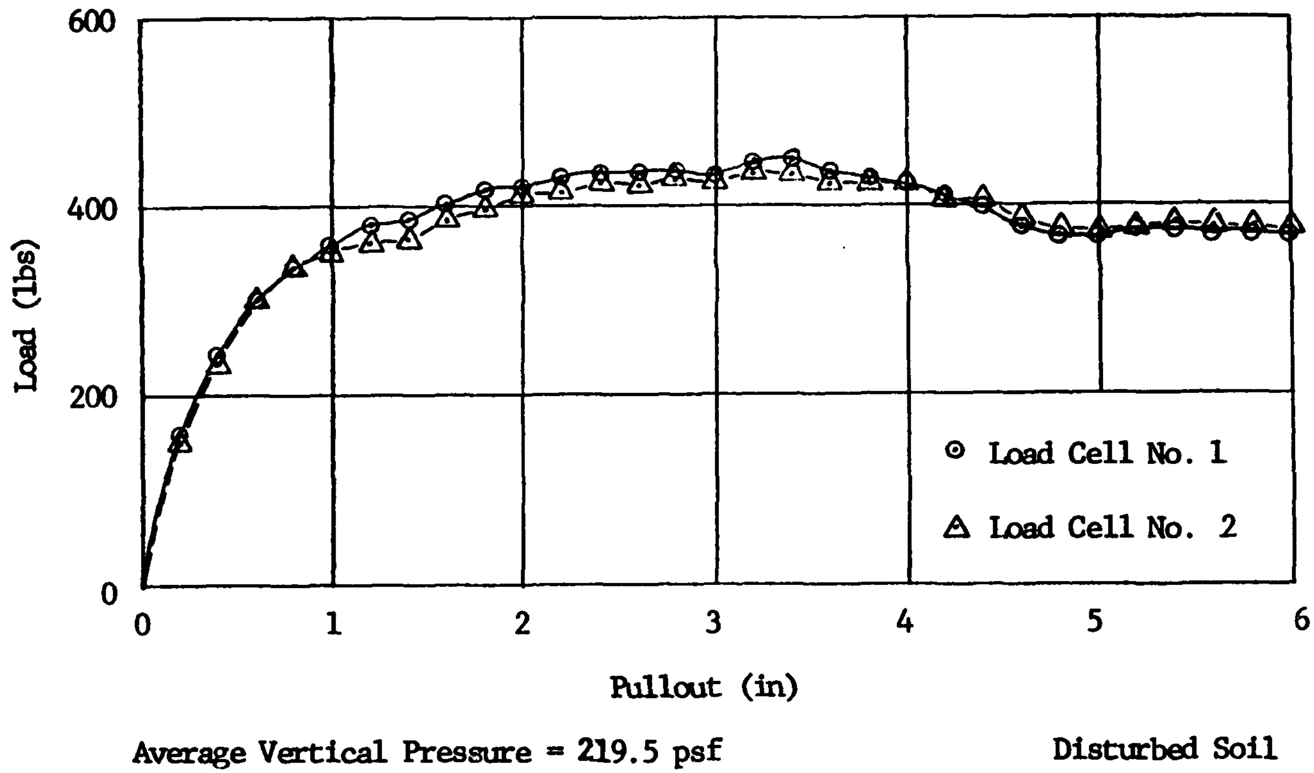
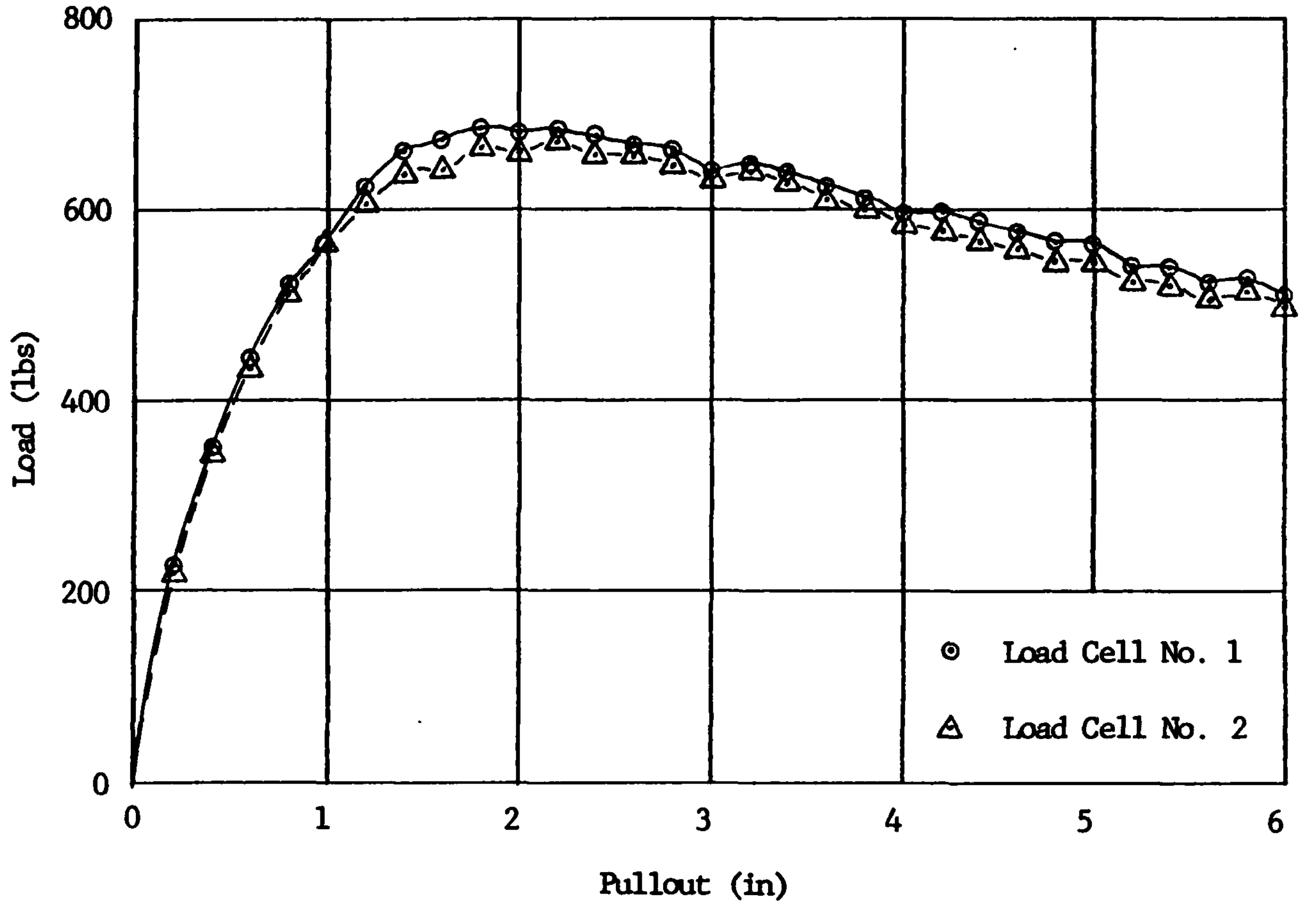


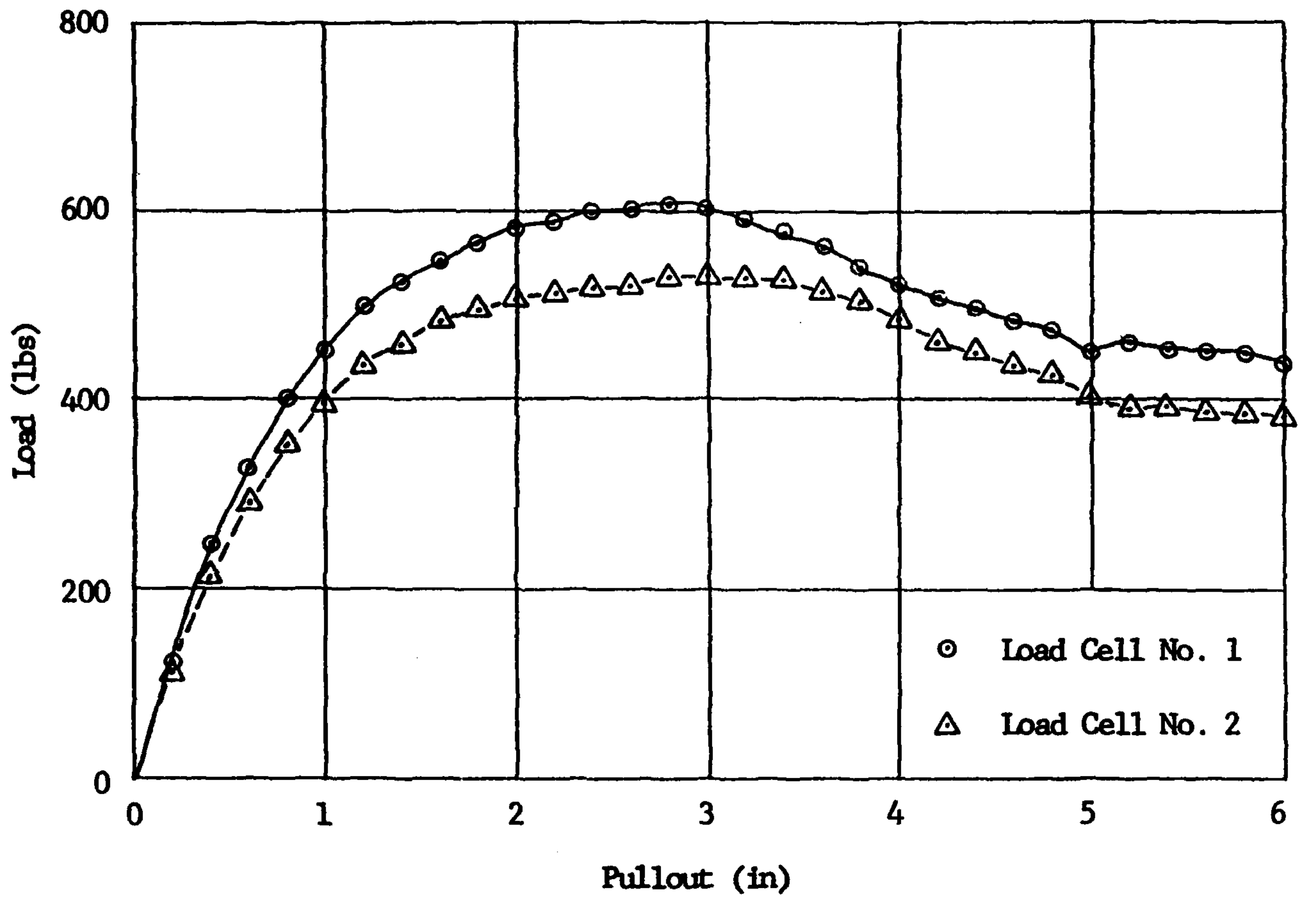
Fig. 15. Load Cells No. 1 and No. 2 vs. Pullout, SS1-2-b



Average Vertical Pressure = 277.1 psf

Undisturbed Soil

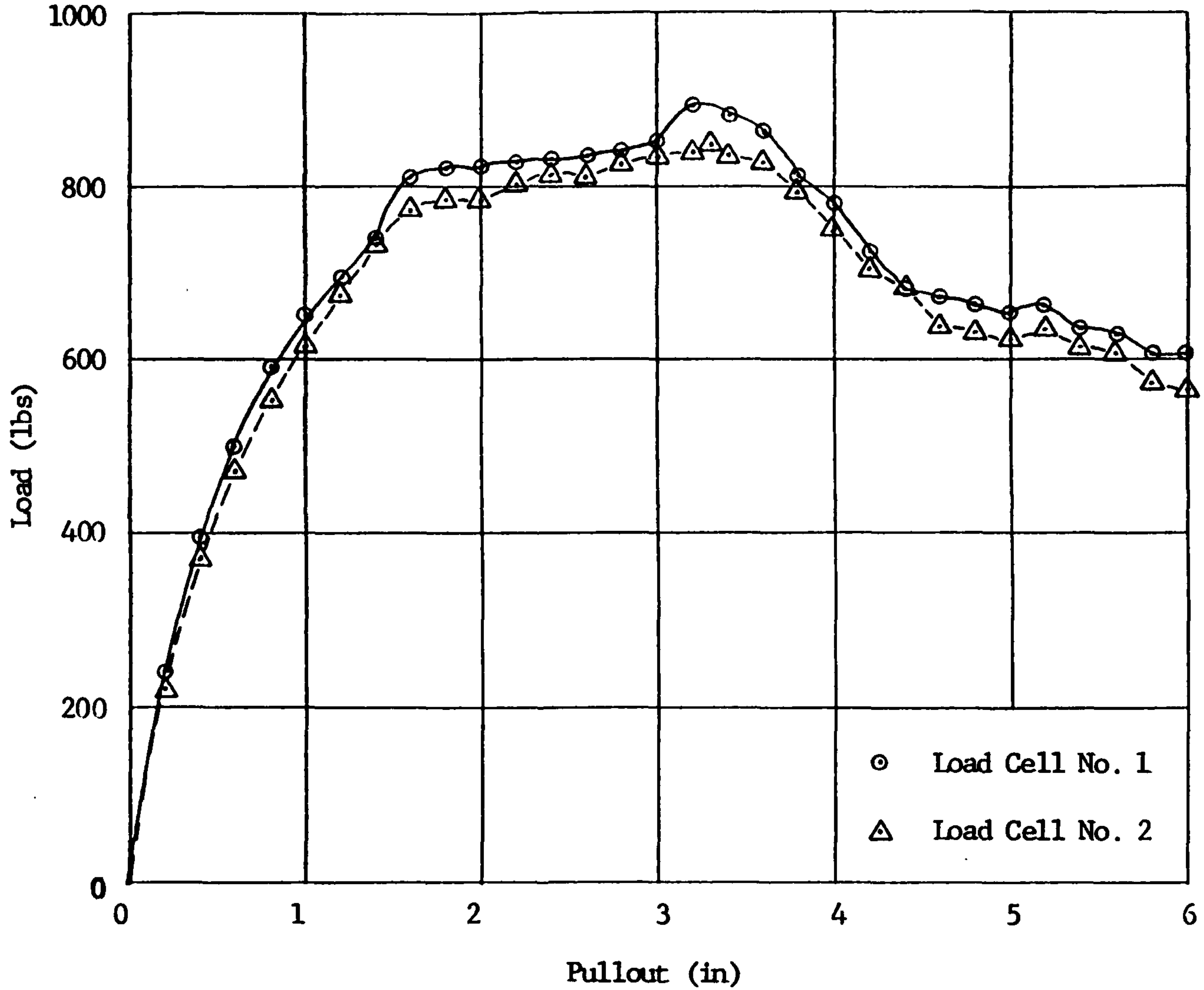
Fig. 16. Load Cells No. 1 and No. 2 vs. Pullout, SS1-2.5-a



Average Vertical Pressure = 277.1 psf

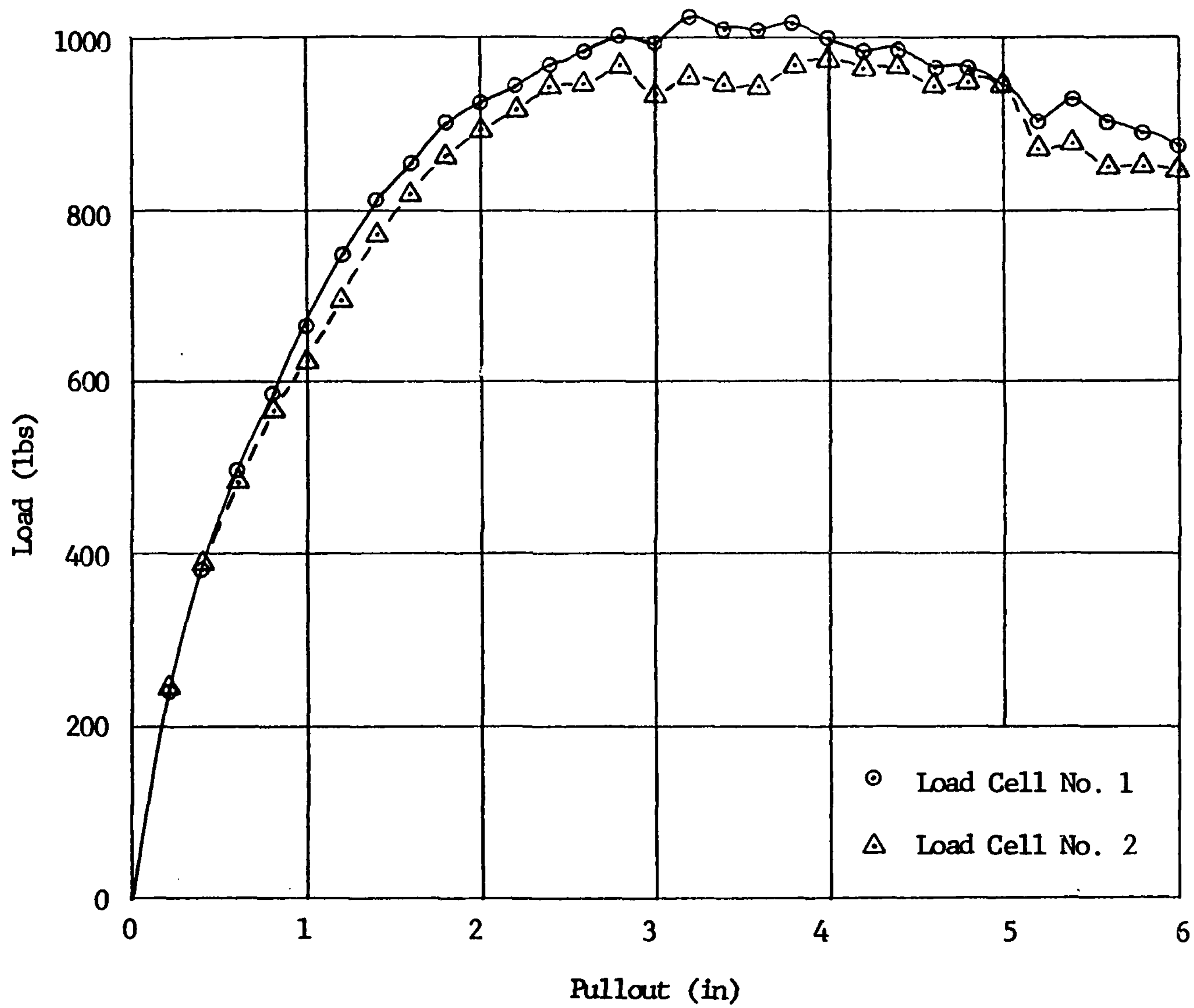
Disturbed Soil

Fig. 17. Load Cells No. 1 and No. 2 vs. Pullout, SS1-2.5-b



Average Vertical Pressure = 330.2 psf Undisturbed Soil

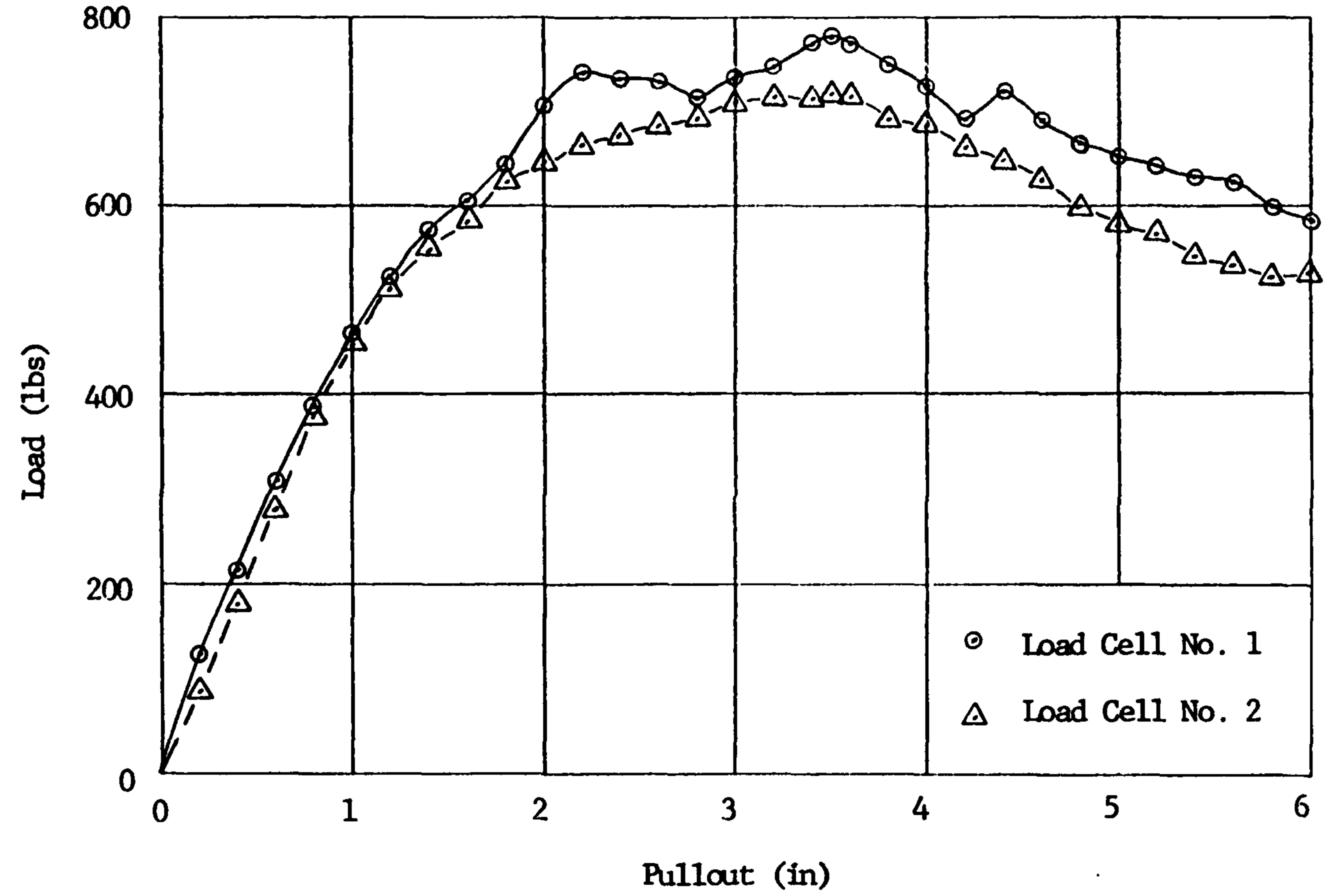
Fig. 18. Load Cells No. 1 and No. 2 vs. Pullout, SS1-3-a



Average Vertical Pressure = 418.8 psf

Undisturbed Soil

Fig. 19. Load Cells No. 1 and No. 2 vs. Pullout, SS1-3.8-a



Average Vertical Pressure = 418.8 psf

Disturbed Soil

Fig. 20. Load Cells No. 1 and No. 2 vs. Pullout, SS1-3.8-b

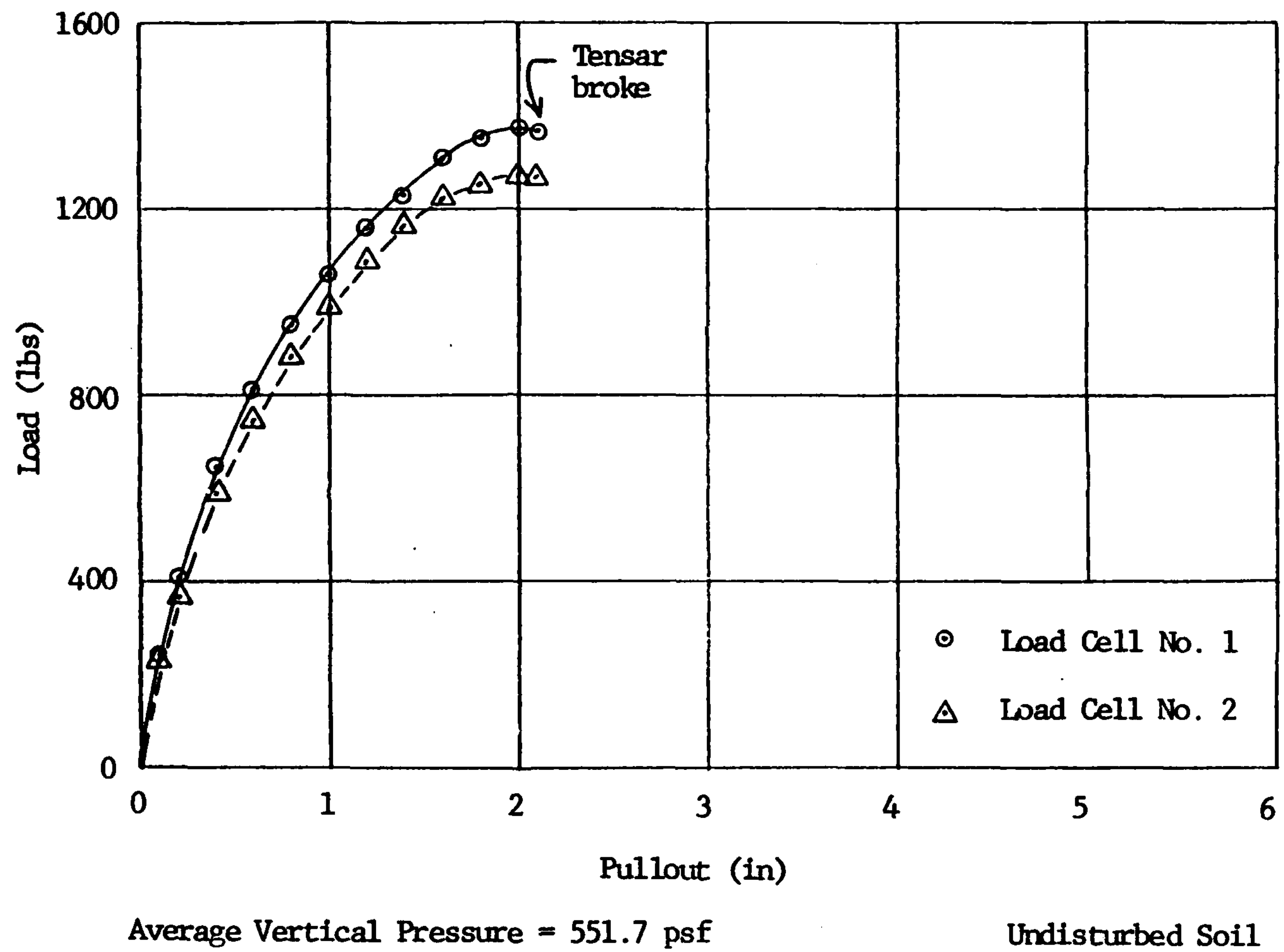
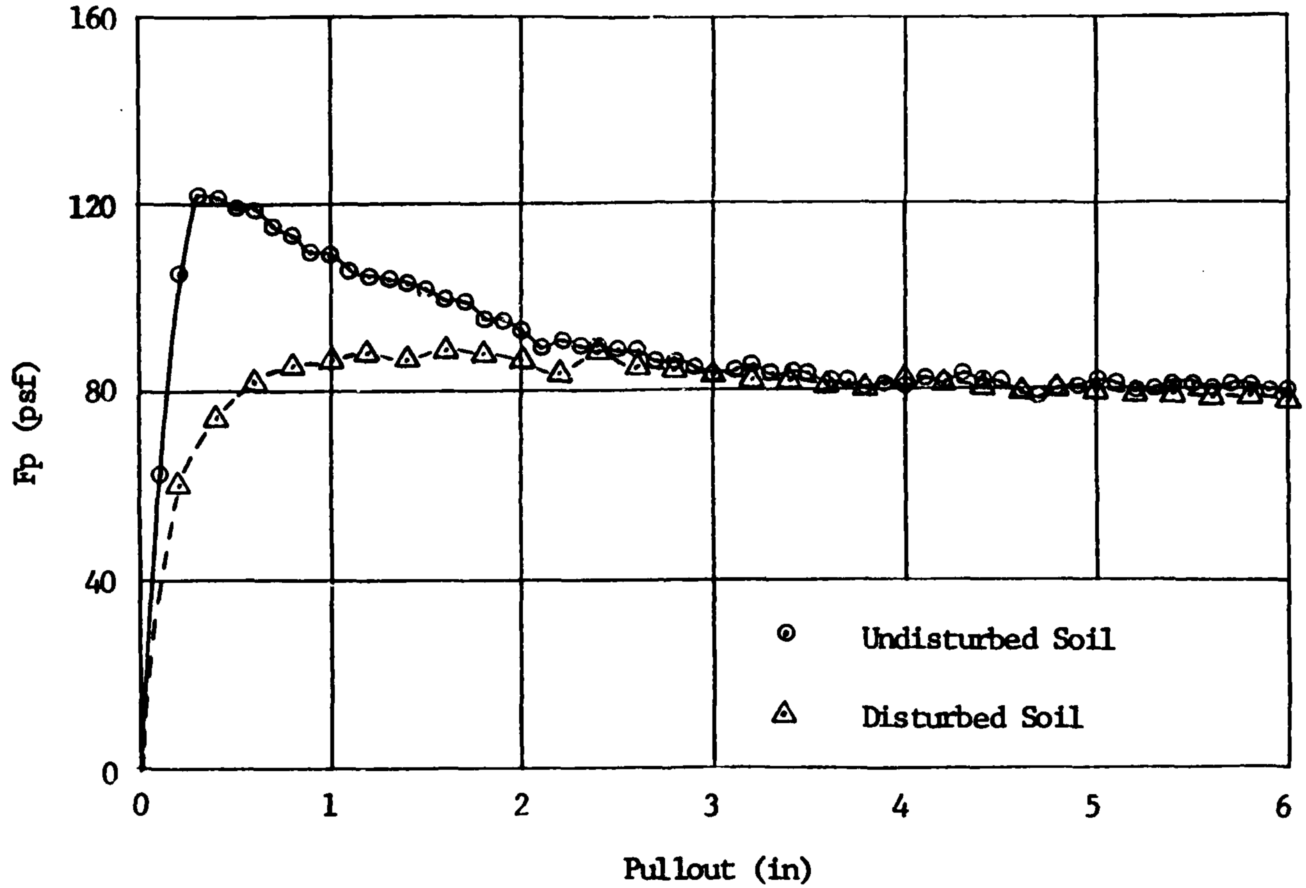
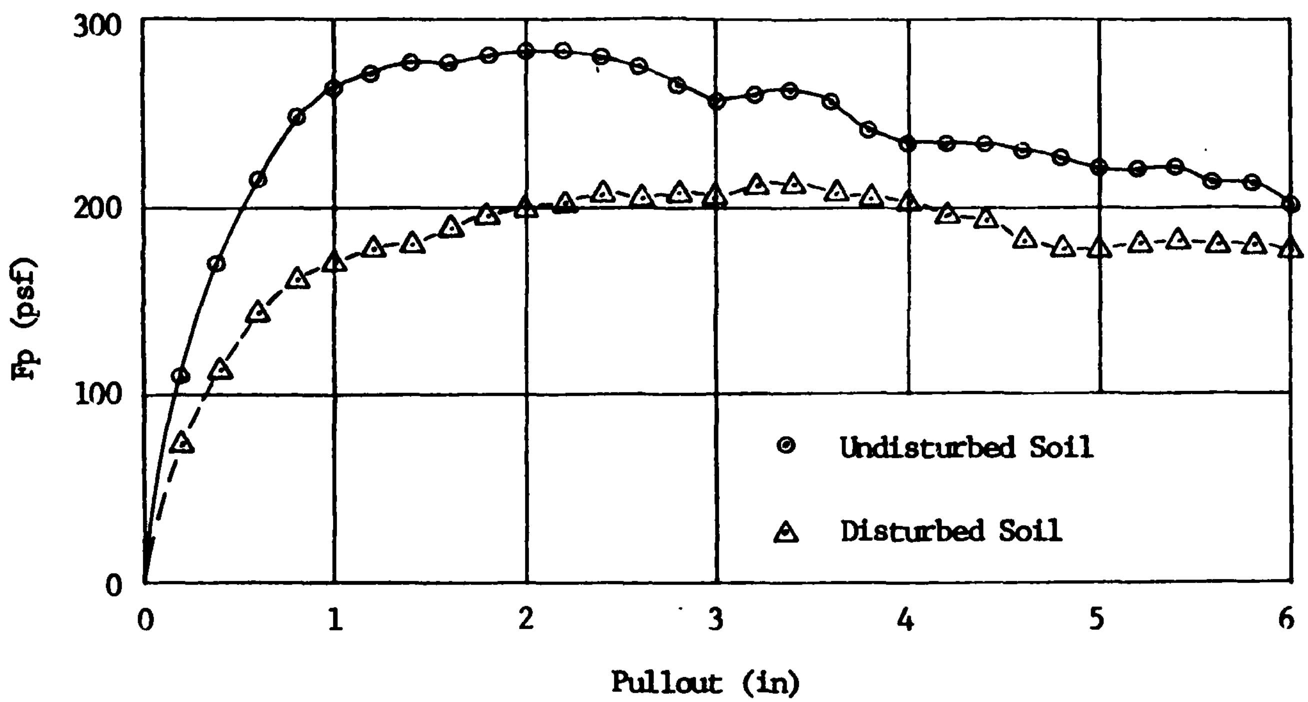


Fig. 21. Load Cells No. 1 and No. 2 vs. Pullout, SS1-5-a



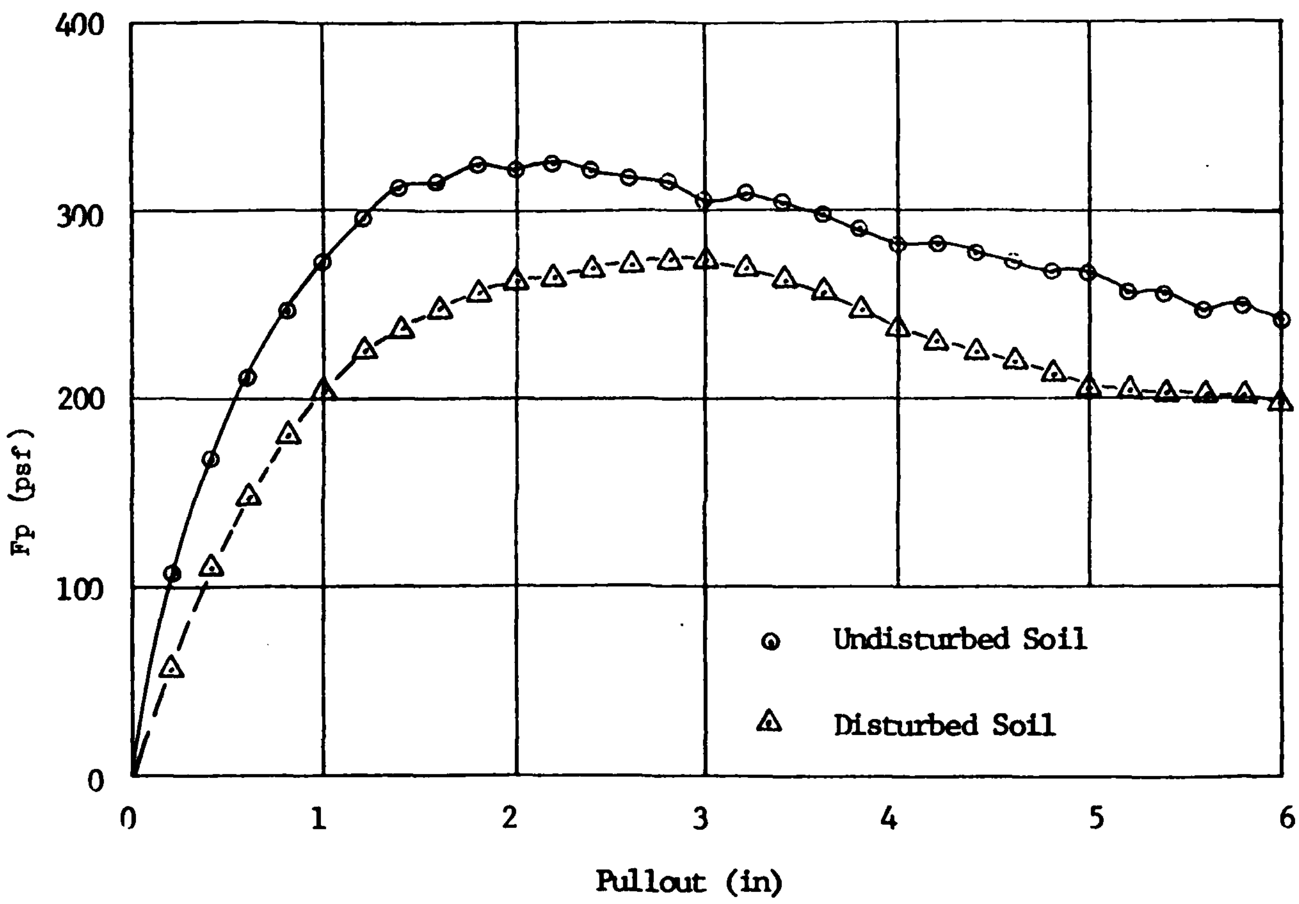
Average Vertical Pressure = 101.5 psf

Fig. 22. Fp vs. Pullout, SS1-1-a and SS1-1-b



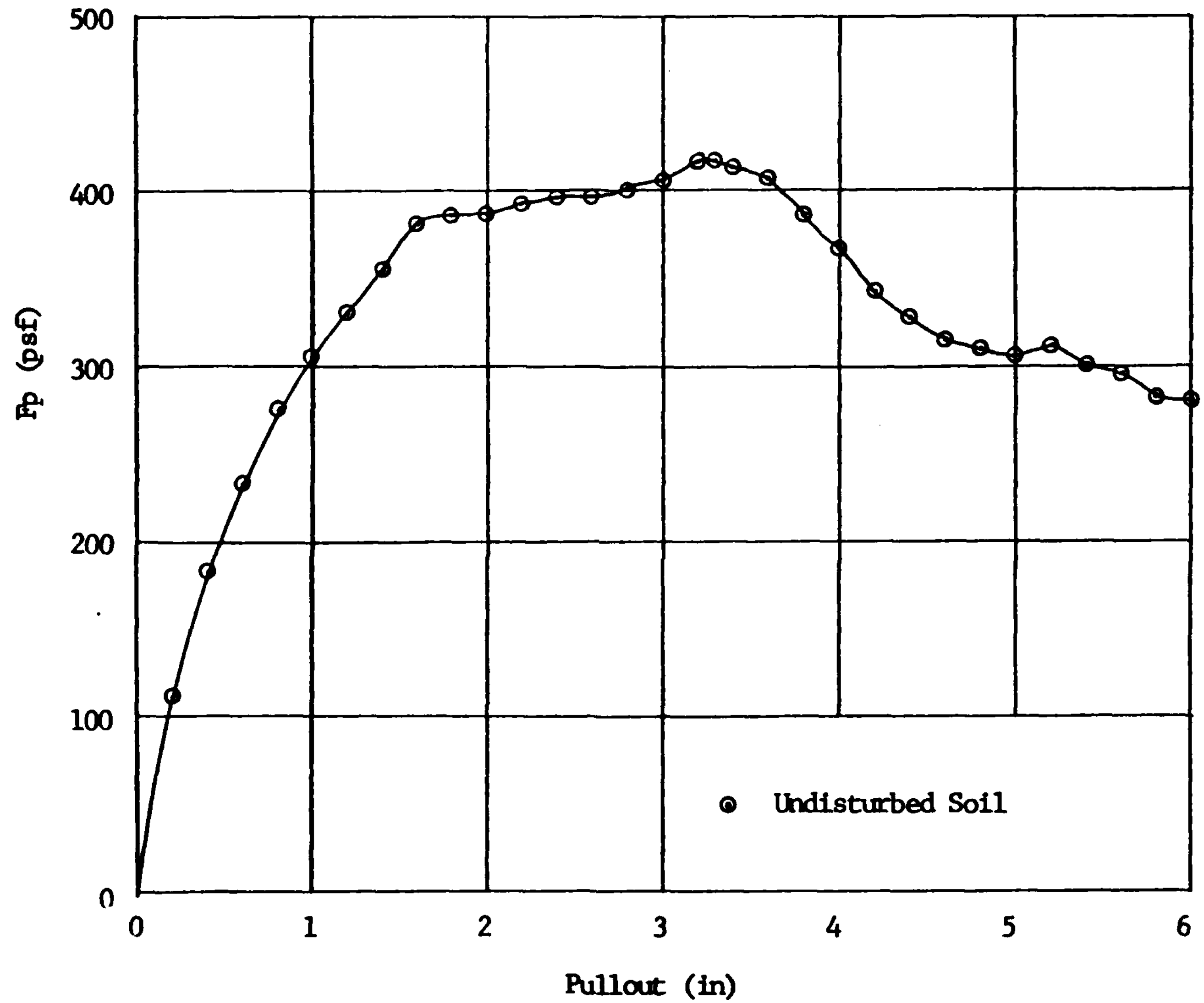
Average Vertical Pressure = 219.5 psf

Fig. 23. Fp vs. Pullout, SS1-2-a and SS1-2-b



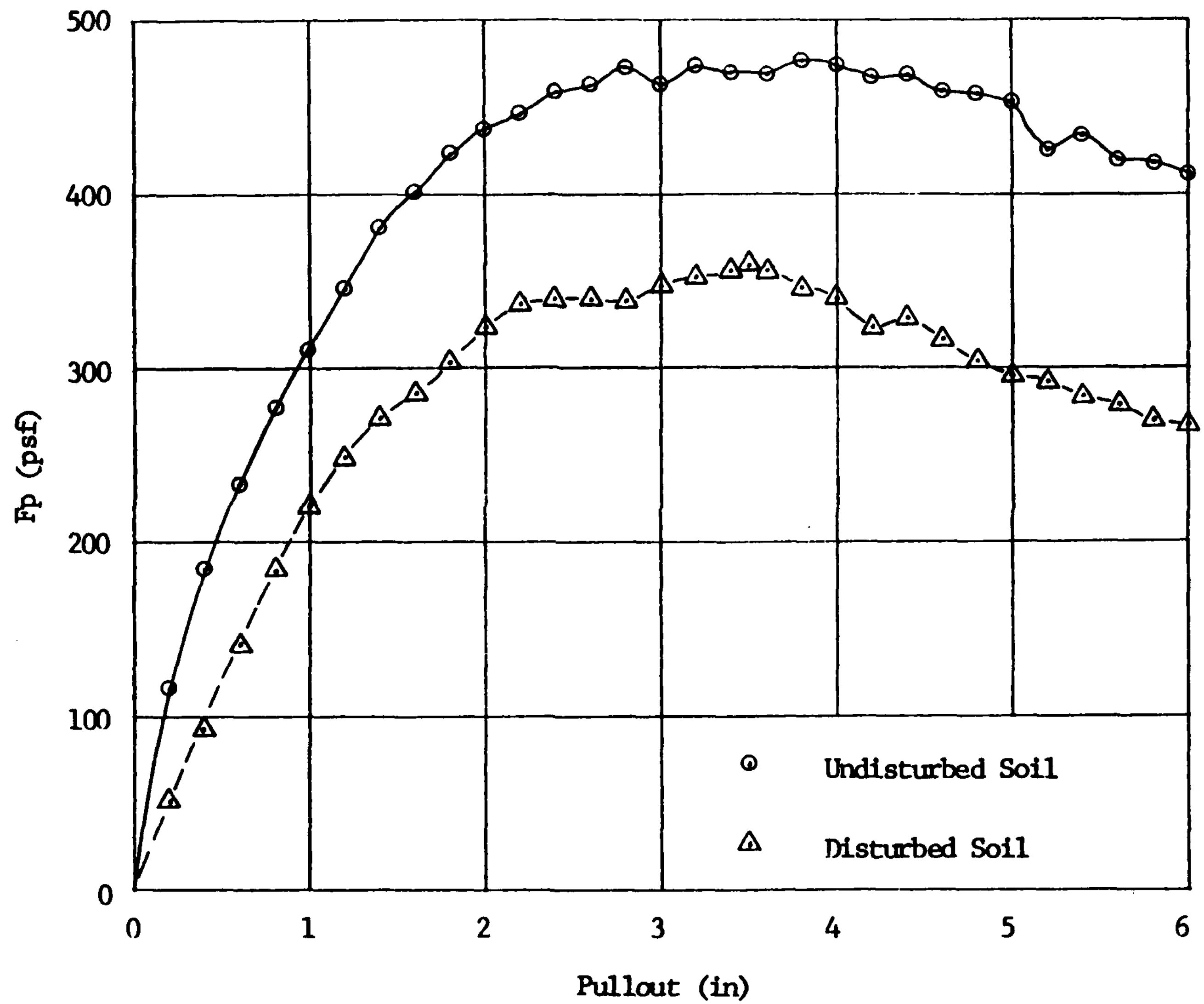
Average Vertical Pressure = 277.1 psf

Fig. 24. Fp vs. Pullout, SS1-2.5-a and SS1-2.5-b



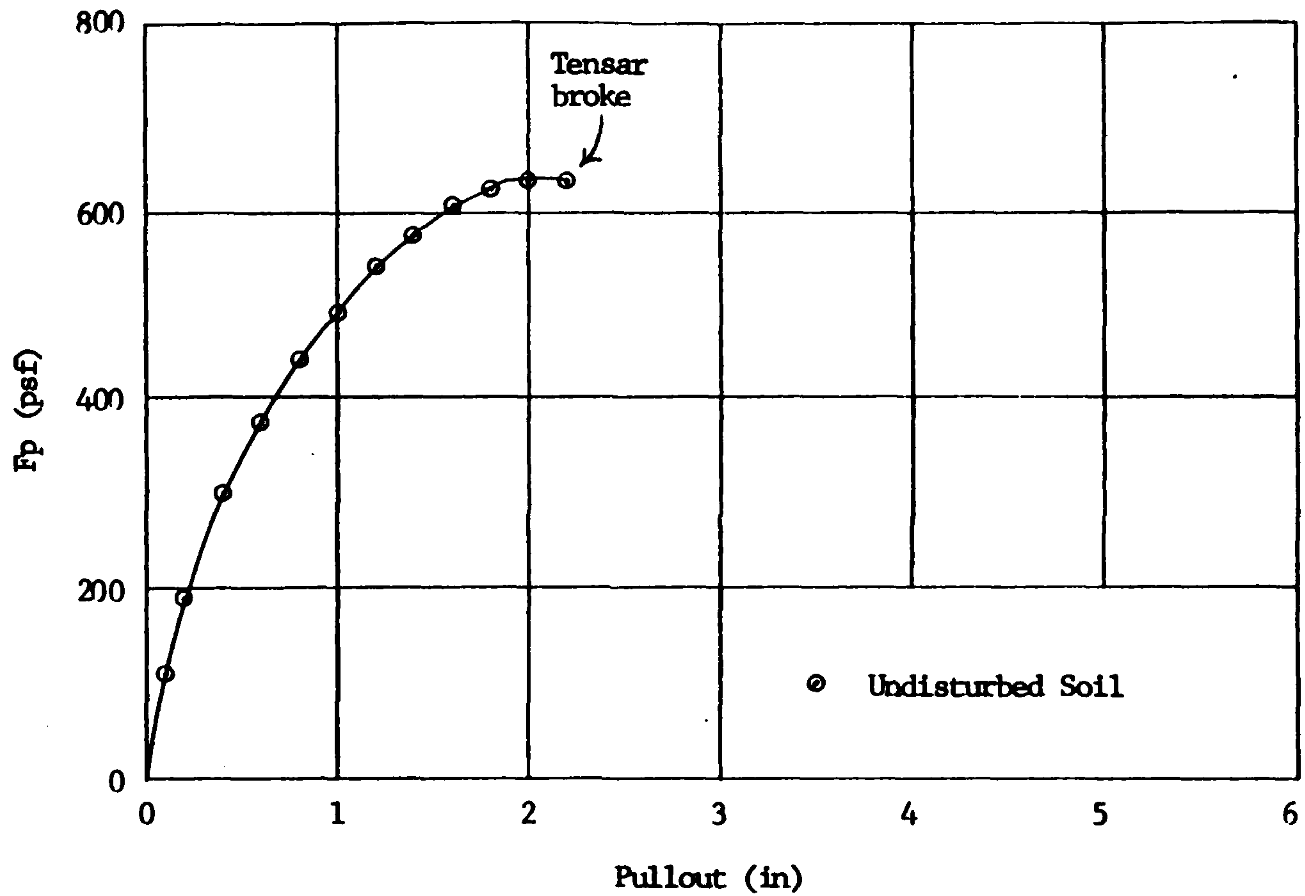
Average Vertical Pressure = 330.2 psf

Fig. 25. Fp vs. Pullout, SS1-3-a



Average Vertical Pressure = 418.8 psf

Fig. 26. Fp vs. Pullout, SS1-3.8-a and SS1-3.8-b



Average Vertical Pressure = 551.8 psf

Fig. 27. Fp vs. Pullout, SS1-5-a

Along with the stress distribution of the mat, the linear dimensions can be obtained. This information can be combined with the modulus of the SS-1 material to give the % elongation at various points along the mat or total elongation of the entire mat at anytime during the testing. These elongation values can be subtracted from the corresponding pullout value to give you the amount of movement that took place between the Tensar and the soil at various places in the chamber.

No attempt is made to seperate this elongation of the Tensar in presenting the test results. A sample calculation of the SS1-2-a test, however, reveals that when Fp peaked at 2.2 inches of pullout, the average movement between the Tensar and the soil was only about 1.3 inches. This calculation is illustrated in the Appendix. The elastic rebound was also measured on several tests by releasing the jacks and thus the stress in the mat after 6 inches of pullout and measuring the distance the clamp traveled backwards. This rebound distance was proportionate with the stress in the Tensar at the time of release, ranging from .75 inches after the SS1-2.5-b test to 2.0 inches for the SS1-3.8-a test. These values demonstrate the magnitude of elongation for the SS-1 material.

Analysis of Test Results

The results just presented are now used to analyse different aspects of the tests and determine certain relationships. Table 2 is a summary of peak Fp's for each test on both undisturbed and disturbed soil. These are plotted in Figure 28 versus Pv.

Table 3 is a summary of estimated ultimate Fp's. These estimates were made by assuming that the ultimate Fp for both the disturbed and undisturbed soil were the same, even though the ultimate may not have been reached at 6 inches of pullout. Figure 29 shows the plot of ultimate Fp versus Pv.

Possibly, more useful than the peak or ultimate pullout strength is the strength at a designated pullout. The Federal Highway Administration Guidlines require that the design pullout strength not be greater than that of .75 inch deformation or the ultimate pullout force divided by 1.5. The Fp's attained in undisturbed soil at .25 inch, .50 inch, .75 inch, and 1.0 inch are listed in Tables 4 and 5. These are plotted against Pv in Figures 30 and 31.

Another interesting result to analyze is at what deformation did Fp peak. The deformation at peak Fp was plotted against Pv for both undisturbed and disturbed soil. Significant scatter was evident. This could be attributed to Fp being close to peak for a long time before or after reaching peak, especially at the higher Pv values. It was thought there would be less scatter if 95% or 90% of peak Fp was used. Thus, the measured deformations for undisturbed and disturbed soil at peak, 95% of peak, and 90% of peak Fp are listed in Tables 6 and 7. These are then plotted versus Pv in Figures 32 and 33.

Table 2 Summary of Peak Fp

Pv (psf)	Peak Fp (psf) Undisturbed	Peak Fp (psf) Disturbed
101.5	121.6	89.6
219.5	285.1	212.2
277.1	325.6	274.6
330.2	418.1	-
418.8	475.9	359.5
551.7	+634.0 *	-

* Peak load at time of rupture (load was still increasing)

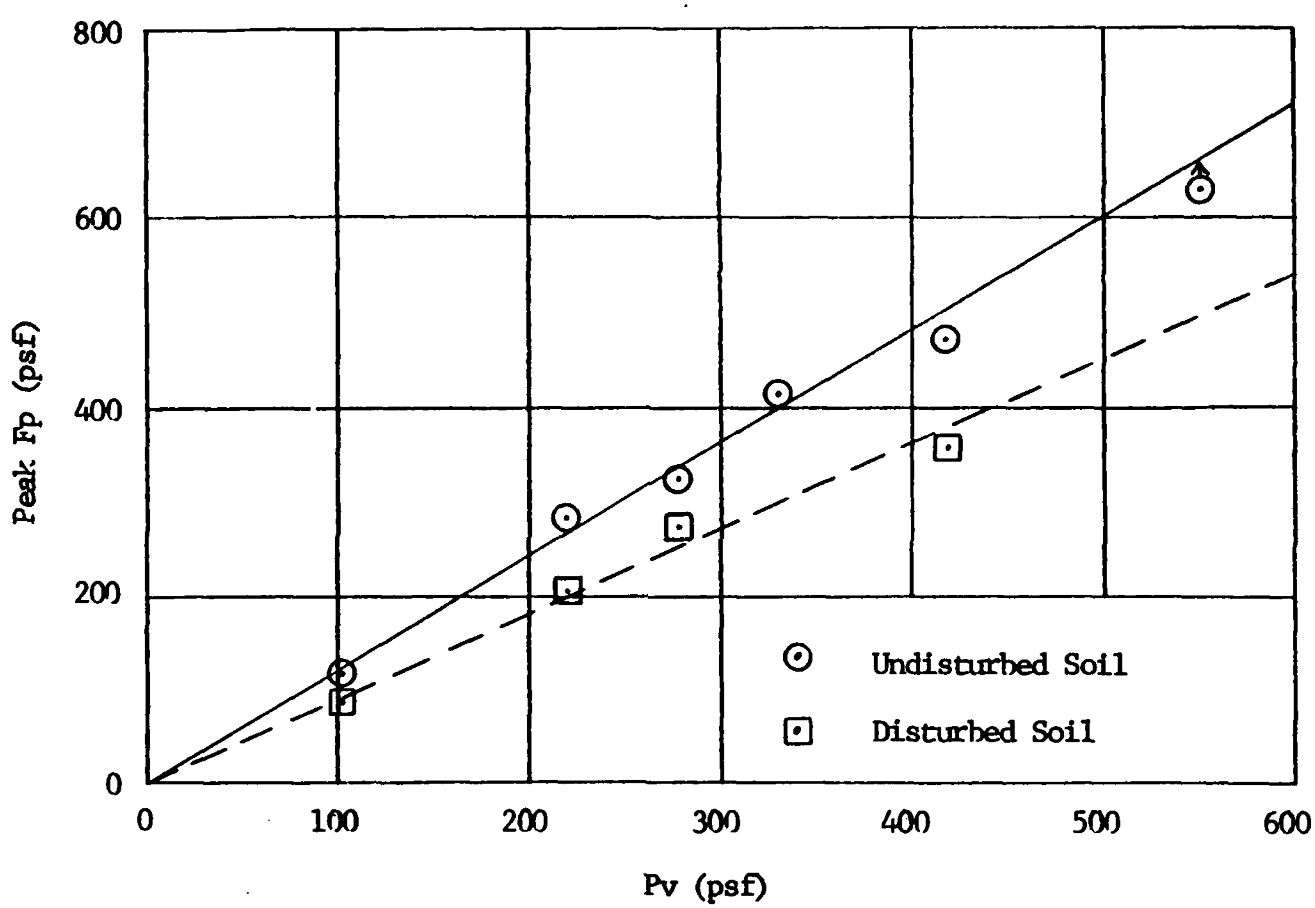


Fig. 28. Peak Fp vs. Pv

Table 3 Summary of Ultimate Fp

Pv (psf)	Ultimate Fp (psf)
101.5	80
219.5	180
277.1	200
330.2	225
418.8	250

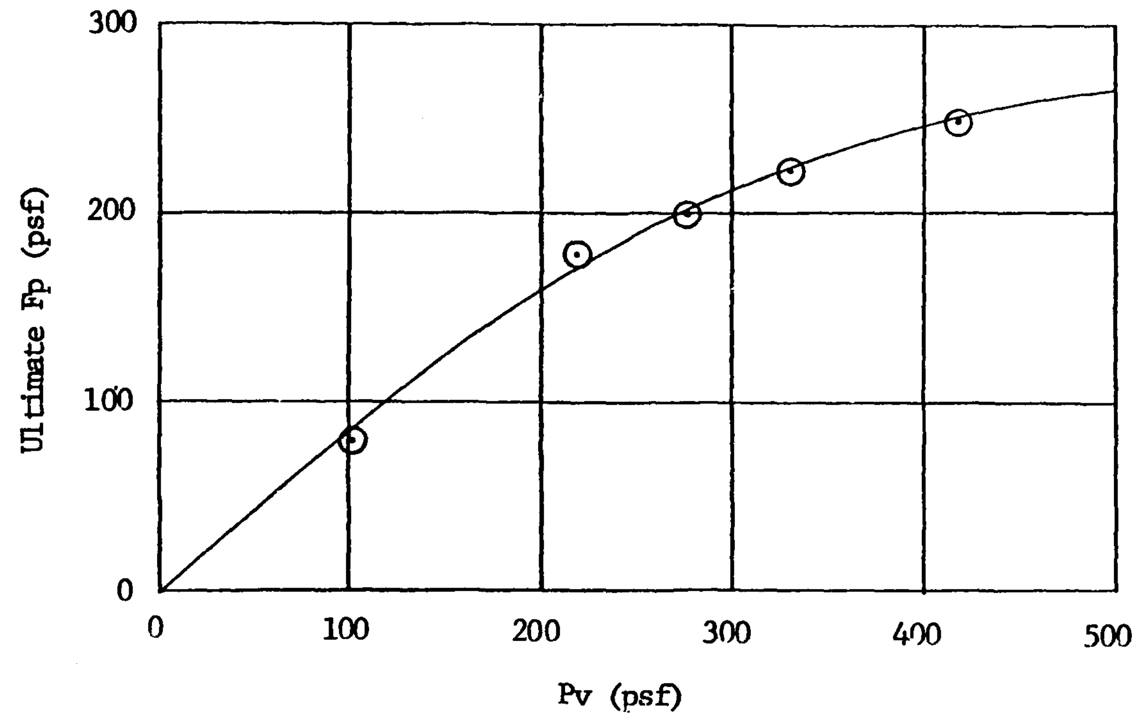


Fig. 29. Ultimate Fp vs. Pv

Table 4 Fp at .25 in. and .50 in. Deformation

Pv (psf)	Fp (psf) of Undisturbed Soil at: .25 in.	.50 in.
101.5	114	119
219.5	130	195
277.1	125	190
330.2	135	210
418.8	135	210
551.7	210	340

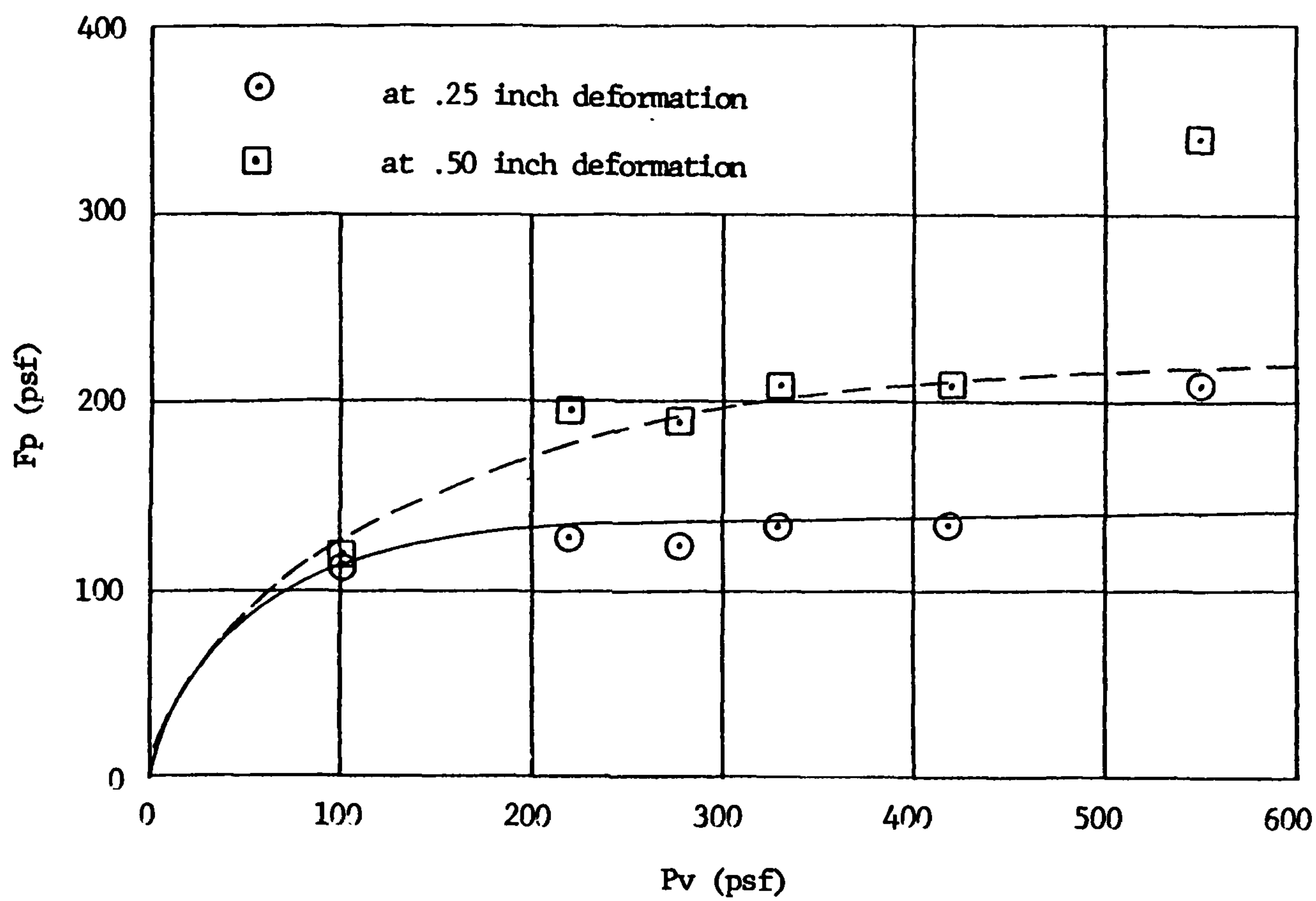


Fig. 30. Fp at .25 in and .50 in Deformation vs. Pv

Table 5 Fp at .75 in. and 1.0 in. Deformation

Pv (psf)	Fp (psf) of Undisturbed Soil at: .75 in.	1.0 in.
101.5	114	110
219.5	240	264
277.1	238	272
330.2	266	306
418.8	267	311
551.7	425	493

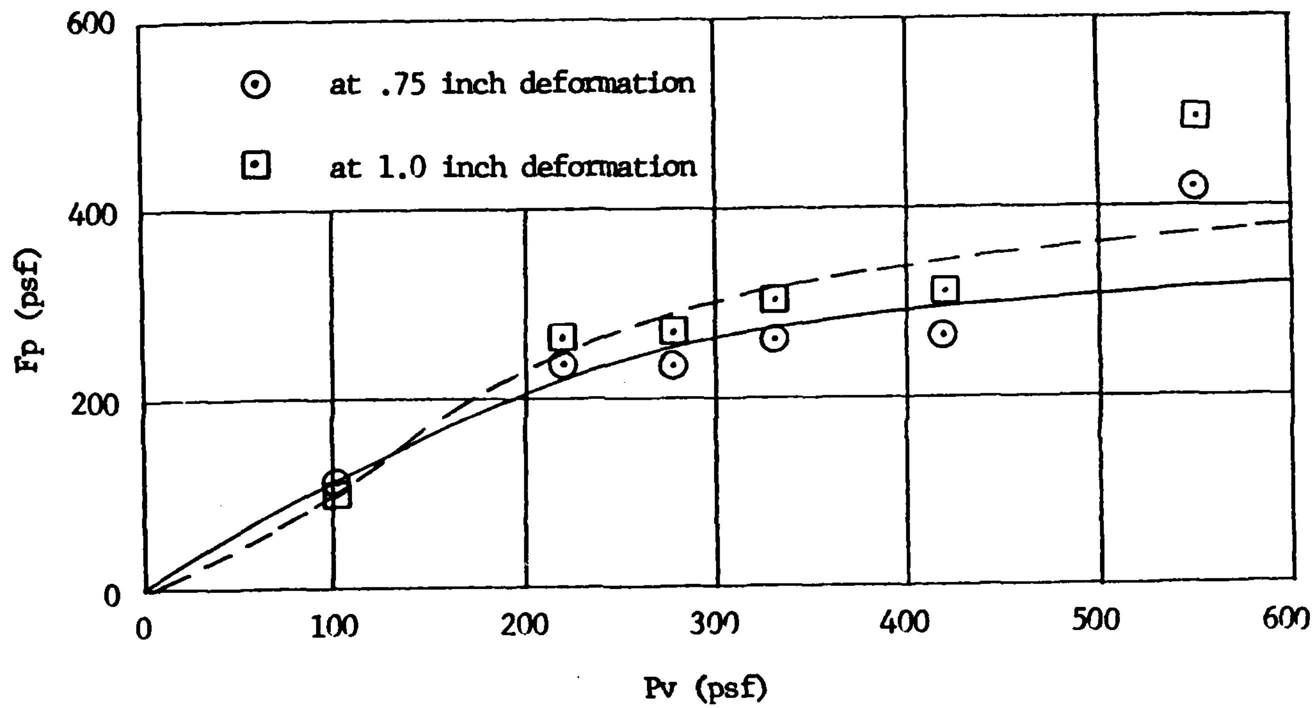


Fig. 31. Fp at .75 in. and 1.0 in. Deformation vs. Pv

Table 6 Measured Deformations for Undisturbed Soil

Pv (psf)	Deformation (inches) for Undisturbed Soil at: Peak Fp	95% of Peak Fp	90% of Peak Fp
101.5	.3	.3	.2
219.5	2.2	1.1	.9
277.1	2.2	1.4	1.1
330.2	3.3	2.4	1.5
418.8	3.8	2.3	1.9

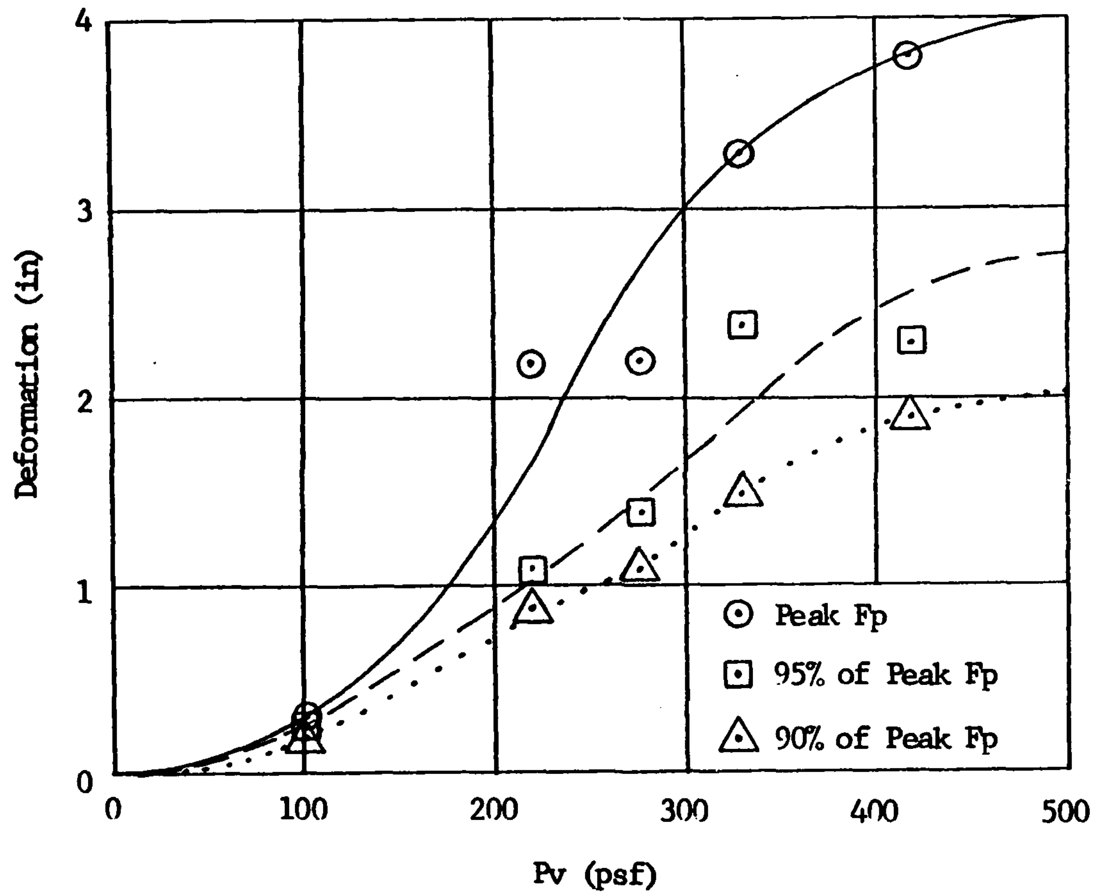


Fig. 32. Measured Deformation vs. Pv for Undisturbed Soil

Table 7 Measured Deformations for Disturbed Soil

Py (psf)	Deformation (inches) for Disturbed Soil at: Peak Fp	95% of Peak Fp	90% of Peak Fp
101.5	1.1	.8	.5
219.5	3.2	2.0	1.6
277.1	2.9	2.0	1.6
418.8	3.5	2.9	2.0

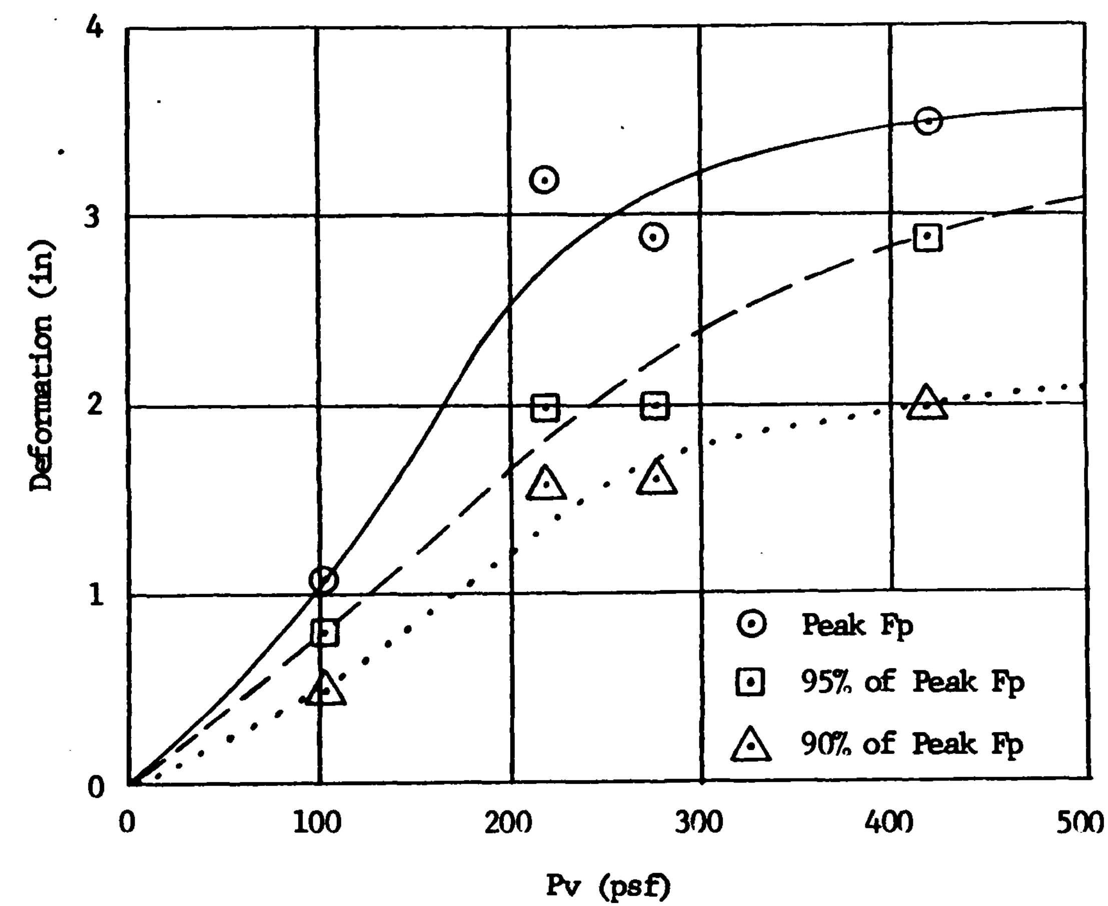


Fig. 33. Measured Deformation vs. Pv for Disturbed Soil

Conclusions and Comparisons

A linear relationship between peak Fp and Pv appears to be standard for all reinforcing members in sand. The linearity for the SS-1 grid in the undisturbed sand tested can be developed from Figure 28 and expressed in equation form as:

$$F = 1.21\ (Pv)\ (A) \qquad \text{(equation 1)}$$

where: F = Peak pullout force in lbs.

Pv = Vertical pressure in psf.

A = Area of grid embedded in sand in square ft.

Mr. Wittrock established a linear relationship when he tested the welded wire mesh. (reference 9) Yuen Zehong's preliminary results of the SR-2 Tensar grid also show a linear relationship between peak Fp and Pv using the same expression as equation 1 except the proportional constant of 1.21 was changed to 1.10. The increase in pullout strength of the SS-1 grid versus the SR-2 grid can be attributed to the different grid geometries of the two materials and their corresponding interaction with sand.

Jewell, Milligan, and Sarsby developed a qualitative model which theoretically relates soil and reinforcement parameters to peak pullout strength. (reference 4) They define a term called the coefficient of bond strength which is the same as the proportional constant used in equation 1. The various geometric parameters of the SS-1 material and the concrete sand parameters that are used by the model were determined. As shown in the Appendix, a theoretical bond strength coefficient of 1.28 was reached using their method. This compares to the constant of 1.21 attained thru this testing.

The testing also suggests that a linear relationship exists between peak Fp and Pv when the SS-1 grid is pulled thru disturbed sand. (Figure 28) Ultimate Fp versus Pv does not appear to be linear according to the testing performed. As Pv got higher, ultimate Fp leveled off. (Figure 29)

As mentioned earlier, several design methods limit the amount of deformation allowed in computing pullout strengths. Figures 30 and 31 suggest that Pv has little effect on Fp at a designated deformation. Once a small overburden pressure exists, the pullout strength for a specified deformation remains relatively constant, even at high overburden pressures.

The mat failure during the SS1-5-a test illustrates another point. The specified peak tensile strength of the SS-1 material in the direction tested was 1400 lbs. per linear foot of mat. At only 5 ft. of overburden and 30 in. of embeddment depth, the interlock or pullout resistance of the mat in sand exceeded the tensile strength. The SR-2 material tested had a much higher tensile strength and could thus be tested at greater overburden pressures.

References

1. Wittrock, M. A., Soil Reinforcement, University of New Mexico, Spring, 1986.

2. Tumay, M. T., Reinforced Earth and Soil Nailing Seminar, Louisiana State University, April 13-15, 1983.

3. McGown, A., Paine, N. and Dubois, D., Use of Geogrid Properties in Limit Equilibrium Analysis, University of Strathclyde, Binnie and Partners, Netlon Limited.

4. Jewell, R., Milligan, R., Sarsby, R. and Dubois, D., Interaction between Soil and Geogrids, Binnie and Partners, University of Oxford, Bolten Institute of Higher Education, Netlon Limited.

5. Tensar Technical Note: BR2, Flexible Pavement Design Guide.

6. McGown, A., Andrawes, K. Y. and Dubois, D., The Load-Strain-Time Behavior of Tensar Geogrids, Department of Civil Engineering, University of Strathclyde, Netlon Limited.

7. Various Tensar Sales and Technical Literature, Tensar Corporation.

8. Carney Jr., J. B., A Test Chamber for the Determination of the Pullout Resistance of Soil Reinforcing Members, Department of Civil Engineering, University of New Mexico, March, 1986.

9. Wittrock, M. A., Evaluation of Welded Wire Mesh Soil Reinforcing Elements in a Load Test Chamber, University of New Mexico, March, 1986.

Appendix

Tensar

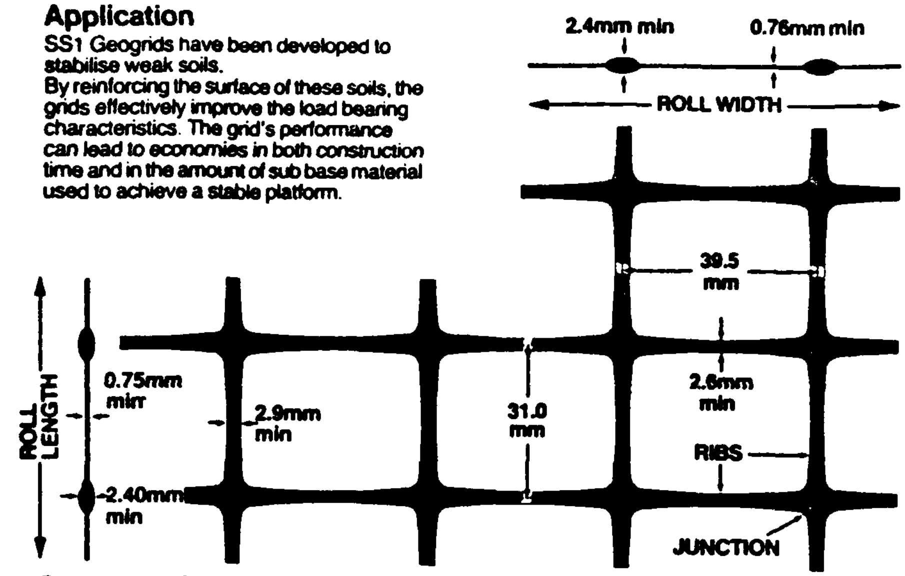
FROM NETLON ®

Provisional Data Sheet

SS1

Application

SS1 Geogrids have been developed to stabilise weak soils.
By reinforcing the surface of these soils, the grids effectively improve the load bearing characteristics. The grid's performance can lead to economies in both construction time and in the amount of sub base material used to achieve a stable platform.



Structural Characteristics:

Roll length (m)	: 50.0
Roll width (m)	: 3.0
Approximate grid weight (g/m²)	: 203.0
Colour	: Black

Mechanical Properties:

	Across Roll width	Along Roll length
Characteristic tensile strength per metre width (kN/m)	20.9	12.6

NB. Samples, 3 junctions long and 1 rib wide were extended at a constant rate of 50mm/min, at a temperature of 20±1°C.

Raw Material – Physical & Chemical Properties:

Polymer		: Polypropylene
Shore Hardness D	(Din 53505)	: 74
Vicat Softening Point	(Din 53460) (°C)	: 148
Impact Strength	(Din 53453) (kJ/m²)	: 4.5
Abrasion Resistance	(Din 53754E) (mm³/100 revs)	: 14.0
Chemical Resistance		: Resistant to all natural occuring alkaline and acidic soil conditions
Biological Resistance		: Resistant to attack by bacteria, fungi and vermin
Sunlight Resistance		: Resistant to UV attack

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Ref: 1/82

STRENGTH VALUES OF SS-1 GEOGRIDS

MD - Machine Direction which is along roll legnth.

CMD - Cross Machine Direction which is across roll width

The testing of the SS-1 grid was performed in the Cross Machine Direction

Peak Tensile Strength	MD	840 lbs/fft
Tensile Strength at 2% Strain	MD	280 lbs/ft
Tensile Strength at 5% Strain	MD	570 lbs/ft
Initial Tangent Modulus	MD	30,000 lbs/ft
Peak Tensile Strength	CMD	1400 lbs/fft
Tensile Strength at 2% Strain	CMD	400 lbs/ft
Tensile Strength at 5% Strain	CMD	830 lbs/ft
Initial Tangent Modulus	CMD	49,000 lbs/ft

SAMPLE CALCULATION OF TENSAR ELONGATION FOR SS1-2-a TEST

At 2.2 in of pullout (peak)

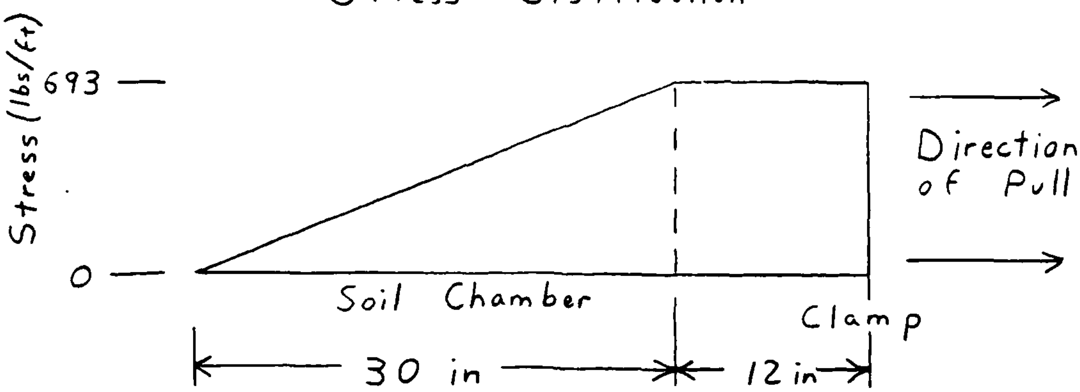
$$\left.\begin{array}{l}\text{load cell \#1} = 605 \text{ lbs} \\ \text{load cell \#2} = 550 \text{ lbs}\end{array}\right\} \text{Fig. 14}$$

mat width = 20 in

$$\text{Max. Stress} = \frac{605 \text{ lbs} + 550 \text{ lbs}}{20 \text{ in}} = 57.75 \frac{\text{lbs}}{\text{in}} = 693 \frac{\text{lbs}}{\text{ft}}$$

$$\text{Avg. Stress in Soil Chamber} = \frac{693 + 0}{2} = 346.5 \frac{\text{lbs}}{\text{ft}}$$

Stress Distribution



Elongation Modulus: reference CMD values on previous page

$$\text{Initial Tangent Modulus} = 49{,}000 \text{ lbs/ft}$$

$$\frac{400 \text{ lbs/ft}}{.02\ \%\ \text{Strain}} = 20{,}000 \text{ lbs/ft}$$

$$\frac{830 \text{ lbs/ft}}{.05\ \%\ \text{Strain}} = 16{,}600 \text{ lbs/ft}$$

Use modulus of 17,500 lbs/ft for stress = 693 lbs/ft

Use modulus of 25,000 lbs/ft for stress = 346.5 lbs/ft

$$\text{Total Elongation} = \left[\frac{693 \text{ lbs/ft}}{17{,}500 \text{ lbs/ft}} \times 12 \text{in}\right] + \left[\frac{346.5 \text{ lbs/ft}}{25{,}000 \text{ lbs/ft}} \times 30 \text{ in}\right]$$

$$= .48 \text{ in} + .42 \text{ in} = .90 \text{ in}$$

Movement between Tensar and Soil = 2.2 in − .9 in = $\underline{\underline{1.3 \text{ in}}}$

CALCULATING THEORETICAL BOND STRENGTH COEFFICIENT $(f_b \cdot \tan \phi)$ USING REFERENCE 4

$$f_b = \alpha_s \frac{\tan \delta}{\tan \phi} + \frac{\sigma_b'}{\sigma_n'} \frac{B}{S} \frac{\alpha_b}{2 \tan \phi} \quad \text{(eq. 18, reference 4)}$$

$\phi = 45°$ (angle of friction of sand - obtained from reference 9)

$\frac{\sigma_b'}{\sigma_n'} \approx 60$ (Fig. 15, reference 4) (ratio of effective bearing stress over normal effective stress between soil and reinforcement)

$B \approx .07$ in (thickness of bearing members)

$S \approx 1.55$ in (spacing between bearing members)

$\frac{\delta}{\phi} = .5$ to $.8$ (p. 19, reference 4)

$\delta = 22.5°$ to $36°$, Use $30°$ (angle of friction between soil and the reinforcement surface)

$\alpha_b = \frac{1.0}{1.2} = .83$ (% of grid width over which bearing surface extends)

$\alpha_s = \frac{.2\,(1.2 + 1.55)}{1.55 \times 1.2} = .2957$ (Method 1)

$\alpha_s = \frac{(1.2 \times 1.55) - (1.0 \times 1.35)}{1.55 \times 1.2} = .2742$ (Method 2)

Avg. $\alpha_s = .28$ (% of grid surface area that is solid)

$$f_b = .28\frac{\tan 30^\circ}{\tan 45^\circ} + 60\left(\frac{.07}{1.55}\right)\frac{.93}{2\tan 45^\circ} = 1.28$$

($f_b \times \tan \emptyset = 1.28$) correlates with the proportional constant obtained in equation 1 of this paper (1.21)

END

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