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Pullout Resistance of Welded Wire Mats Embedded in Soil

Mark R. Nielsen

Utah State University

PULLOUT RESISTANCE OF WELDED WIRE MATS
EMBEDDED IN SOIL

by

Mark R. Nielsen

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Mark R. Nielsen

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ABSTRACT

Pullout Resistance of Welded Wire Mats Embedded in Soil

by

Mark R. Nielson, Master of Science

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Major Professor: Dr. Loren R. Anderson

Department: Civil Engineering

Welded wire mesh has been used in the past as soil reinforcement in the construction of reinforced soil embankments. Involved in the design of these embankments is the external and internal stability. The internal stability has two failure mechanisms; tension failure and pullout failure of the welded wire mesh.

This paper presents the results of laboratory tests on different sizes of welded wire mats embedded in different types of soils. These tests were performed on mats that are much larger than in previous tests. These tests measured the pullout resistance as a function of the number of embedded wires, the diameter of the wire, and the overburden pressure. This data is plotted to allow design of reinforced soil embankments in various types of soils and to compare the results with theoretical relationships.

(137 pages)

CHAPTER I

INTRODUCTION

Problem

Background of reinforcing soil

Reinforcing soil is a method of construction which consists of soil, reinforcements, and facing elements. Reinforcements include rods, fibers, strips, and meshes. Reinforced soil acts very similarly to reinforced concrete, except for the fact that soil is more spatially variable in its properties than concrete. The reinforcement in the soil adds tensile strength to the soil in much the same way as reinforcing bars add strength to concrete.

Reinforced soil is not a new idea, but has been used many times in the past. The quality of adobe brick has been improved by adding straw. Roads through swampy areas are often constructed on a foundation of small tree trunks and branches called corduroy roads. Many miles of low dikes were built in Canada using mud and sticks. Vidal (1969) was the first person to formalize a rational design procedure for reinforcing soil. His patented process is referred to as Reinforced Earth.

Since its formalization by Vidal (1969), Reinforced Earth has become a popular construction method. There have been hundreds of Reinforced Earth structures built throughout the world. New knowledge, new techniques, new hypotheses, and new theories are con-

tinually being added to the basic concepts which were pioneered and promoted by Vidal. Recently many new methods for reinforcing soil have come about which use different types of reinforcement than in Reinforced Earth.

Welded wire wall

One of the methods of reinforcing soil is to use welded wire mats, made of smooth wires and bars, as the reinforcement. Mr. William Hilfiker was the originator of this concept and patented his idea in 1978. Since the first patent in 1978, many different ideas have come about for using the welded wire mats.

The first walls which were constructed consisted of 9 gage or 7 gage wires welded in a 2 inch by 6 inch mesh. The facing elements were also composed of the same wire mesh. Since the first walls were built, the meshes have increased in size and different facing elements are now used. Bars as large as 3/8 inch diameter are being used and precast reinforced concrete panels are used as facing elements.

Many tests have been run on the welded wire walls (Bishop and Anderson, 1979; Peterson and Anderson, 1980). The first instrumented wall was built in the Angelan National Forest for Southern California Edison Company. Since that first wall, laboratory tests have been conducted and other walls have been instrumented. Figure 1 shows a pictorial sketch of a welded wire retaining wall and the different components which make up the wall.

Statement of the problem

Lee (1978, p. 66) stated, "To my mind, the most fundamentally

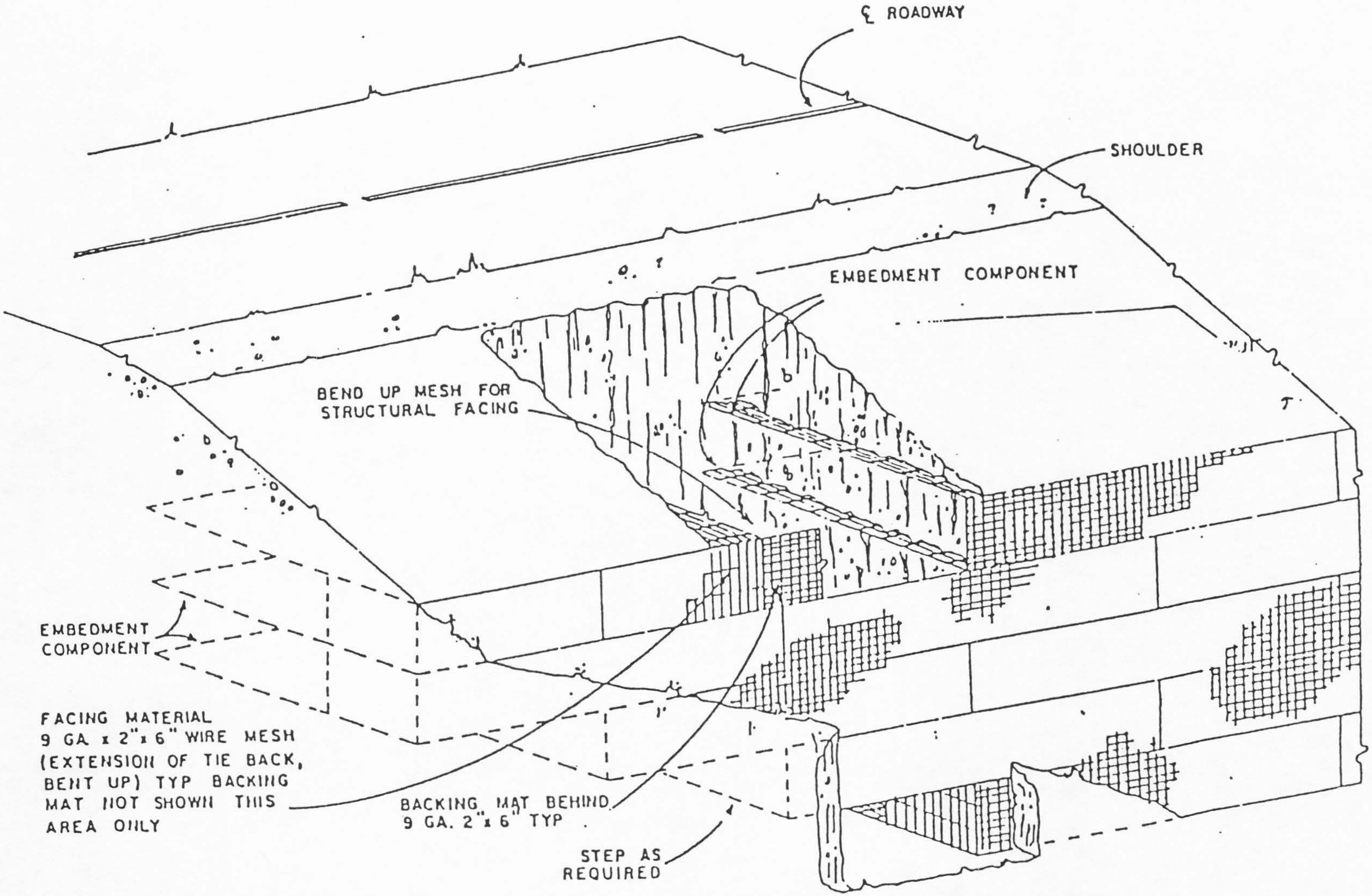


Figure 1. Pictorial sketch of welded wire wall
After Peterson and Anderson (1980)

important, the most critical, and the least understood aspect of reinforced earth in any form is the mechanism of sliding shear resistance between soil backfill and the tensile reinforcing elements." This statement clearly indentifies pull out resistance of the reinforcement as the number one topic for research on reinforcing soil.

Lee (1978) also indentified other topics which need to be addressed pertaining to reinforcing soil. These topics include:

1. Fundamental behavior mechanisms and practical design procedures.
2. Long term durability or corrosion of reinforcing materials.
3. Backfill of cohesive soil or soil with fines.

Along with these topics, Lee also stressed the need for research on all different types of reinforced earth. Much of the research which has been done is for strip reinforcement and may not apply to other types of reinforcement. Therefore, the need for research on the welded wire wall is necessary to ensure correct design procedures and safe retaining walls.

Welded wire retaining walls are used extensively in some parts of the country. This study which will look at the pull out resistance of welded wire mats in different types of soils will be beneficial in the design of future walls.

Purpose of Study

Objectives

There are four objectives involved in this study of pull out resistance.

1. Find the best relationship for pull out resistance for each soil type.

2. Verify that the pull out resistance is a function of the number of transverse wires.

3. Verify that the pull out resistance is a function of the diameter of wire used.

4. Relate the pull out resistance to basic soil parameters.

Scope

In satisfying these four objectives, twenty-nine welded wire mats were pulled in a full scale testing apparatus. A total of ninety-three tests were run on four different diameters of wire mesh. The majority of the tests were run with 1/4 inch diameter and 3/8 inch diameter wire mesh. These tests were run with overburden pressures ranging from 600 pounds per square foot (psf) to 4000 psf.

With the data collected from the pull out tests, many plots were made and theoretical relationships were examined to obtain the best relationships between pull out resistance and mat parameters, such as diameter and number of transverse wires. With this data, design procedures were developed for pull out resistance in granular soils.

CHAPTER II

LITERATURE REVIEW

Introduction

Since its formalization by Vidal (1969), earth reinforcement has had a wide and varied range of applications. Structures have been constructed using earth reinforcement for many different applications, ranging from retaining walls and bridge abutments to roads constructed over swampy land. The research for earth reinforcement covers a myriad of topics. Some researchers have discussed laboratory tests on reinforcement pullout or friction between earth and reinforcement, others have discussed the intricate mathematics that are involved in the theoretical analysis of a reinforced earth wall, while others have instrumented actual walls to determine how well the lab and theory predict the results.

This paper will not cover all the topics that could be discussed in dealing with earth reinforcement. The main topic that will be covered will deal with the pullout resistance of welded wire mats. Some of the material that comes from research on Reinforced Earth walls, but pertains to the welded wire walls will be presented in this review.

Pullout Resistance of Welded Wire Reinforcement

General

A reinforced soil wall structure needs to meet the requirements of two different design criteria. One is the external stability of the structure which includes sliding, overturning, bearing capacity, and deep stability. The second is the internal stability which deals with the pullout resistance and tension capacity of the reinforcement.

The internal stability of reinforced soil retaining walls consists of two different modes of failure. One mode is when the reinforcement breaks. The size of reinforcement that is necessary for any specific case is easily determined if the load that is to be applied to the reinforcement is known. This type of failure is very critical because when one reinforcement tie breaks, the load is transmitted to the remaining ties and could cause a continuing progressive failure of the entire wall (Binquet, 1978).

The second mode is tie pull out which is far more complicated than the first mode. If one layer of reinforcement fails by pulling out, the entire load will not be transferred to the other reinforcement (Binquet, 1978). Therefore, the failure of one tie may not lead to a total failure of the wall, but may lead to excessive deformations of the wall in certain places.

With the welded wire walls having the transverse wires as well as longitudinal wires, the pullout resistance is more complicated than that which has been researched for Reinforced Earth walls. All of the resistance for Reinforced Earth walls depends on frictional

resistance between the soil and the reinforcement. However, for the welded wire wall, the resistance depends on both friction and the resistance provided by the transverse wires. From past tests, the resistance of th transverse wires is far greater than the frictional resistance of the longitudinal wires (Bishop and Anderson, 1979).

Frictional resistance

The frictional resistance developed with a welded wire mat is due to the friction between the soil and reinforcement on the longitudinal wires. This friction is a function of the overburden pressure, the angle of friction between the soil and reinforcement, and the surface area of the reinforcement. In the case of Reinforced Earth, where thin metal strips are used as reinforcement, the friction is easier to calculate because the strips are very thin and the friction only occurs on the top and bottom of the strips. However, with a wire mat, the reinforcement is circular and the overburden pressure changes over the circumference of the wire.

From a theoretical view point, the frictional resistance could be obtained by using the average stress over the circumference of the wire and the friction factor between the soil and the reinforcement from the published tables. The friction factor could be estimated by using tables such as the one in the Navy Docks Manual (NAVFAC DM-7, 1971). Table 1 is a part of the table found in the Navy Docks Manual which lists the friction factors of different materials on steel sheet piling.

Table 1. Friction factors for dissimilar materials (After NAV reference FAC DM - 7, 1971)

Interface Materials	Friction Factor, tan δ	Friction Angle, δ, Degrees
Steel sheet piles against the following soils:		
Clean gravel, gravel-sand mistures, well-graded rock fill wil spalls	0.40	22
Clean sand, silty sand-gravel mixture, single size hard rock fill	0.30	17
Silty sand, gravel or sand mixed with silt or clay	0.25	14
Fine sand silt, non-plastic silt	0.20	11

The average overburden pressure is obtained by averaging the vertical and horizontal pressures. The average overburden pressure, assuming the coefficient of earth pressure at rest to be 0.50, would be three-fourths of the vertical pressure, which is easily calculated.

$$\sigma_{ave} = \frac{\sigma_v + 0.50\ \sigma_v}{2} = 0.75\ \sigma_v$$

$$F_f = \pi d\ L \tan\delta\ \sigma_{ave}$$

where:

π = 3.1416

d = diameter

L = length of wire

$\tan\delta$ = friction coefficient between soil and reinforcement

σ_{ave} = average overburden pressure

Vidal (1969) used this same general equation as the theoretical solution to the frictional resistance. Vidal (1969) did not give

suggested values for the coefficient of earth pressure or the average overburden pressure. Vidal (1969) said this equation is pessimistic because it does not take into account the arching effect between adjoining reinforcing bars.

There has been a great deal of research about the frictional resistance between the soil and thin strips of various material (Bacot, Iltis, Lareal, Paumier, Sanglerat, 1978; Binquet, 1978; Dash, 1978; Lee, Adams, Vagneron, 1973; Schlosser and Elias, 1978). The variables which have been tested include density, width of strips, and length of strips. Some results contradict each other, while other results are in very good agreement.

Tests performed in France by Bacot, Iltis, Lareal, Paumier, and Sanglerat (1978) examined the frictional resistance from many different types of tests. Pull out tests performed in an experimental soil embankment that was 2.80 meters high showed that a non-compacted fill had more frictional resistance than a compacted fill. The soil was a glacial deposit that was washed by running water. Their results also showed that as the reinforcing strip widened, the resistance decreased; and as the strip length increased, the resistance increased (Bacot, et al, 1978). In general, the experimental frictional resistance was greater than the theoretical equation predicted.

Schlosser and Elias (1978) performed tests on the frictional resistance of reinforcing strips. Their tests used density and width as the variables. They determined that the higher the density of the soil, the greater the frictional resistance between the soil and the reinforcement. They also found that at low densities the theoretical

and experimental values are close and in some tests, the experimental value was less than the theoretical value. Schlosser and Elias (1978) determined that when the height of their experimental wall was less than eighteen centimeters, the frictional resistance decreased as the width increased. For walls greater than eighteen centimeters, there was no definite relationship between the frictional resistance and the width (Schlosser and Elias, 1978). Frictional resistance on thin strips is a difficult variable to determine strictly from theory, and experimental tests probably give closer results to the actual value in a reinforced earth embankment.

For circular bars, there have been some tests performed to determine the friction between the soil and the reinforcement. Chang, Hannon, and Forsyth (1977) performed a great deal of pullout tests on different types of reinforcement. In these tests, they ran two pullout tests on number three rebar with no transverse wires. These results showed that the frictional resistance of these longitudinal bars were greater than predicted by theory. Table 2 summarizes these results and the predicted theoretical values for the two tests which were run. Figure 2 shows the two different mats and the wire geometry.

Peterson and Anderson (1980) also performed pullout tests with longitudinal bars. These bars were smaller diameter than the bars pulled by Chang et al (1977). Peterson and Anderson (1980) obtained similar results in that the pull out resistance of the bars was greater than predicted by the theoretical equation. Therefore, frictional resistance can be either calculated or experimentally determined , but usually the theoretical solution gives results that

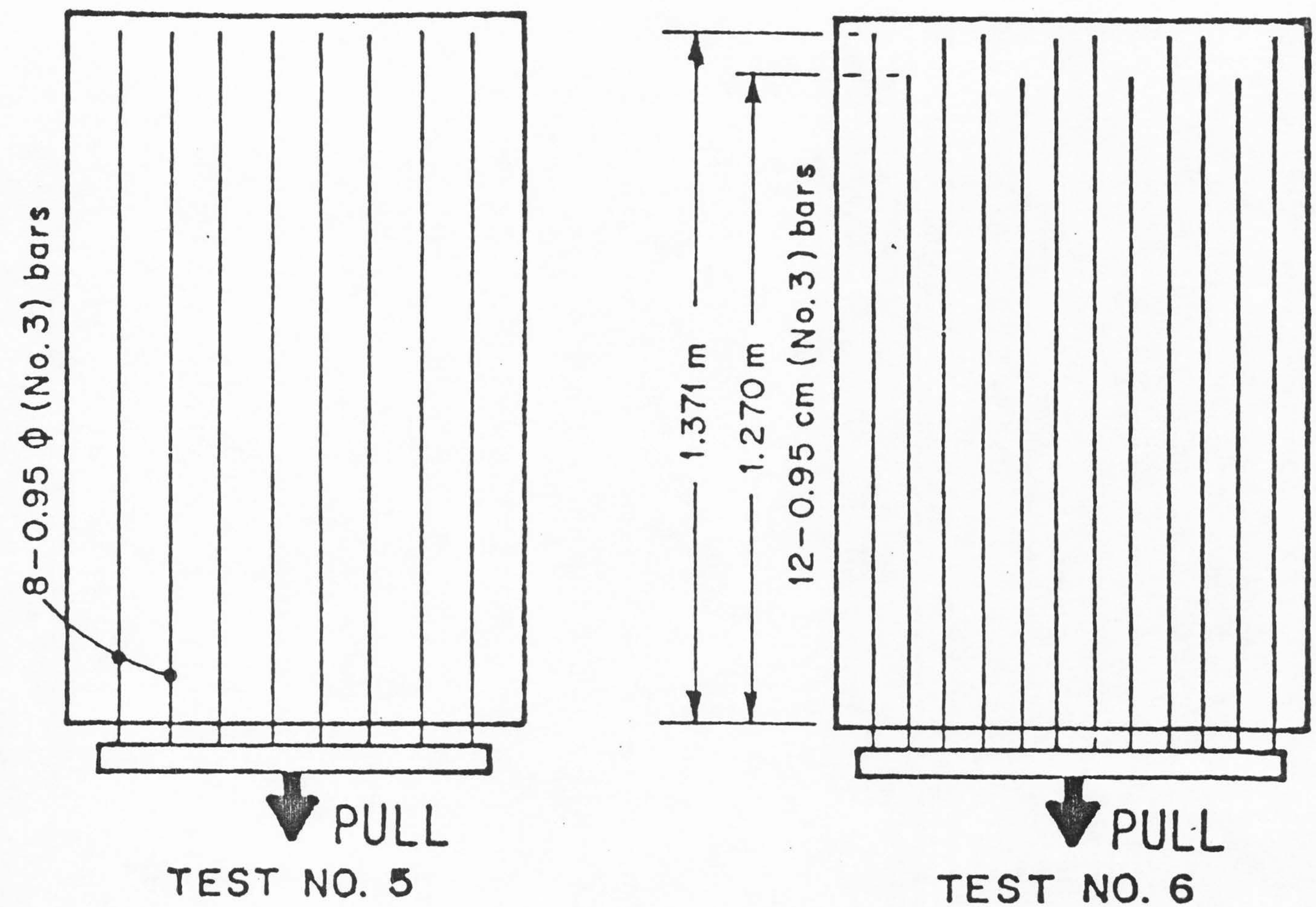


Figure 2. Mat geometry
After Chang, Hannon, Forsyth (1977)

are lower than the actual frictional resistance obtained by laboratory studies.

Table 2. Comparison of theoretical pull resistance to the laboratory pull resistance

Type of Reinforcement	Total Surface Area (ft.^{2})	Under 1462 psf Vertical Pressure (pounds)	Peak Resistance Theoretical Value (pounds)
8 Longitudinal Bar	3.53	3,375	1161
12 Longitudinal Bar	5.17	6,750	1700

Transverse wire resistance

The resistance that is furnished by the transverse wires in a welded wire mat is not well understood. Most of the research which has been done provides empirical results from pullout tests performed in the laboratory. The interaction between the transverse wires and the soil is very complex and is hard to determine theoretically. Unlike the frictional resistance, which is dependent upon the friction angle of the soil, the transverse resistance can be greater for a silty-clay than for a gravelly-sand (Chang et al., 1977).

Chang et al. (1977) performed pullout tests on many different schemes of reinforced earth. Among these were mesh type reinforcing mats. The two different sizes of mats which were used were 3/8 inch diameter bars welded in a 4 inch by 8 inch mesh and in a 5 inch by 14 inch mesh. The pulling aparatus is pictured in Figure 3. The test facility consisted of a rigid steel box, 18 inches high, 36 inches

wide, and 54 inches long. A vertical presure could be applied to simulate up to 50 feet of earth fill. The reinforcing mats were then pulled at a controlled strain rate of two thousandths (0.002) of an inch per minute. The deformations of the reinforcement were measured by two extensometers; one at the front and one at the rear of the mat. The normal and pull loads were measured by load cells.

Some typical results of the pull out tests on the different sizes of mesh are shown in Figure 4a and 4b. Figure 5 shows the relationship between normal load and pull out load for the 4 inch by 8 inch mesh. The results show a linear relationship for the yield, peak, and residual pull loads. This is all of the information that was included by Chang et al. (1977) for mesh reinforcement. Their tests also showed that the peak pulling loads were higher in a dense silty clay soil than in a less dense gravelly sand soil. Also in the same soil, the peak pulling load decreased substantially when the mesh openings increased to the 5 inch by 14 inch mesh. The conclusion of Chang, et al (1977) was that the transverse wires increased the pulling resistance of the reinforcement by about six times compared to the longitudinal bars or strip reinforcement for the same surface area. No design procedures were proposed by Chang et al. (1977) on the basis of their results for bar mesh reinforcement.

The failure mechanism that Chang et al. (1977) observed in their experiments was that with bar mesh reinforcement the reinforcing failed by development of a cone shaped soil wedge while the longi tudinal bars and strips failed by slippage. The development of a soil wedge indicated full mobilization of soil resistance. This

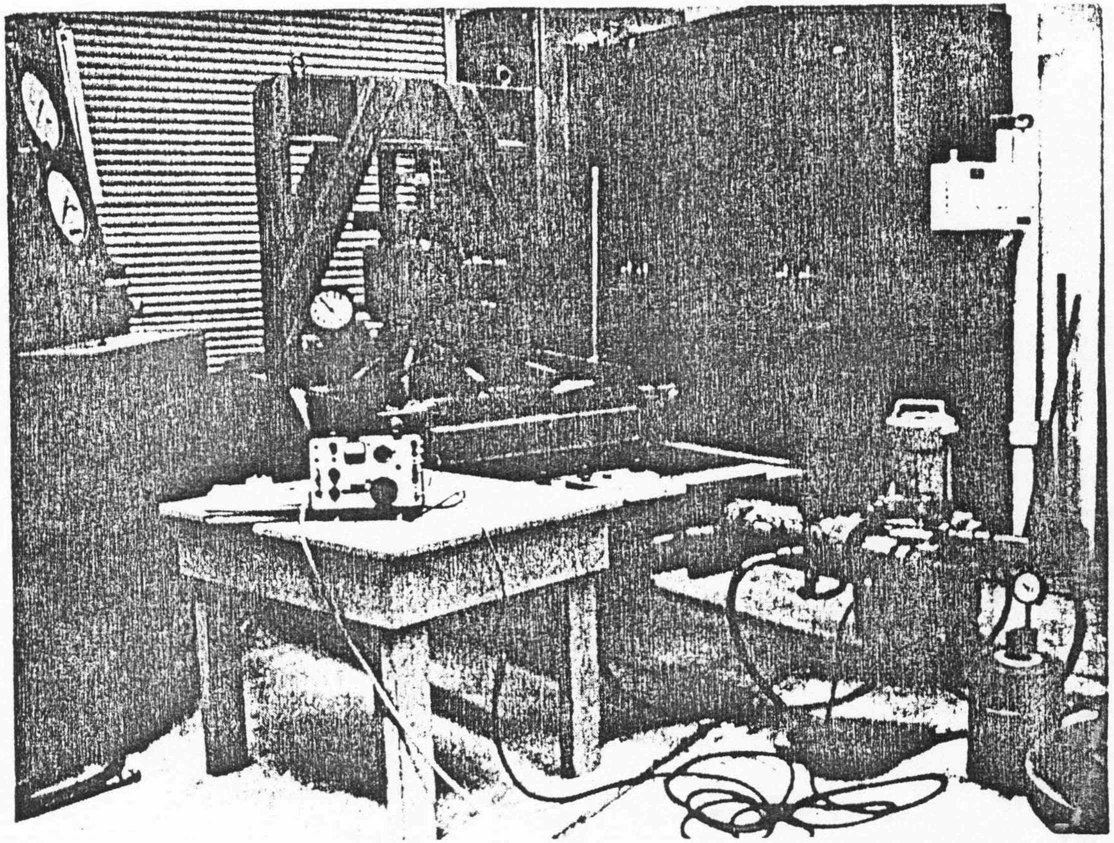


Figure 3. Pulling apparatus
After Chang, Hannon, Forsyth
(1977)

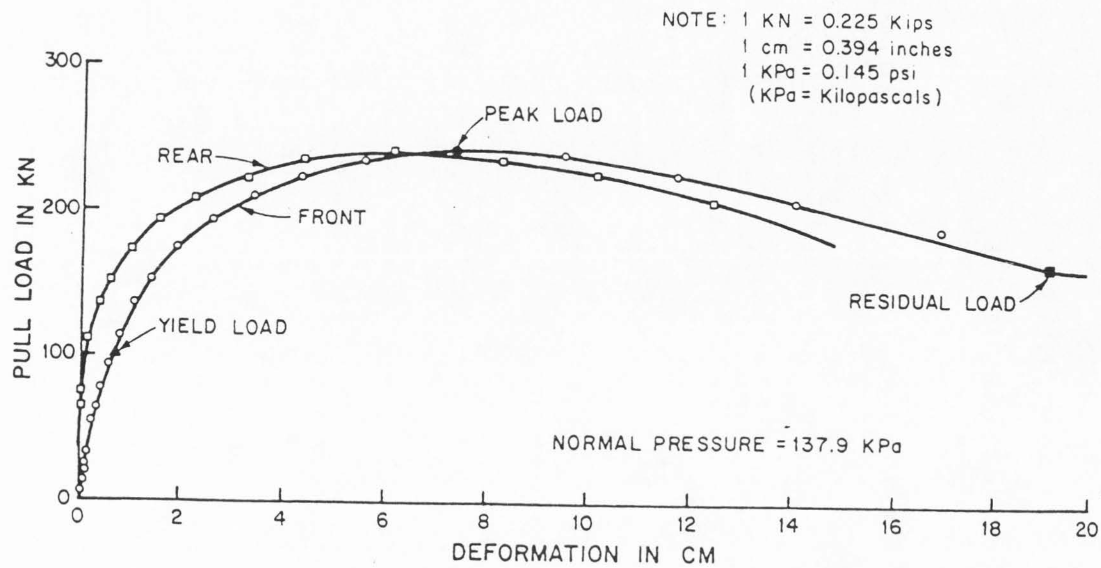


Figure 4a. Load-displacement curves

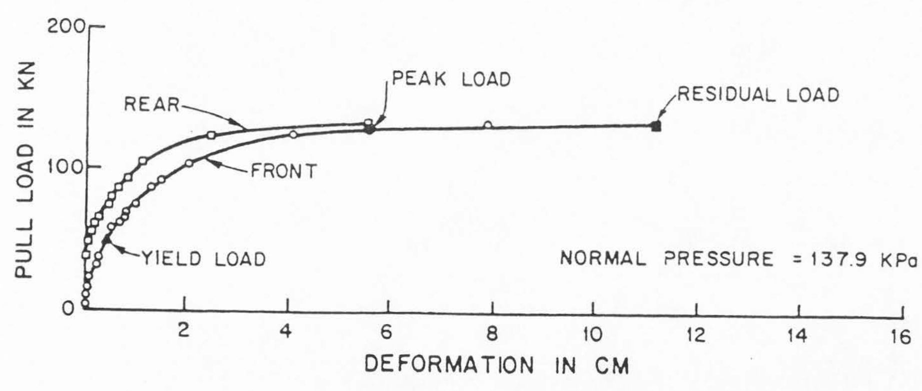


Figure 4b. Load-displacement curves
After Chang, Hannon, Forsyth (1977)

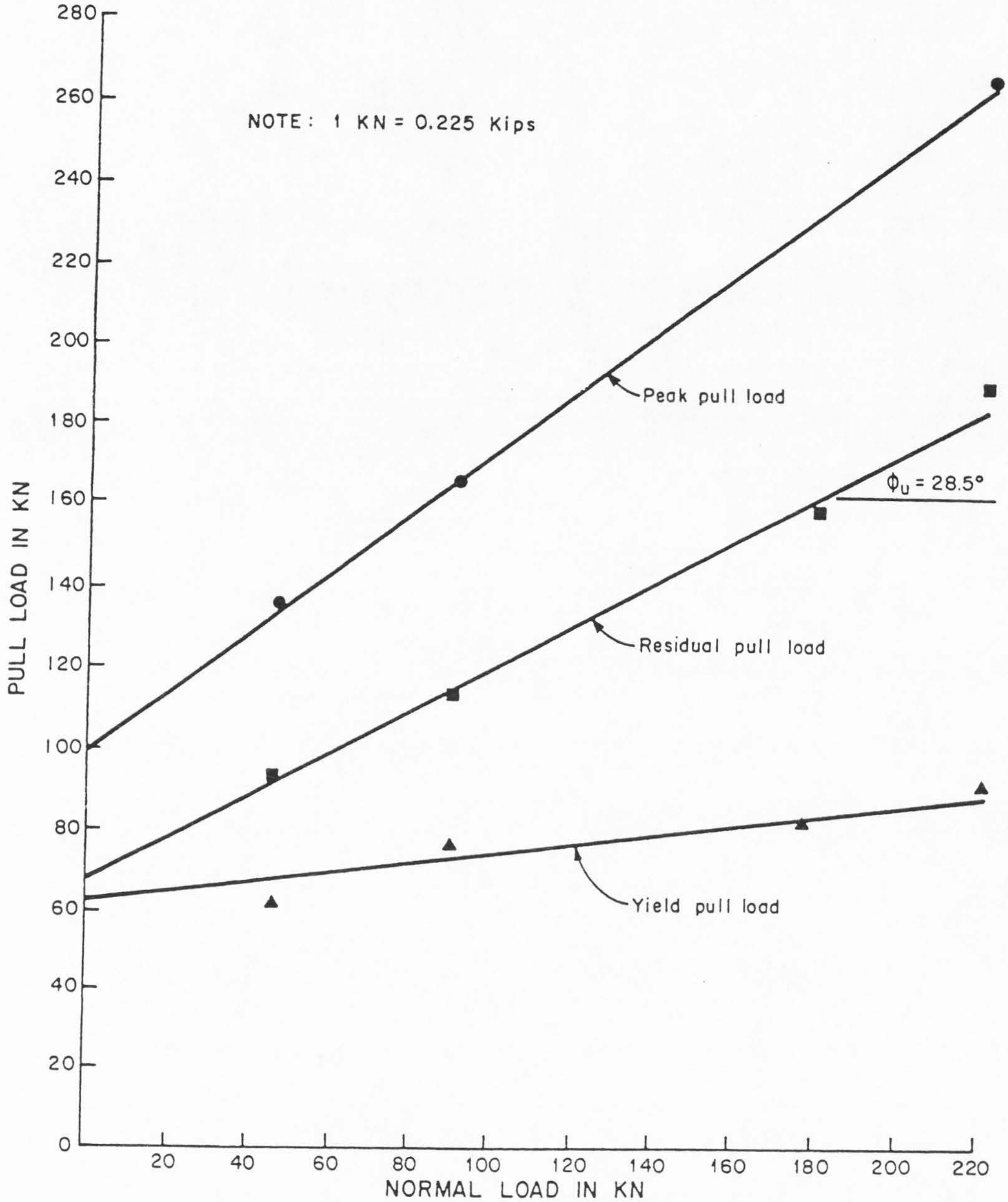


Figure 5. Relationship between normal load and pull load
After Chang, Hannon, Forsyth (1977)

means that the soil and bar mat reinforcement failed as a unit and not individually as did the slippage of the strips.

Bishop and Anderson (1979) also performed some preliminary pull out tests on twelve different mats. These mats consisted of three longitudinal wires with transverse wires welded at various spacings. Bishop and Anderson (1979) plotted their data in two different forms. Figures 6 and 7 show the results from their tests. Bishop and Anderson (1979) indicated that more testing was needed for pull out resistance of welded wire mats. Peterson and Anderson (1980) extended these preliminary results.

Peterson and Anderson (1980) ran extensive pull out tests in a silty sand soil using welded wire mats. The tests were run on mats that were welded into a 2 inch by 6 inch mesh. The wire sizes which were used included 9 gage (0.15 inch), 7 gage (0.177 inch), 5 gage (0.207 inch), and 0.252 inch diameter wire. A total of 73 tests were run and each test was pulled until the mat displaced approximately one-half inch. The tests were run in the buried structures test facility at Utah State University (Figure 8).

The tests were stress controlled and were run by applying a 500 pound seating load, reading the deflection every 30 seconds, and then adding 500 or 1000 pounds increments to the pull out force. The pull out force was measured by a load cell and the normal force was converted from the hydraulic pressure of the cylinders applying the vertical load. The deflection was measured by a dial gage which was accurate to a thousandth (0.001) of an inch.

Peterson and Anderson (1980) plotted their results in a different manner than Chang, et al. (1977) because many different

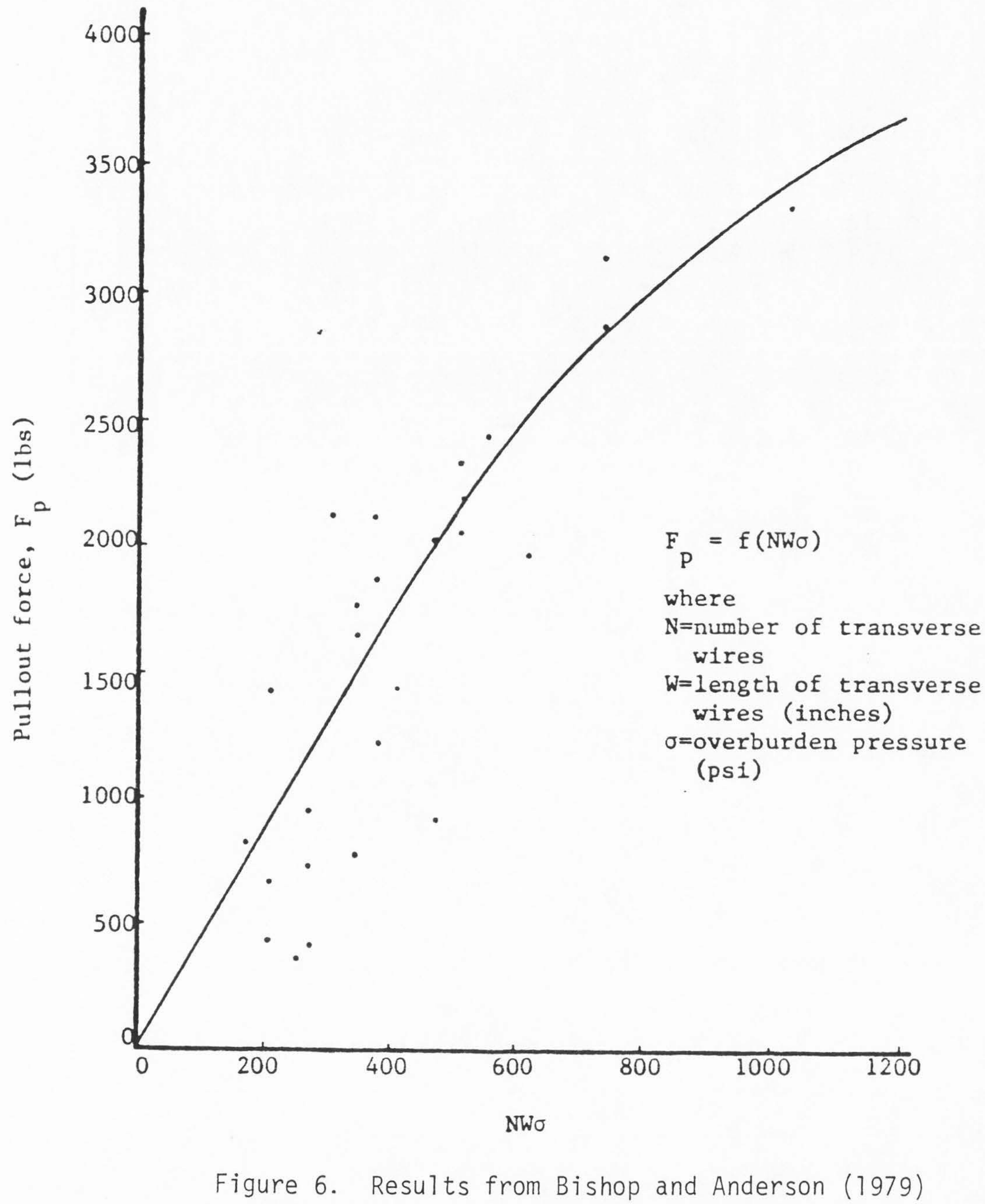


Figure 6. Results from Bishop and Anderson (1979)

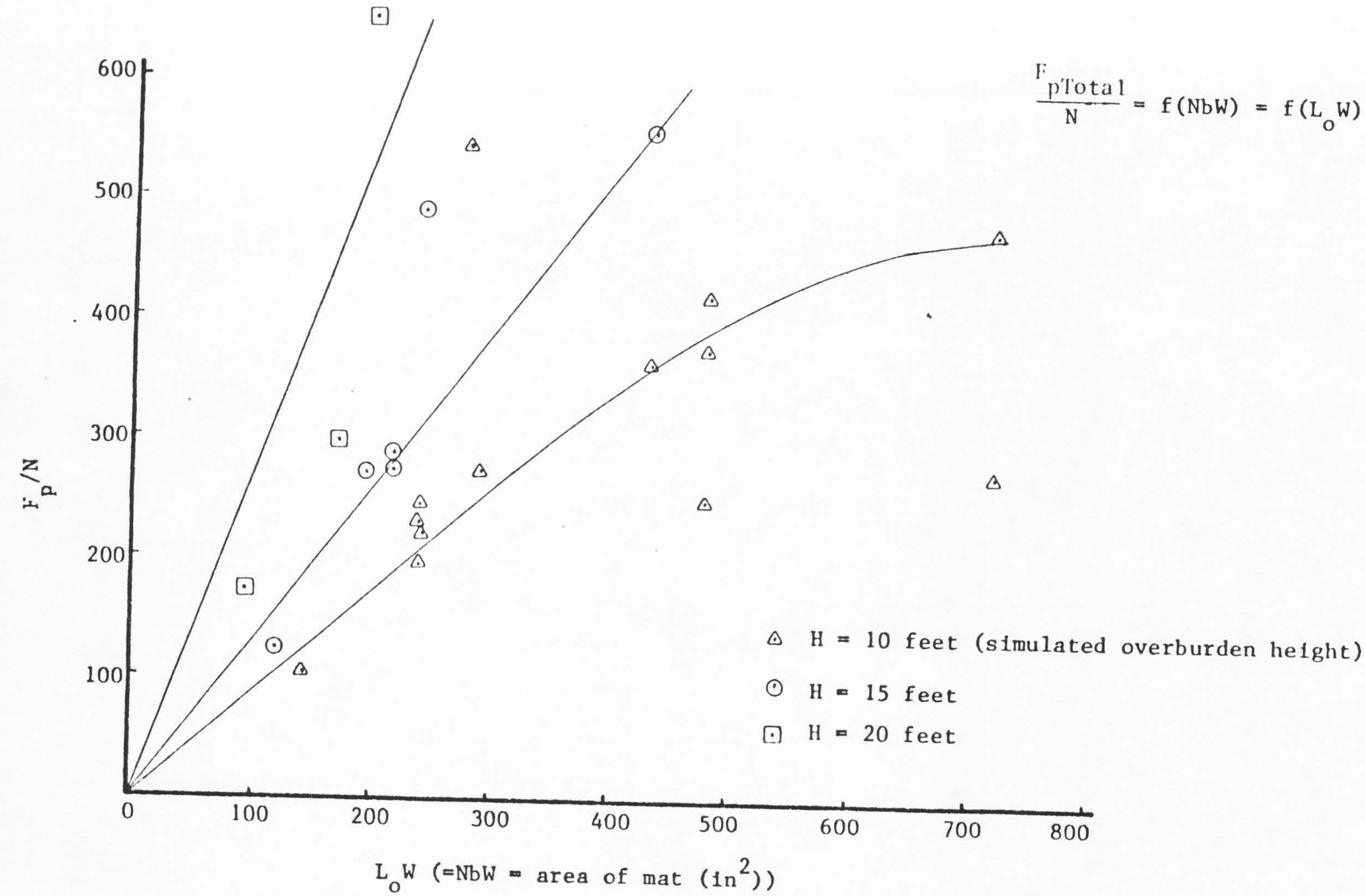


Figure 7. Results from Bishop and Anderson (1979)

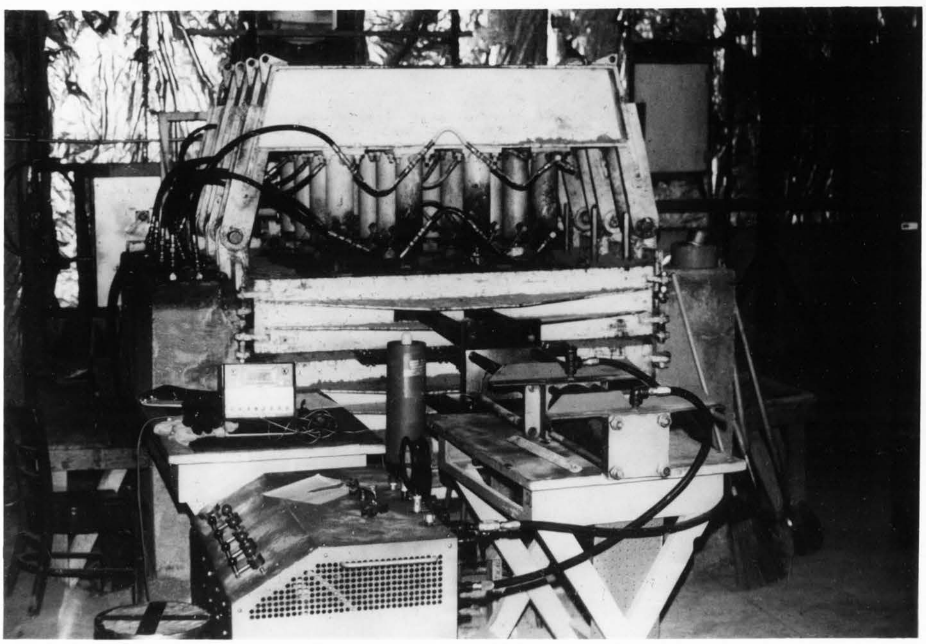


Figure 8. Buried structures test facility

diameter wires were used. A typical load-deformation curve is shown in Figure 9. The results of all the tests were plotted using the pull out force per unit width as a function of (with the friction component subtracted out so all the resistance was due to the transverse wires), the number of transverse wires times the overburden pressure times the diameter of the wire. The pull load was taken as the load at two tenths 0.200 of an inch displacement, which was usually close to the yield load. Figure 10 is the plot obtained by Peterson and Anderson (1980) from their pull out tests. By plotting the yield pull load versus the overburden pressure from these tests, the results are identical to Chang, et al (1977) in that the relationship shows a linear function when the pull out force of a constant diameter wire mat is plotted for varying overburden ranges (Figure 11).

Peterson and Anderson (1980) suggested a design procedure for bar mesh reinforcement. However, the soil must be similar to the soil used in the pull out tests. From Figure 10, the equation for the best fit straight line was used and then the frictional resistance was added to give the total pull out force for the mat (Equations 1 and 2).

$$Ft = \sigma_v * d(PI * L * M * \tan\delta + 15.58 * N) \quad N\sigma_v d \leqslant 300 \text{ lb/ft} \qquad (1)$$

$$Ft = 1380 + \sigma_v * d(PI * L * M * \tan\delta + 10.65N) \quad N\sigma_v d \geqslant 300 \text{ lb/ft} \qquad (2)$$

where:

σ_v = Overburden Pressure (psf)

d = diameter (ft.)

PI = 3.1416

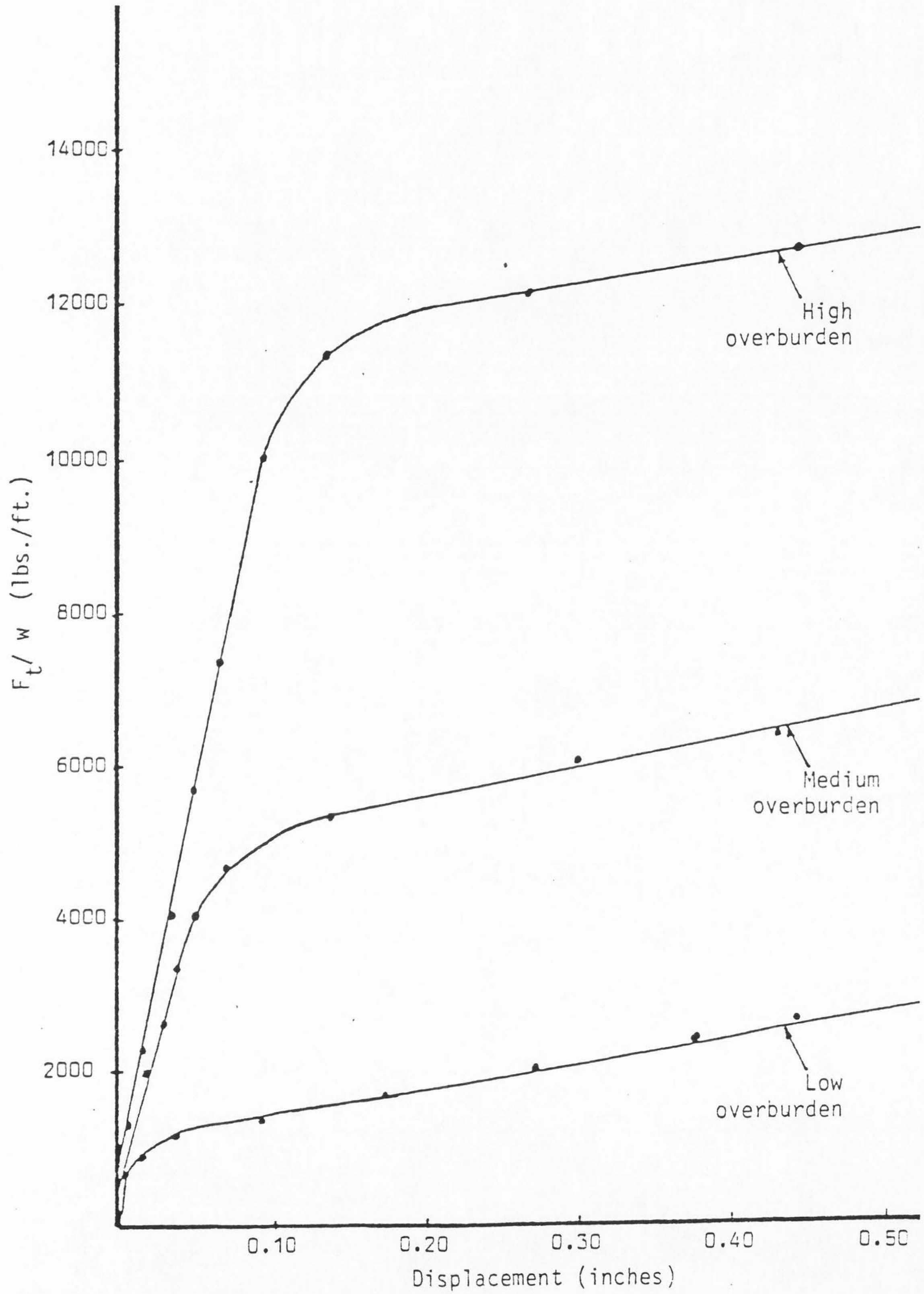


Figure 9. Load-deformation curve
After Peterson and Anderson (1980)

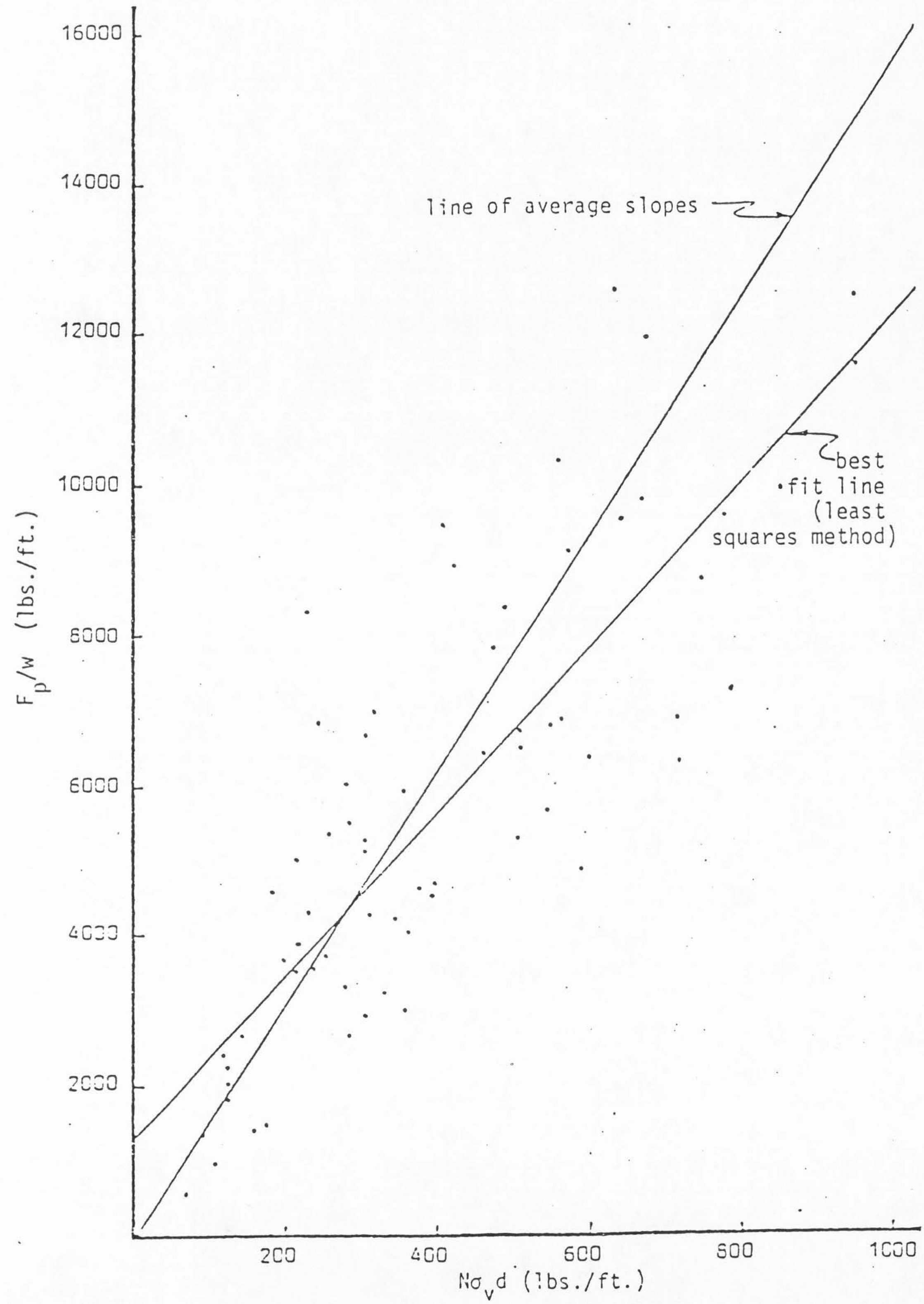


Figure 10. Pullout results from Peterson and Anderson (1980)

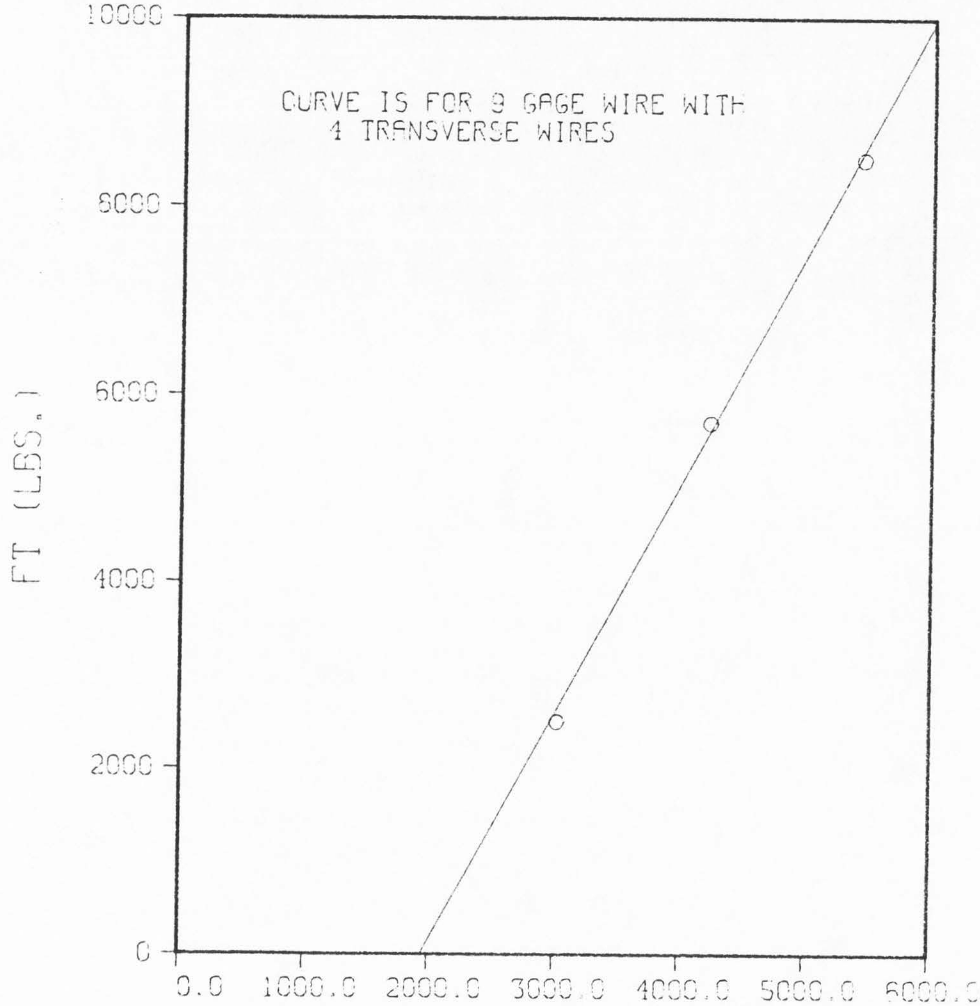


Figure 11. Relationship between pull load and normal load.

L = Length

M = Number of longitudinal wires

δ = Friction angle between soil and wire

N = Number of transverse wires

The reason for two equations is because theoretically with no overburden pressure, there would be no pull out resistance. The tests did not indicate this, and so the line of average slopes of all the points was used to adjust the lower values so that there was no pull out resistance when no overburden pressure was present.

The failure mechanism proposed by Peterson and Anderson (1980) for the transverse wires was a bearing capacity failure. The transverse wires would behave as a strip footing being pulled through the soil. The diameter of the wire would be the width of the strip footing and the equation to represent the pull out force would be the Terzaghi-Buisman bearing capacity equation (Equation 3).

$$Qult = BcN_c + \frac{1}{2}\gamma D_f B^2 N_\gamma + \gamma D_f N_q B \qquad (3)$$

where:

B = footing width

c = cohesion

N_c = cohesion factor

γ = unit weight of soil

D_f = height of overburden

N_q = surcharge factor

Because the diameter is small and the cohesion is usually small for soils used in reinforced earth structures, the equation will reduce to Equ. 4 with F_p/Nw substituted for Qult. Peterson evaluated N_q as

$$\frac{F_p}{Nw} = T_v DNq \qquad (4)$$

obtained from the plot of TvD versus Fp/Nw using his test results and compared it to the value of N_q as determined theoretically from the Terzaghi-Bearing capacity factor. These two values for N_q were very close which suggests perhaps this could be the phenomenon that is occuring. The hypothesized failure planes for the soil are shown in Figure 12. These planes are probably not sufficient to cause the cone shaped failure wedge that Chang, et al. (1977) observed in their pull out tests, but if the total effect of the longitudinal wires were included, perhaps the same result would be observed. This would provide a very good method of relating the pull out resistance to a soil parameter (Peterson and Anderson, 1980).

Another method of determining the pull out resistance of a welded wire mat is using numerical methods, such as the Finite Element technique. Gerrard (1982) discusses a numerical scheme for analysing a reinforced earth wall using strips or mesh reinforcement. The general method used is the equivalent material concept. Every layer where there is reinforcement is replaced by an equivalent layer of material which has the properties of the soil and reinforcement combined. From this program, the actual pull out resistance is not determined as it is with the laboratory experiments; however, the displacements and stresses in the equivalent material are given which could be of some use. Considering the time and expense of running a finite element program, this technique is probably not useful for practical applications.

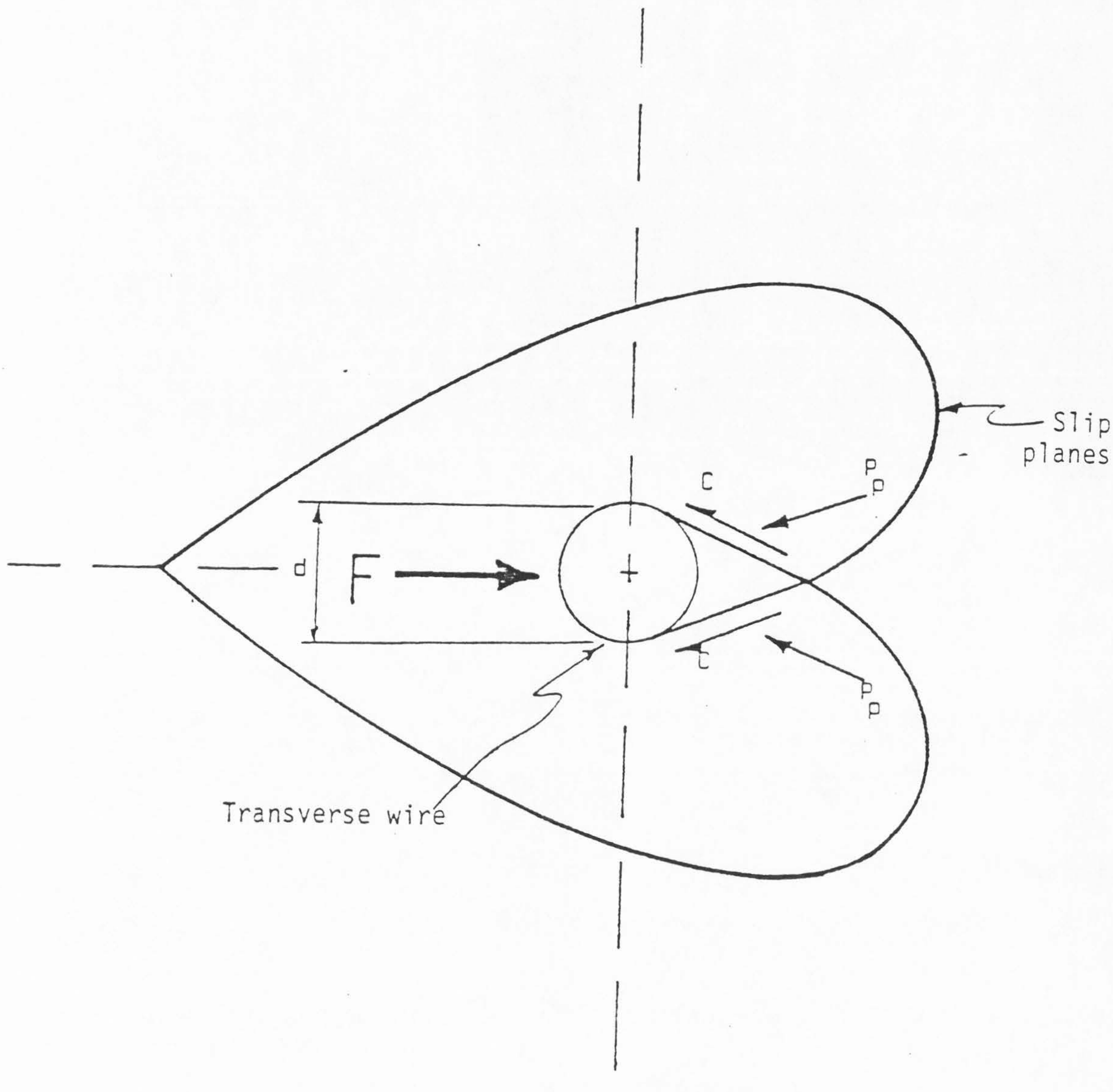


Figure 12. Failure planes
After Peterson and Anderson (1980)

The results obtained from welded wire mats seem to agree very well thus far. There has not been a great deal of research done on this specific subject; however, from the favorable comparison of Chang, et al. (1977) and Peterson and Anderson (1980), it would indicate that the research is headed toward finding the mechanism for transverse wire resistance.

Pull out resistance related to soil parameters

The largest work on relating pull out resistance to a soil parameter has been done with respect to the strip reinforcing. Peterson and Anderson (1980) did come up with a mechanism as discussed earlier which would relate pull out resistance to a soil parameter. However, there were not enough tests performed on other soils to show the relationship to hold for different soils.

Because of the nature of the strip reinforcement, pull out resistance is fairly easily related to the friction angle of the soil. With strip reinforcement it has been shown that cohesive soils do not work as well as granular soils. Given a granular soil with a certain friction angle, the pull out resistance depends on the coefficient of friction between the soil and the strip. For most types of soils this coefficient has been well documented.

CHAPTER III

METHODOLOGY

Introduction

To evaluate the pull out resistance of welded wire mats, a facility large enoguh to allow full scale pulling tests was needed. Three different soil types were used and the equipment necessary for compacting and moving the soil was required. The Buried Structures Test Facility at Utah State University was used to conduct the tests.

Pullout Tests

Equipment

The test cell that was used is pictured in Figure 8 and the dimensions are shown in Figure 13. The test cell is open at the front and top to allow filling and emptying with a fron end loader. The test cell has four steel gates which open and close on the front of the cell. When the cell is filled with soil, a one inch thick steel plate is placed on top of the soil to allow relatively even distribution of the load throughout the cell. There are sixteen hydraulic cylinders which are then positioned on top of the steel plate. These cylinders have a bore of four inches and a stroke of ten inches. Four cylinders are welded to an individual wide-flange beam, and the beams are attached to the cell by steel pins. The

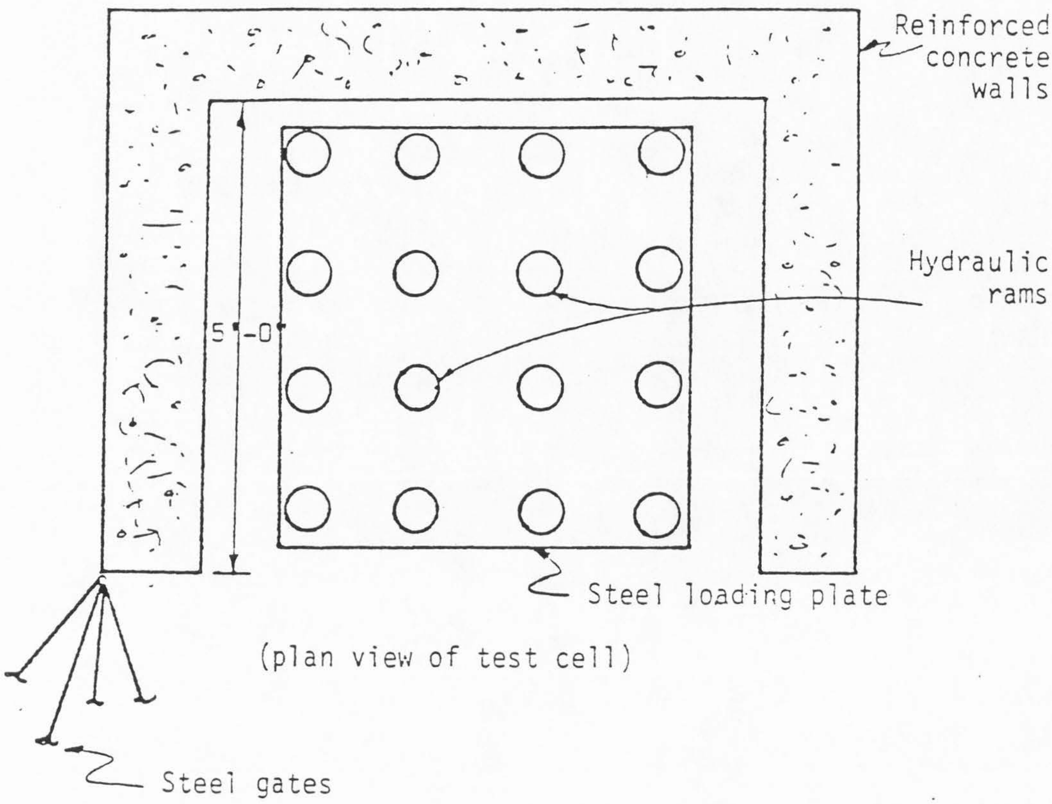
hydraulics are capable of applying up to 150 feet of overburden pressure (18000 psf).

The three soils that were used are stored in concrete storage bins. The three different soils were a silty sand, a washed sand, and a pea gravel. The soil properties are presented in Appendix A and include soil gradations, maximum density and optimum moisture (standard procter test), and the shear strength from direct shear tests. Triaxial shear tests were also run on the silty sand.

The soils were compacted in the test cell to 90 percent of their maximum dry density except for the pea gravel which was usually approximately 100 percent of its maximum dry density. The soil was compacted in one foot lifts with a Whacker tamping maching (Figure 14). The density was then measured with a Troxler nuclear gage (Figure 15).

Mats were made using four wire sizes including 1/4 inch, 3/8 inch, 9 gage and 7 gage. The yield point strength of all mats was 60 ksi. The 3/8 inch bar mats were made of deformed bars. The mats were six feet long and 2.5 feet wide. The longitudinal spacing of the 1/4 inch and 3/8 inch mats was six inches. Three transverse spacings were used including: 12 inches, 18 inches, and 24 inches. The 9 gage and 7 gage mats had wire spacings of 2 inches by 6 inches.

Two half inch thick steel plates were bolted on each side of the portion of the mat extending from the test cell (Figure 16) The plates had a V-groove machined into them to allow four contact points with each wire. The plates were bolted together as shown in Figure 17. The plates were attached to a large hydraulic ram by means of a clevis and a 3/4 inch diameter pin. The clevis was threaded onto a



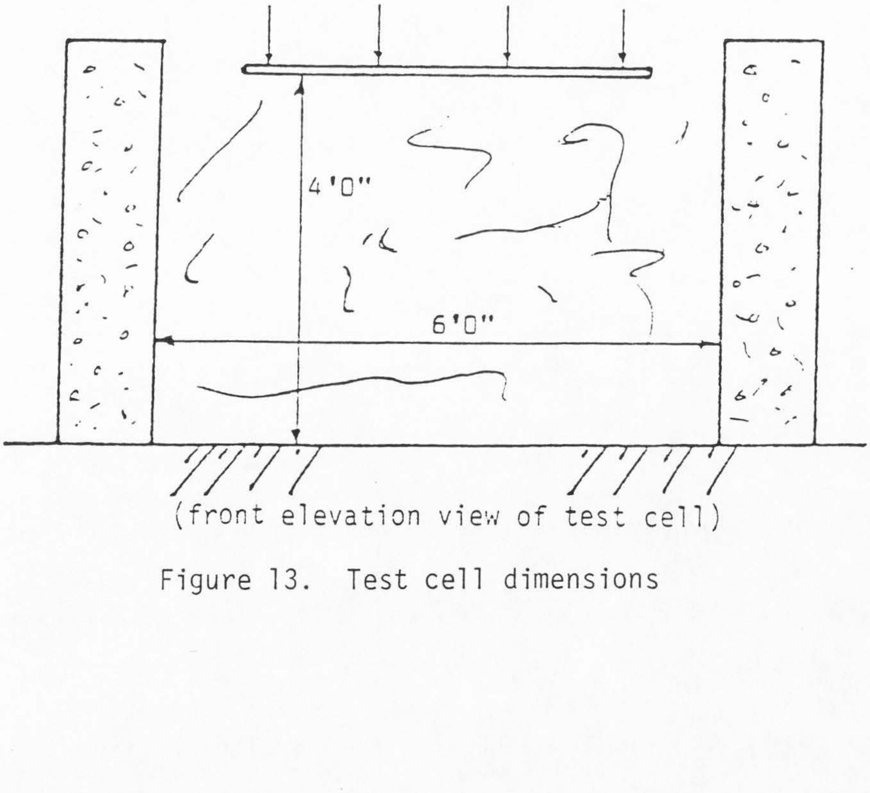


Figure 13. Test cell dimensions

Figure 14. Whacker tamping machine

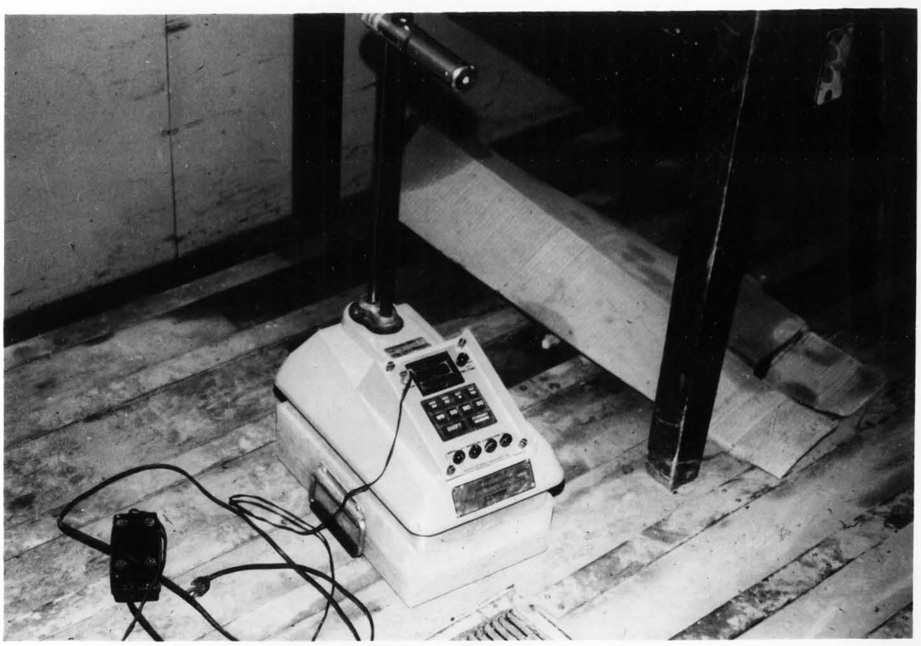


Figure 15. Troxler nuclear gage

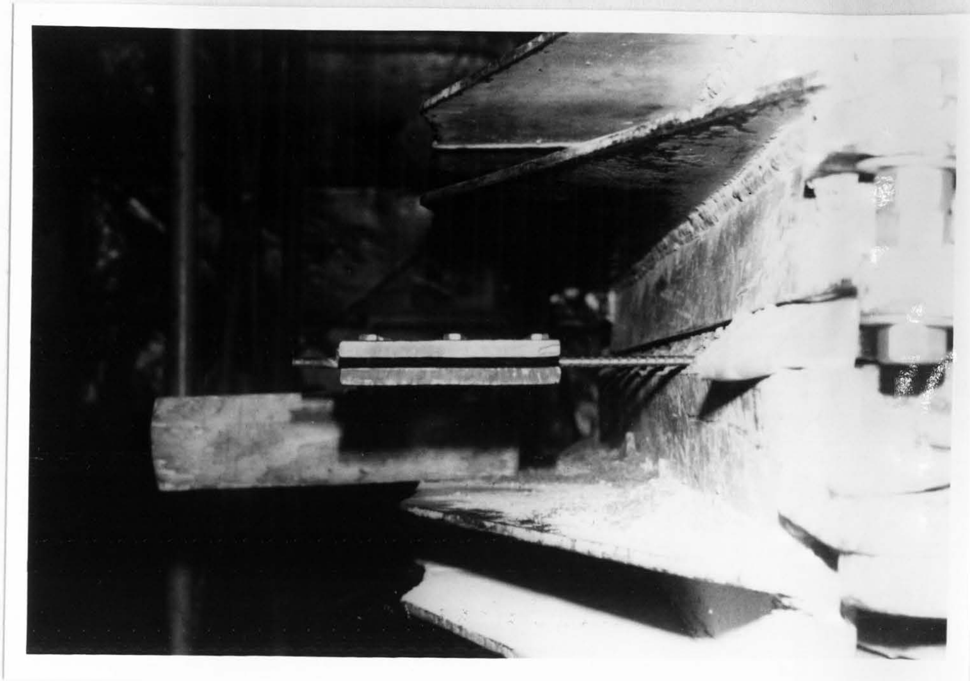


Figure 16. Mat and clamping plates

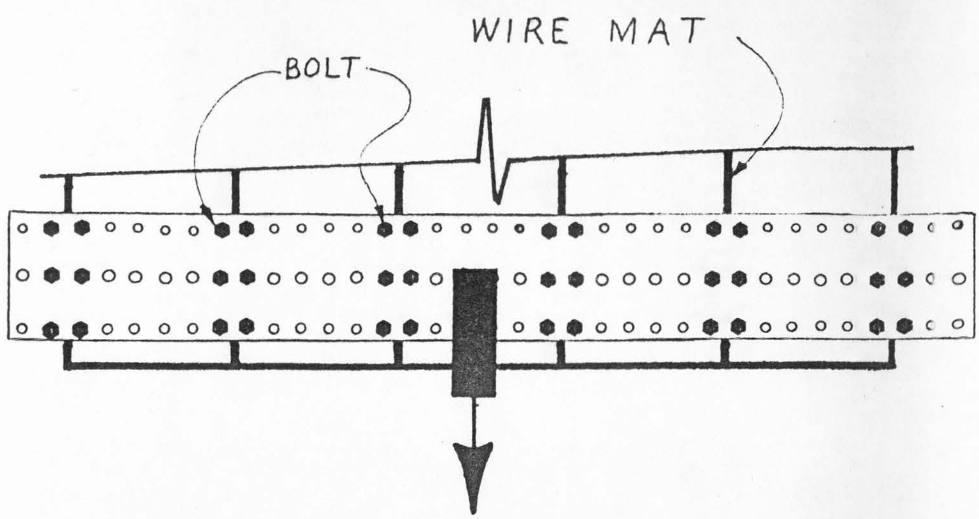


Figure 17. Top view of clamping plates and mat

one inch round rod which was also threaded into the hydraulic ram. The rod was first made of A-36 steel and later changed to a rod made of 4340 cold-rolled steel to allow greater pulling loads. The hydraulic ram sat on a table and jacked against the gates of the test cell. The hydraulic ram was run by a hydraulic pump which was independent of the overburden hydraulic system. The pump was capable of supplying 3000 pounds per square inch (psi) to the cylinder. The entire set up is shown in Figure 18.

The tests were monitored for both pull out force and displacement. The pull out force was monitored by means of two strain gages attached to the one inch round rod. These strain gages were mounted on opposite sides of the rod and wired in series to cancel any bending stresses induced in the rod and therefore, the strain gages would only measure the axial tension in the rod. The strain gages were hooked to a Vishay-Ellis digital stain indicator using a half bridge circuit (Figure 19). The rod and strain indicator had been calibrated to read directly in pounds. This calibration was performed on a Tinius-Olsen testing machine in the laboratory.

The displacement of the mat was measured by a dial gage positioned near the face of the mat (Figure 20). The dial gage was accurate to a thousandth (0.001) of an inch and permitted a maximum displacement of one inch.

Set up

The soil was put in the test cell in one foot lifts and compacted to approximately 90 percent of the maximum dry density of the soil at a moisture content near optimum. The test cell was filled to

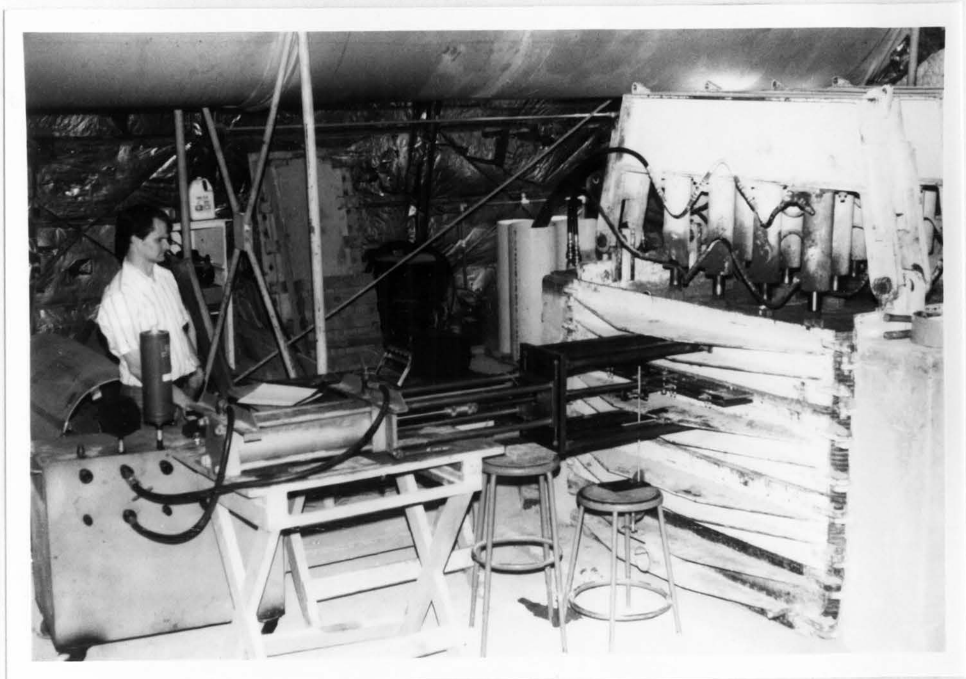


Figure 18. Complete pulling appartus

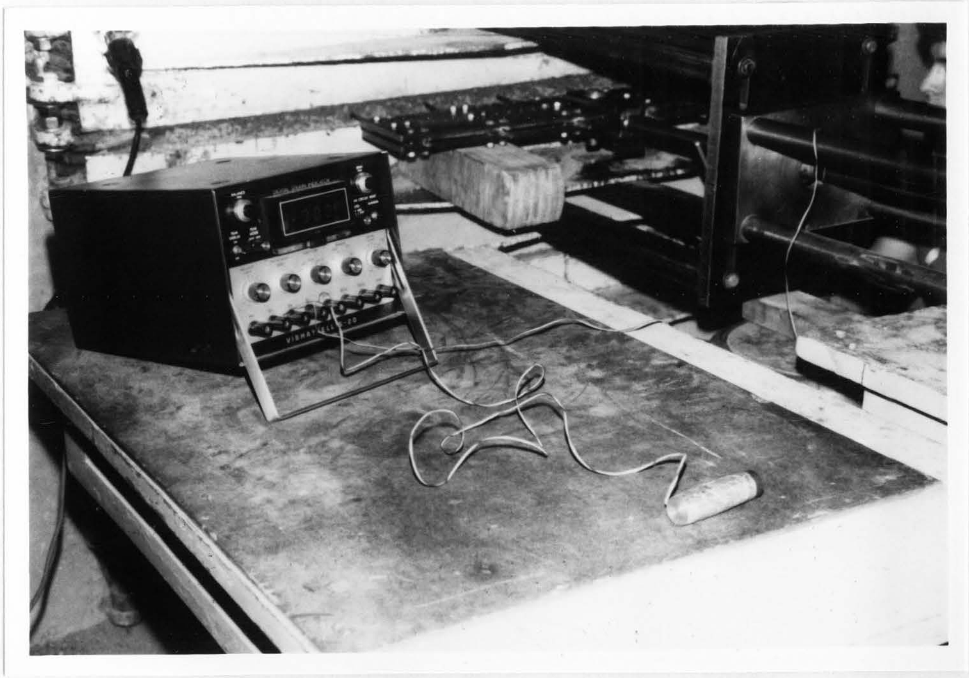


Figure 19. Vishay-Ellis digital strain indicator

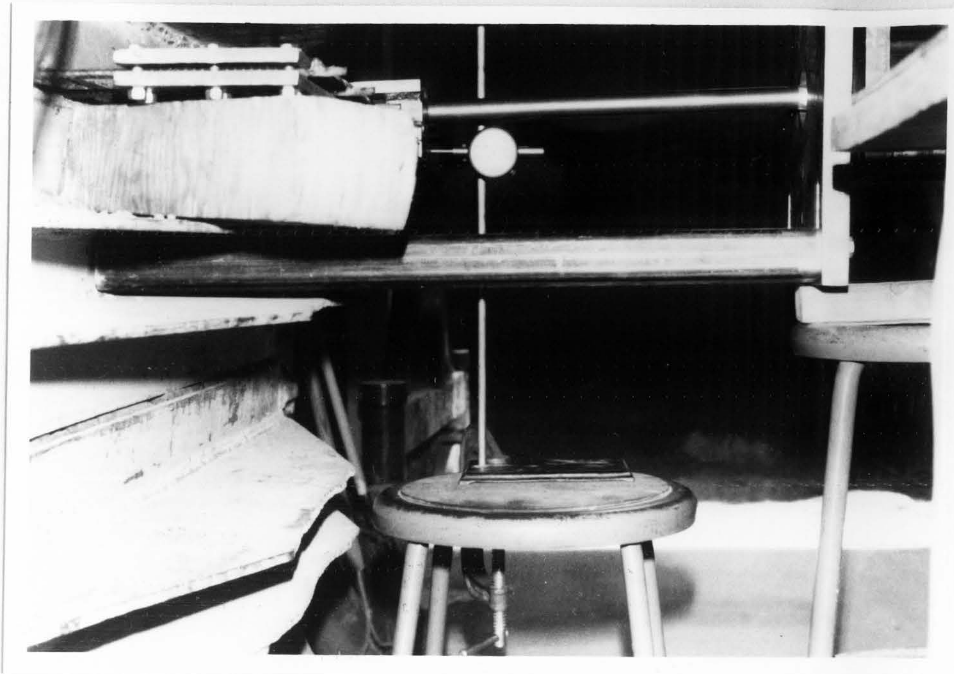


Figure 20. Dial gage arrangement

the top of the third gate which is about three to four feet of soil. After the soil was compacted the density was checked. In tests using other soils besides the silty sand, the cell was filled half way with the compacted silty sand soil, and then the rest of the cell was filled with the soil that would be used for the test. This provided about one foot of soil above and below the welded wire mat.

After the test cell was filled to the top of the third gate, the mat was inserted and centered in the cell. The mat was checked with a carpenters level to ensure that it was horizontal. After the mat was centered and level, more soil was put on top of the mat and compacted to the desired density. Then the one inch thick steel plate was positioned on top of the soil, and the hydraulic cylinders were placed on top of the steel plate.

Two 1/2 inch thick steel plates were fastened to the wire mat which protruded between the third and fourth gates. The plates were fastened together with 3/8 inch diameter bolts. The pulling equipment was then positioned in fron of the cell. Then the pump was hooked to the cylinder and the load cell was screwed into the cylinder. The clevise was then screwed onto the load cell and hooked to the steel plates with a steel pin. Then a square was used to align the cylinder and the mat so that the mat was being pulled straight out.

After the mat was tested, the pulling equipment was removed and the hydraulic cylinders and steel plate were removed. Then the soil was dug out by hand until the mat was exposed. The mat was taken out and marked to identify the test and was then stored. The soil under the mat was then checked for compaction to ensure the vertical load

had not compacted the soil to a great extent. If the soil had been compacted by the vertical load, then the next 8 to 12 inches were removed and recompacted in the cell. Then another mat was installed and the procedure was repeated.

Running the test

In running the test, the desired vertical pressure was applied and allowed to come to equilibrium. This ussually required about ten to twenty minutes. A 500 pound seating load was then applied with the ram to take up any slack in the system and to ensure the cylinder was securely against the steel gates. After the seating load was applied, the dial gage was set up to measure the horizontal displacement. The pull out load was then applied at a displacement rate of approximately 4 inches per hour. The load was read directly in pounds from a Vishay strain indicator. The operator tried to keep the mat pulling at a constant strain rate of 4 inches per hour (0.0333 inch per minute). The force readings were taken at every 0.02 inch displacement. A person was watching the dial gage and recording the time every 0.02 inch to keep the strain rate relatively constant. The mat was pulled to a deflection of one inch and then the test was stopped.

After the test was stopped, the vertical overburden was increased and allowed to come to equilibrium again. The cylinder was checked to ensure that it was pulling the mat straight and was adjusted if necessary. Then the seating load was applied and the test was run again. The tests were run in this manner until the wire broke in tension or the mat pulled out crooked and the test needed to

be stopped. Many times the welds holding the transverse wires to the longitudinal wires would break and the mat would pull out very crooked.

CHAPTER IV

PRESENTATION AND ANALYSIS OF RESULTS

Introduction

Ninety-three pull out tests were performed at the buried structures test facility at Utah State University on various welded wire mats in different types of soils. Of these 93 tests, 19 failed by tension failure of the longitudinal wires. It took five or six tests to work out operational problems and to get the tests running smoothly.

Pullout Tests

Load-displacement curves

For all the 93 tests that were performed, a load-displacement curve was plotted. A sample of some of these curves is shown in Figures 21, 22, & 23. All of the curves have a yield point where the curve flattens out or where it requires less load to cause the same displacement. The slopes of these different sections of the curves have nearly the same slope for all the tests in the same soil As indicated by Figure 21, the portion of the curve after yielding is nearly parallel for all values of N and overburden pressure. The different soils produced different slopes for the curves, but the curves for every type of soil looked very similar.

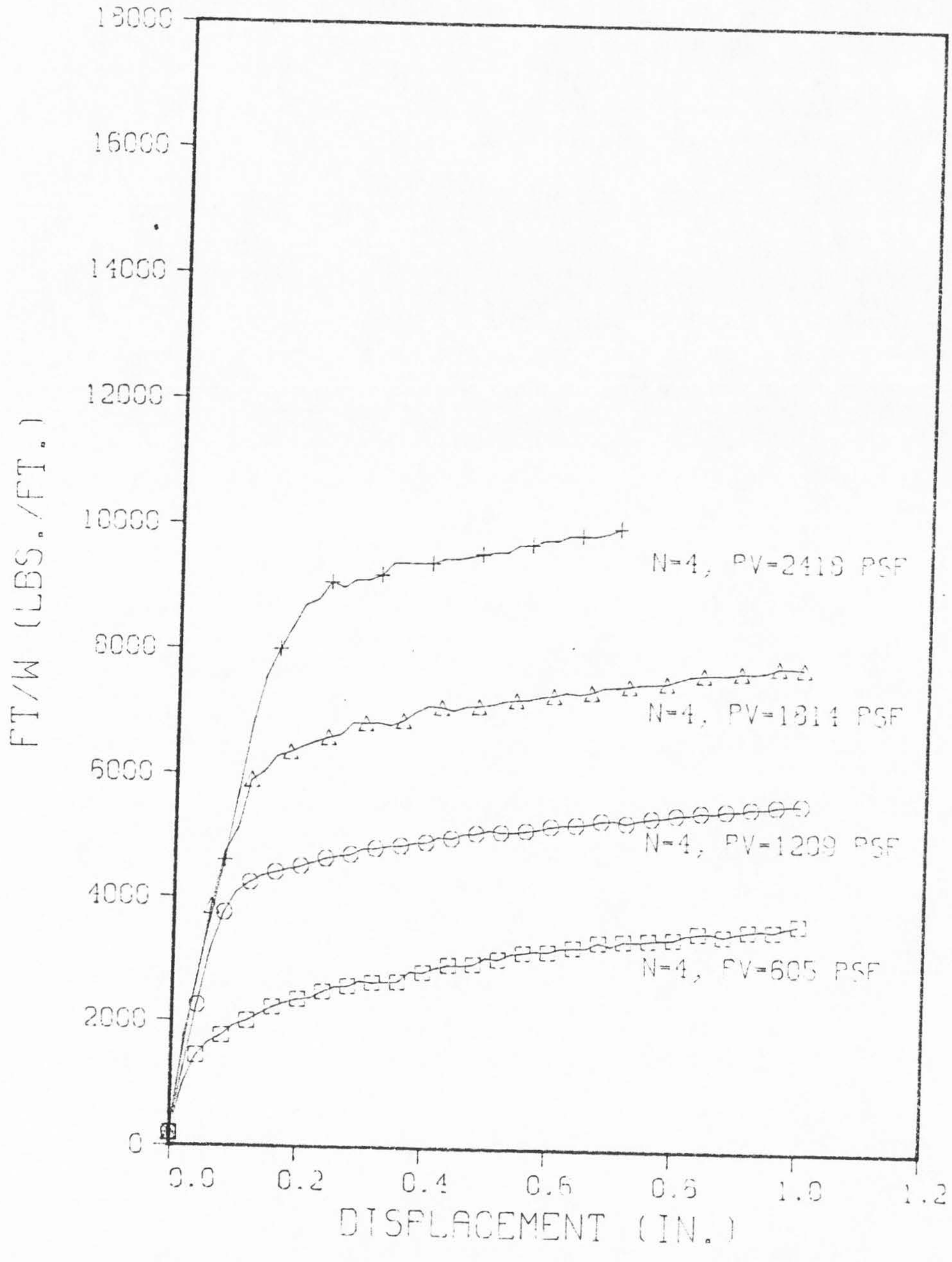


Figure 21. Load-displacement curve - silty sand

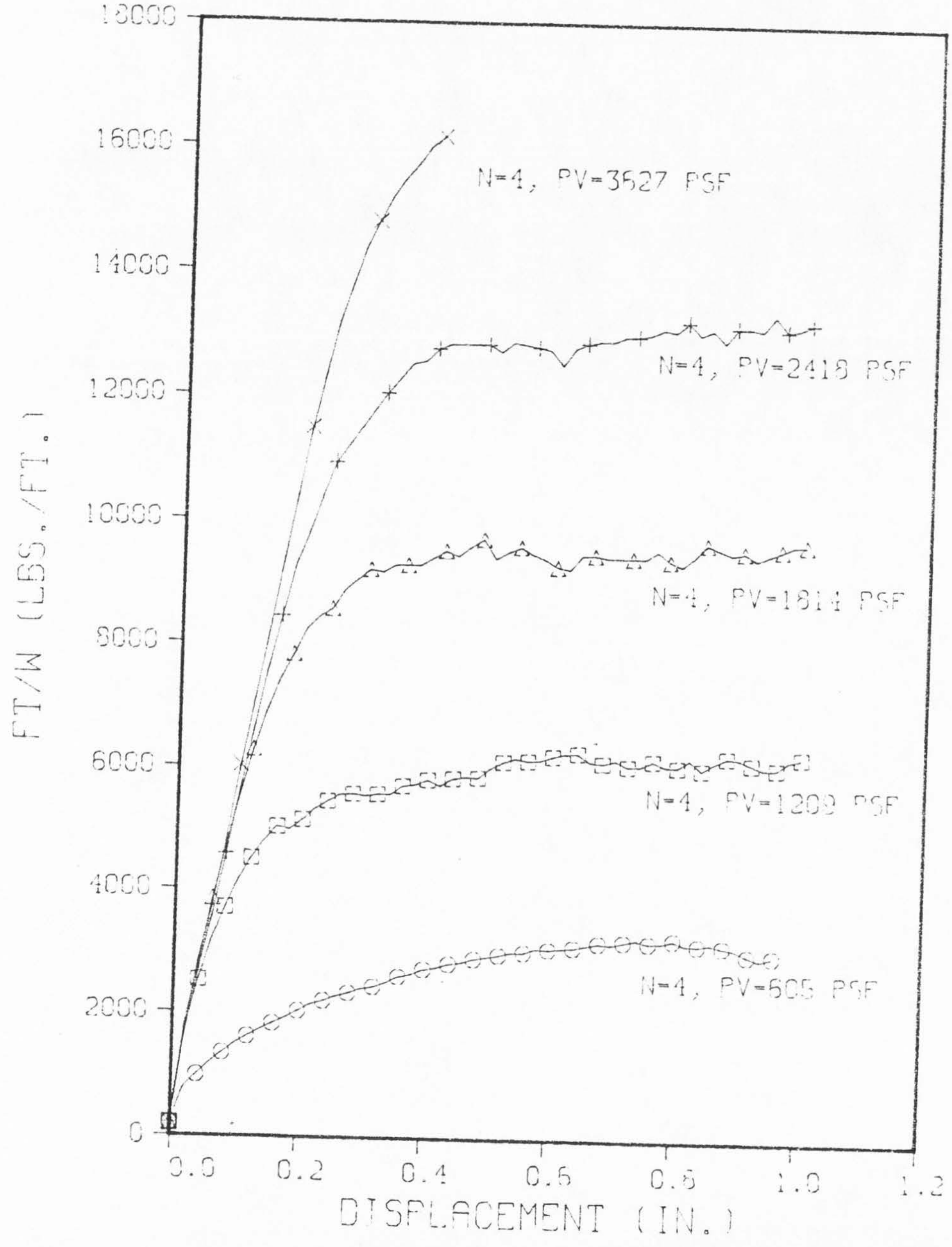


Figure 22. Load-displacement curve - pea gravel

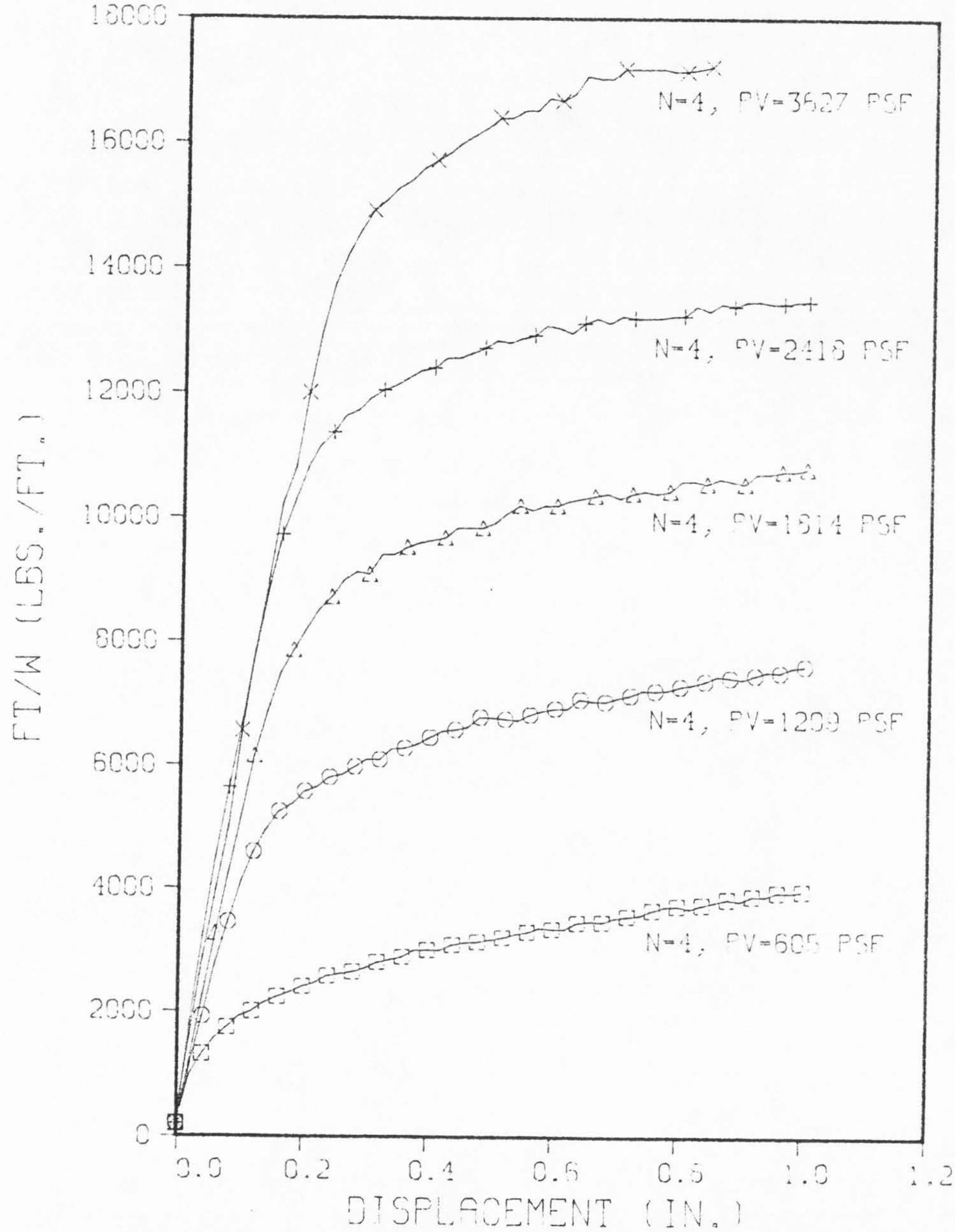


Figure 23. Load-displacement curve -
washed sand

All of the load displacement curves are in Appendix B. These curves are labeled with the number of transverse wires and the overburden pressure at which each test was run.

In some tests there was a weld that would break causing the transverse wire to become unattached to the longitudinal wire. The load-displacement curve also showed this phenomenon because when the weld would break, the force would drop. Figure 24 is an example of such a test where a weld broke and the curve depicted this fact. There were times when the force would take a sudden drop without a weld breaking and it cannot always be deduced what happened to cause the anomaly in the curve.

Effect of number and diameter of transverse wires

To determine the relationship between the pull out force and the number of embedded wires, plots were made showing the pull out force per unit width (Fp/w) versus the overburden pressure for constant diameter wire mats (Figures 25-30). Qualitative reasoning would predict that if the number of embedded wires increased, then the pull out force should also increase. The pull out resistance in this case, is the resistance due only to the transverse wires and does not include the frictional resistance of the longitudinal wires.

From the plots for the silty sand (Figures 25 & 26) it can be seen that if the number of transverse wires is increased from three to four, the pull out resistance does increase for both diameters of wire. The increase from four to five embedded wires, however, is not as well defined. With the 3/8 inch diameter wire mat the pull out resistance is nearly the same for either four or five embedded

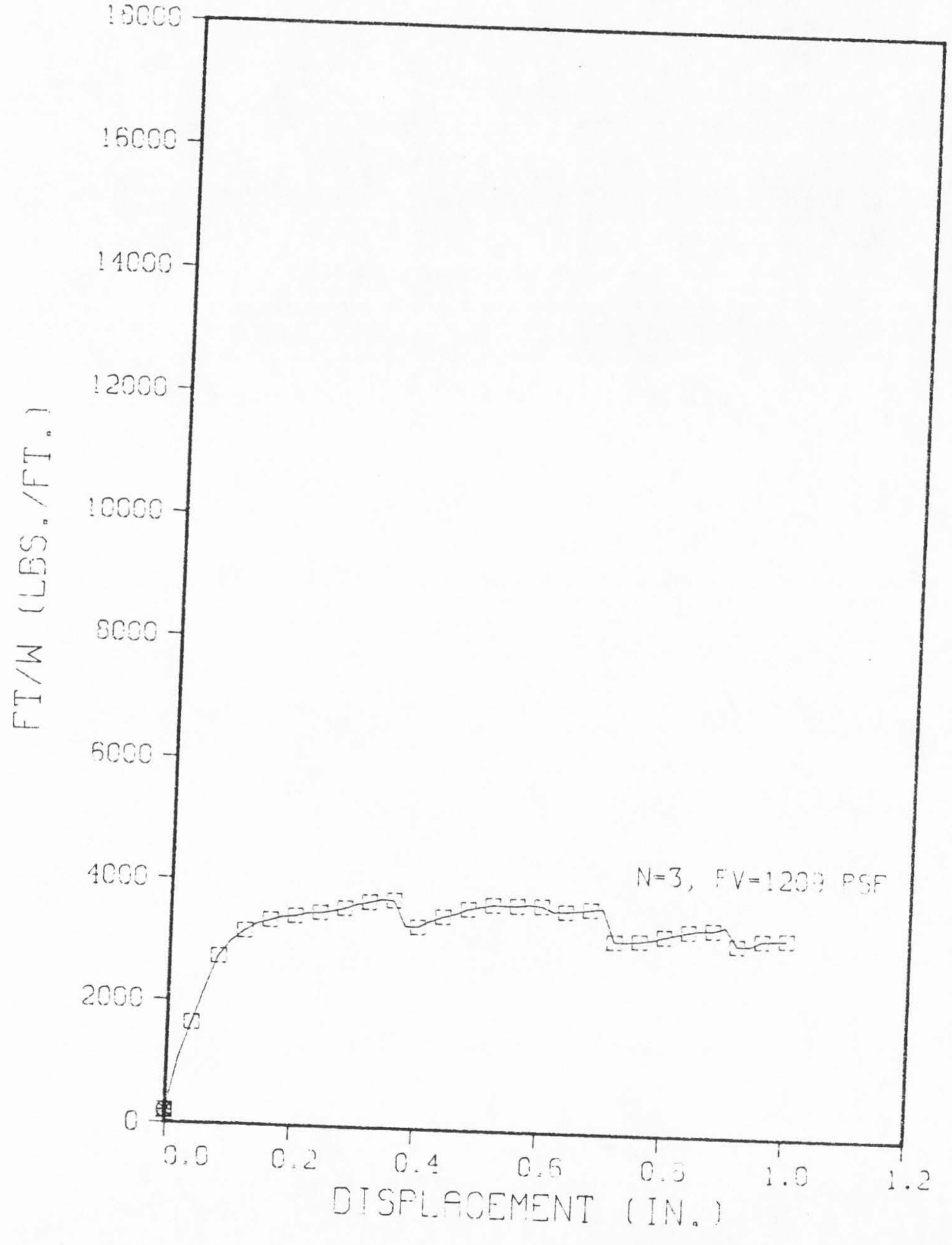


Figure 24. Load-displacement curve depicts broken weld

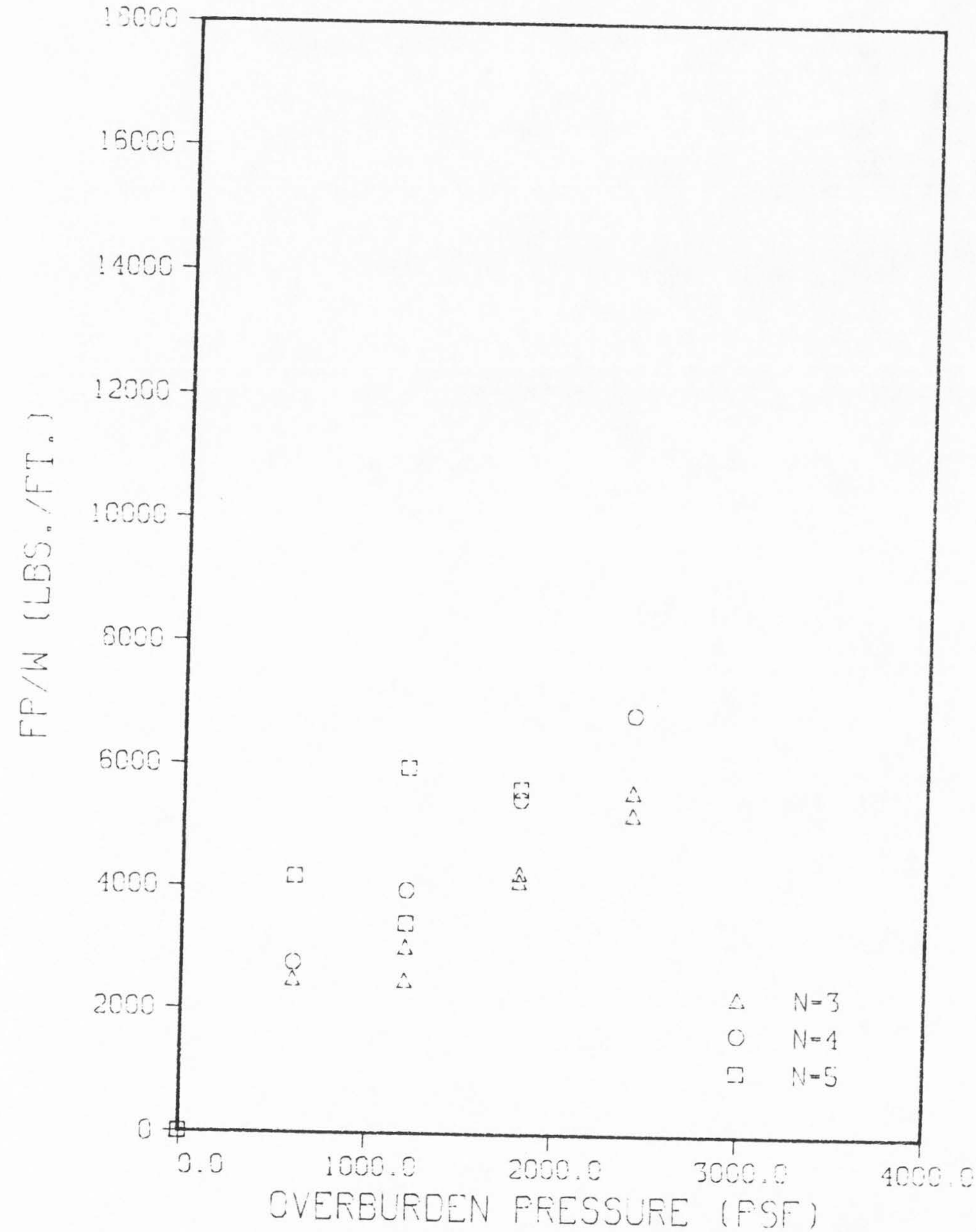


Figure 25. Affect of number of transverse wires. Silty sand - 1/4 inch diameter

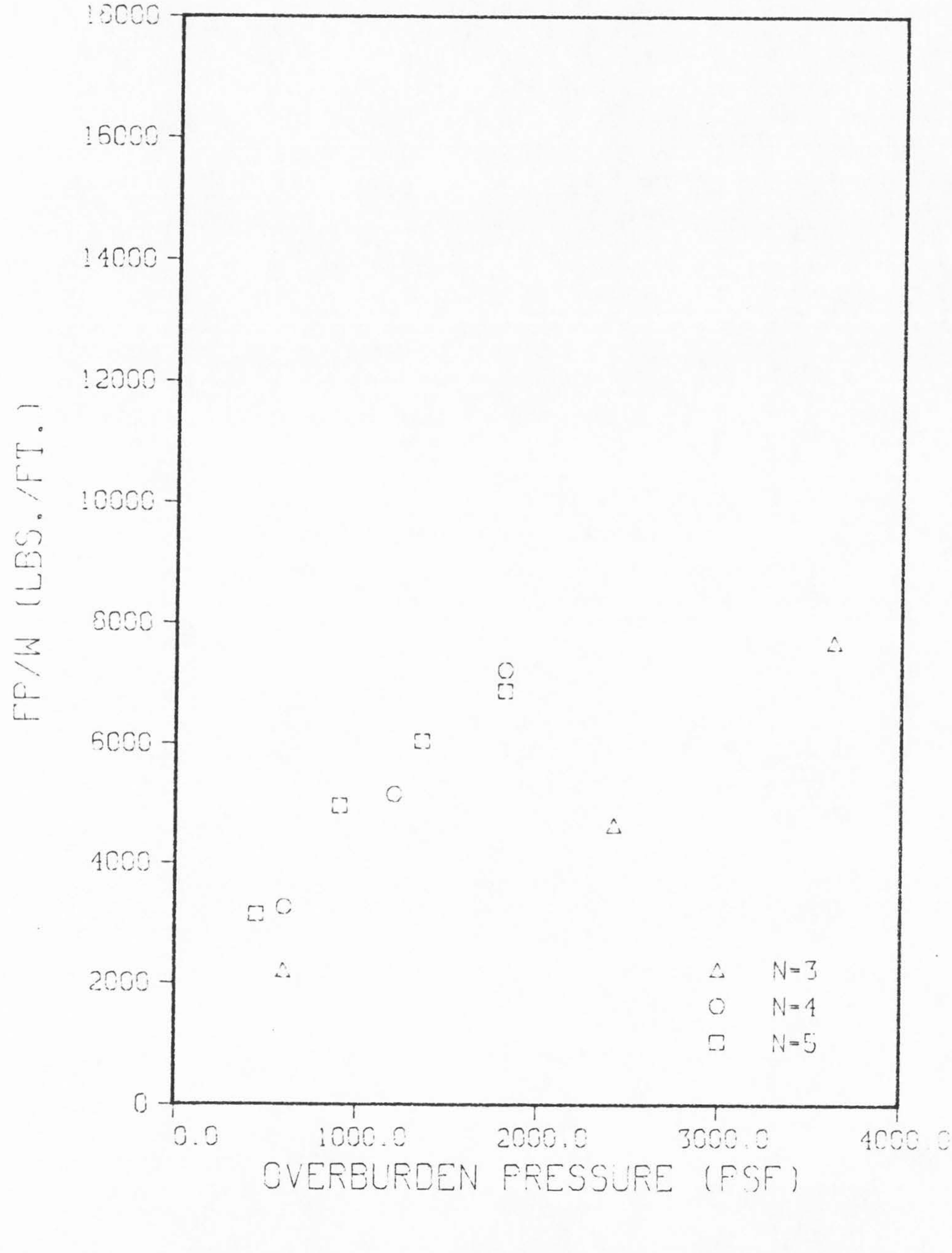


Figure 26. Affect of number of transverse wires.
Silty sand - 3/8 inch diameter.

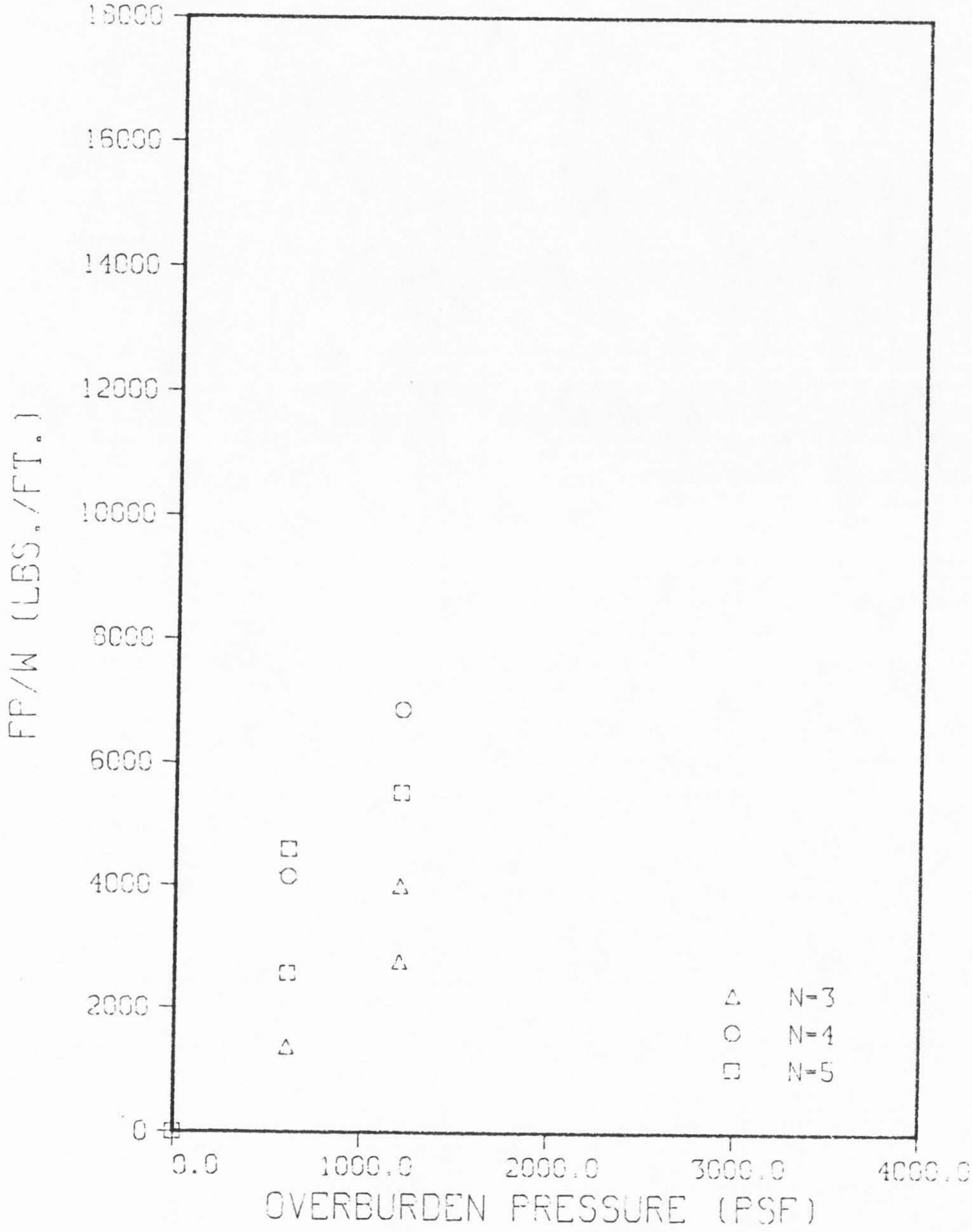


Figure 27. Affect of number of transverse wires.
Pea gravel - 1/4 inch diameter

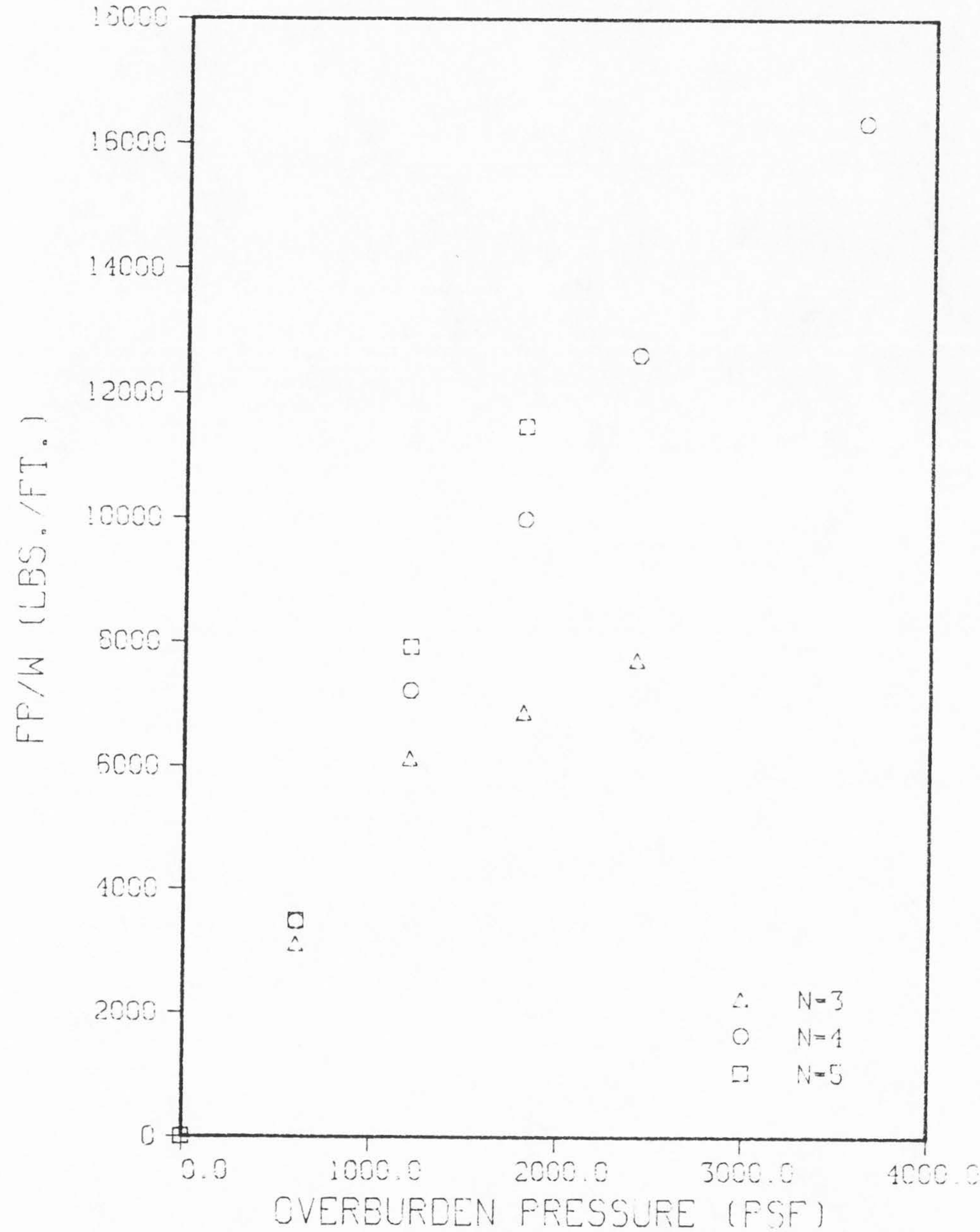


Figure 28. Affect of number of transverse wires.
Pea gravel - 3/8 inch diameter

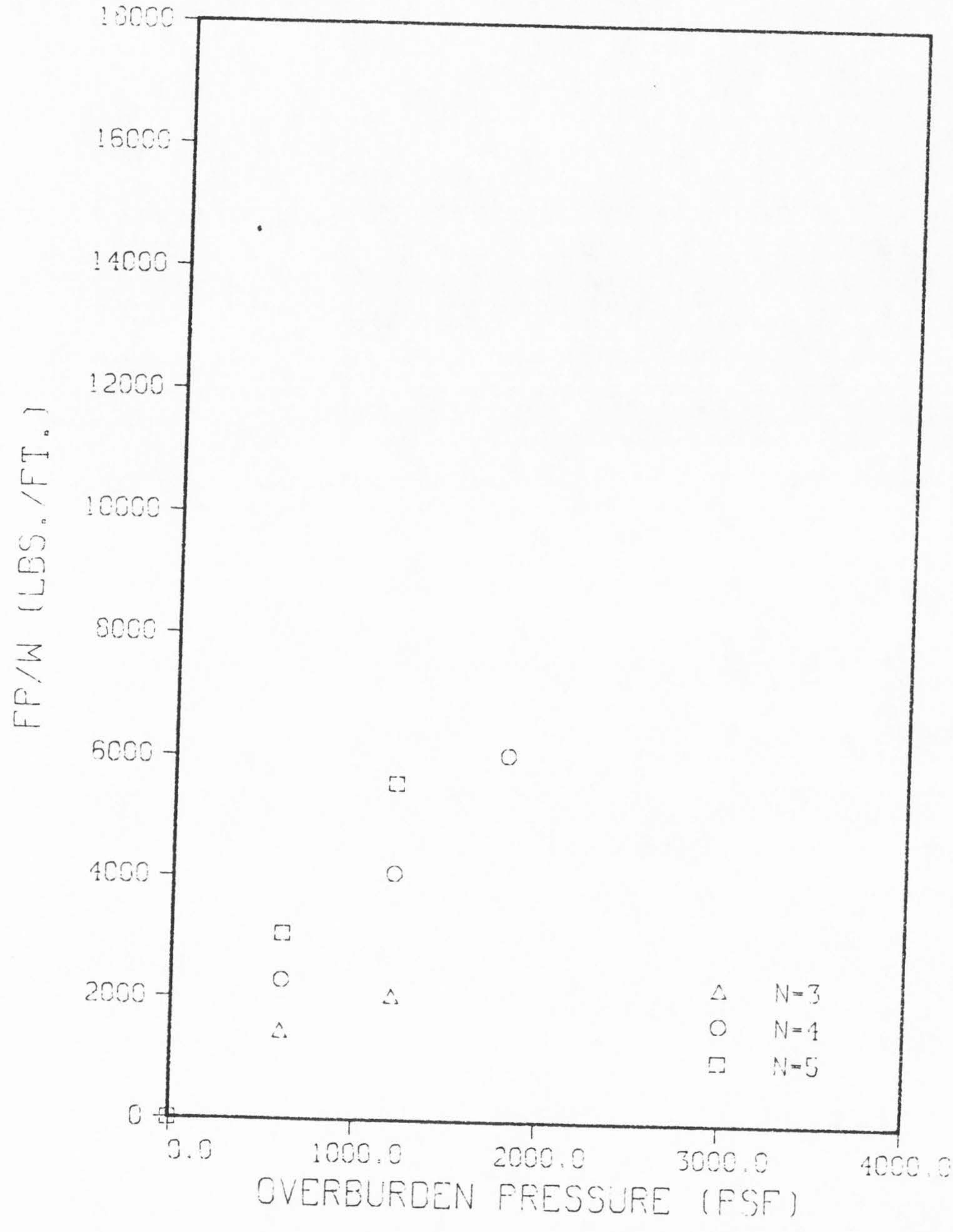


Figure 29. Affect of number of transverse wires.
Washed sand - 1/4 inch diameter

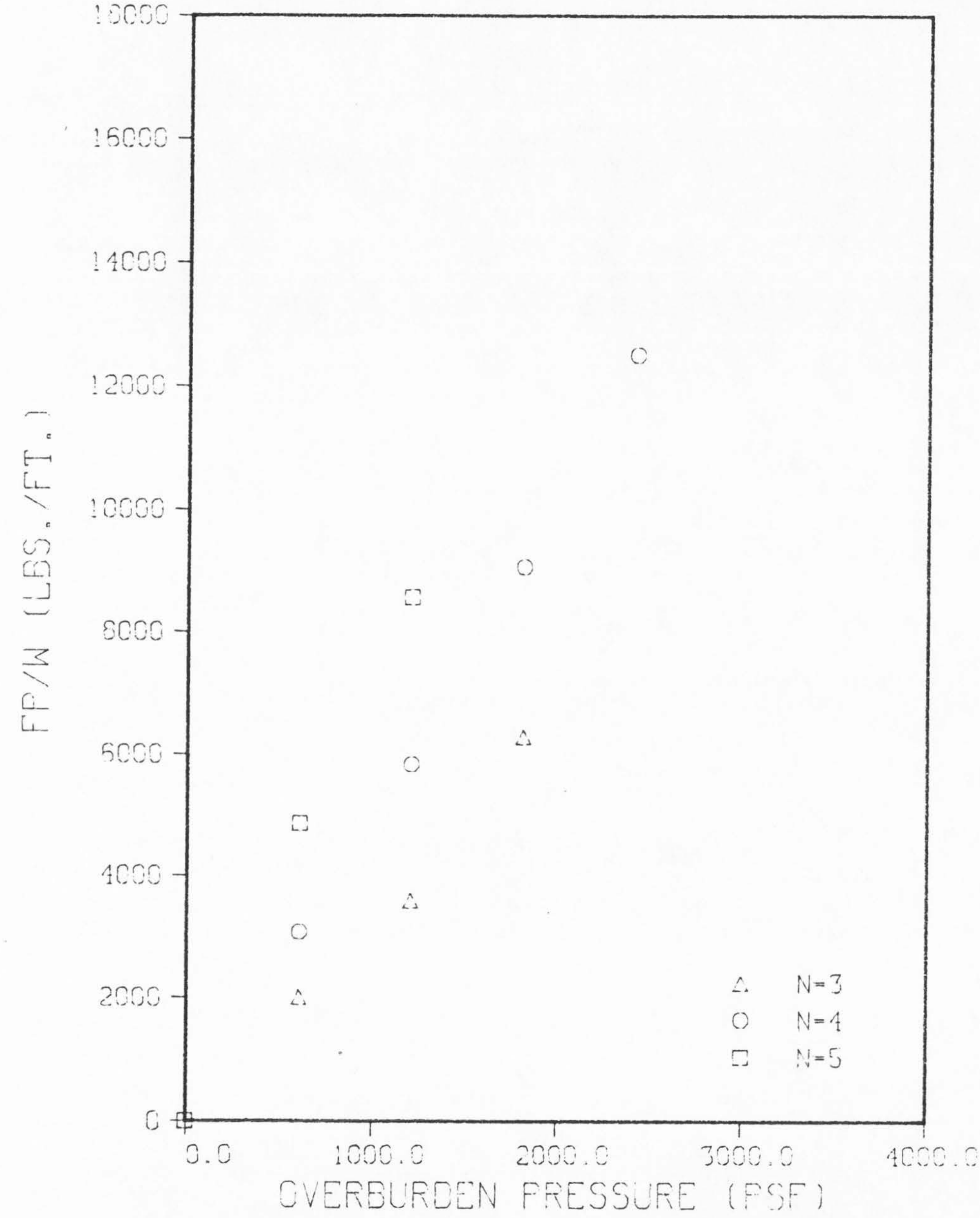


Figure 30. Affect of number of transverse wires.
Washed sand - 3/8 inch diameter

wires. For the 1/4 inch wire, the pull out force is greater for one set of tests, but about equal for another set. This could be due to experimental error.

For the pea gravel (Figures 27 & 28) and the clean sand (Figures 29 & 30) the results are positive in that all but one case shows an increase in the pull out resistance for an increase in the number of embedded wires. The only one which was questionable was in the pea gravel soil with four or five embedded wires. The pull out resistance shows that the five embedded wires had less resistance than with four embedded wires.

The logical expectation of increasing the diameter of the wire would be to increase the pull out resistance, but by a small amount as compared to the transverse wire pull out. By increasing the diameter, the failure planes around the wire should increase in total length, thus causing more utilization of the internal friction of the soil. The same plots were made for the wire mats except the number of transverse wires was held constant. Only three of the plots are shown in Figures 31-33, one for each soil type. The rest of the plots are in Appendix B.

The plots for the silty sand again are not clear cut. The plot for four embedded wires does show a definite increase in pull out resistance for 3/8 inch diameter wire compared to 1/4 inch diameter wire. This difference in pull out resistance increases as the overburden pressure increases. At 3000 pounds per square foot (psf) overburden pressure, the difference between 3/8 inch diameter wire and 1/4 inch diameter wire is greater than the difference at 500 psf. For five embedded wires, the 3/8 inch diameter wire is greater

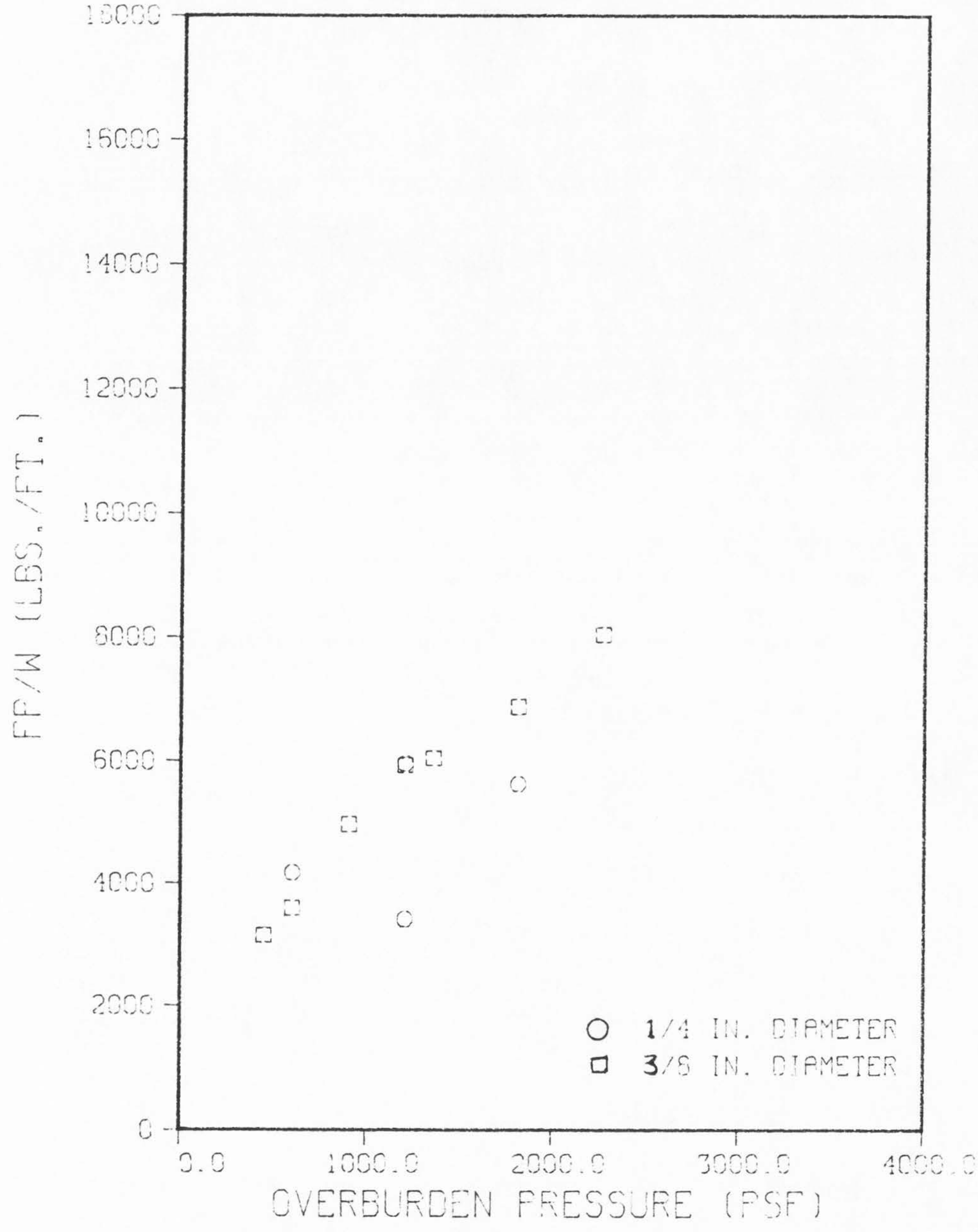


Figure 31. Affect of diameter. Silty sand - N=3

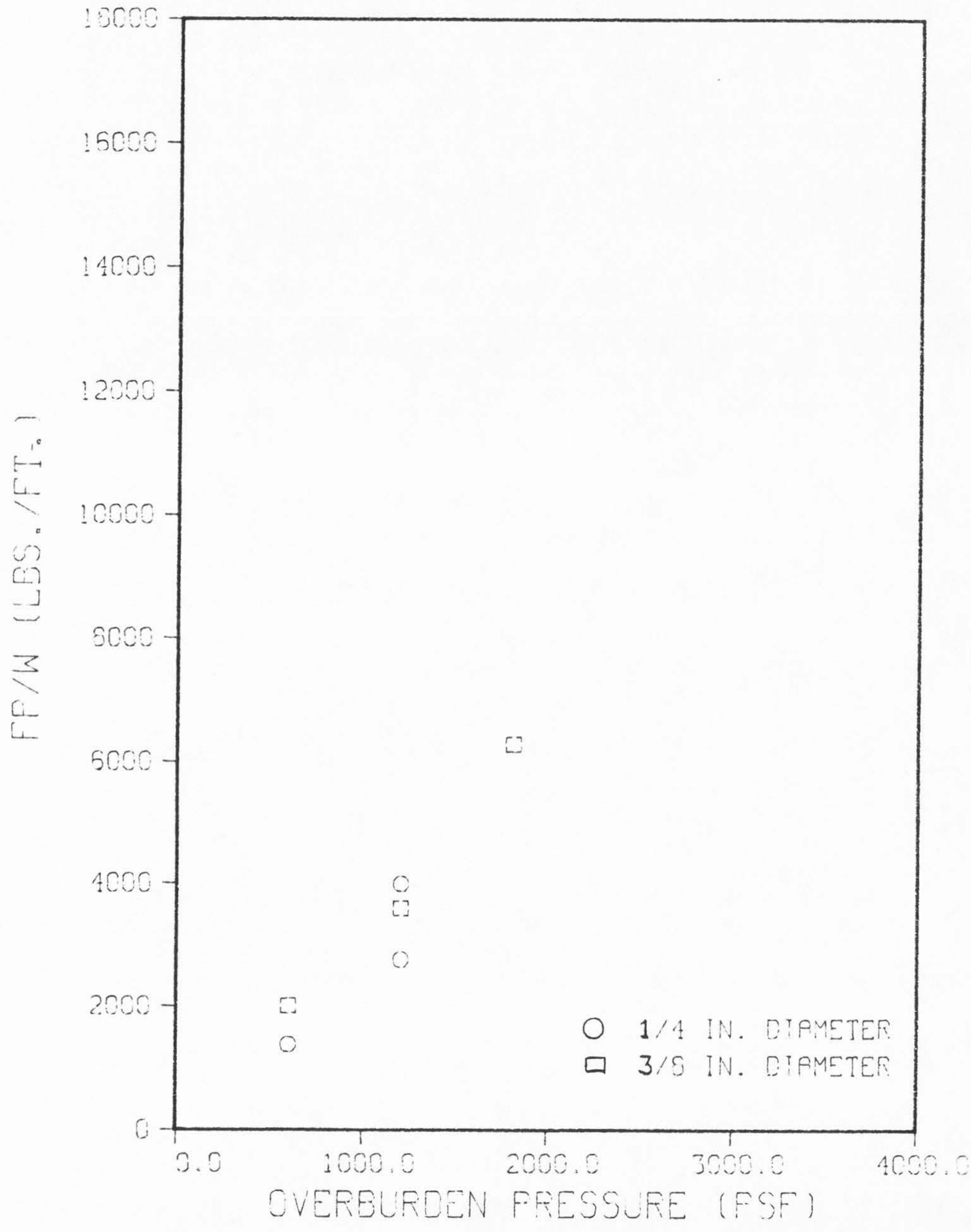


Figure 32. Affect of diameter. Pea gravel - N=3

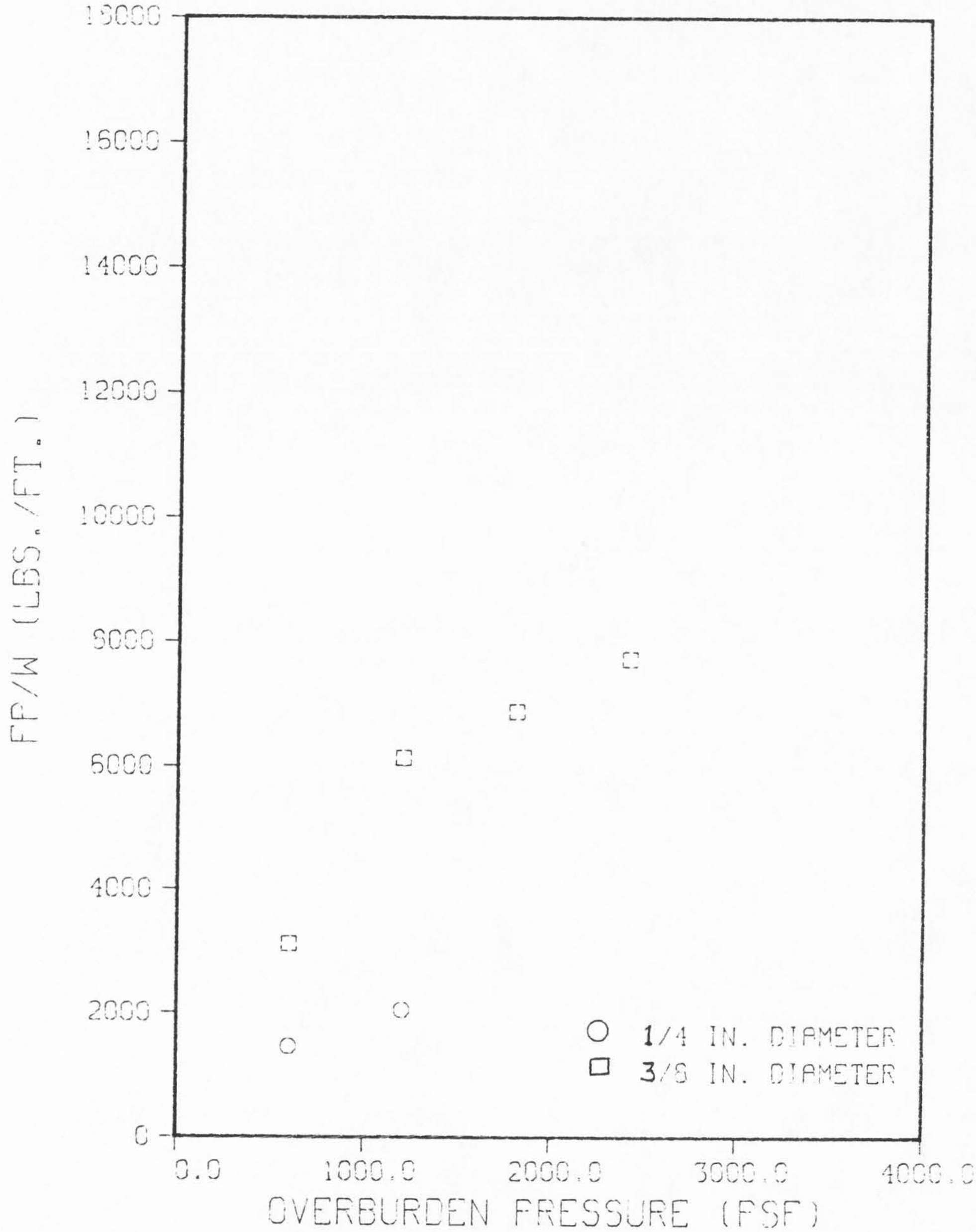


Figure 33. Affect of diameter. Washed sand - N=3

than the 1/4 inch diameter wire except in one set of tests which show about the same pull out resistance. With three transverse wires, the pull out resistance shows about the same no matter what diameter. This again could be due to experimental error.

For the pea gravel and the clean sand, the results show an increase in pull out resistance with an increase in wire diameter. Only one test does not show this and this test was done with two different types of rods. The 1/4 inch diameter wire was pulled using an A-36 steel rod and the 3/8 inch was pulled using a 4340 alloy steel rod. With the difference in materials and new strain gages, this could account for the difference in readings.

On the average, the tests showed an increase in pull out resistance with an increase in the number of embedded wires and also with an increase in the diameter of wire. The pea gravel and the clean sand definitely showed this relationship to be true. The silty sand was not as decisive, but did show that it was true for some tests.

Best relationship for each soil type

The relationship between the pull out resistance and the mat parameters (number of transverse wires, overburden pressure, diameter, length of embedment, etc.) was believed to be linear from earlier studies performed by Peterson and Anderson (1980). Many different types of plots were tried to find out which one was best suited for all the test results. Plots were tried which plotted the following: $N*\sigma_v*L$ versus Fp/w, σ_v versus Ft/w, $N*\sigma_v$ versus Fp/w, σ_v versus Fp/wL, and $N*\sigma_v*d$ versus Fp/w, where:

Fp = pull out force due to transverse wires,

w = width of mat,

N = number of transverse wires,

σ_v = overburden pressure,

L = length of embedment,

d = diameter,

Ft = total pull out force.

A plot of σ_v versus Fp/wL is shown for the silty sand soil in Figure 34. The coefficient which is used as an indication of how good the data points fit a straight line is called the coefficient of determination. For this plot the coefficient was 0.30. A coefficient of 1.0 is a perfect line and a coefficient of 0.0 is a terrible line. This plot and most of the other plots did not plot very well as a straight line.

All of these plots were plotted with the point of failure taken at a displacement of the mat of 0.75 inch. This number was obtained from the Federal Highway Administration in meetings held with Professor Loren Anderson and Mr. William Hilfiker. This point was used on all plots to obtain the failure point.

All the different plots were tried without including the tests where welds were broken. The plots with no broken welds included did not vary significantly from the plots that included all the tests. There were eight mats that had broken welds. The fact that 25 percent of the tests had broken welds indicates that care is needed in making sure the mats are welded securely so that welds will not break in an actual embankment.

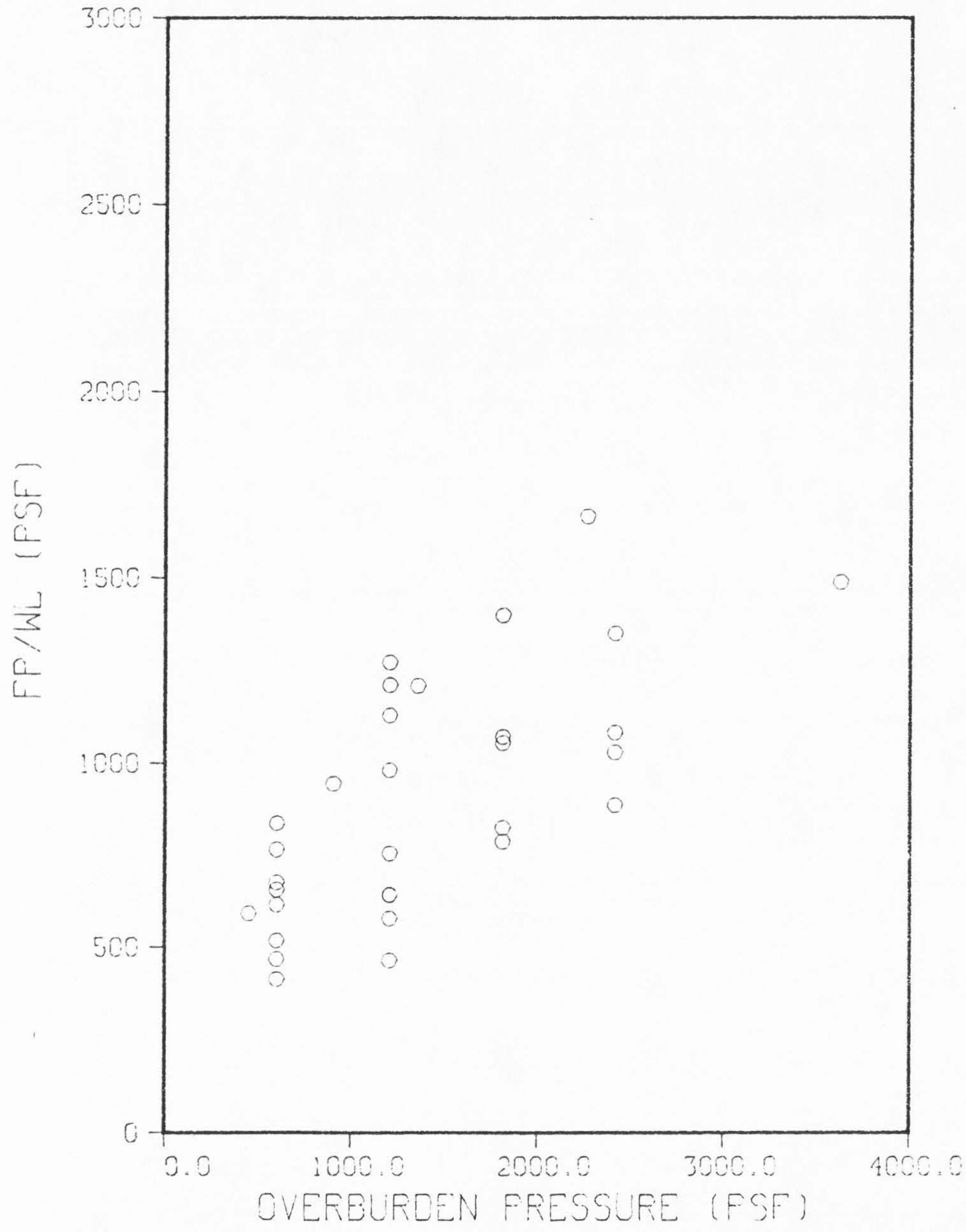


Figure 34. σ_v versus Fp/wL

The best plot for the data points was $N*\sigma_v*d$ versus Fp/w. The plots for all the tests in the three different soils are shown in Figures 35, 36, & 37. All of the tests which were run in one type of soil are plotted together. The coefficient of determination for the silty sand, pea gravel, and washed sand respectively are 0.78, 0.88, and 0.97. The scatter of the data is the worst for the silty sand, which is consistent with the other results discussed earlier for effects of diameter and number of transverse wires. This is probably due to experimental error. The plots are very good for the different soils.

One of the uses of these plots is for design purposes. Each soil type could have a design equation to determine the pull out resistance given the diameter of wire to be used, the position of the mat in the embankment, and the number of transverse wires behind the failure plane. If a soil was to be used in an embankment which was close to one of these three types, the pull out resistance could be calculated by the equation for the best fit line. The equations which could be used are listed below.

Silty Sand:

$$F_t = 2143 + \sigma_v d(\pi LM\tan\delta) + 17.61N) \qquad N\sigma_v d \geqslant 113.6 \qquad (5)$$

$$F_t = \sigma_v d(\pi LM\tan\delta + 36.47\ N) \qquad N\sigma_v d \leqslant 113.6 \qquad (6)$$

Washed Sand:

$$F_t = 633 + \sigma_v d(\pi LM\tan\delta) + 36.8\ N) \qquad (7)$$

Pea Gravel:

$$F_t = 712 + \sigma_v d(\pi LM\tan\delta) + 38.1N) \qquad (8)$$

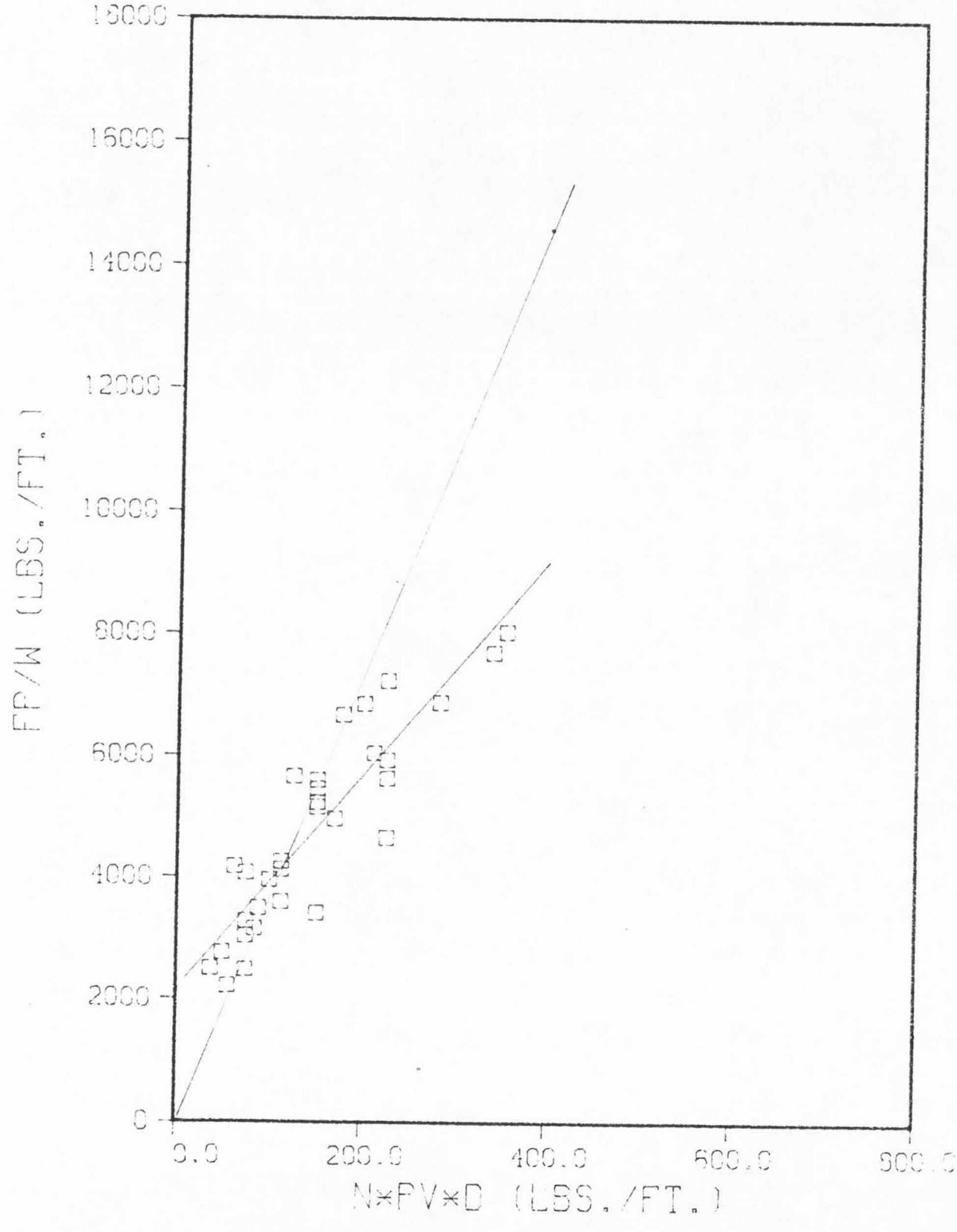


Figure 35. Plot of Nσvd versus Fp/w for silty sand

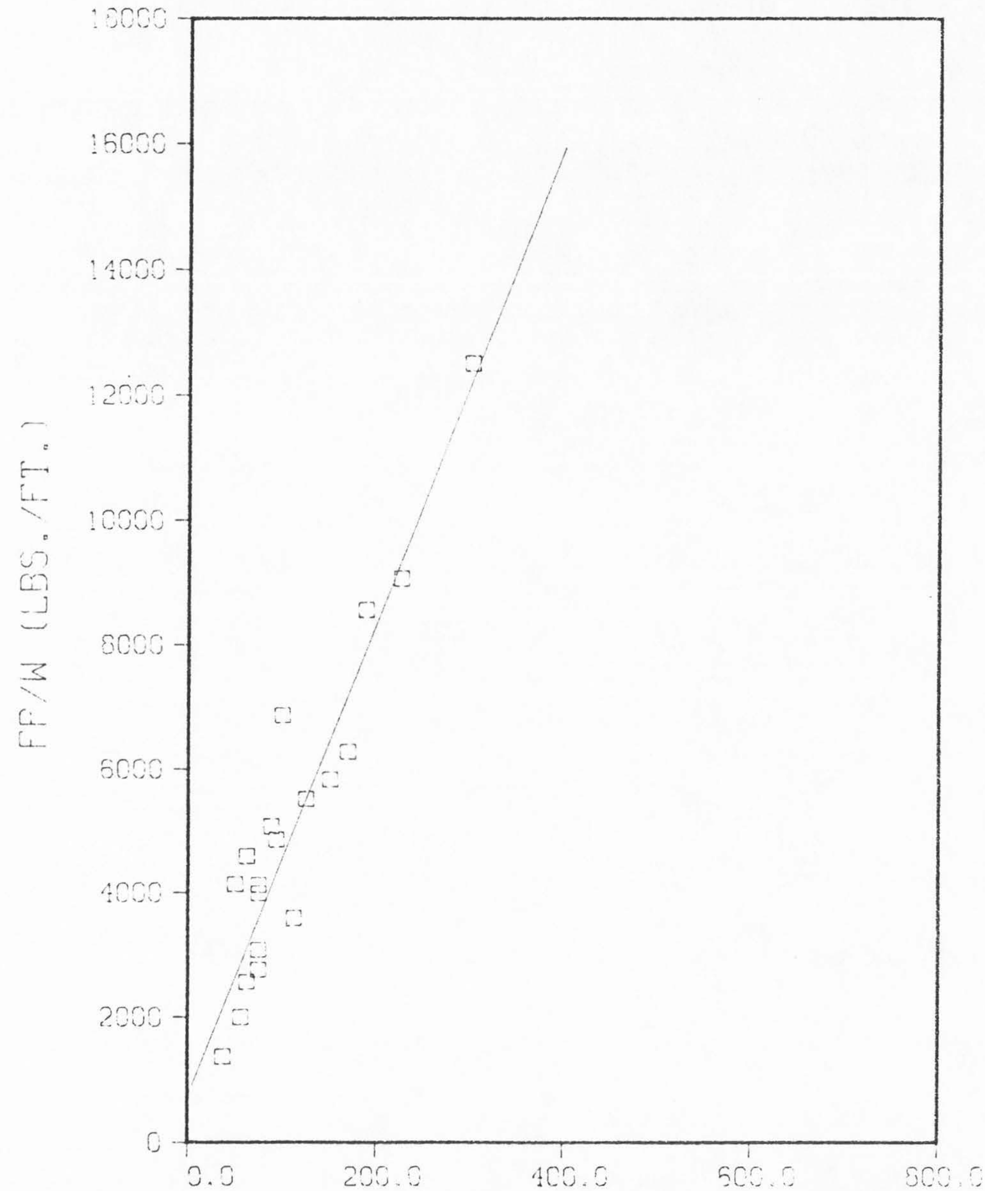


Figure 36. Plot of Nσvd versus Fp/w for pea gravel

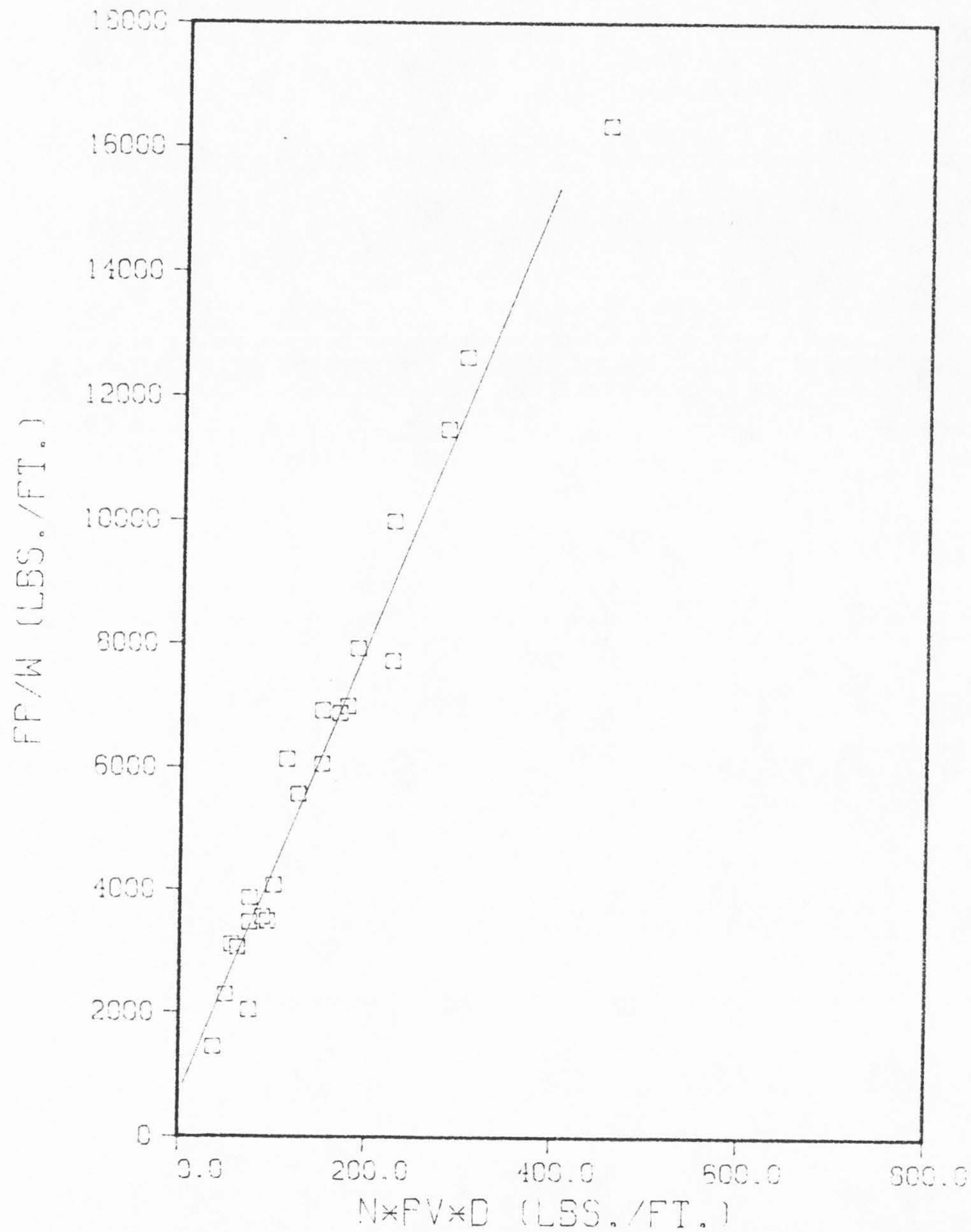


Figure 37. Plot of Nσvd versus Fp/w for washed sand

where:

F_t = total pull out force

σ_v = vertical overburden pressure

d = diameter

π = 3.1416

L = length of embedment

M = number of longitudinal wires

$\tan\delta$ = friction factor between soil and reinforcement

N = number of transverse wires

The reason the silty sand has two equations is because the best fit line intersects the axis at 2143 pounds per foot. This would mean that a mat with no soil above it would have a pull out resistance of over 2000 pounds per foot of mat. Since the pull out resistance should be zero at zero overburden pressure, a second line which is the line of average slopes is used to correct the lower values to give more reliable results.

Relationship between soil parameters and pull out resistance

Direct shear tests were performed on the three different soil types. These results are in Appendix A. From these tests the angle of internal friction and the cohesion intercept were found. Using the initial calculations suggested by Peterson and Anderson (1980) the pull out resistance is easily related to the friction angle and the cohesion intercept by using the Terzaghi-Buisman Bearing Capacity Equation.

$$\frac{Fp}{Nwd} = cNc + \sigma_v Nq \tag{9}$$

where:

Nc = cohesion factor evaluated at 0.86ϕ.

Nq = factor evaluated at 0.86ϕ.

A comparison of this theoretical equation with the actual results is found in Figures 38, 39, & 40. In these plots, the solid line is the best fit line and the dashed line is the fit obtained theoretically. As shown by the plots, the washed sand and the silty sand agree very well with the equation. These two curves could be very easily used for design instead of referring to the individual plots of N*Pv*d versus Fp/w (Figures 35-37).

The correlation for the pea gravel was not as good. One reason for the poor correlation could be due to incorrect strength parameters. The pea gravel which was used in the test did not have any cohesion; however, the test showed a cohesion of 1882 psf which is more than the silty sand had. This could be due to the size of grains in the gravel which were too large to be tested in the direct shear machine. The results of equation 9 are conservative and could still be used if necessary.

By measuring the angle of repose of the pea gravel used in the tests, a different value of friction angle was obtained which was much closer to the actual data. The plot of this line using the angle of repose and a cohesion of 0.0 psf is also shown in figure 38. As can be seen from the plot, the line is very close to the best fit line of the data points, which suggests that the direct shear tests are off, and more important that this relationship does hold for the pea gravel and could be used as a design procedure.

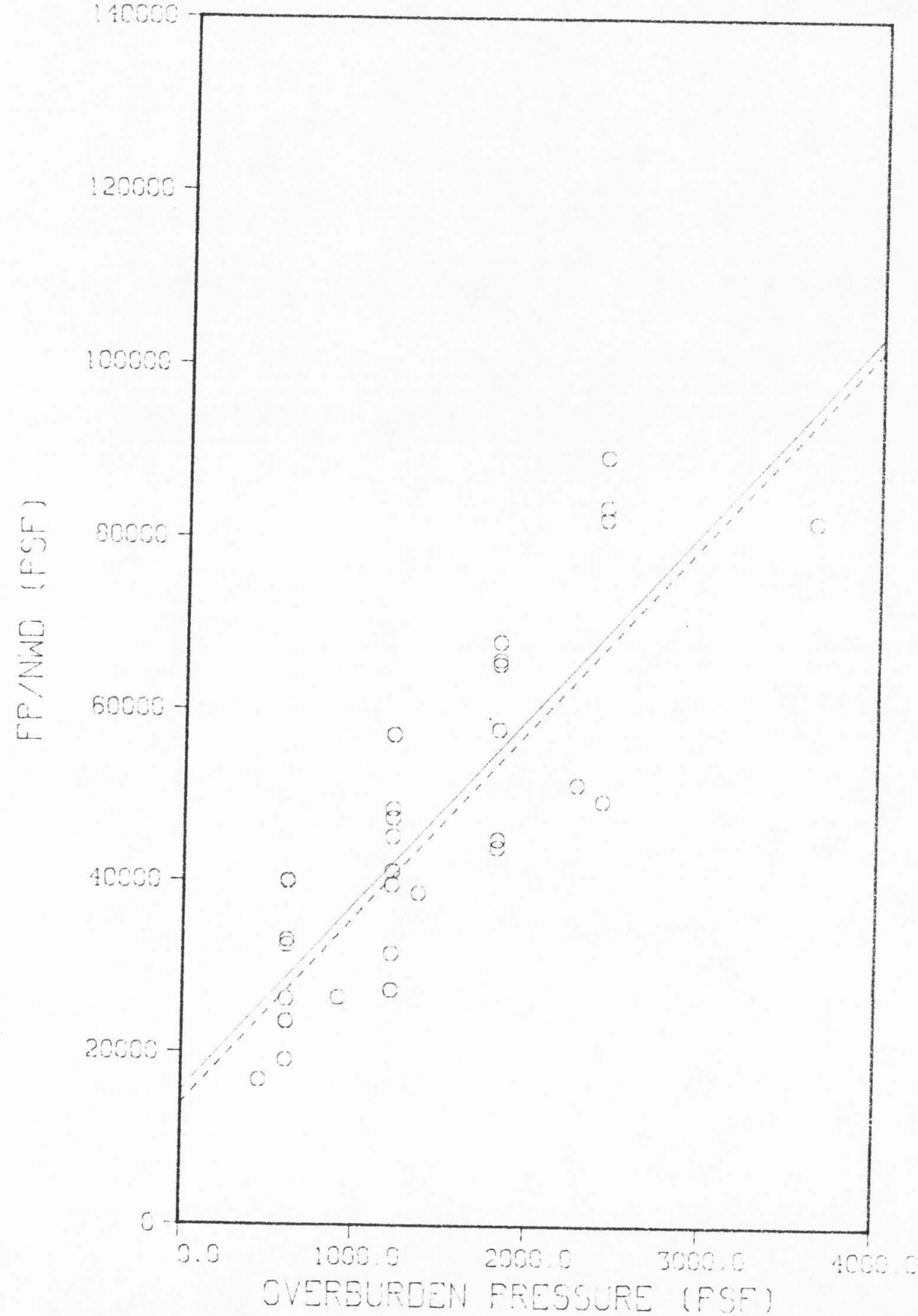


Figure 38. Plot of σv versus Fp/NwD for silty sand

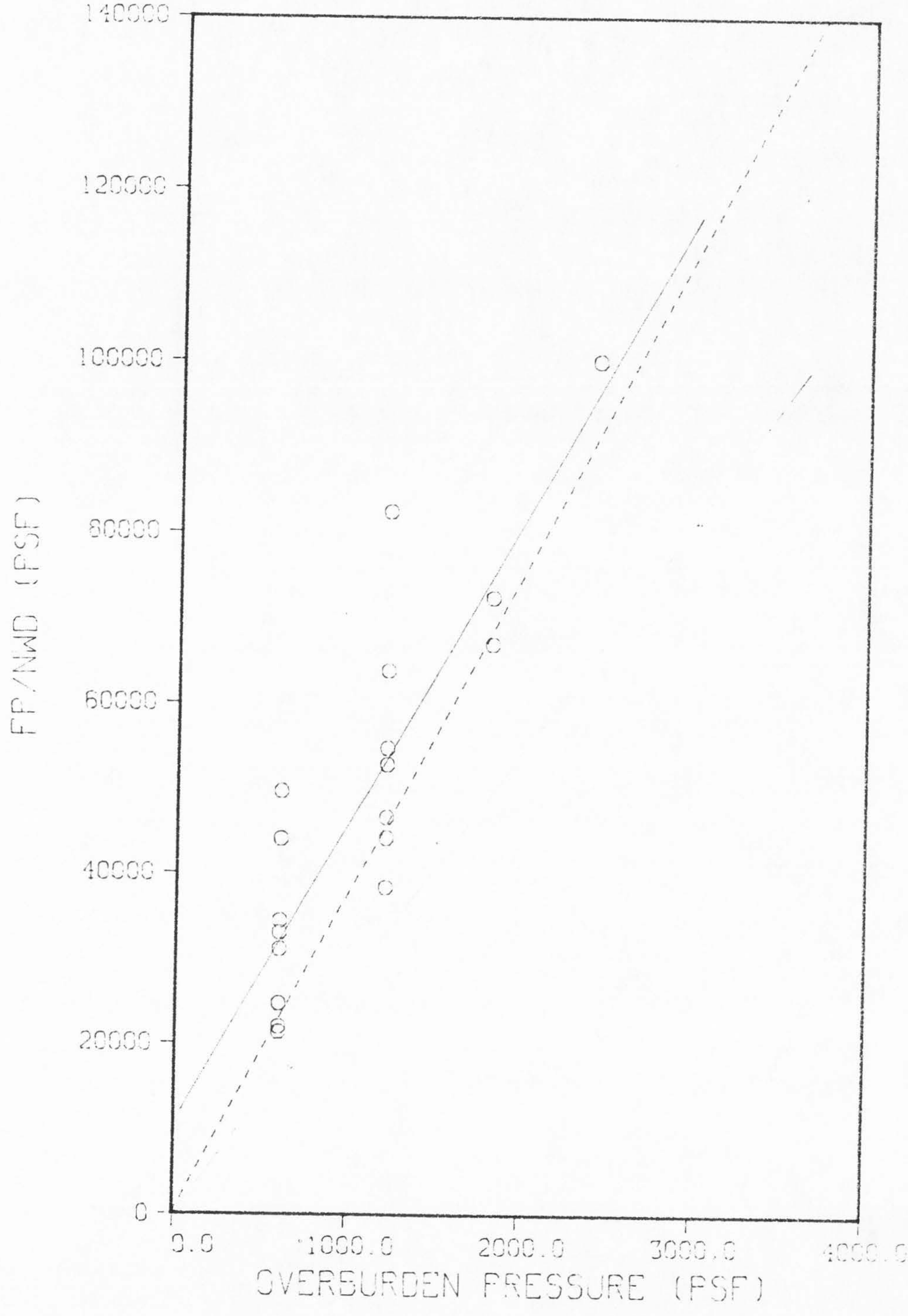


Figure 39. Plot of σv versus Fp/NwD for pea gravel

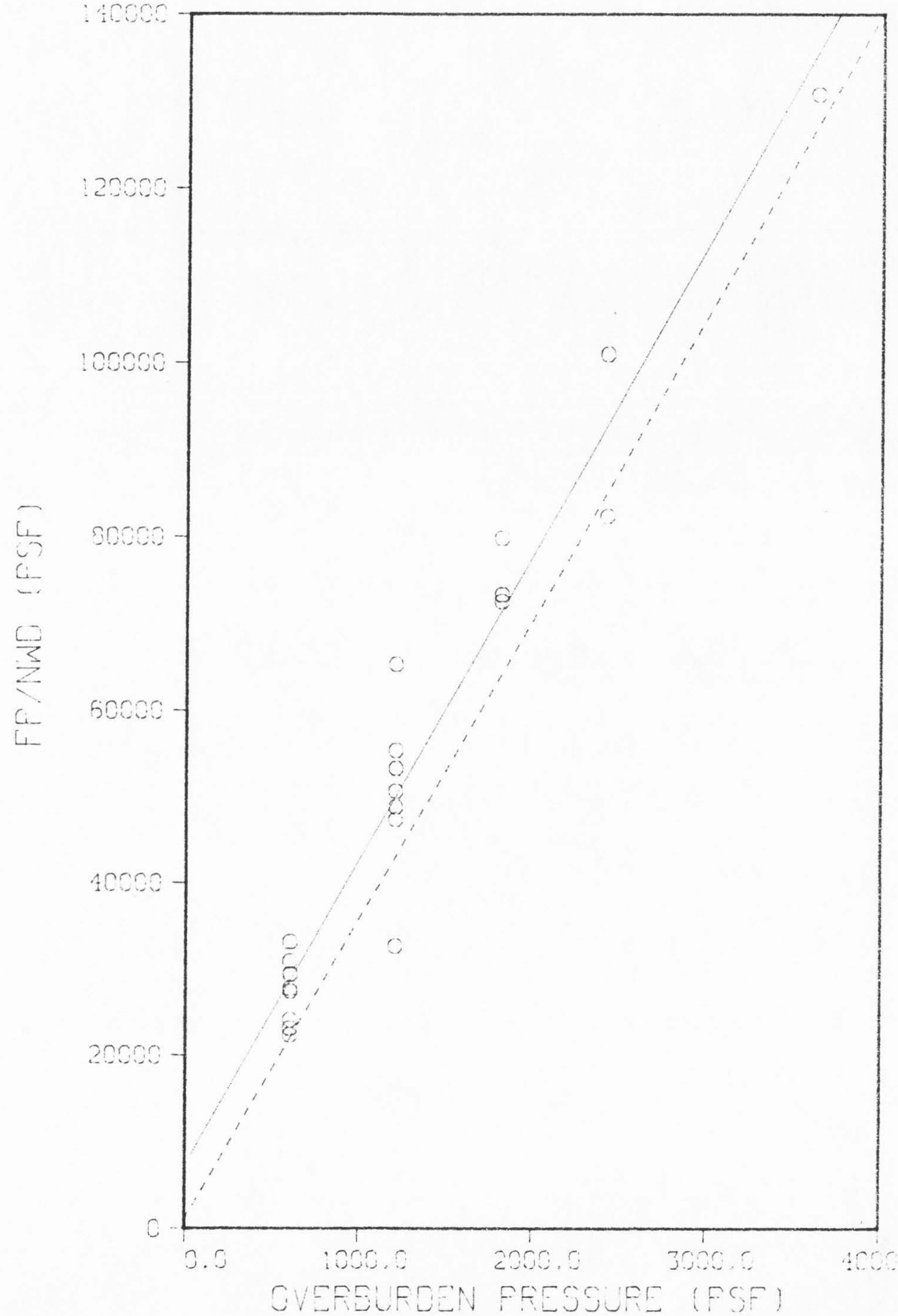


Figure 40. Plot of σv versus Fp/NwD for washed sand

The reason for using a factor of 0.86 for the friction angle in determining Nc & Nq for equation 9 is that the silty sand had triaxial tests performed on the soil and the friction for these tests were 0.86 times the direct shear test results. The cohesion was similar for both tests. The results using the friction angle obtained from the triaxial tests were in very good agreement with the data points from the pull out tests. Because there were no other methods of obtaining the friction angle for the pea gravel and the washed sand, a factor was introduced to compensate for the high values obtained from the direct shear machine.

The real problem in using this relationship for pull out resistance is in obtaining the quantities of the friction angle and the cohesion intercept. Different types of machines may give different friction angles and cohesion intercepts which could mean drastic changes in the pull out resistance. If this problem could be worked out, this would be an excellent method of getting the pull out resistance for all kinds of soils.

To use equation 9 for design purposes, all that is needed is the soil parameters phi and c. The quantities Nc and Nq are determined from phi. The quantity c*Nc is the intercept for the vertical axis and the quantity Nq is the slope of the line. With this line determined, a value of Fp/Nwd can be obtained for any overburden pressure. With this value the number of transverse wires and diameter can be multiplied to obtain the pull out force per unit width of the mat. Example 1 demonstrates how this is done for given soil parameters.

Example 1: Given: If $\phi = 36°$

$c = 500$ psi

Solution: $Nc = 50.6$

$Nq = 37.7$

$cNc = 25{,}300$

$$Fp/NwD = 25{,}300 + 37.7\ \sigma_v$$

Plot is shown in Figure 41.

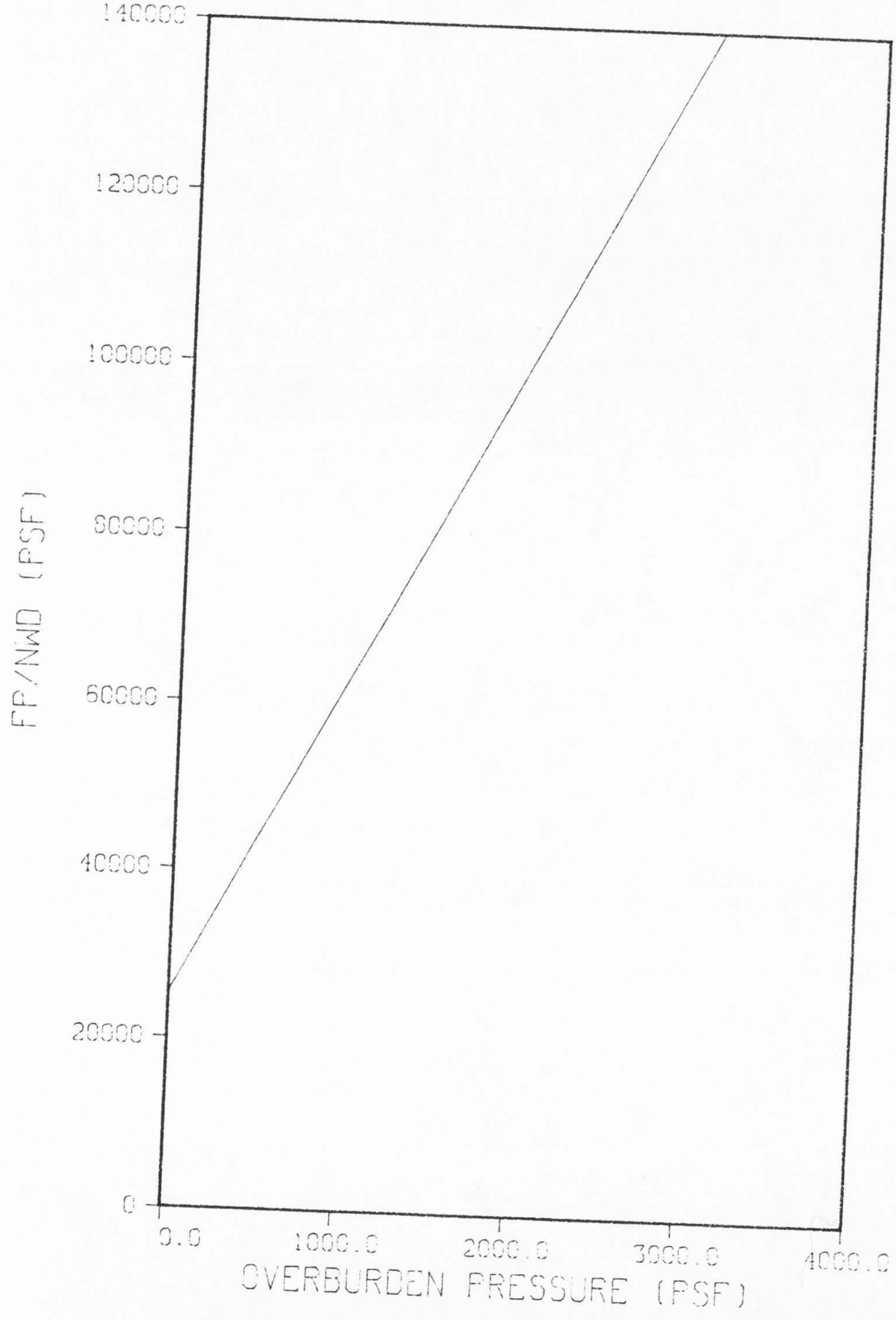


Figure 41. Plot from Example 1.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

Introduction

Purpose of study

The purpose of this study was to evaluate the pull out resistance of welded wire mats in different soils. The main objectives were as follows:

1. Find the best relationship for each soil type.

2. Verify that the pull out resistance is a function of the diameter and the number of transverse wires.

3. Relate the pull out resistance to a soil parameter.

Method of procedure

To fulfill the objectives, ninety-three pull out tests were performed on four different diameters of wire mesh reinforcement. This data was then plotted to identify the relationships between the soil and the reinforcement.

Conclusions

Based on the test data and the plots which have been made, the following conclusions can be drawn.

1. The relationship that yields the best straight line approximation for the results is a plot of Fp/w versus $N*\sigma_v*d$. All tests in

each soil type can be plotted and the results are very close to a straight line.

2. The plots of Fp/w versus N*σv*d can be used for design in the cases where the soil to be used is close to the type used in the laboratory tests.

3. The pull out resistance is definitely a function of the number of embedded transverse wires. As the number of embedded transverse wires increases, the pull out resistance also increases.

4. The pull out resistance is also a function of the diameter of the wire. If the diameter of wire increases, then the pull out resistance also increases. This increase for small diameter changes can also be quite small.

5. The pull out resistance can be related to the friction angle and cohesion intercept of the soil. This relationship is dependent on how the friction angle and cohesion intercept of the soil is determined. Care MUST be exercised in using the plot given for granular soils to ensure the friction angle and cohesion intercept is compatible with the methods used to obtain these parameters presented in this paper.

Recommendations

Design recommendations

There are two choices to determine the pull out force which can be tolerated in an embankment. If the soil is similar to one which was used in the laboratory pull out tests, the pull out force can be calculated by using one of the following equations:

Silty Sand:

$$F_t = 2143 + \sigma_v d(\pi LM\tan\delta) + 17.61N) \quad N\sigma vd \geqslant 113.6$$

$$F_t = \sigma vd\ (\pi LM\tan\delta + 36.47N) \qquad N\sigma vd \leqslant 113.6$$

Washed Sand:

$$F_t = 633 + \sigma vd(\pi LM\tan\delta) + 36.8N)$$

Pea Gravel:

$$F_t = 712 + \sigma vd(\pi LM\tan\delta) + 38.1N)$$

where:

F_t = total pull out force

σ_v = vertical overburden pressure

d = diameter

π = 3.1416

L = length of embedment

M = number of longitudinal wires

$\tan\delta$ = friction factor between soil and reinforcement

N = number of transverse wires

The other alternative for a granular soil is to determine the angle of internal friction and the cohesion intercept, and use an equation which would give the pull out resistance for different values of overburden.

$$Fp/Nwd = cNc + \sigma_v Nq \qquad (10)$$

It is important to note that the number of transverse wires and the length are obtained for the portion of the mat which is behind the

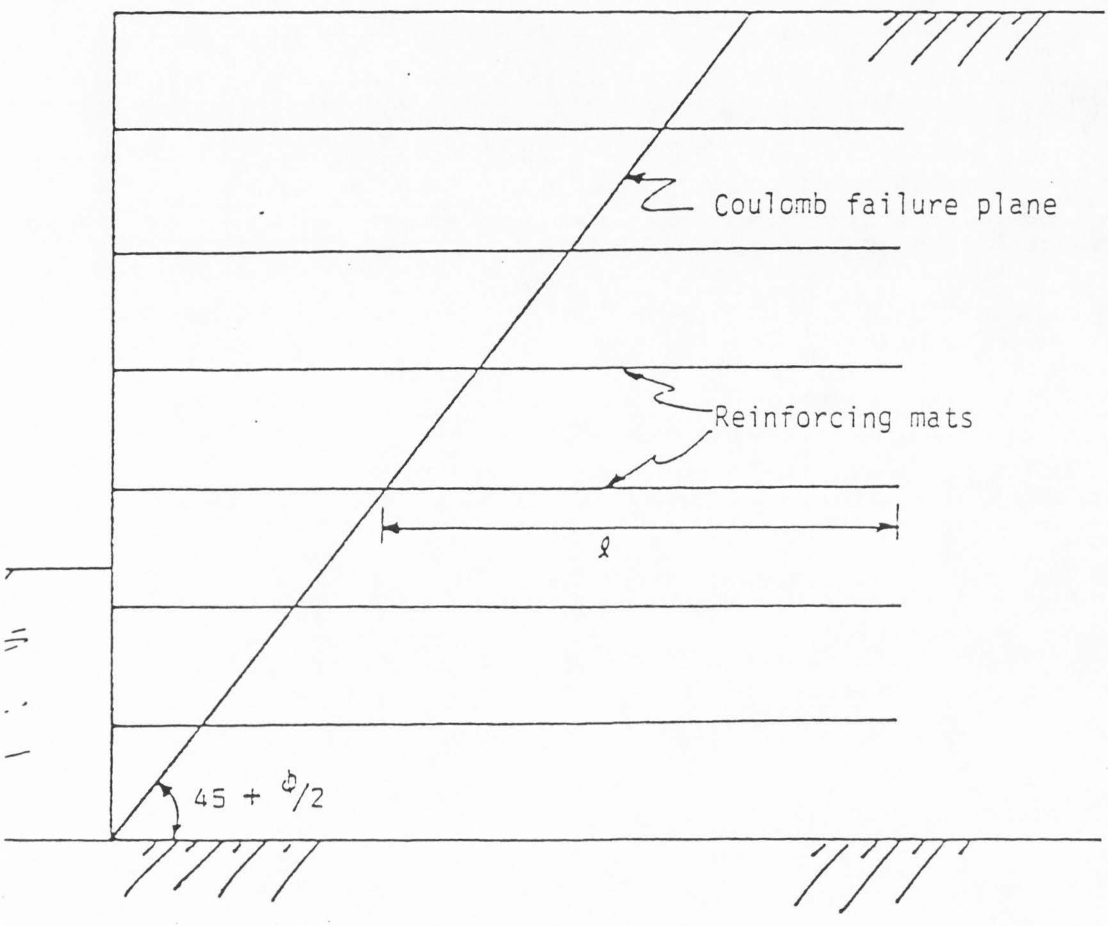


Figure 42. Coulomb failure plane

Coulomb failure plane (Figure 42). The recommended allowable pull out resistance is obtained by using a factor of safety of two.

$$F_{allowable} = \frac{F_t}{2}$$

Additional research recommendations

This study has provided some new results on different wire mats and different soils which will be useful for design purposes. Additional research is definitely needed to determine if the relationship of the angle of internal friction and the cohesion intercept to the pull out resistance is valid for all types of soils. One of the aspects of these tests should determine the best method for determining the friction angle and the cohesion intercept to be used for pull out resistance. Also, the applicability of this procedure to fine grained cohesive soils should also be investigated.

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APPENDIXES

Appendix A

Soil Data

The soils used as backfill material for the pullout tests included a silty sand, pea grave, and washed sand. These soils classify as a SM, GW, SW respectively using the Unified Soil Classification.

Extensive laboratory tests on the silty sand soil were performed in 1978 by W.C. Yu and G.A. Rush. The tests which are included in this Appendix are:

1. Atterberg limits.
2. Confined compression test.
3. Triaxial shear test.
4. Permeability test.

The grain size distribution and compaction tests that are included were performed in the laboratory in 1983 by Eve Jones and were nearly identical to the laboratory tests previously run by Yu and Rush. In 1983 direct shear tests were performed by Jon Bischoff and Mark Nielsen. These results are also included in this Appendix.

The Atterberg limits test revealed the soil was non-plastic with no liquid limit. The compaction test results indicated γ_{dry} = 125.4 pcf at an optimum moisture content of 9.5 percent (Fig. 44). The permeability test showed the soil to have a permeability of 1-4 ft/day.

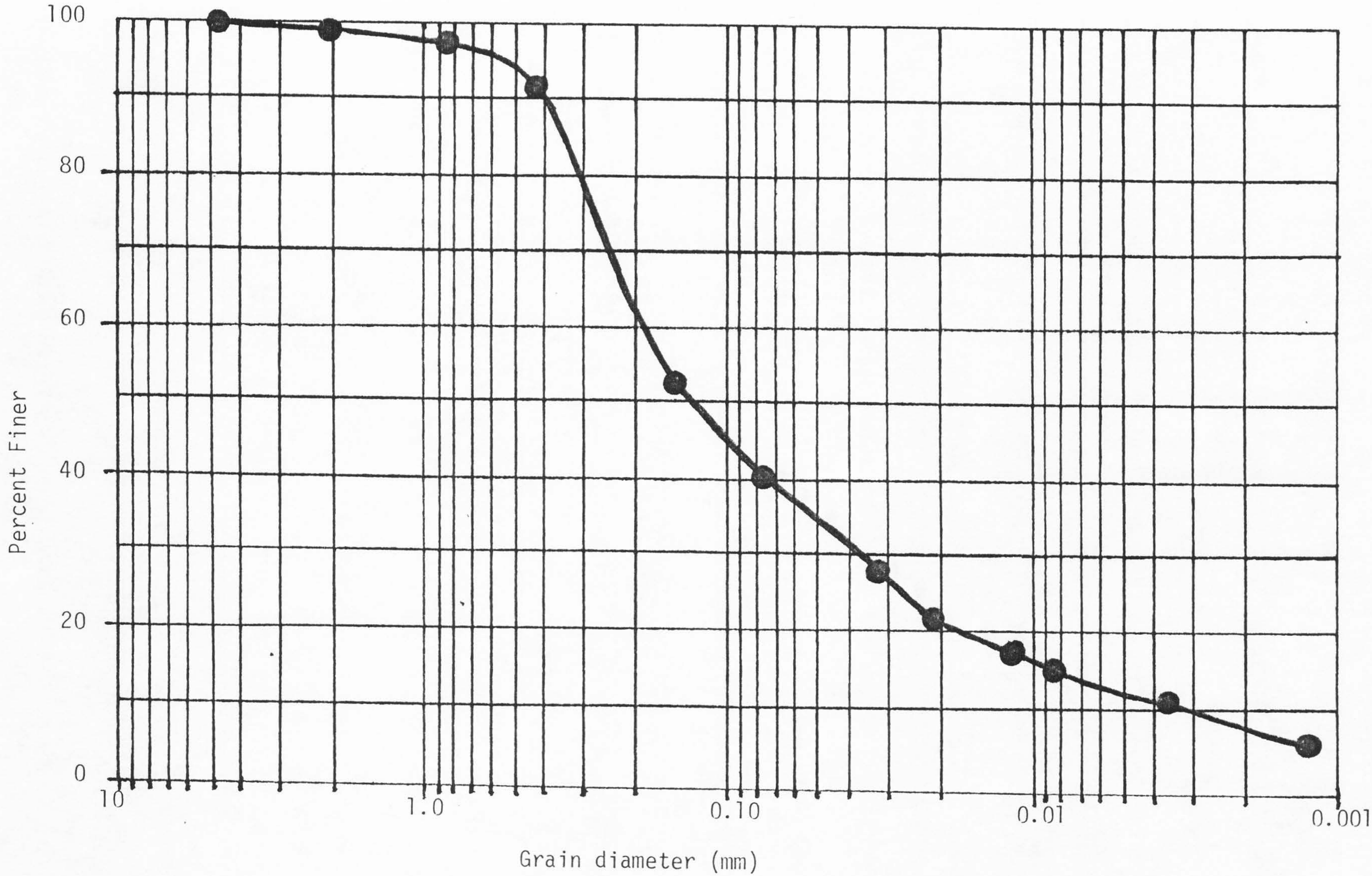

Figure 43. Grain size distribution for silty sand

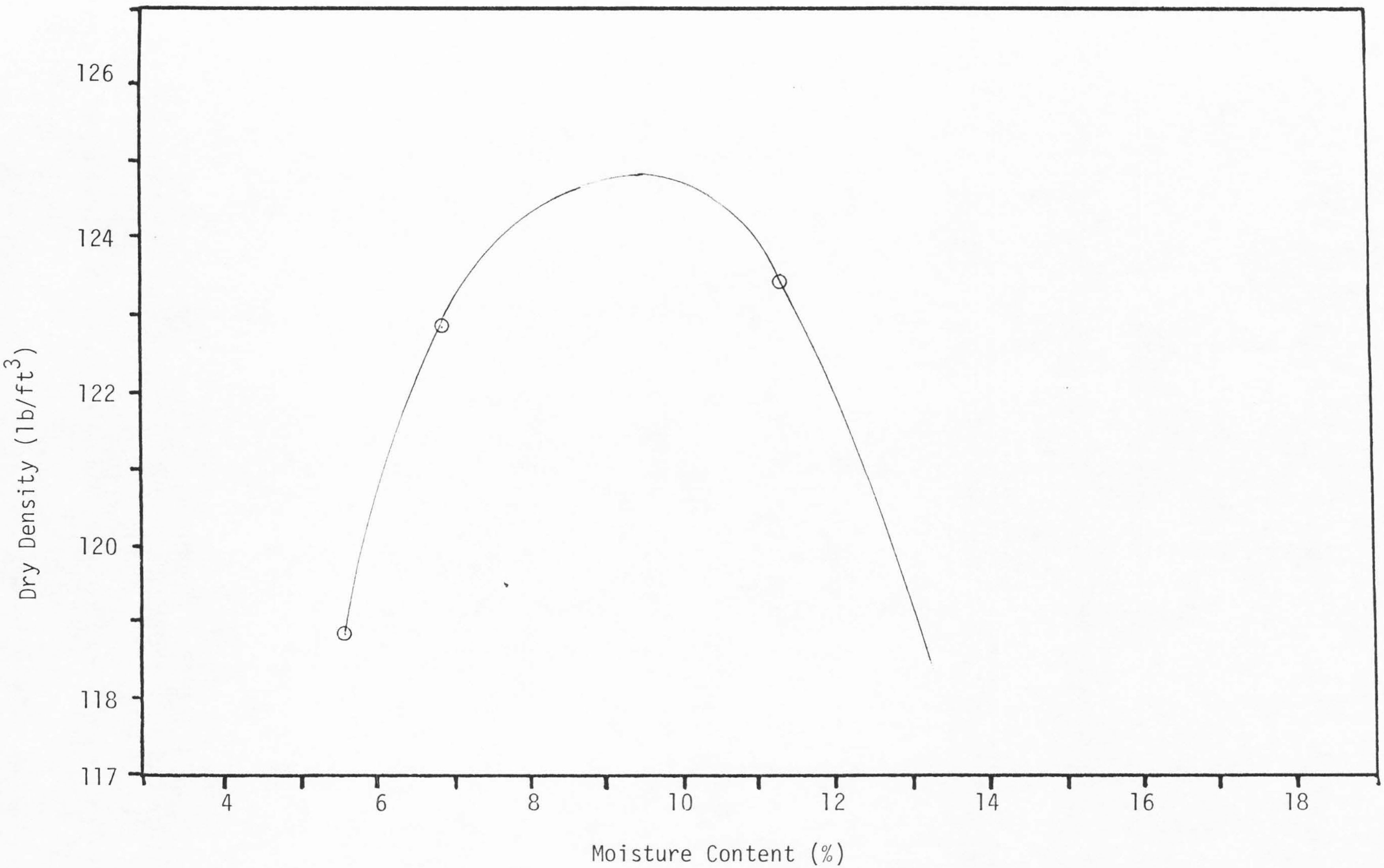

Figure 44. Moisture - density relationship for silty sand

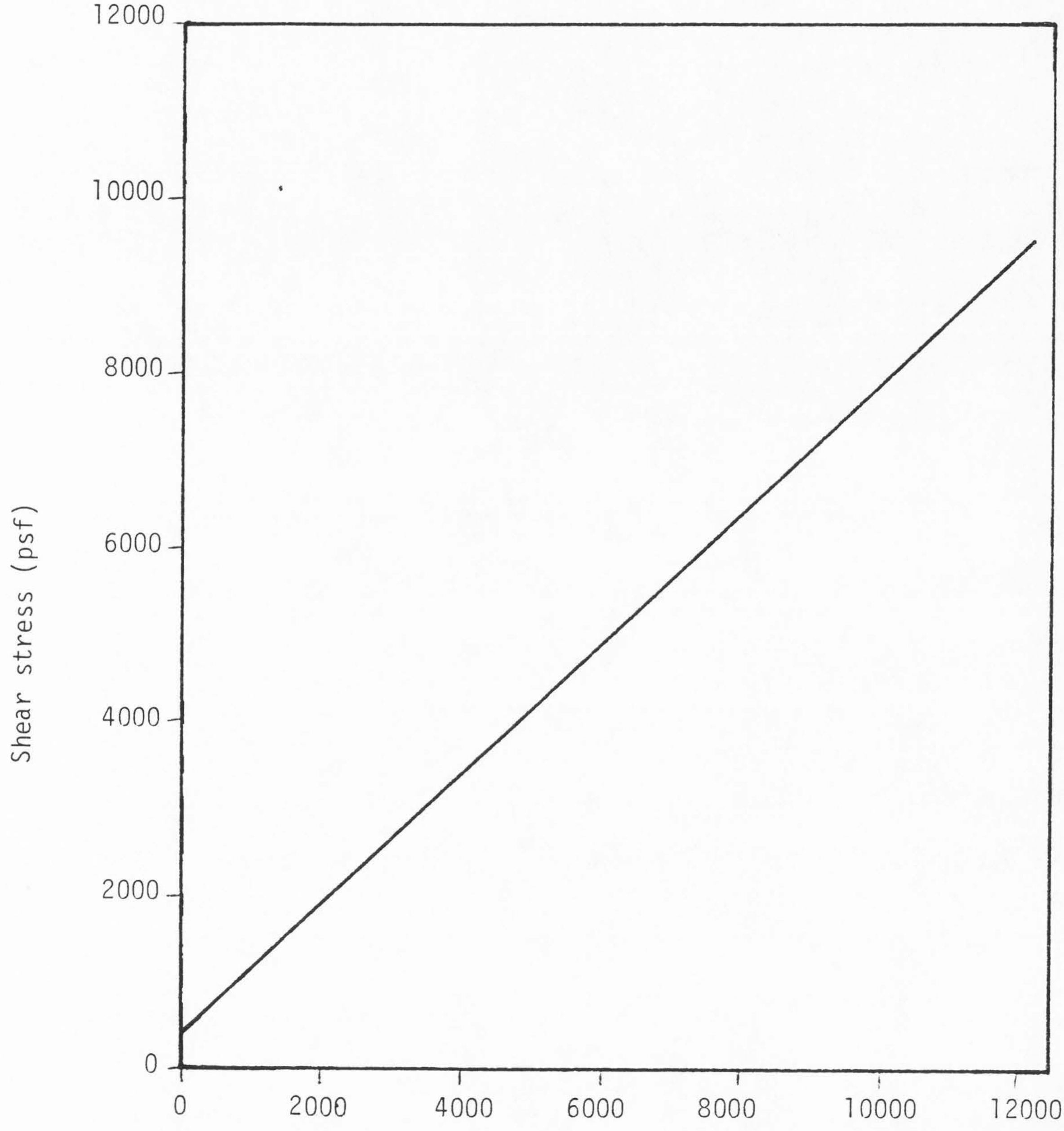

Figure 45. Direct shear test results for silty sand

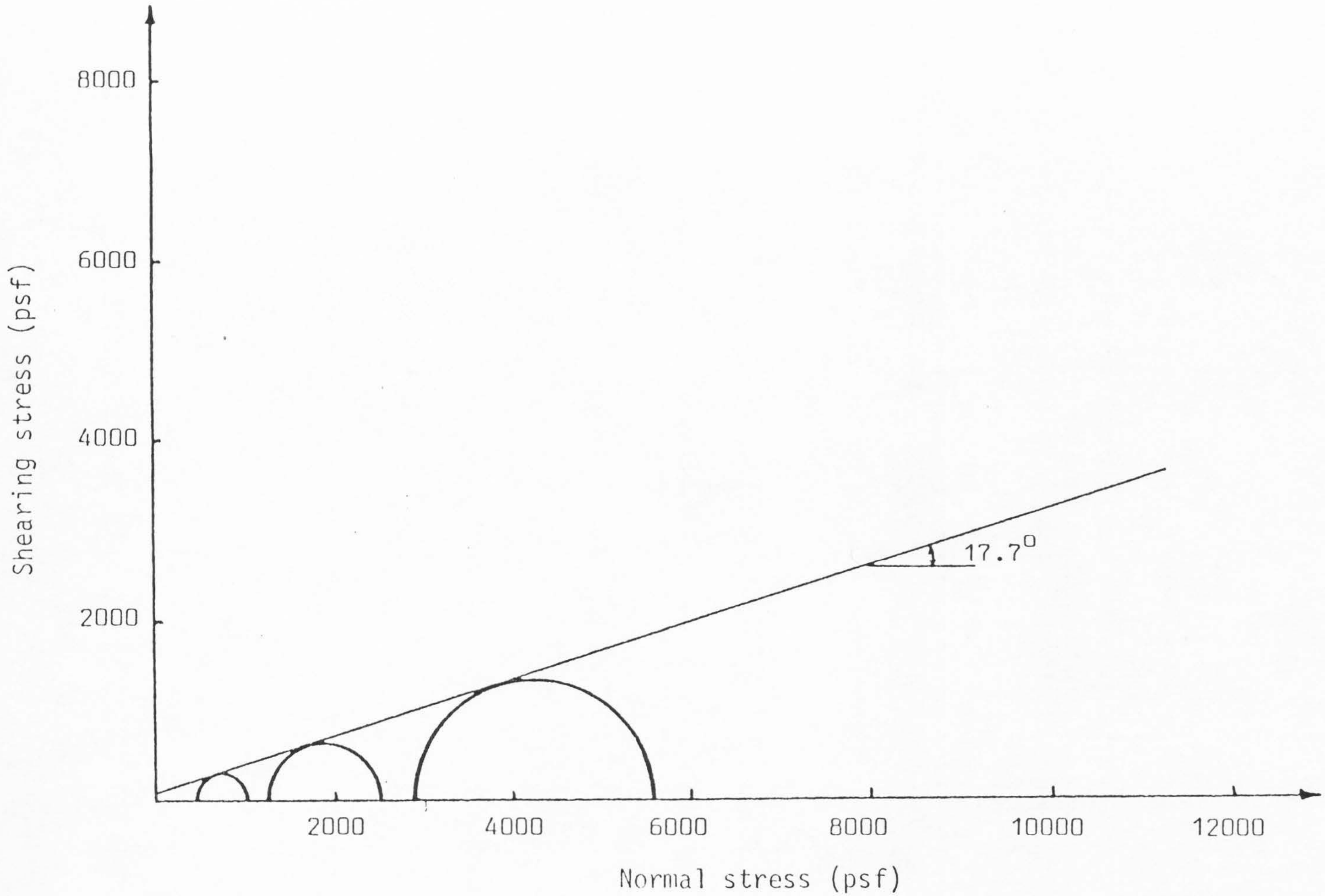

Figure 46. Triaxial shear test results on silty sand (75% compaction)

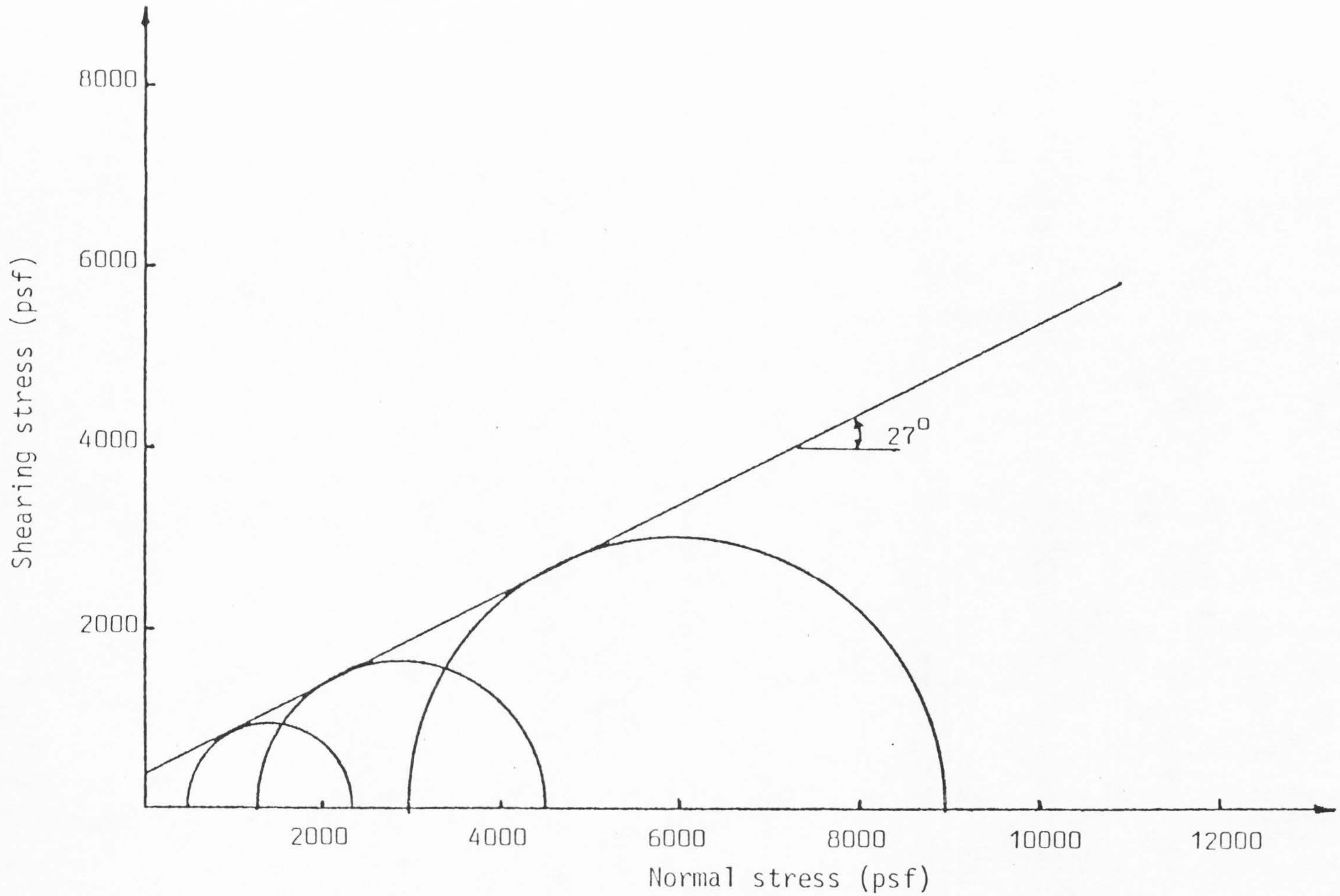

Figure 4.7 Triaxial shear test results on silty sand
(85% compaction)

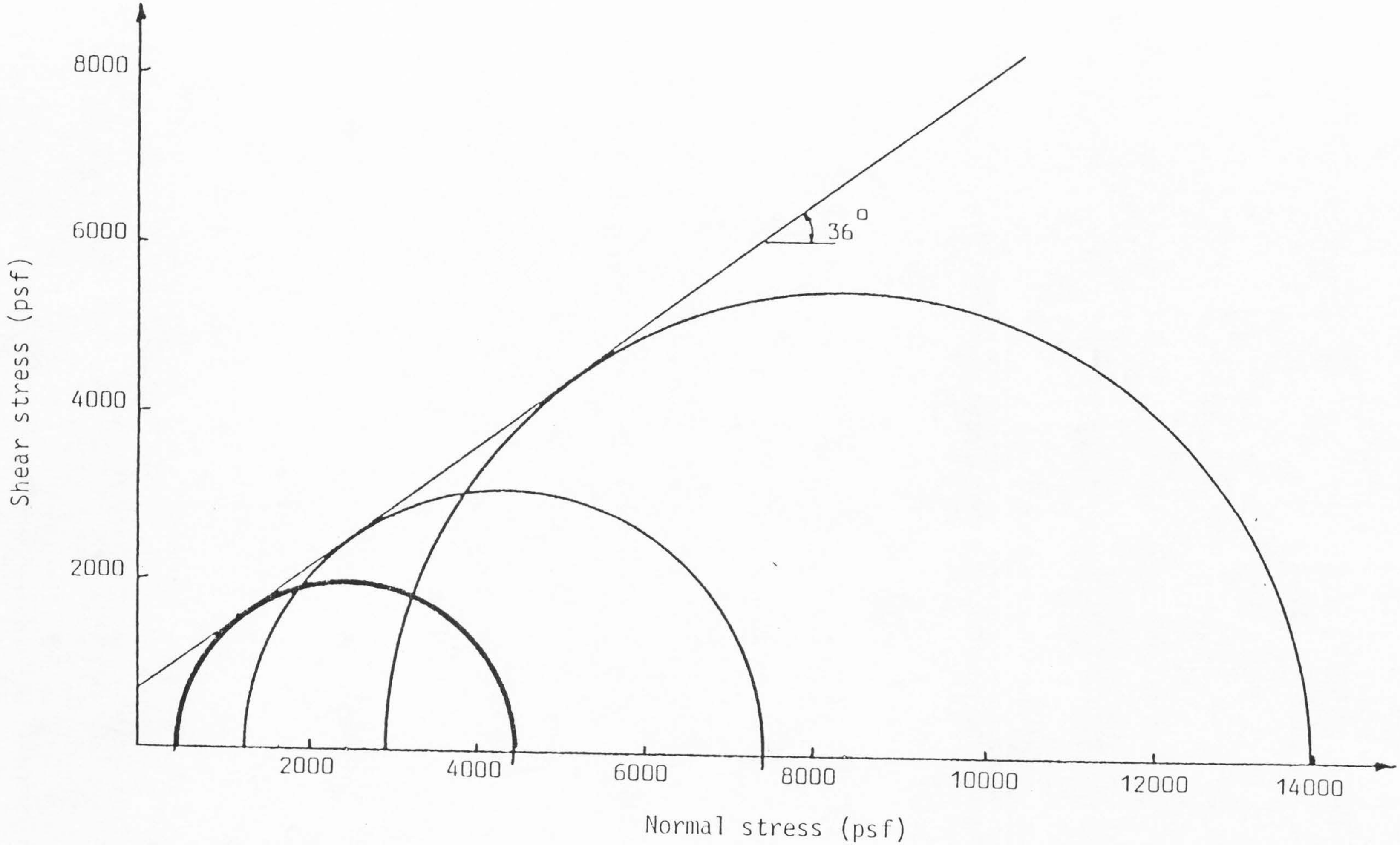

Figure 48. Triaxial shear test results on silty sand (95% compaction)

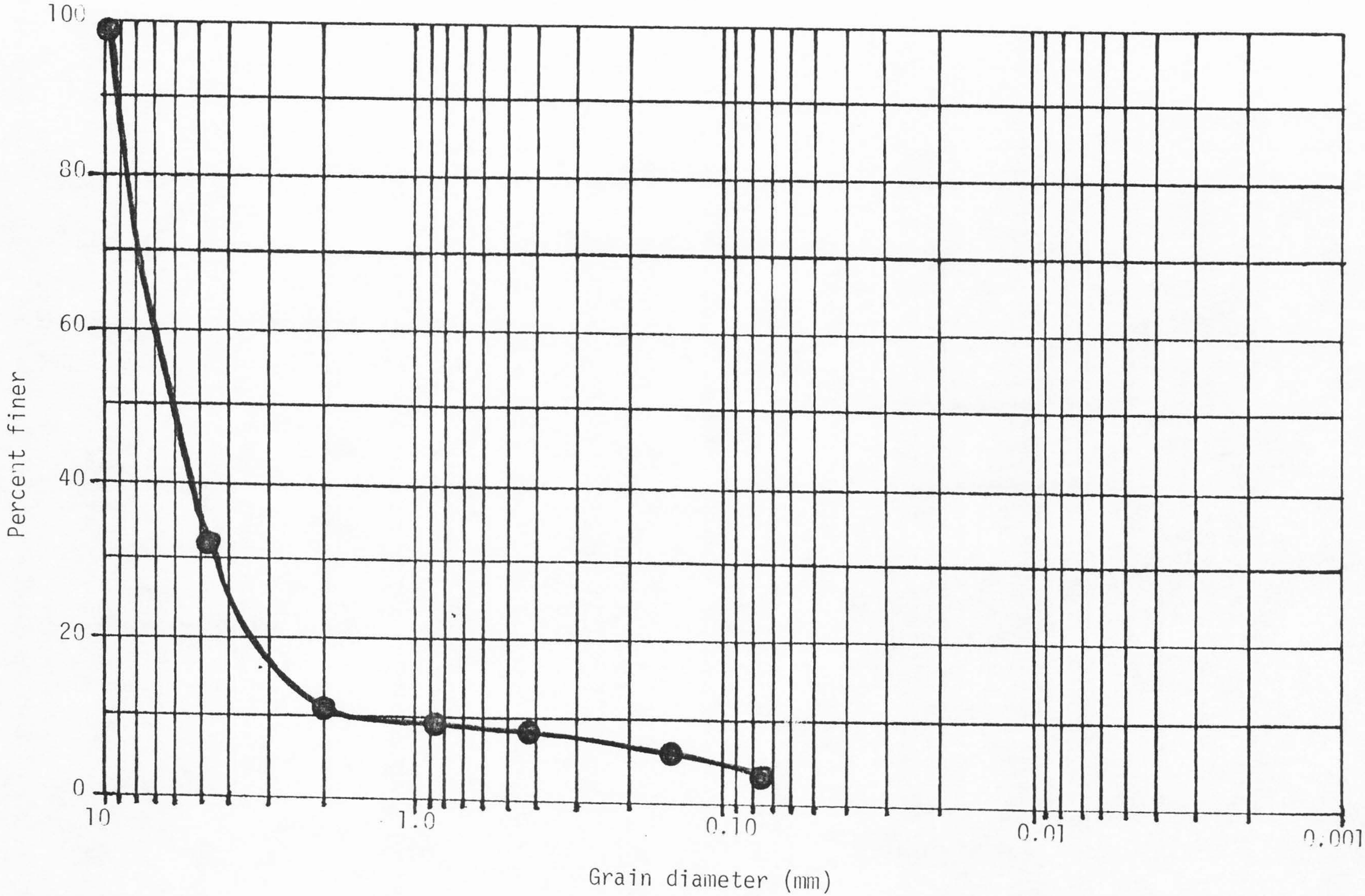

Figure 49. Grain size distribution for pea gravel

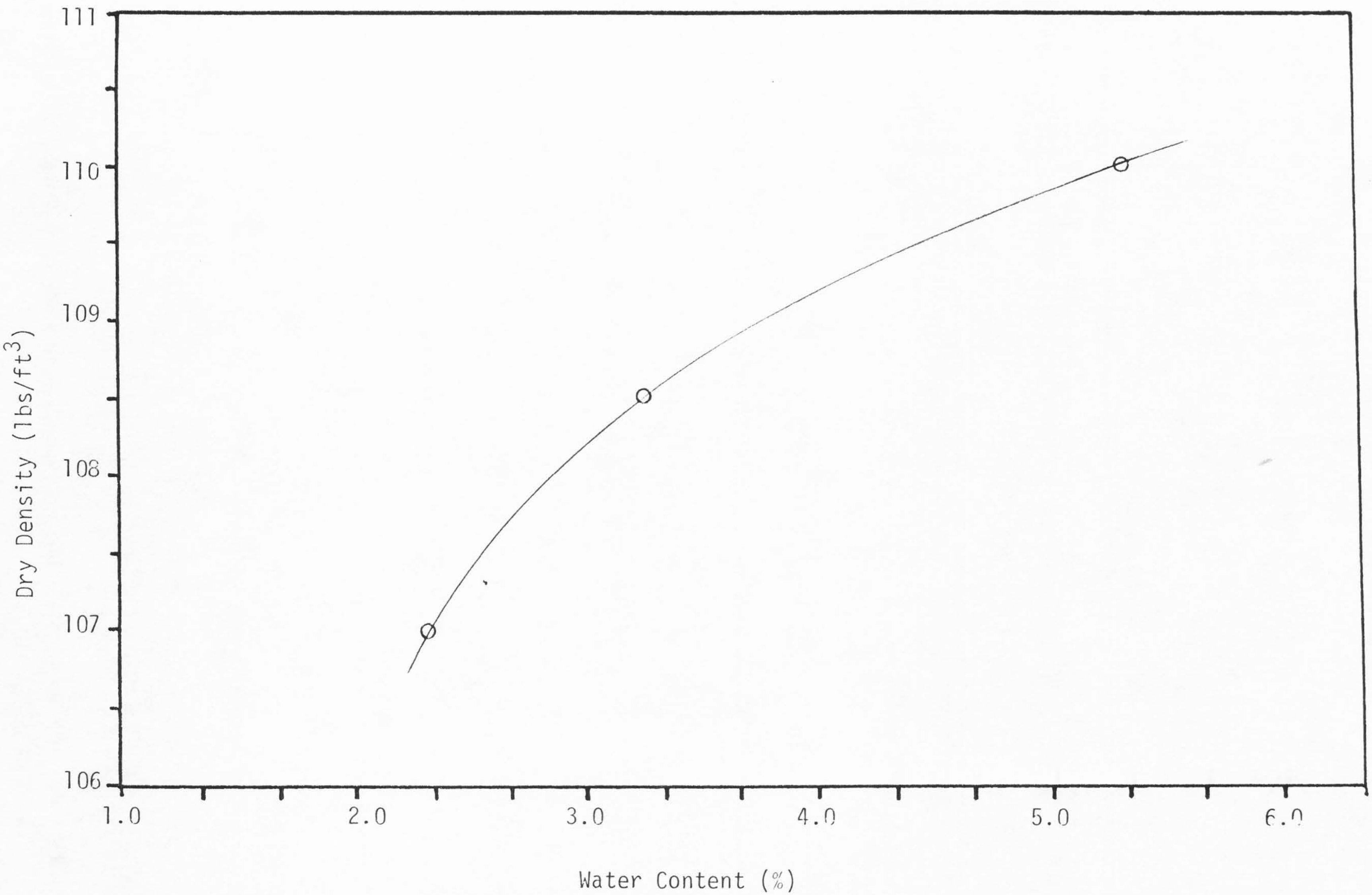

Figure 50. Moisture density relationships for pea gravel

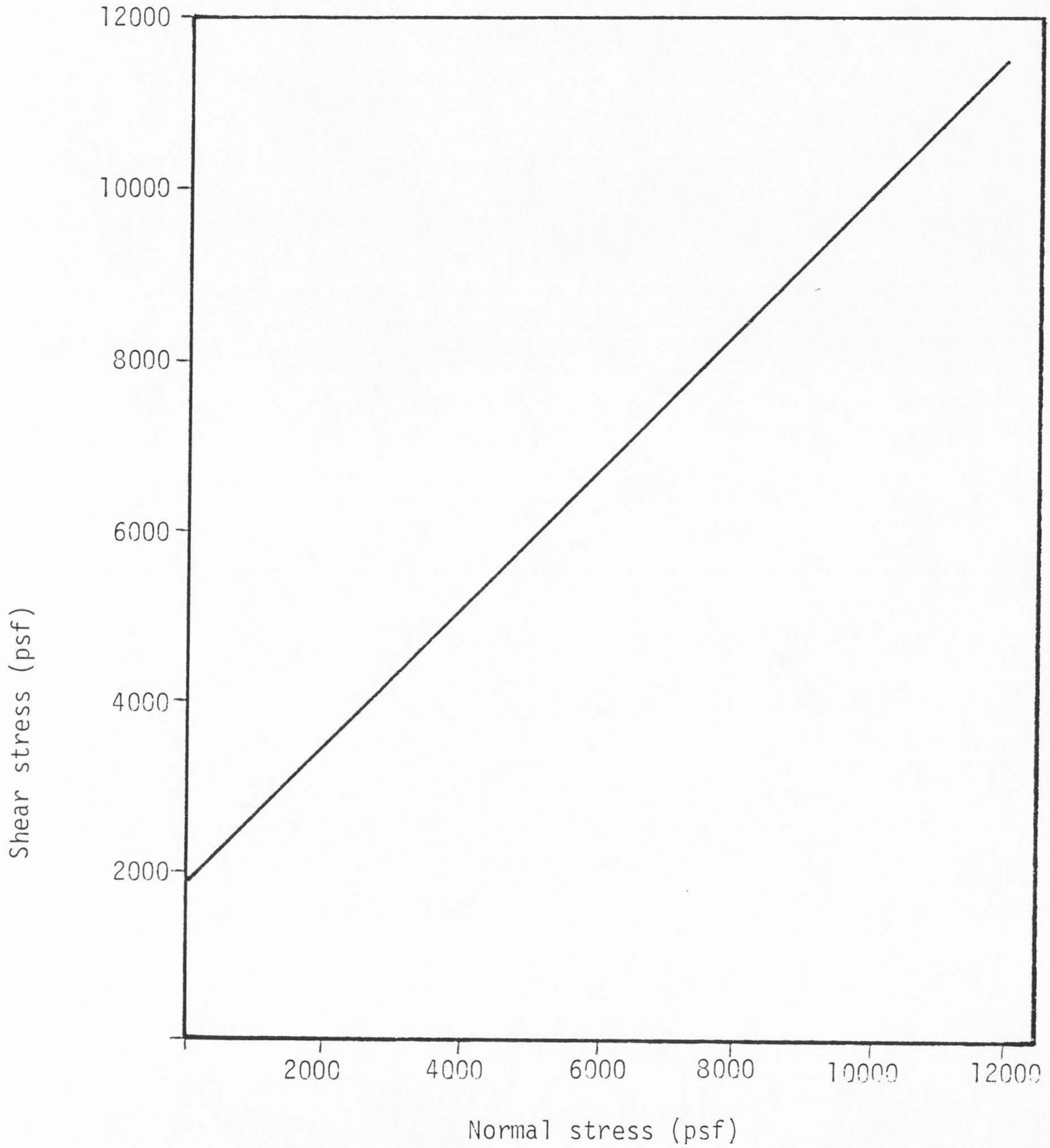

Figure 51. Direct shear test results for pea gravel

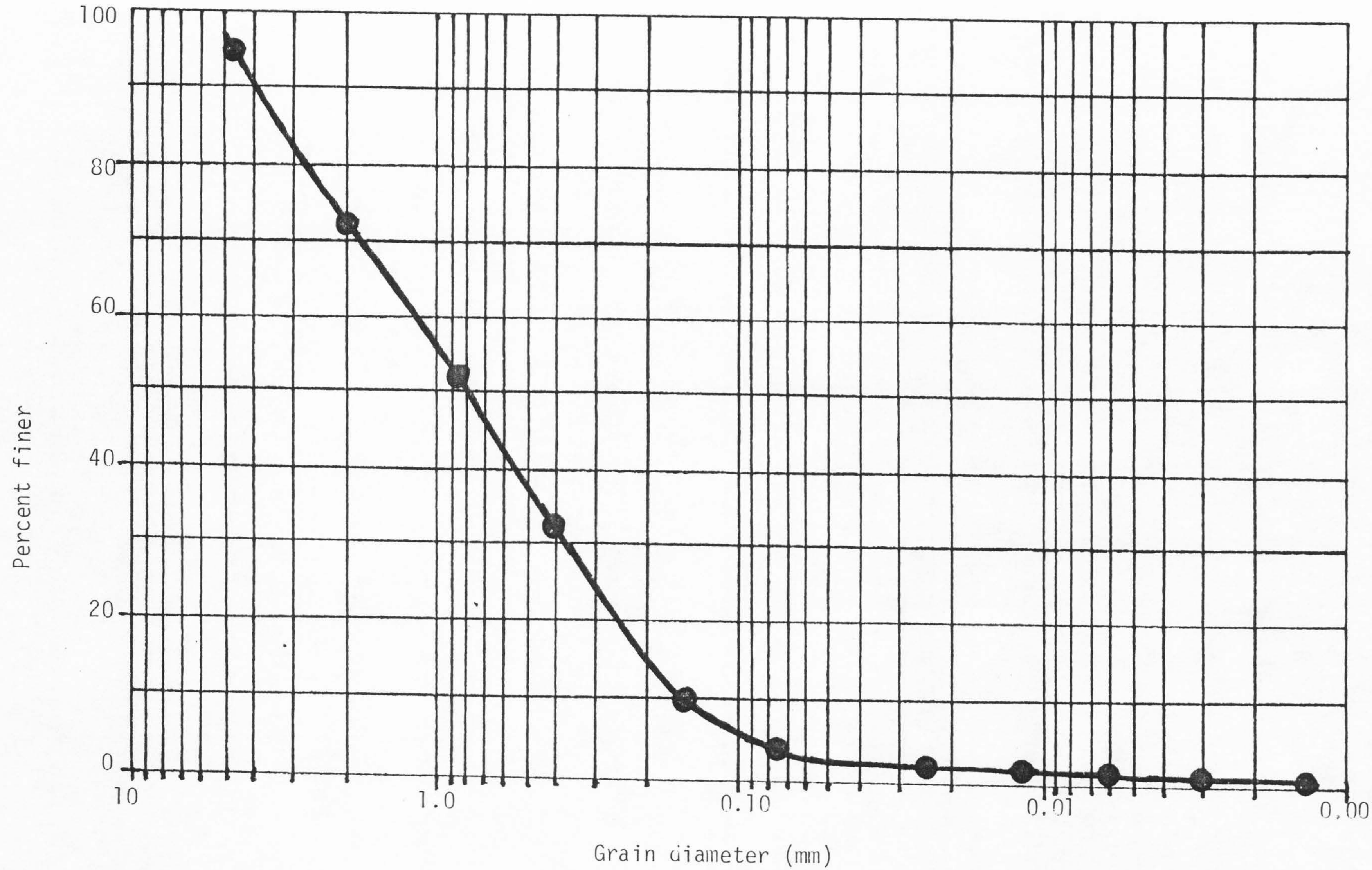

Figure 52. Grain size distribution for washed sand

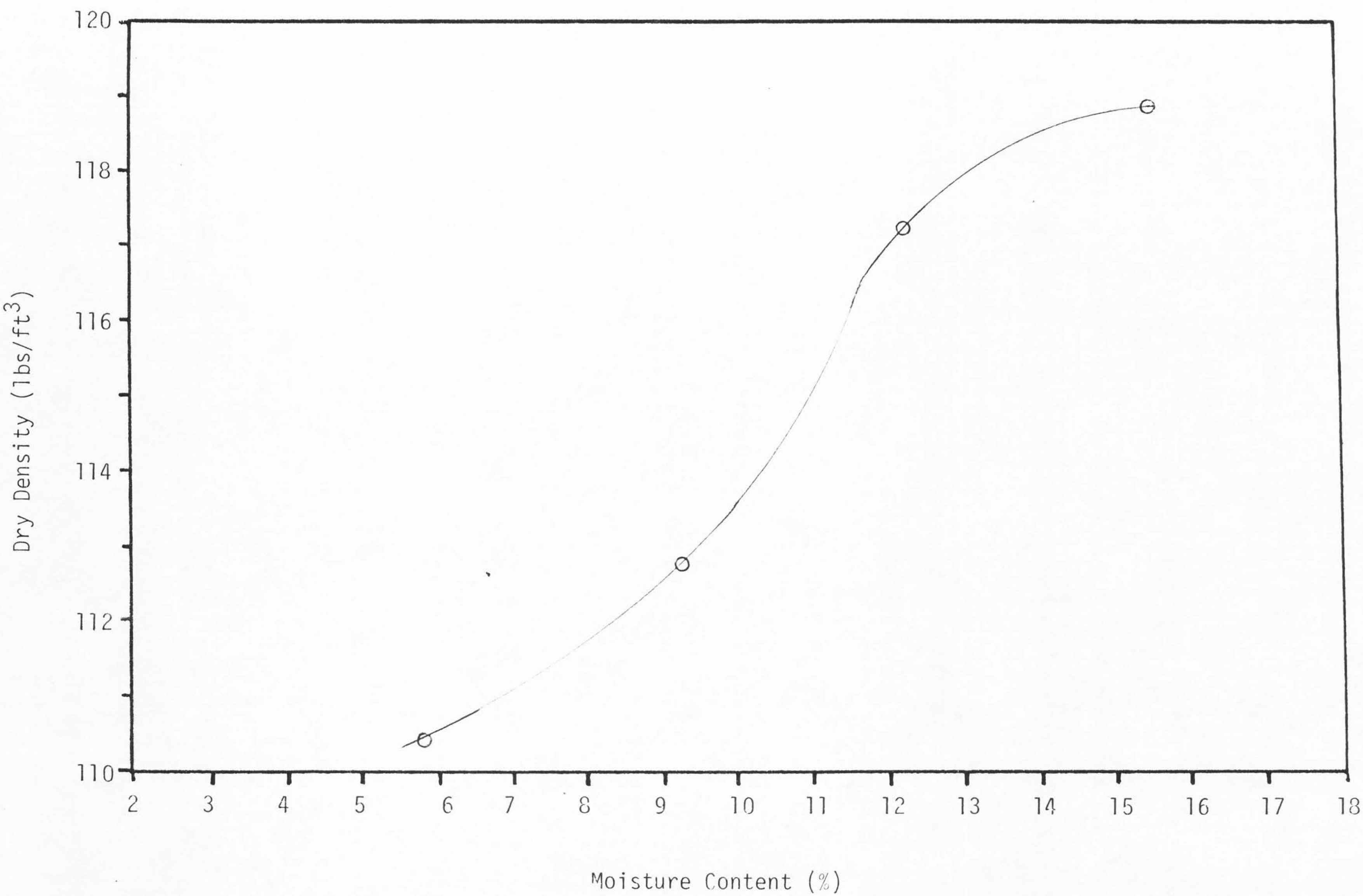

Figure 53. Moisture density relationships for washed sand

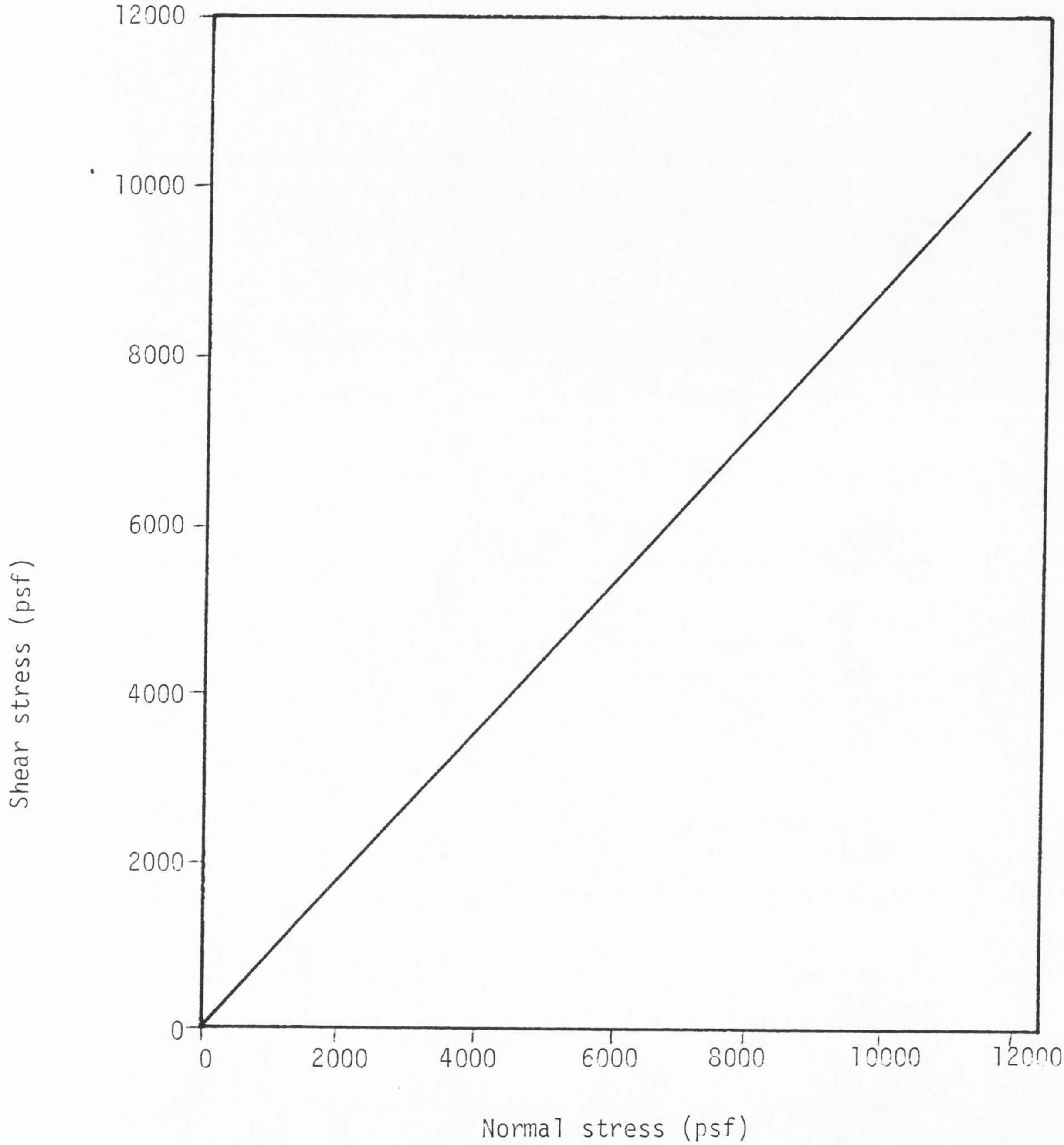

Figure 54. Direct shear test results for washed sand

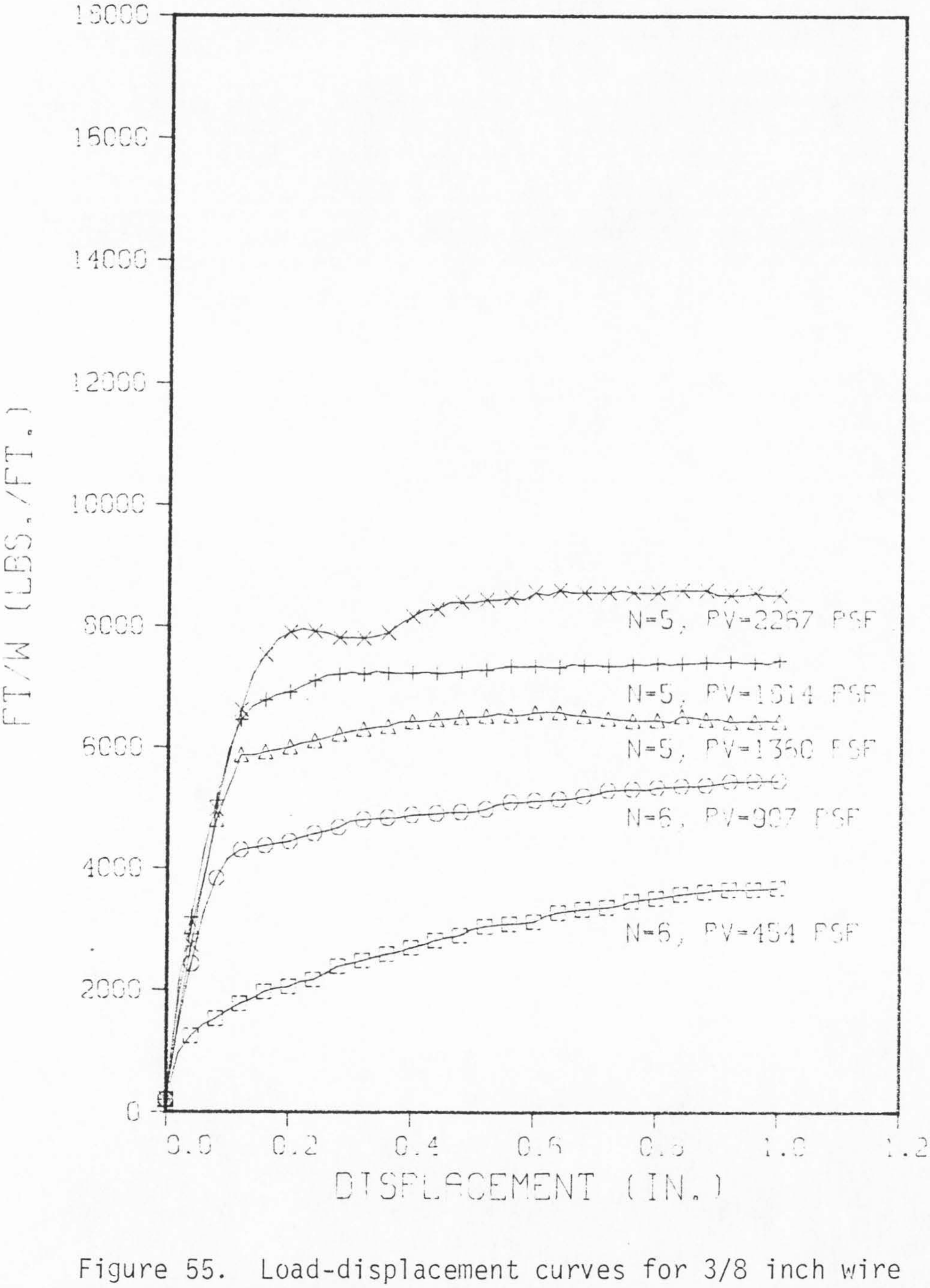

Figure 55. Load-displacement curves for 3/8 inch wire in silty sand

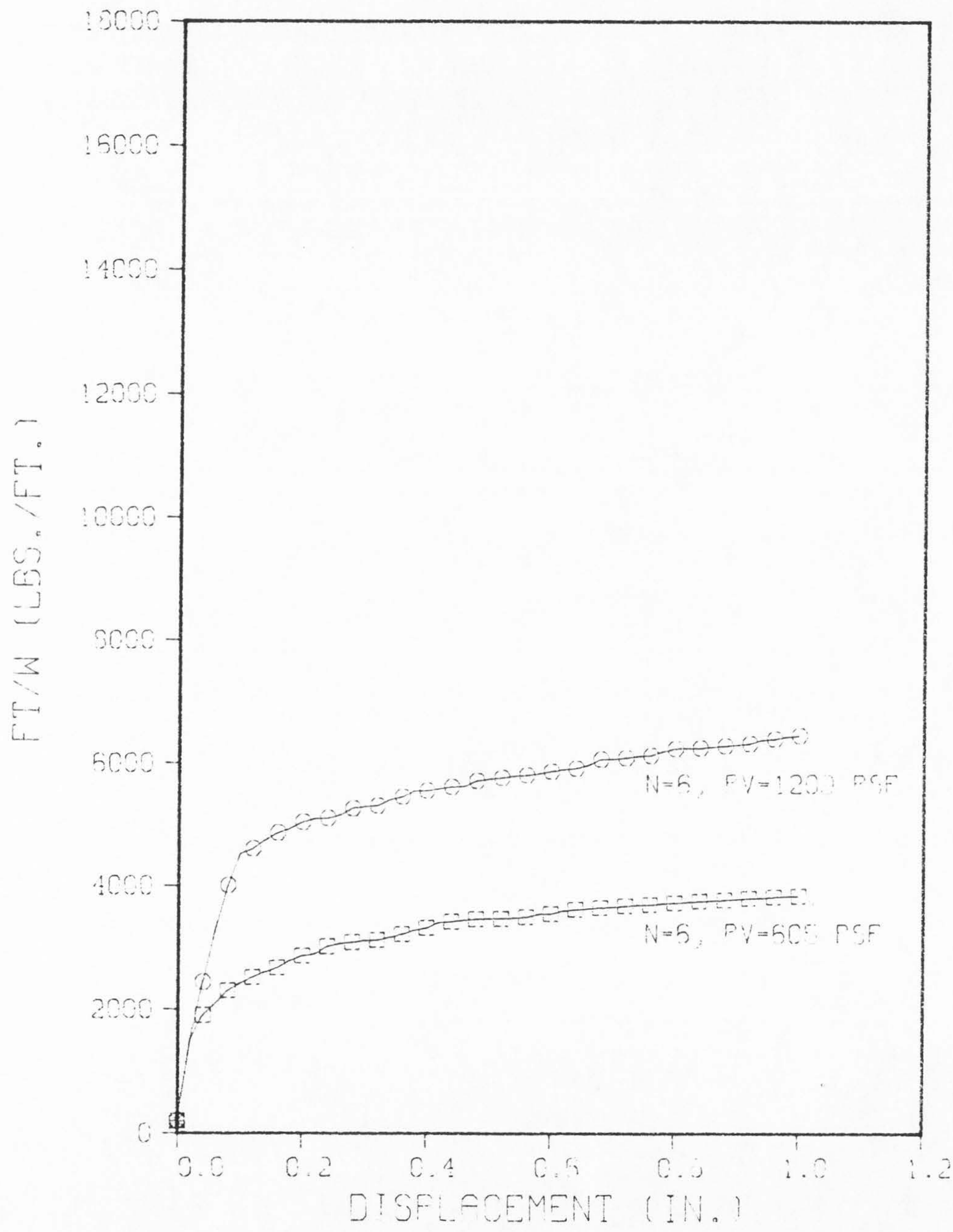

Figure 56. Load-displacement curves for 3/8 inch wire in silty sand

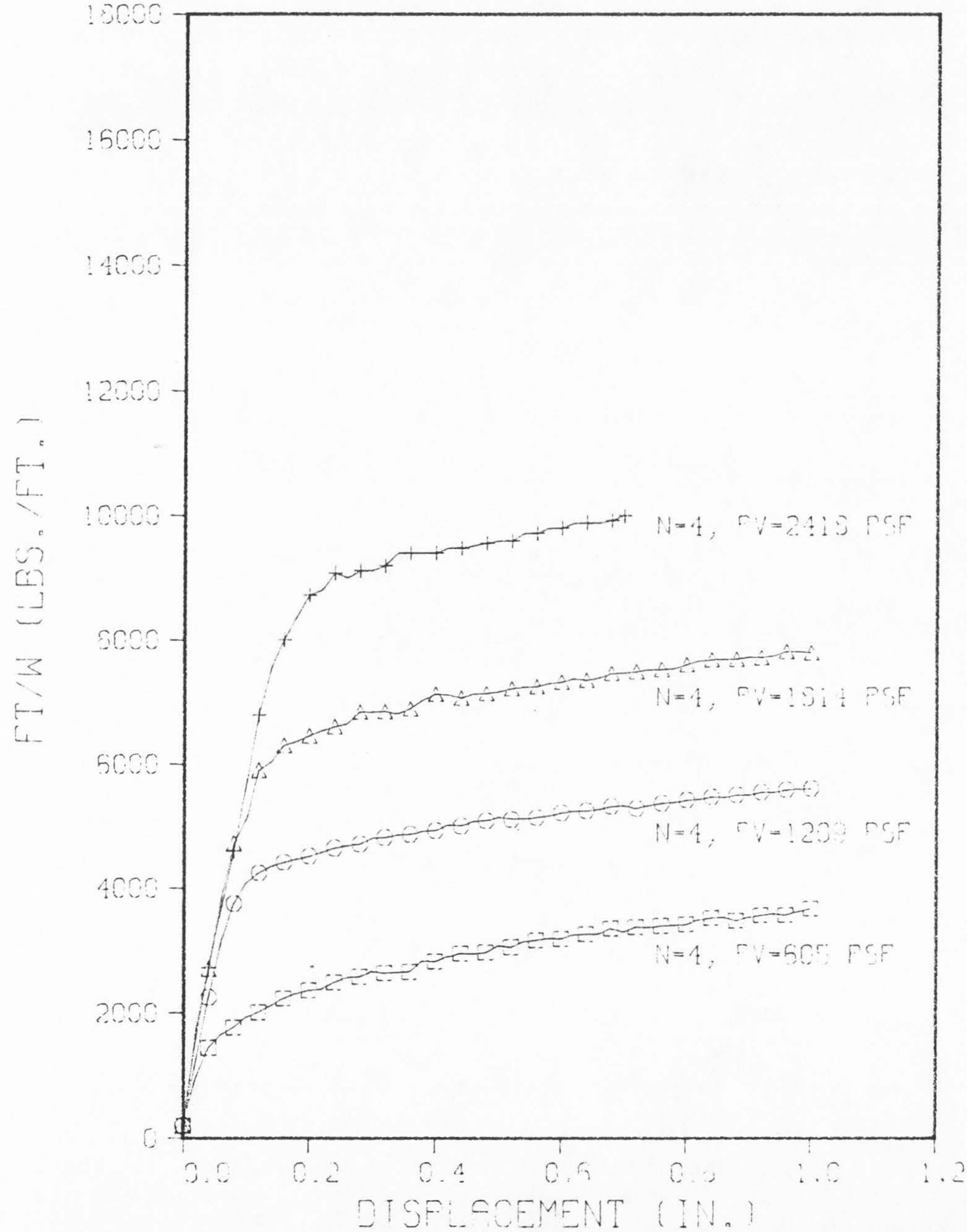

Figure 57. Load-displacement curves for 3/8 inch wire in silty sand

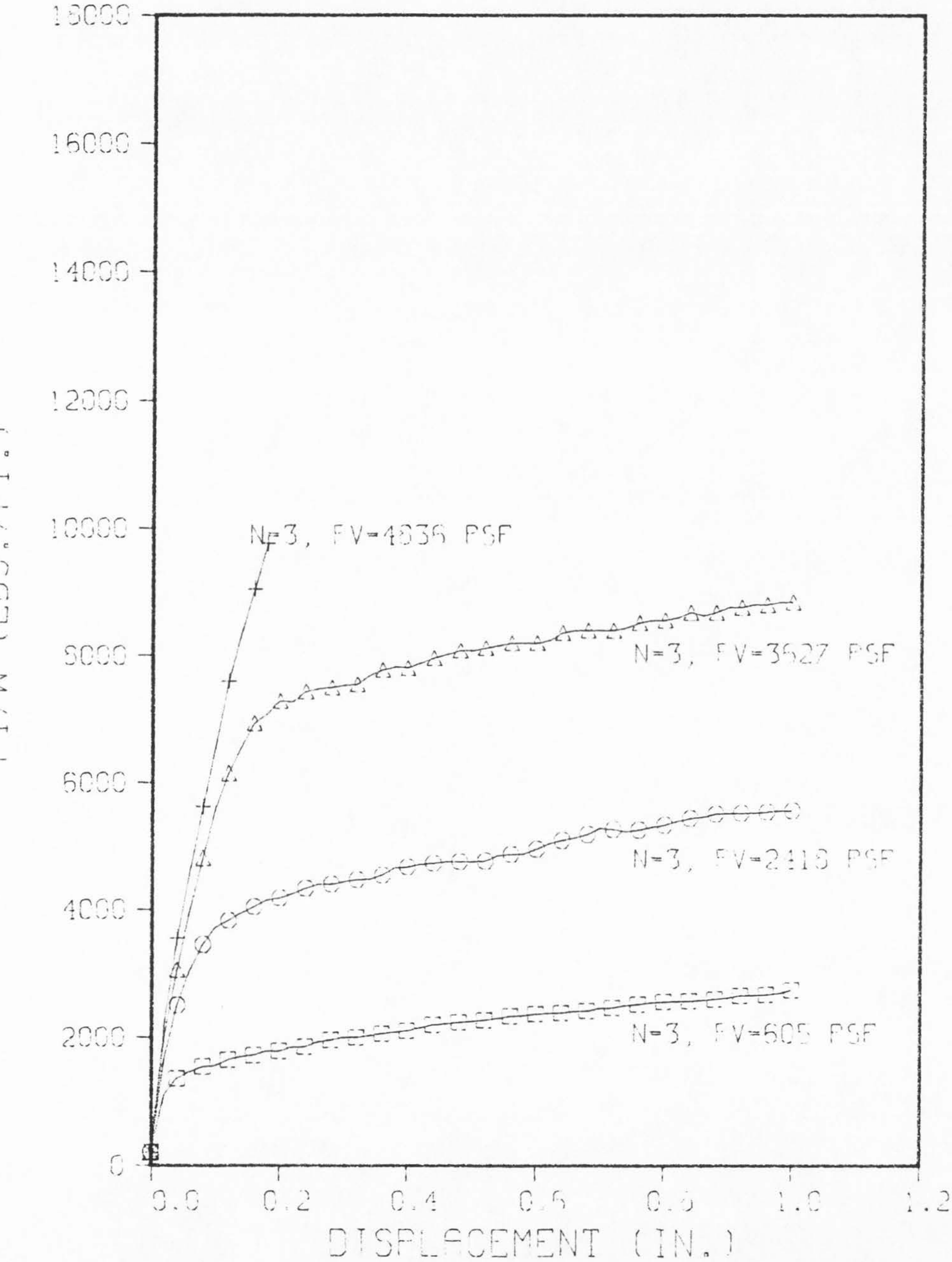

Figure 58. Load-displacement curves for 3/8 inch wire in silty sand

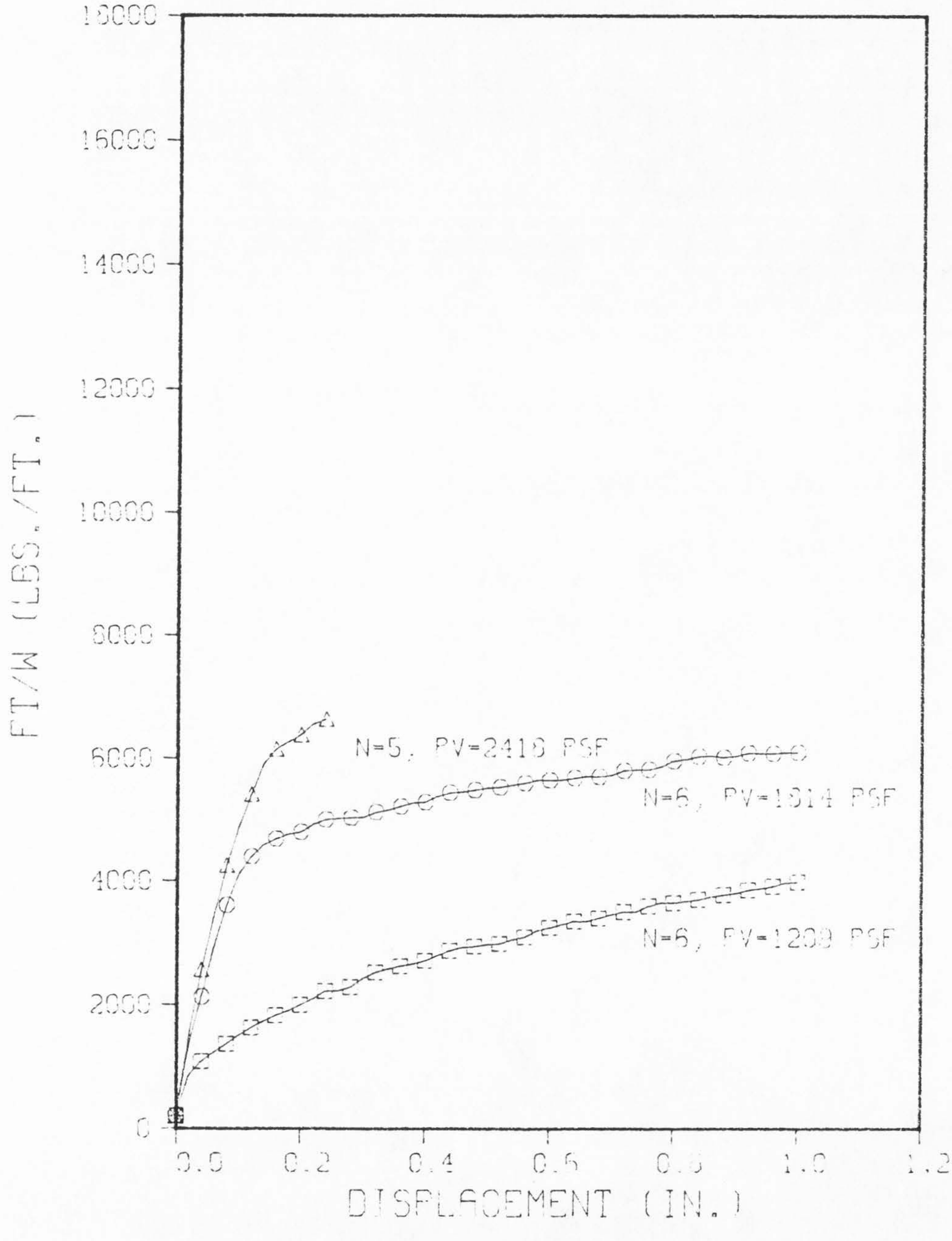

Figure 59. Load-displacement curves for 1/4 inch wire in silty sand

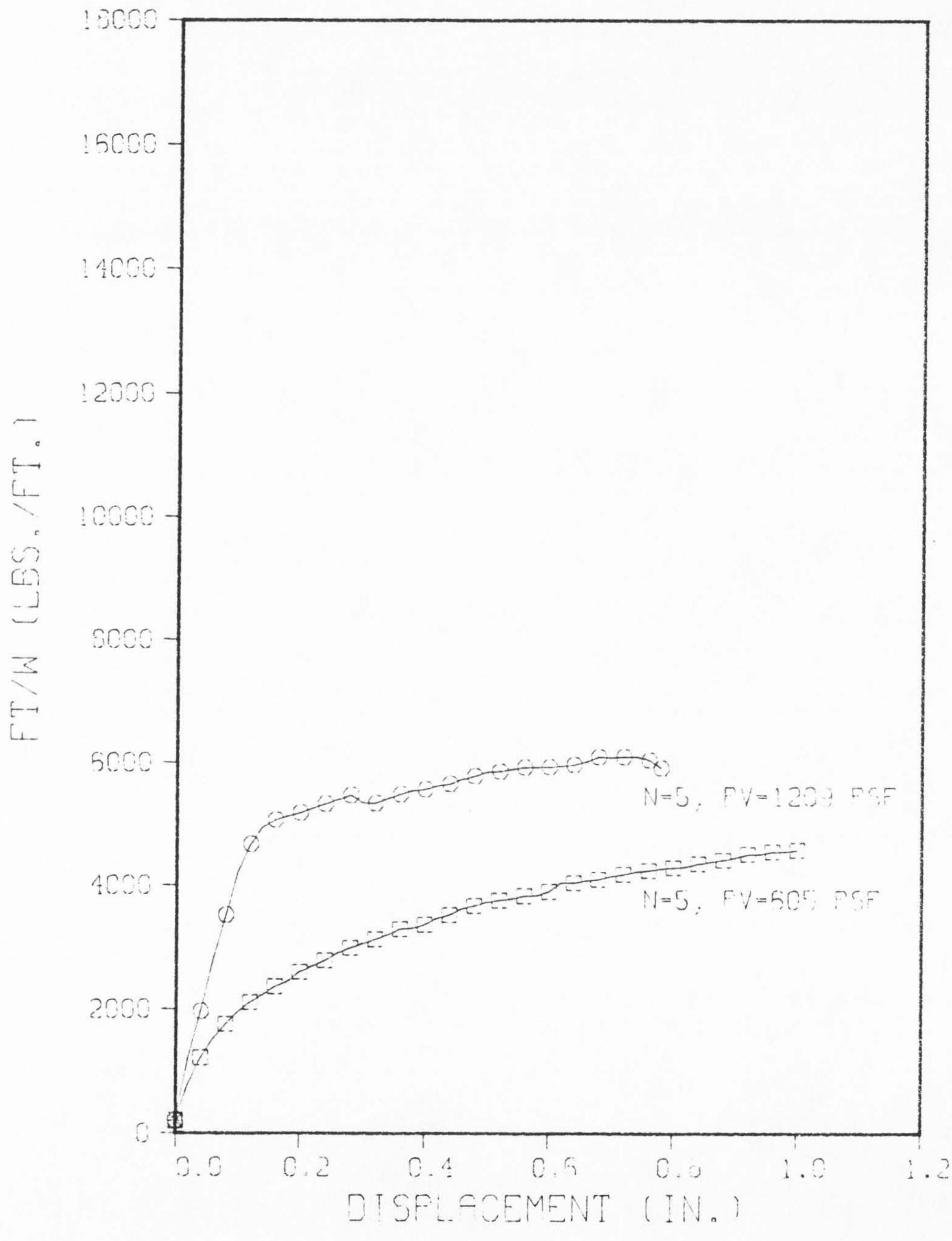

Figure 60. Load-displacement curves for 1/4 inch wire in silty sand

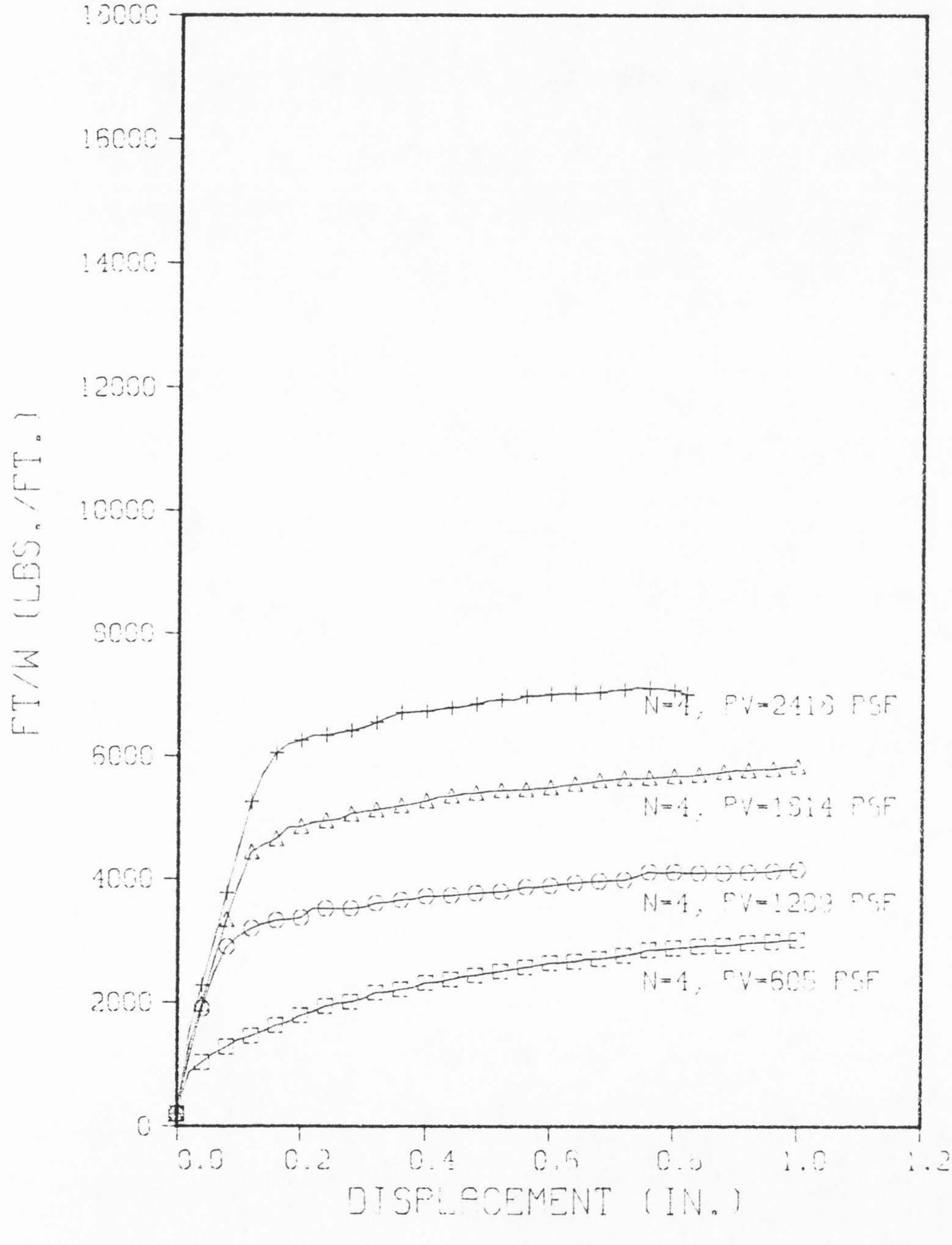

Figure 61. Load-displacement curves for 1/4 inch wire in silty sand

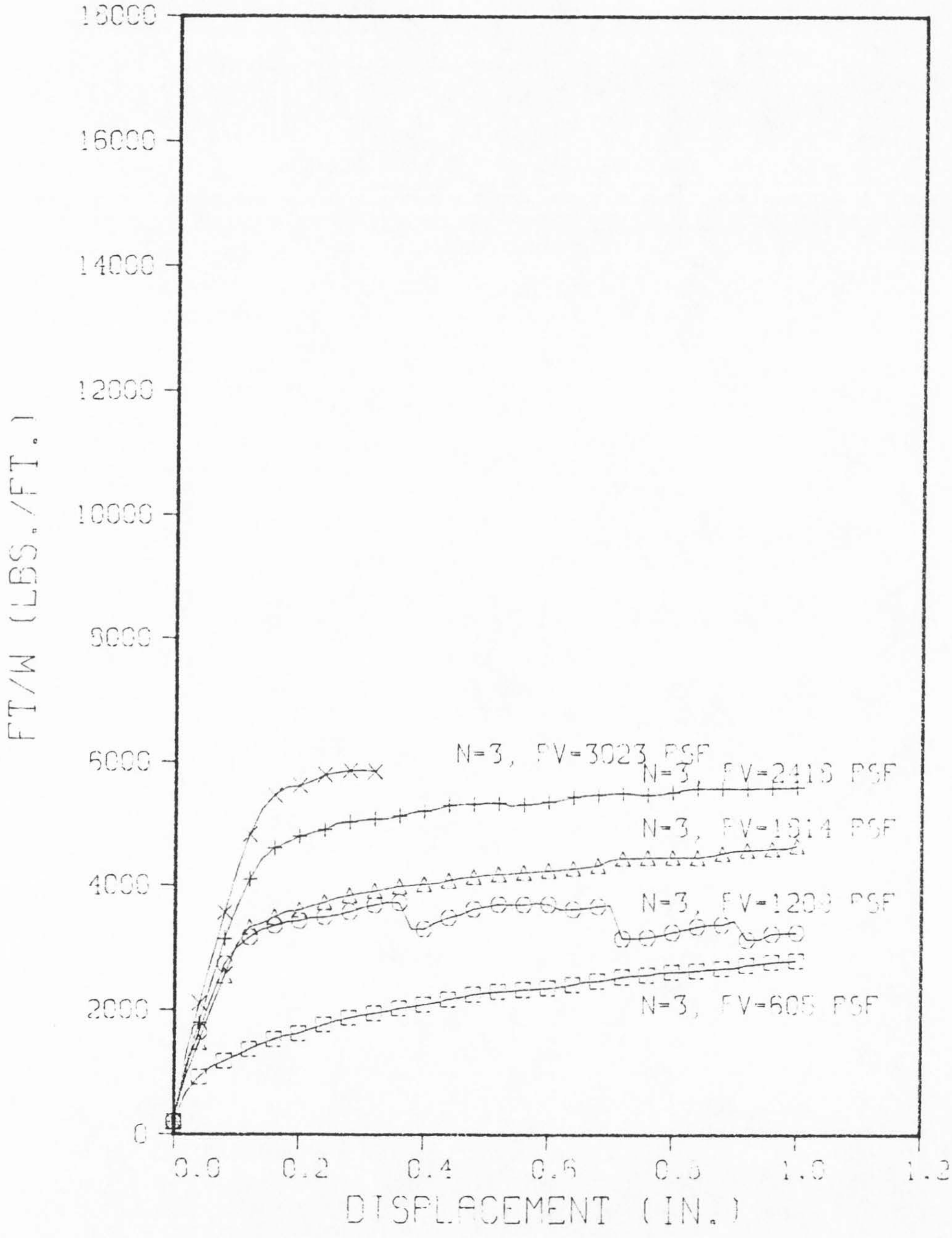

Figure 62. Load-displacement curves for 1/4 inch wire in silty sand

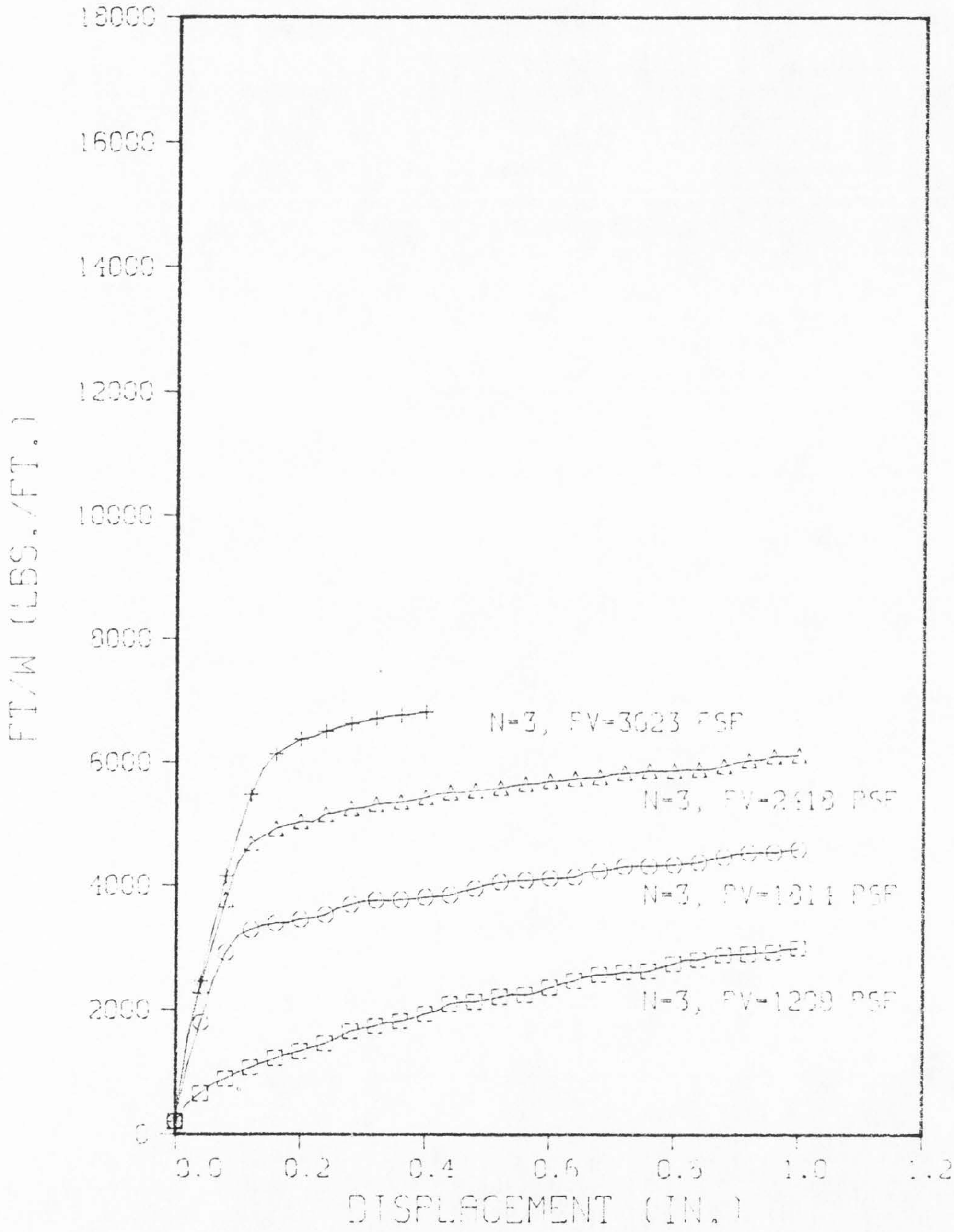

Figure 63. Load-displacement curves for 1/4 inch wire in silty sand

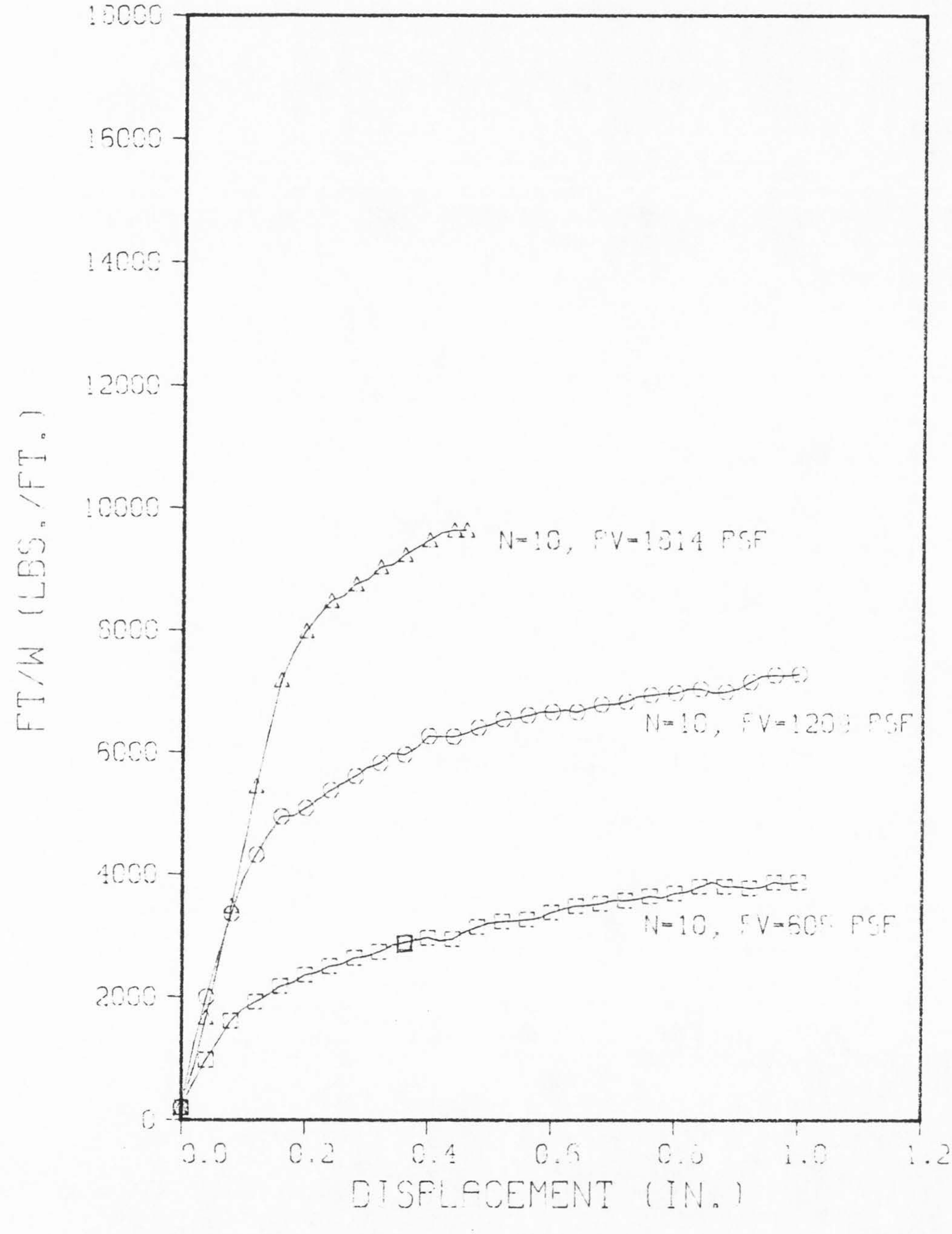

Figure 64. Load-displacement curves for 7 gage wire in silty sand

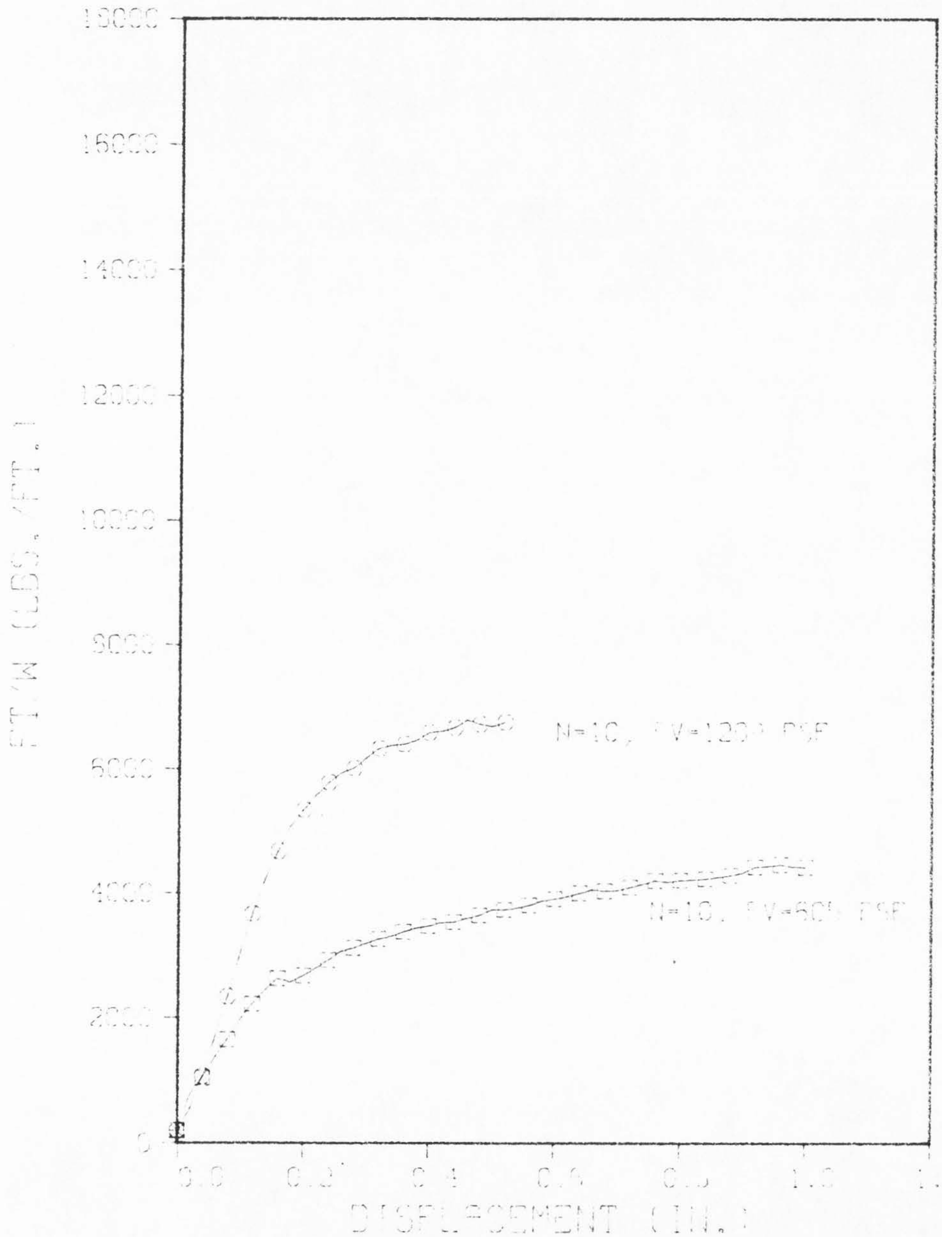

Figure 65. Load-displacement curves for 9 gage wire in silty sand

Appendix B

Load-displacement Curves

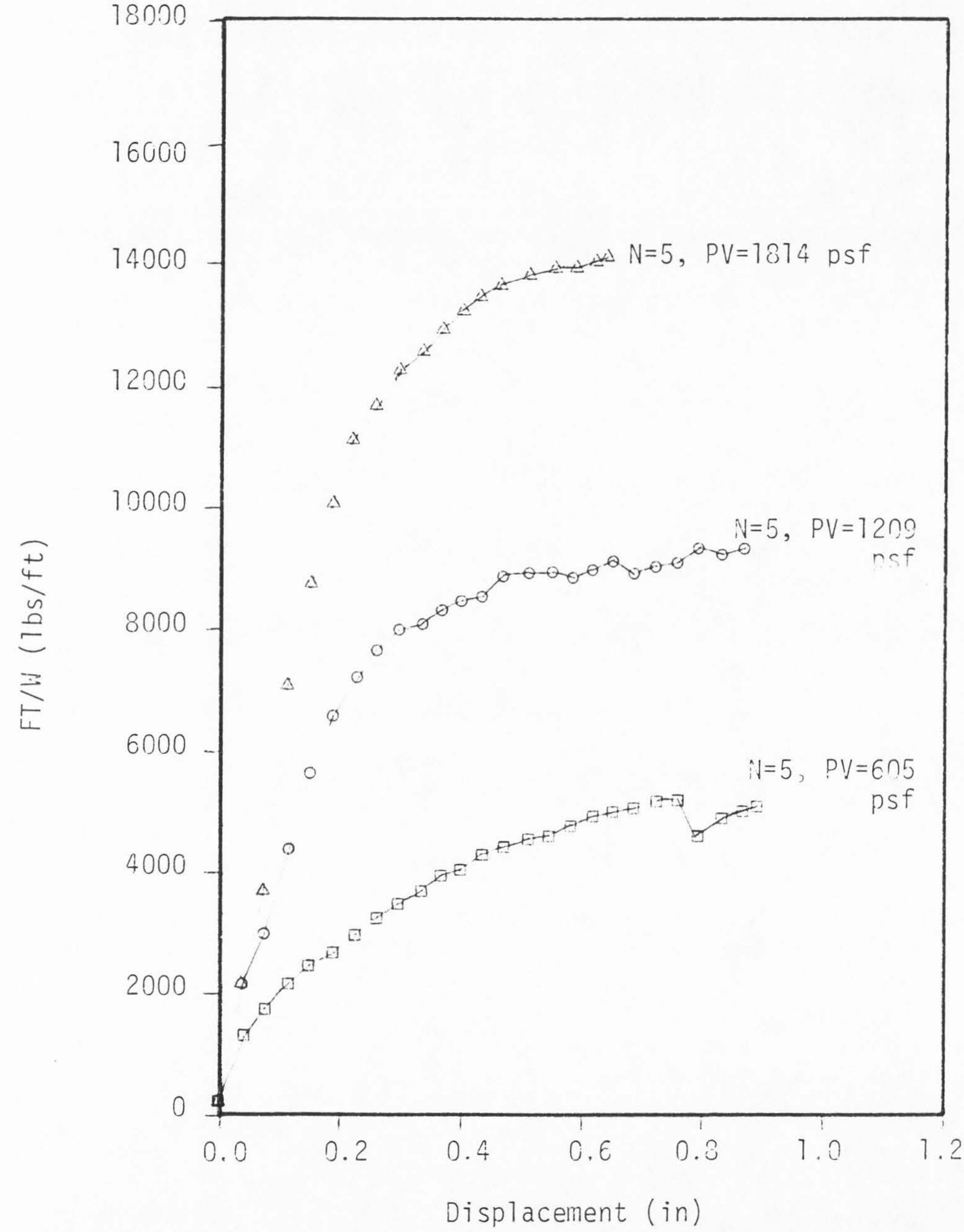

Figure 66. Load-displacement curve for pea gravel 3/8 inch wire

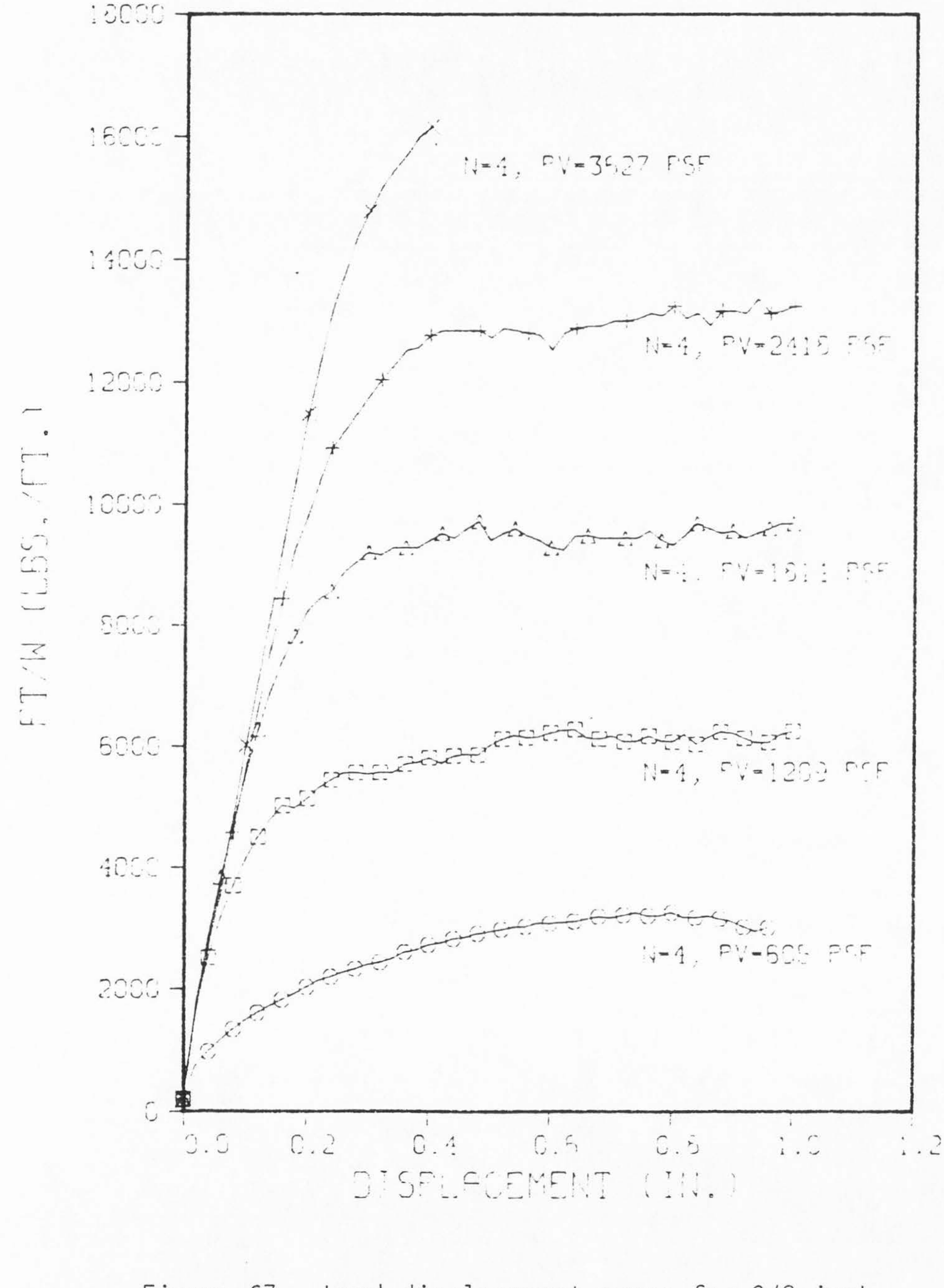

Figure 67. Load-displacement curve for 3/8 inch wire in pea gravel

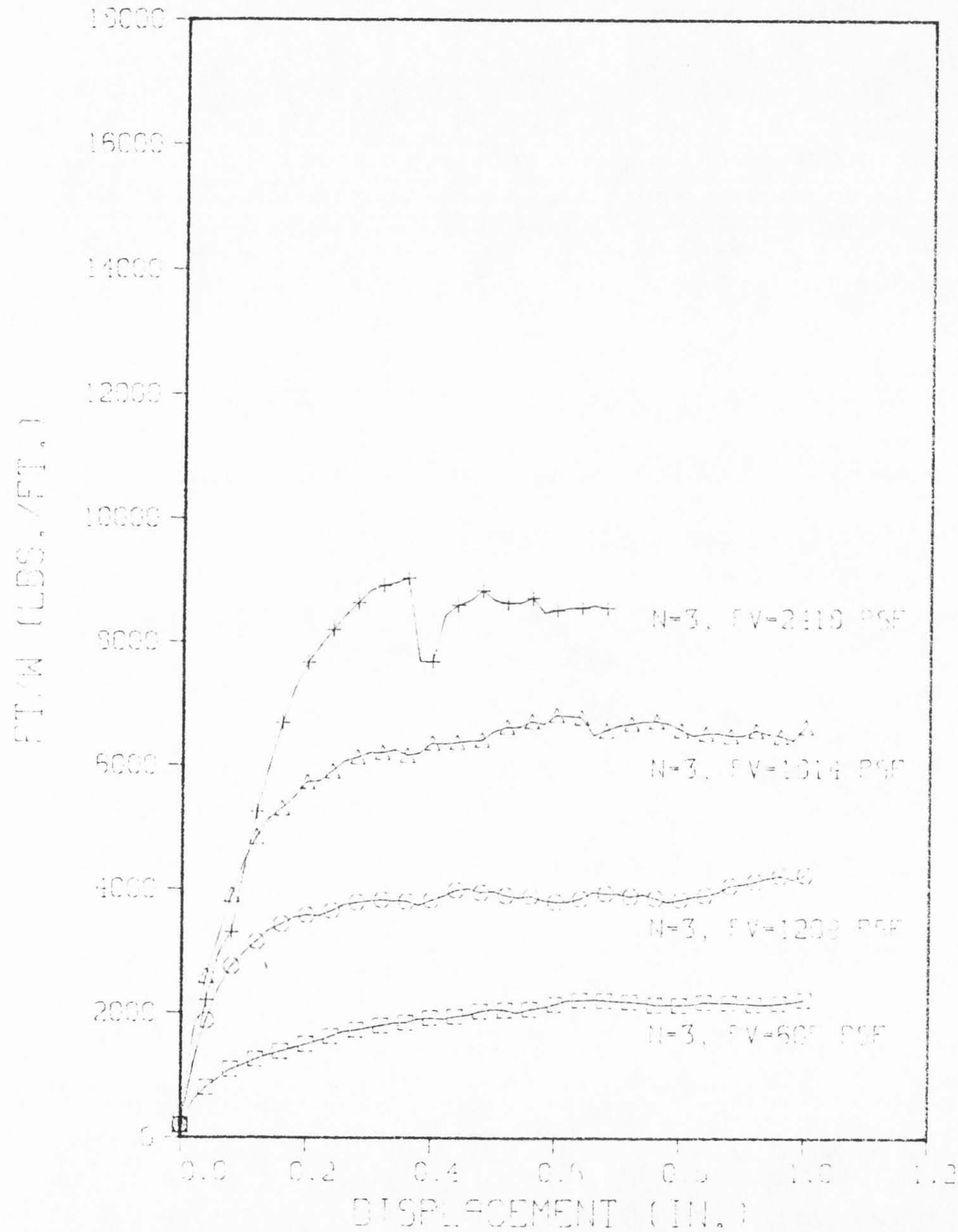

Figure 68. Load-displacement curves for 3/8 inch
wire in pea gravel

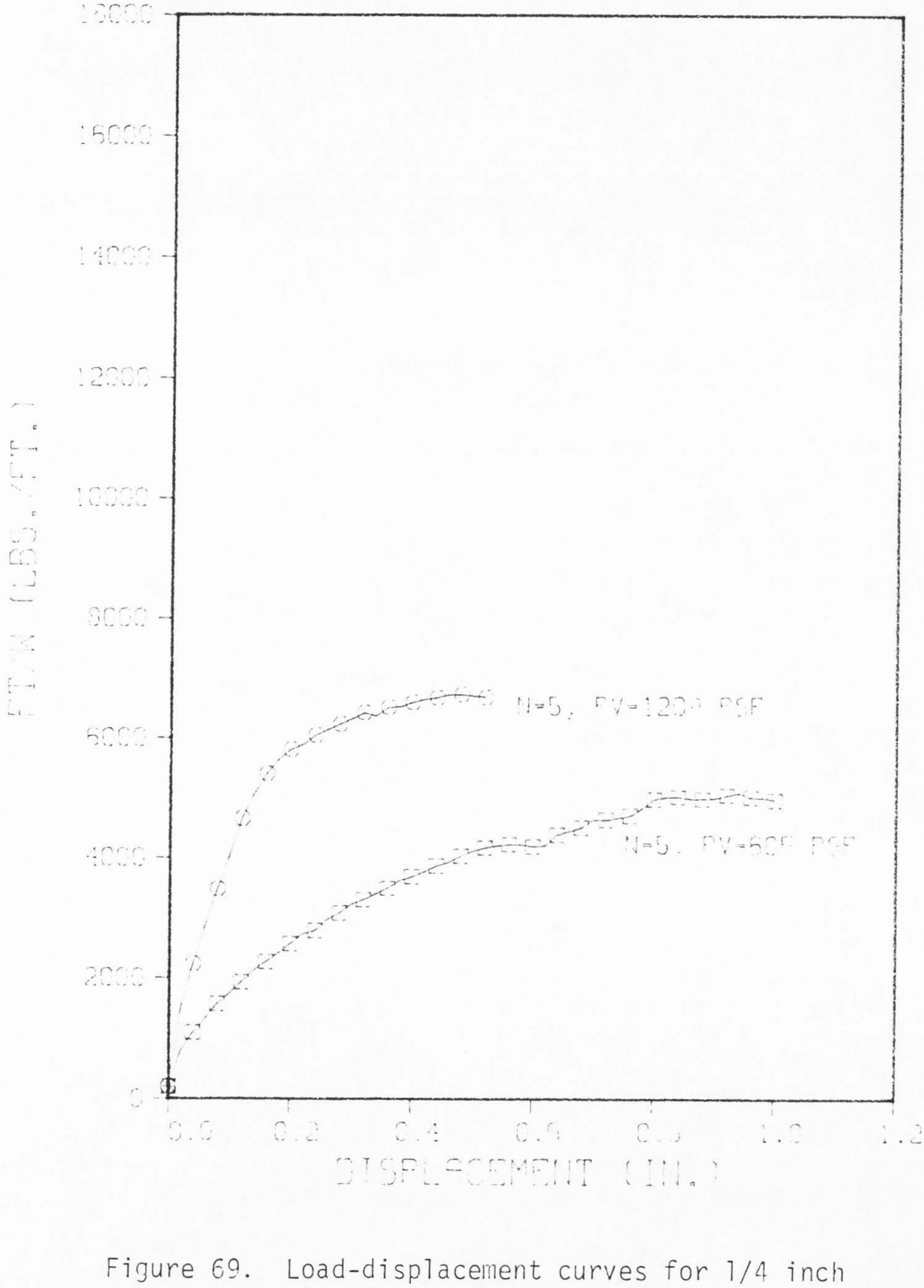

Figure 69. Load-displacement curves for 1/4 inch wire in pea gravel

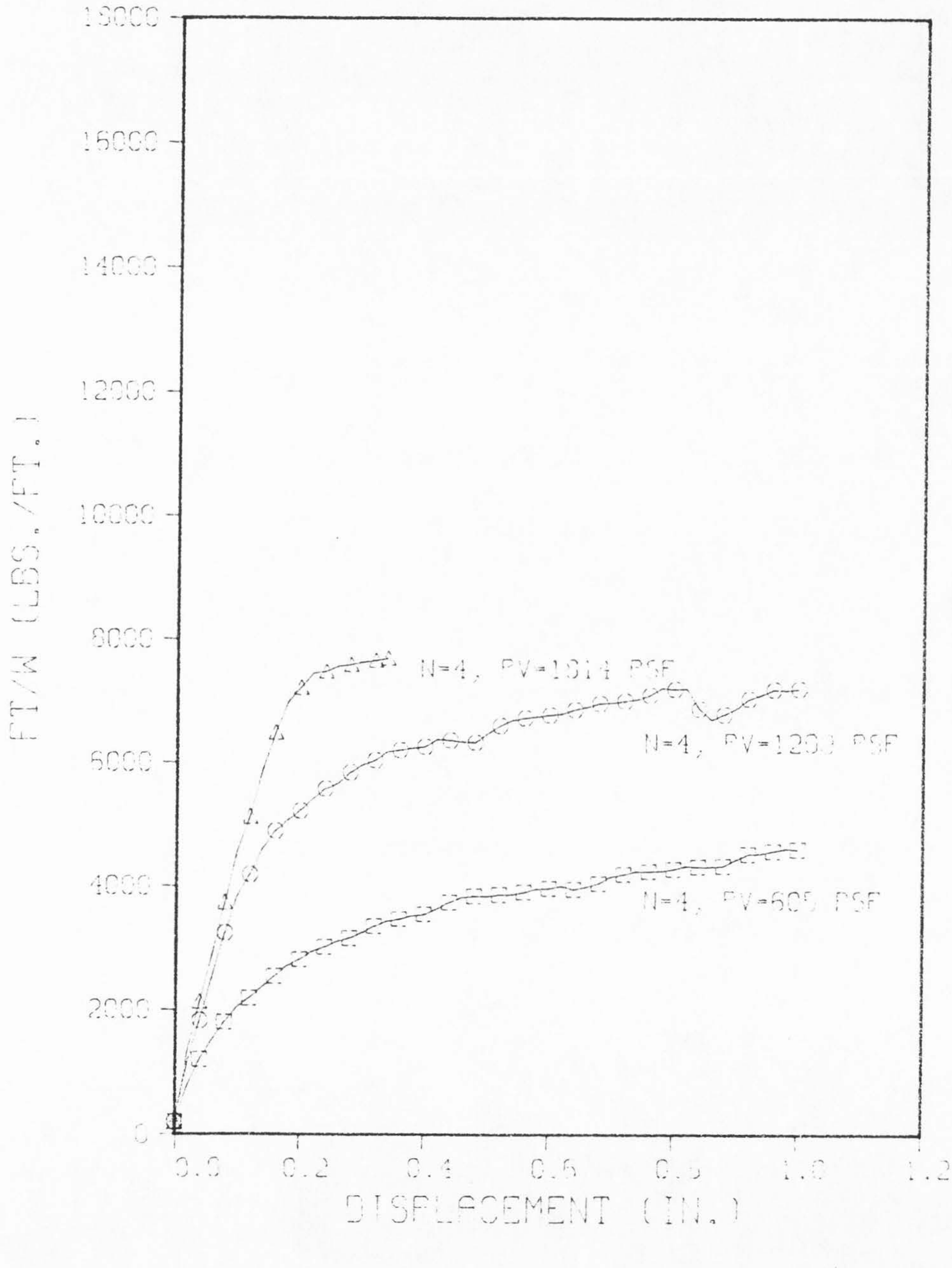

Figure 70. Load-displacement curves for 1/4 inch wire in pea gravel

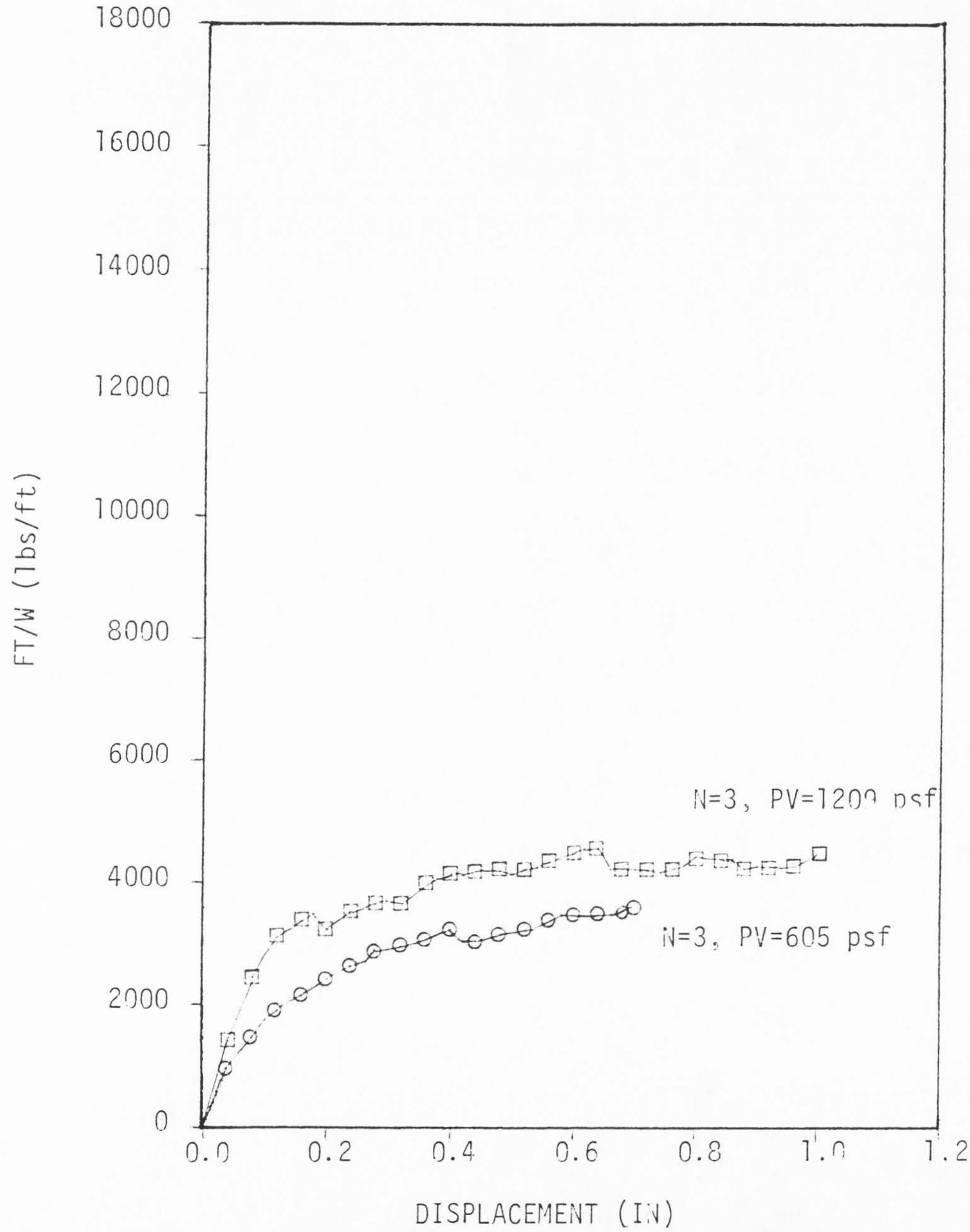

Figure 71. Load-displacement for 1/4 inch wire in pea gravel

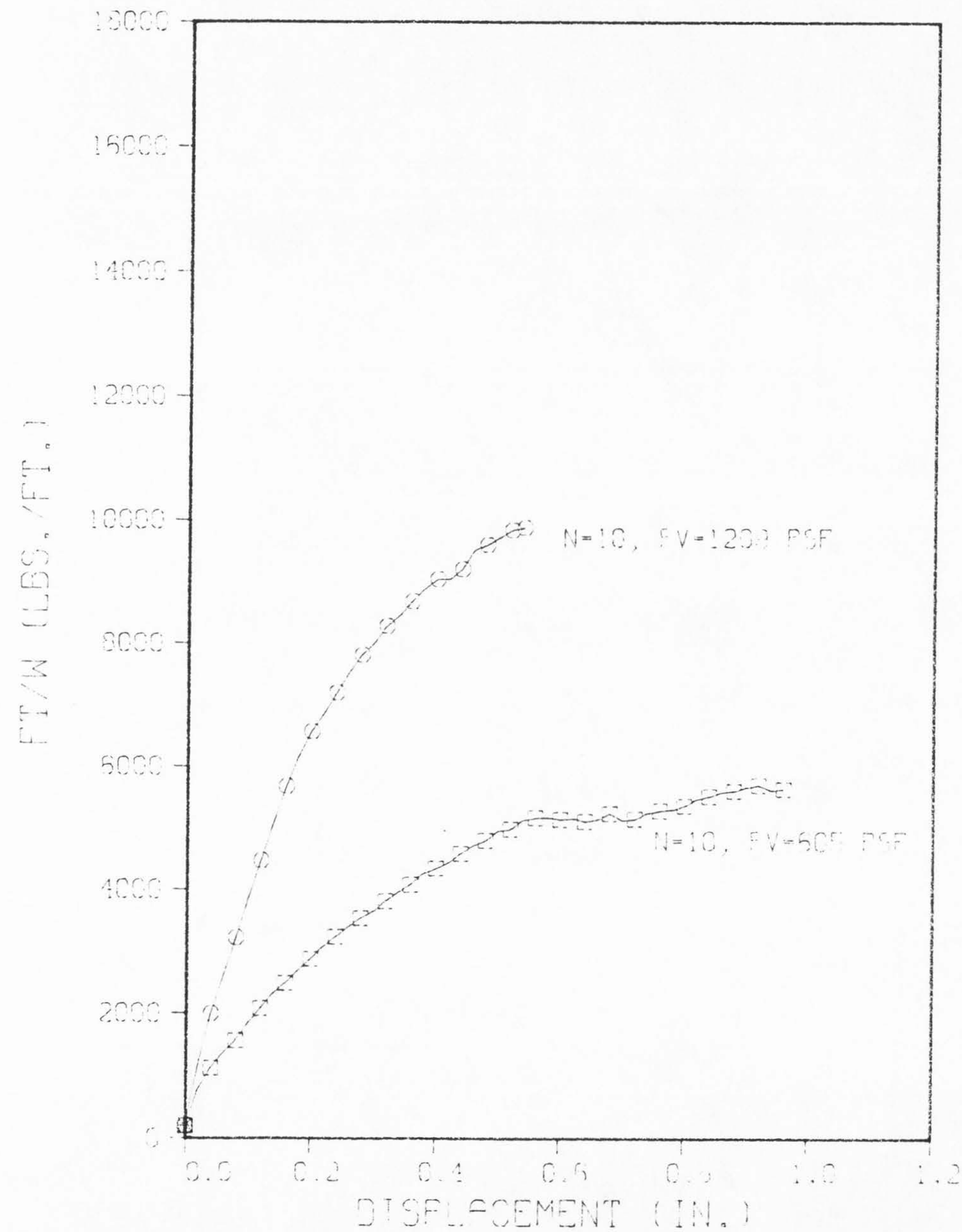

Figure 72. Load-displacement for 7 gage wire in pea gravel

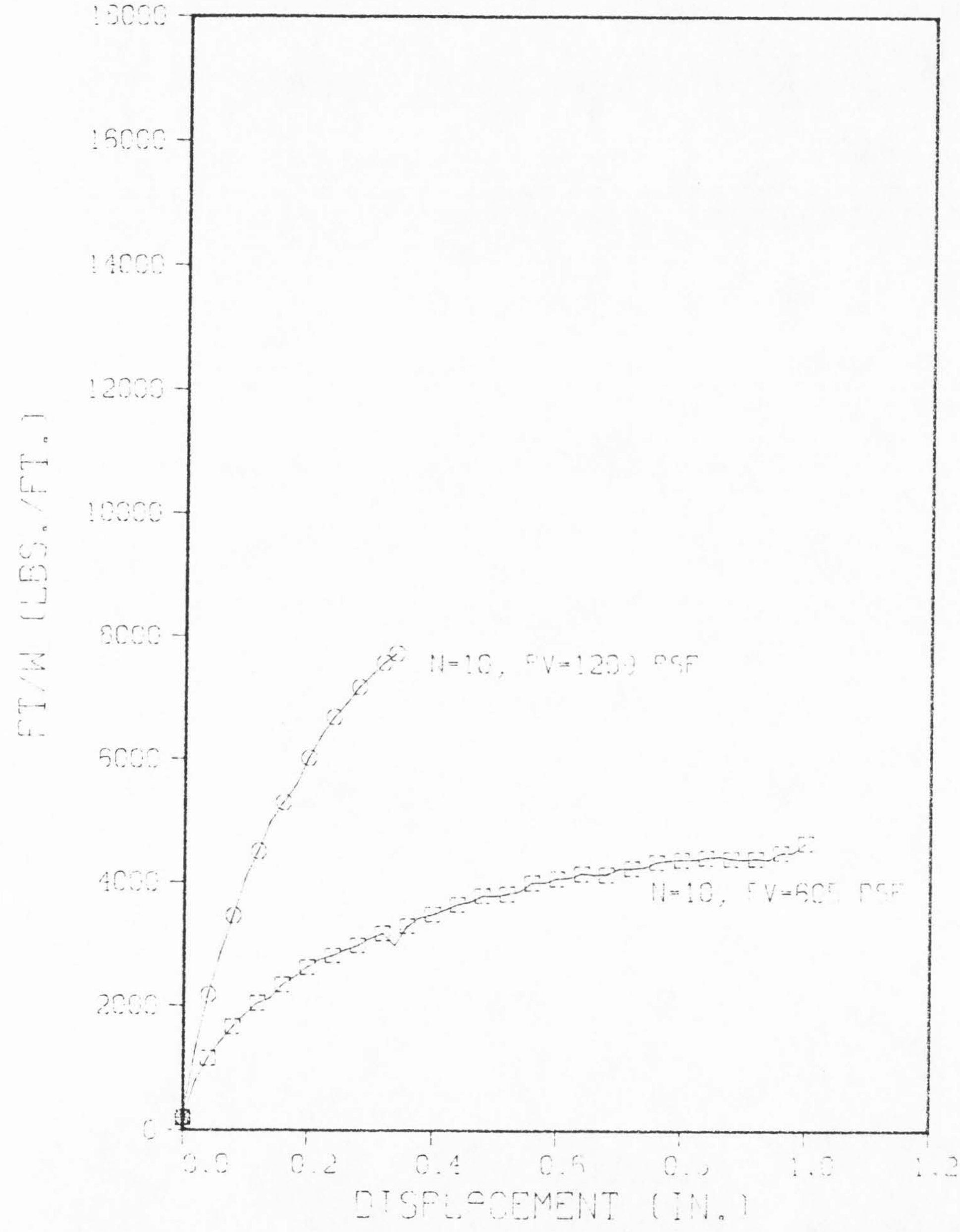

Figure 73. Load-displacement curves for 9 gage wire in pea gravel

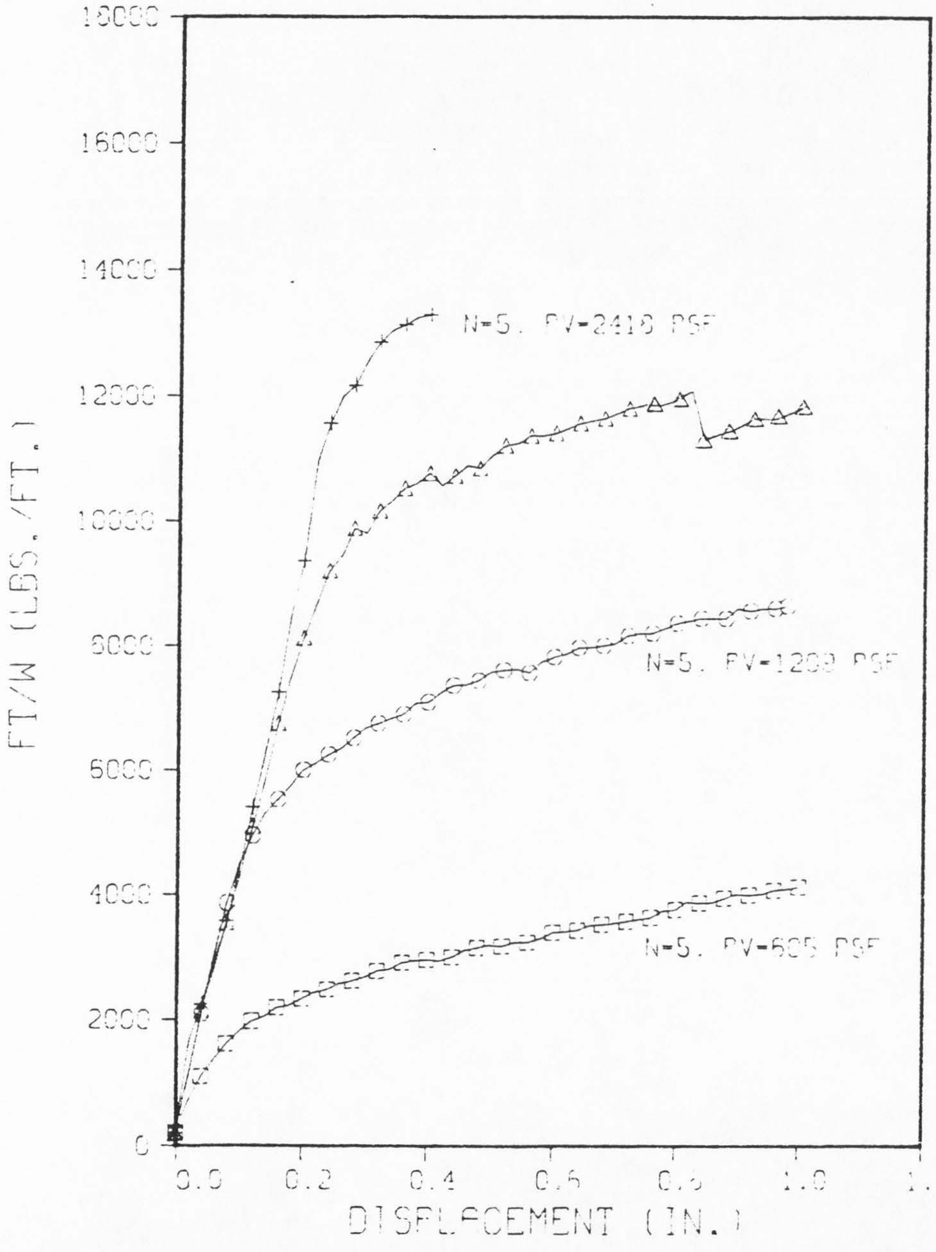

Figure 74. Load-displacement curves for 3/8 inch wire in washed sand

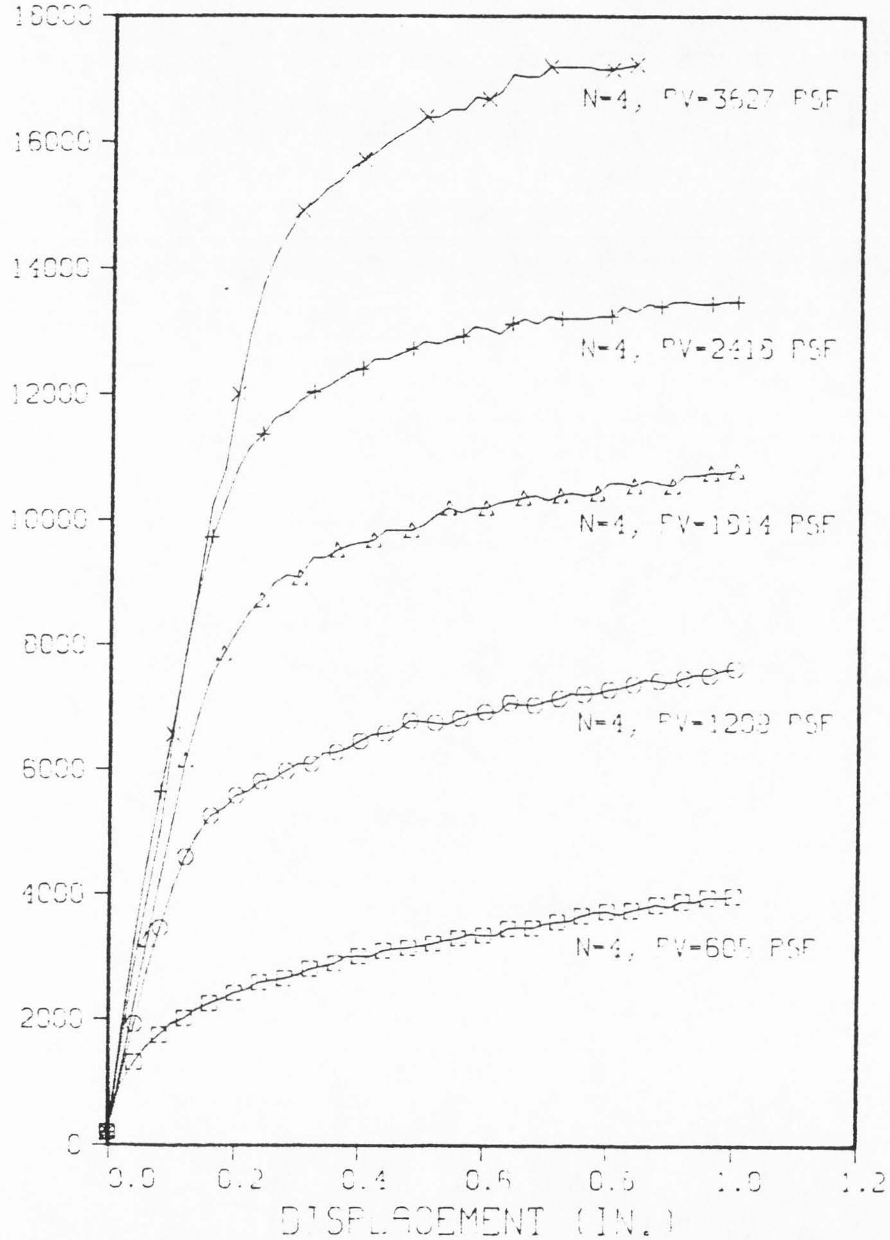

Figure 75. Load-displacement curves for 3/8 inch wire in washed sand

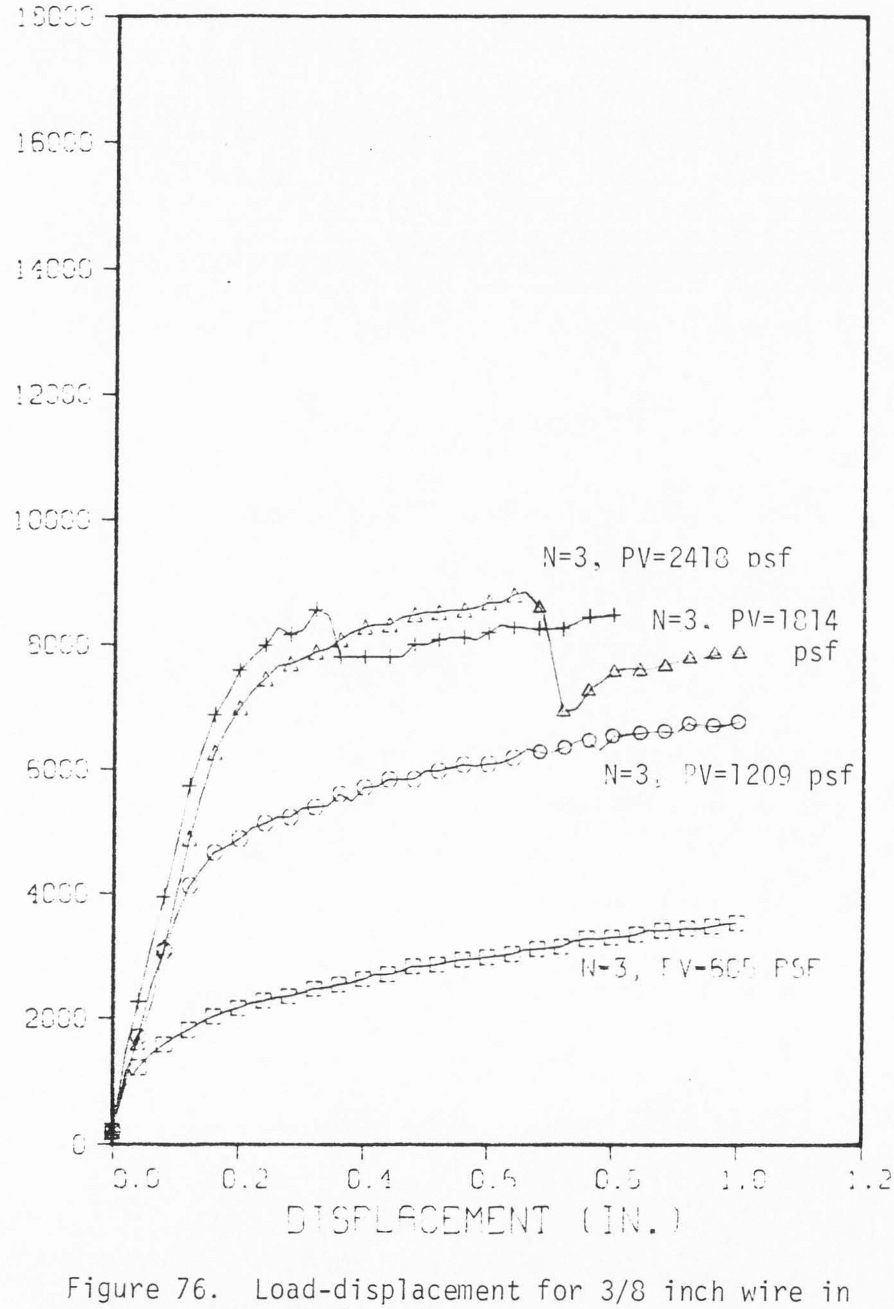

Figure 76. Load-displacement for 3/8 inch wire in washed sand

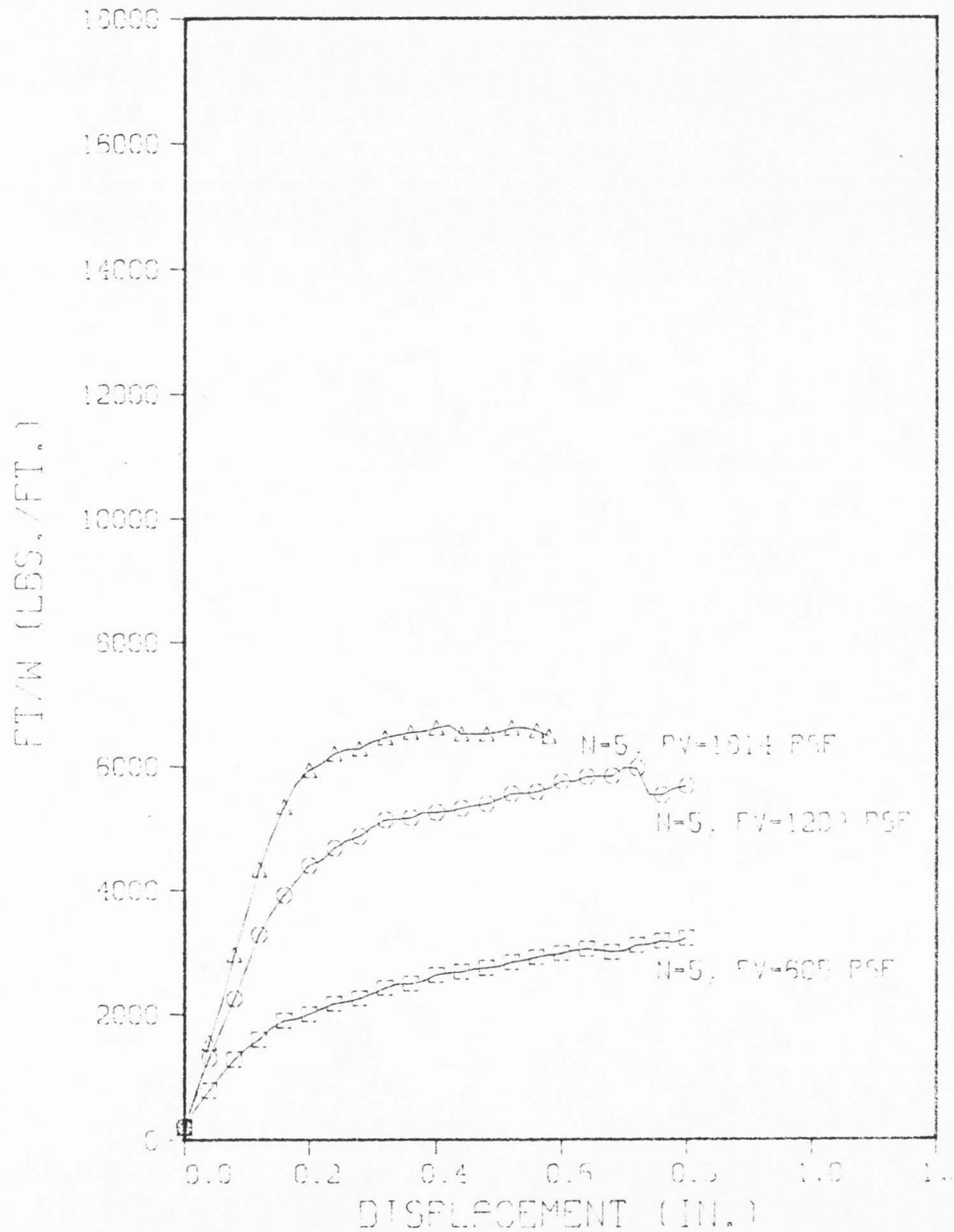

Figure 77. Load-displacement curves for 1/4 inch
wire in washed sand

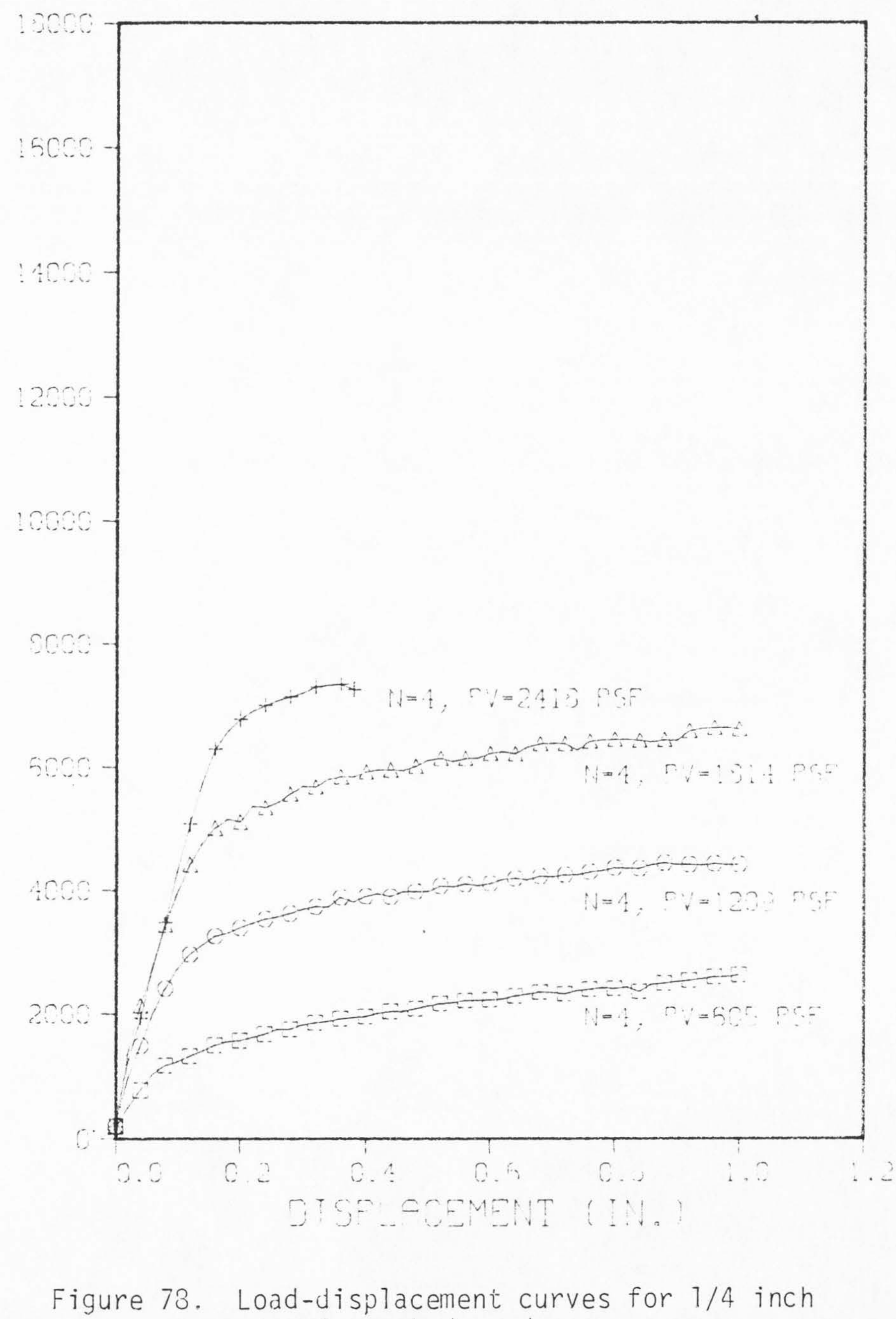

Figure 78. Load-displacement curves for 1/4 inch wire in washed sand

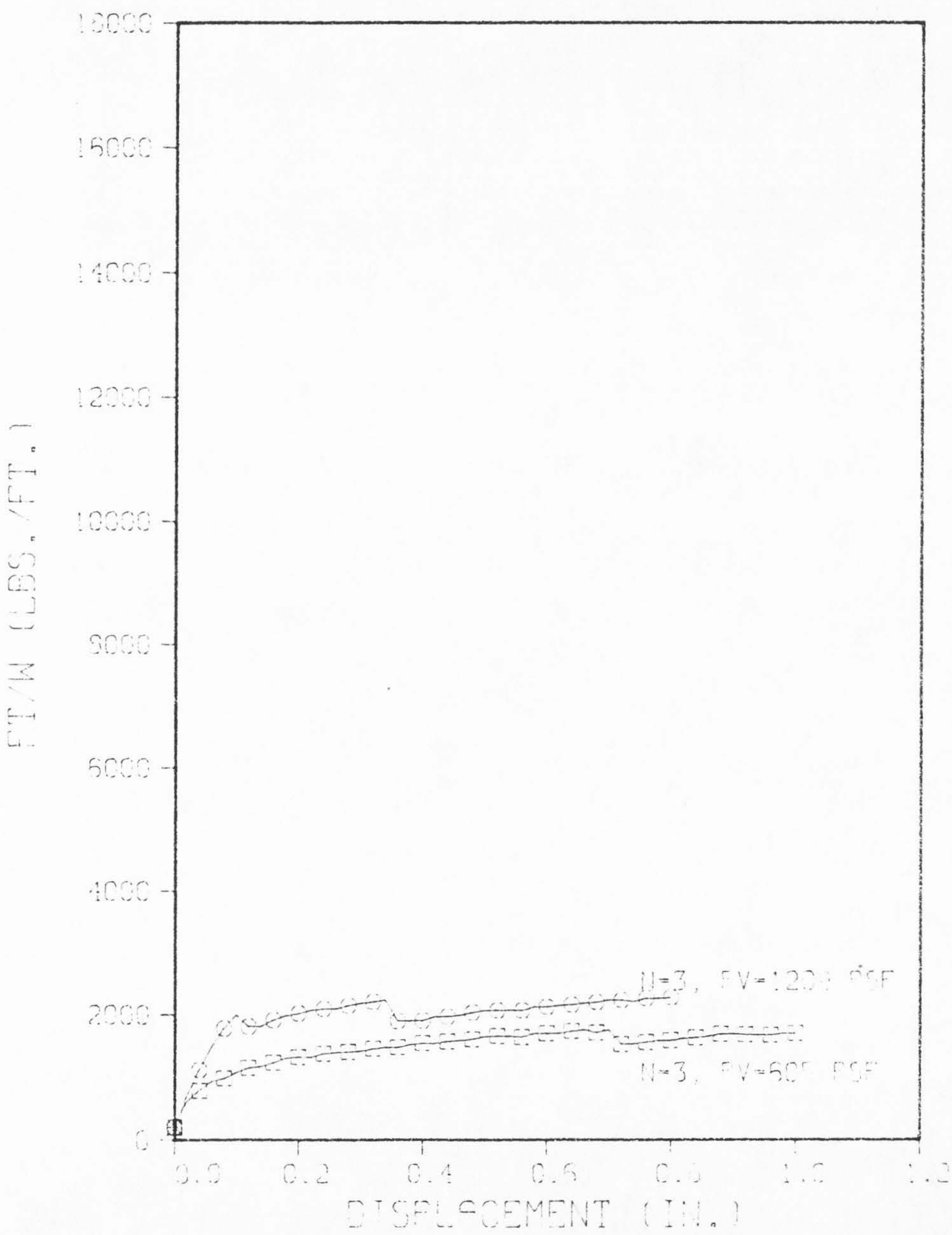

Figure 79. Load-displacement curves for 1/4 inch wire in washed sand

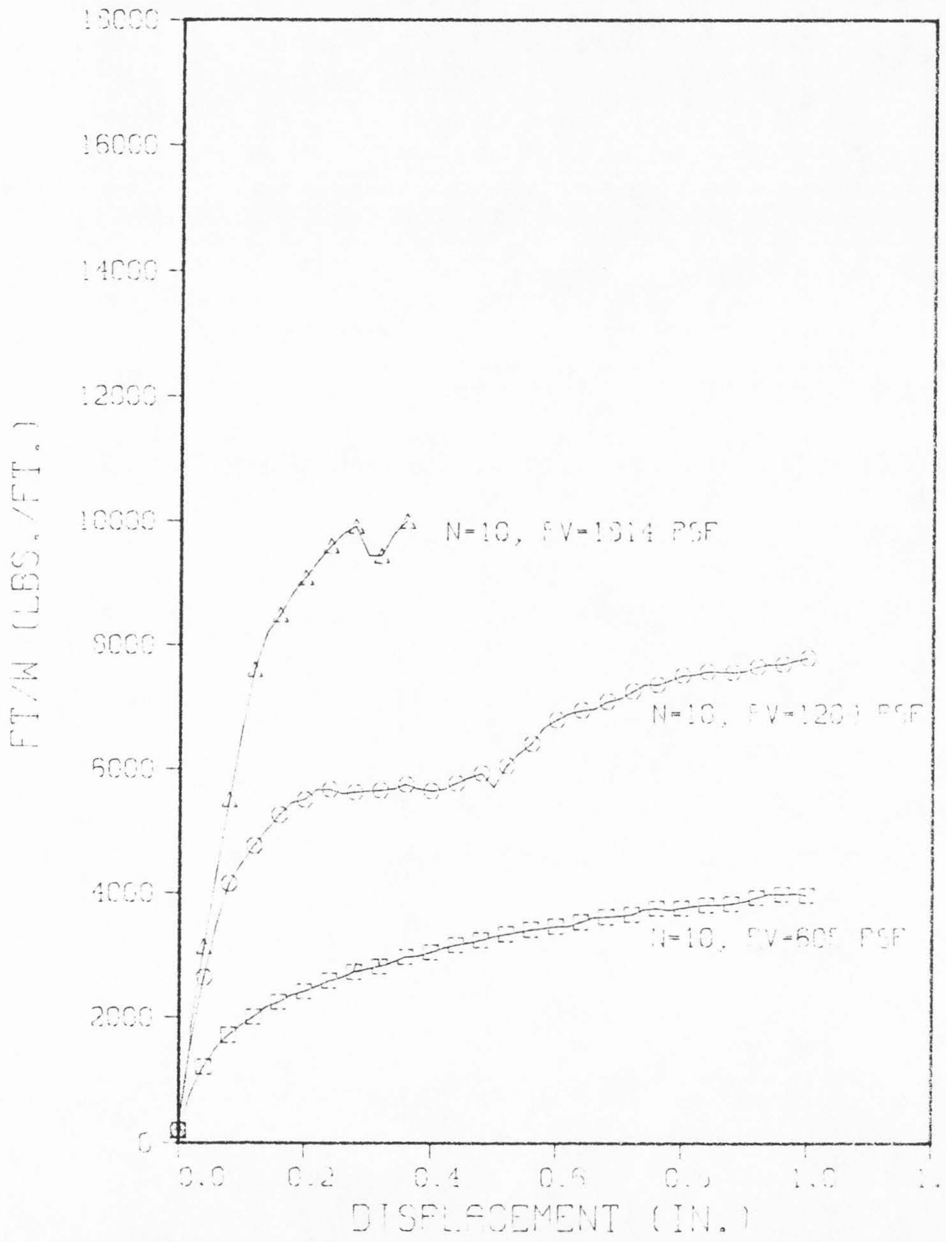

Figure 80. Load-displacement curves for 7 gage wire in washed sand

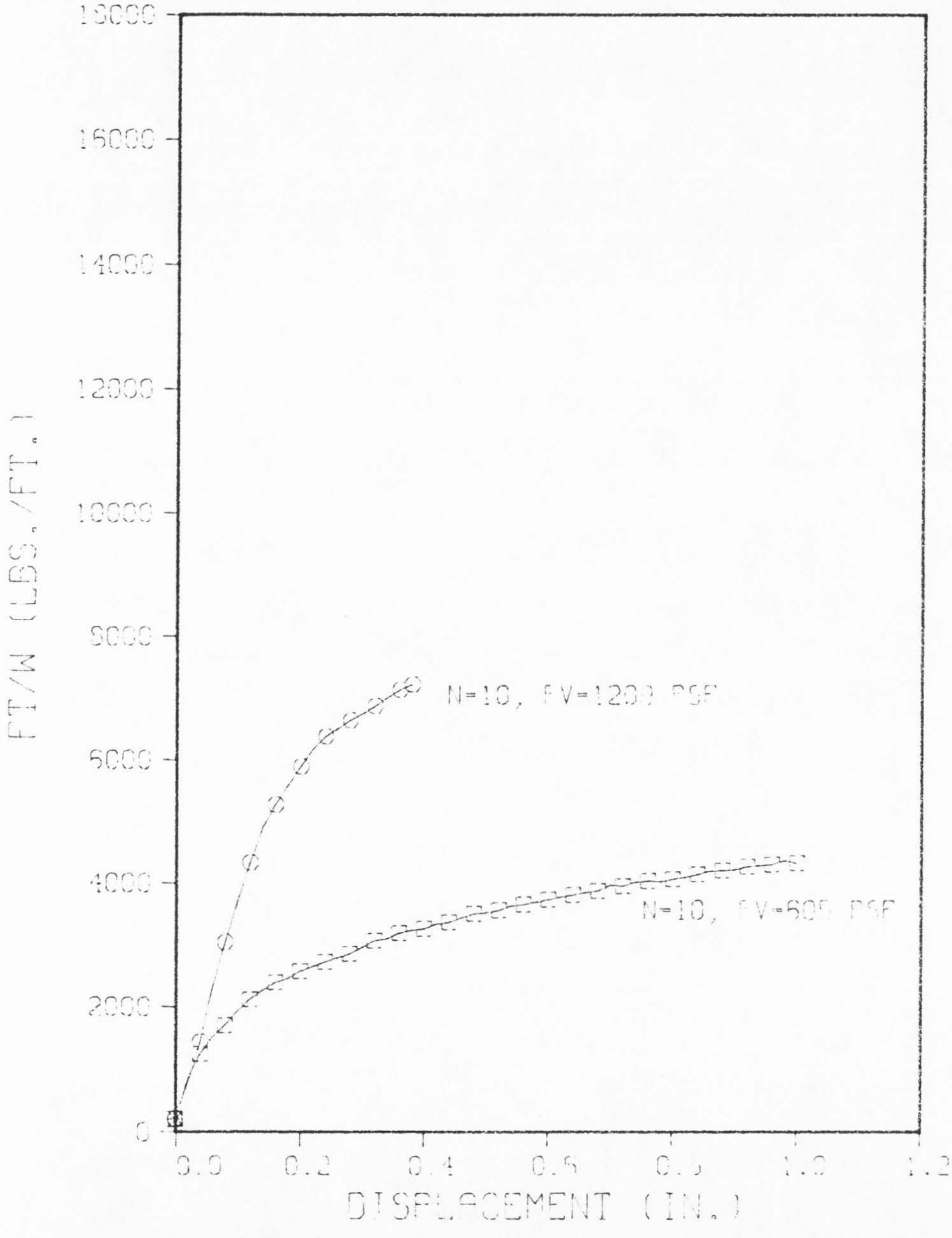

Figure 81. Load-diplacement curves for 9 gage wire in washed sand

Figure 82. Affect of diameter for silty sand and N = 4

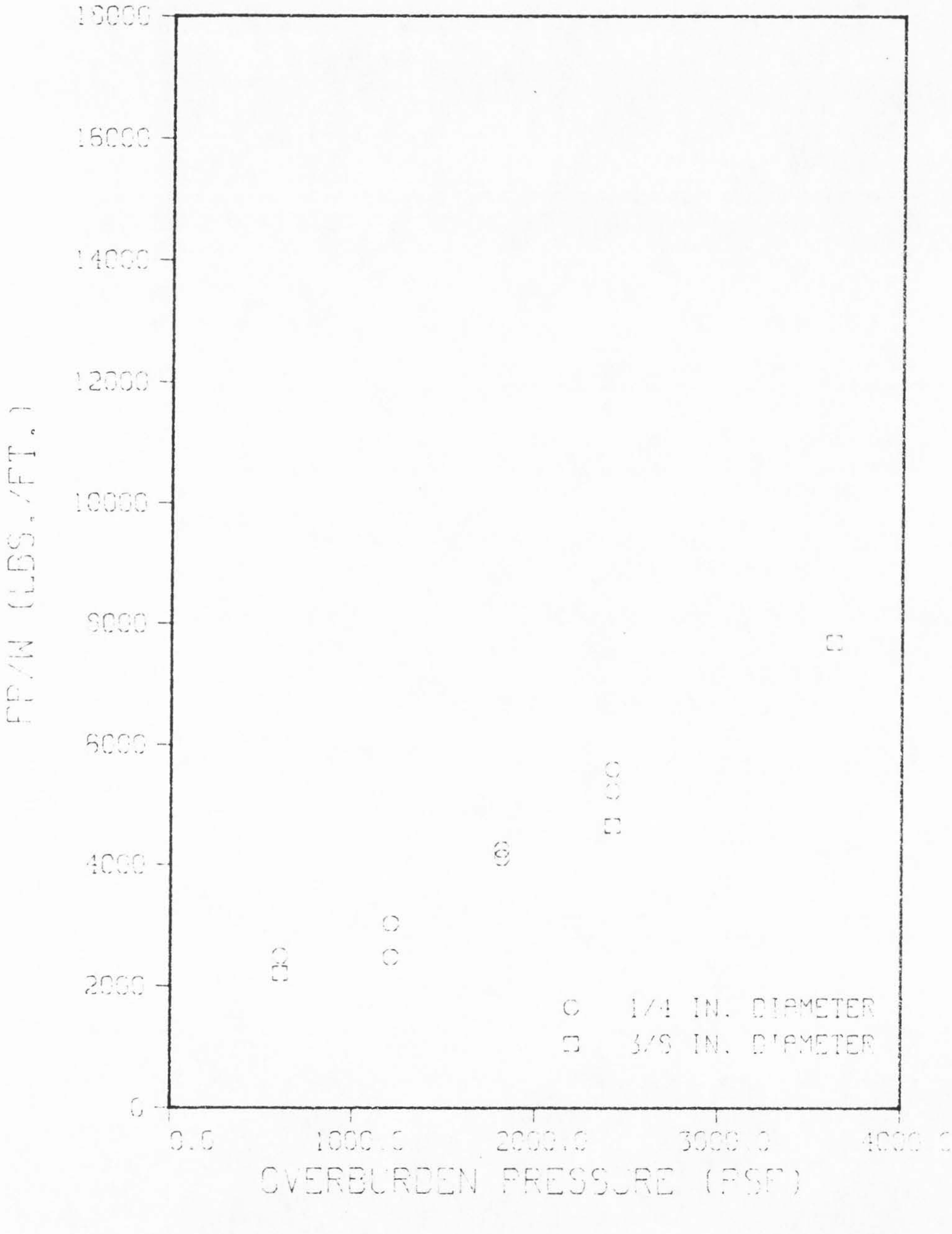

Figure 83. Affect of diameter in silty sand and N = 5

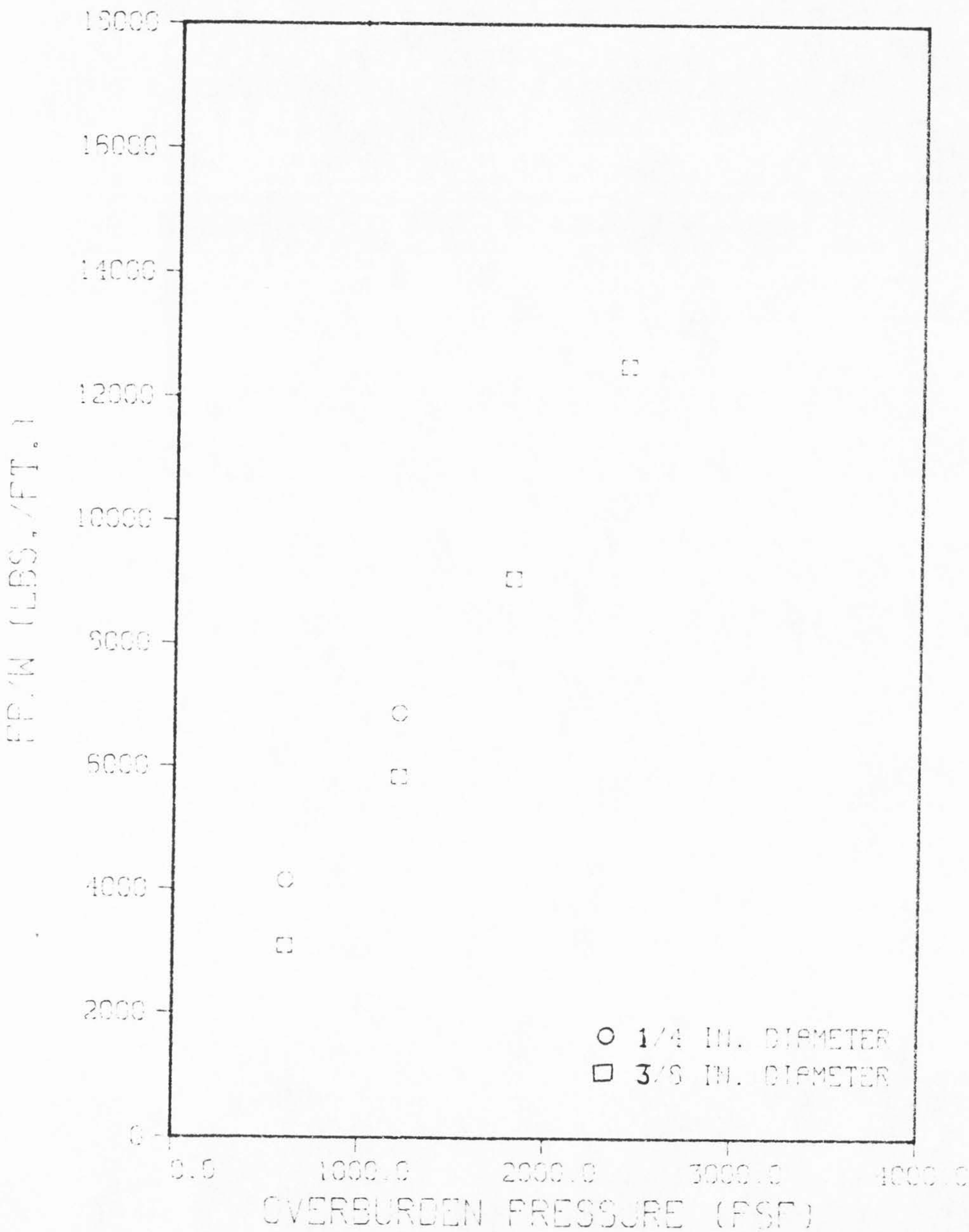

Figure 84. Affect of diameter in pea gravel and N = 4

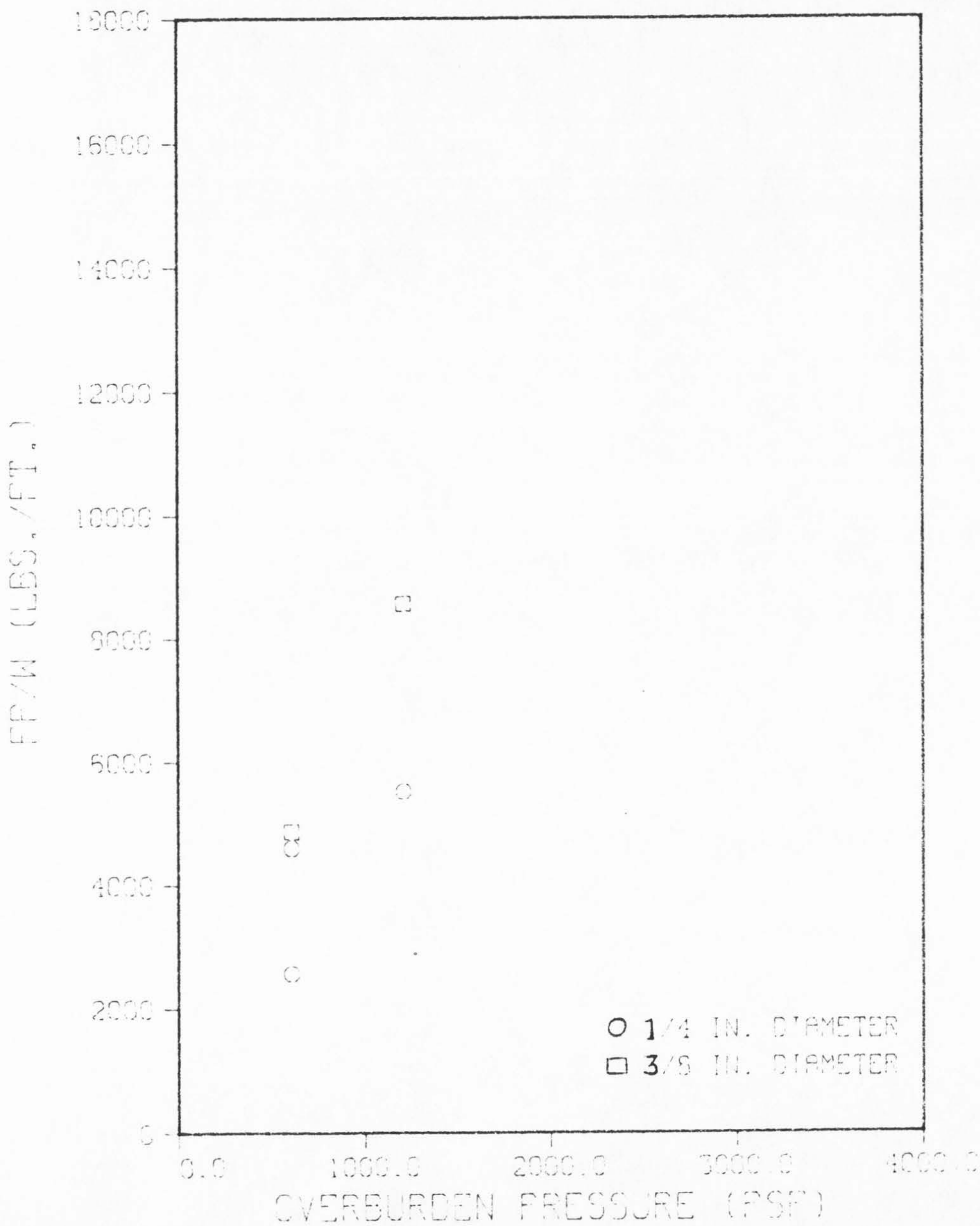

Figure 85. Affect of diameter in pea gravel and N = 5

Figure 86. Affect of diameter in washed sand and N = 4

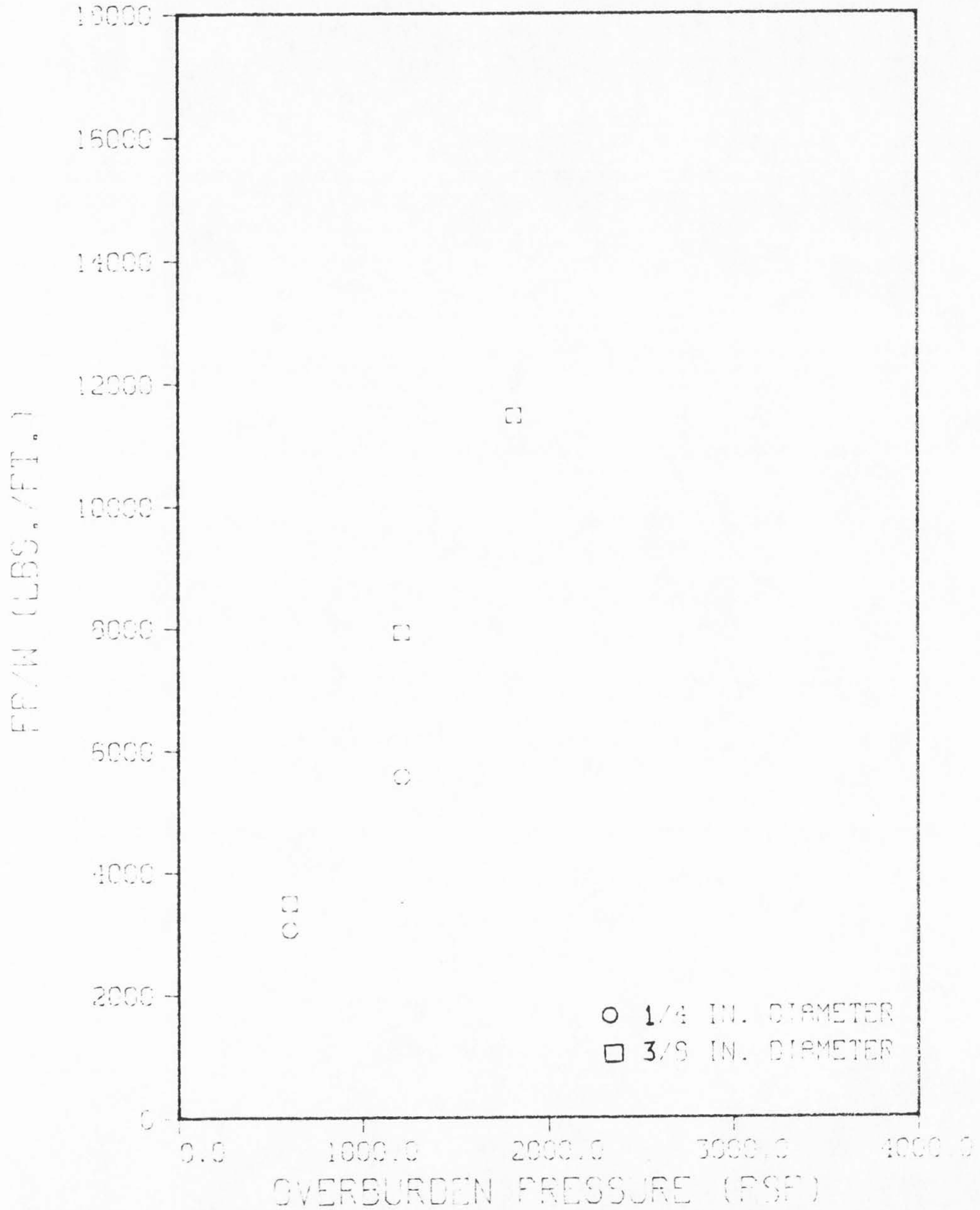

Figure 87. Affect of diameter in washed sand and N = 5

Appendix C

Pullout Test Data

Silty Sand Soil

Date	d (in)	N	W (ft)	σ_v (psf)	M	L (ft)	F_T (lbs)	F_E (lbs)	F_P (lbs)
2/08/83	0.25	6	2.5	1209	6	5.33	8,875	337	8,538
2/08/83	0.25	6	2.5	1814	6	5.25	14,545	449	14,046
2/08/83	0.25	5	2.5	2418	6	5.00	16,590	633	15,957*
2/19/83	0.25	5	2.5	605	6	5.00	10,585	158	10,427
2/19/83	0.25	5	2.5	1209	6	4.92	15,190	311	14,879
2/10/83	0.25	4	2.5	605	6	5.33	7,070	169	6,901
2/10/83	0.25	4	2.5	1209	6	5.25	10,205	332	9,873
2/10/83	0.25	4	2.5	1814	6	5.17	14,115	491	13,624
2/10/83	0.25	4	2.5	2418	6	5.08	17,770	643	17,127
2/09/83	0.25	3	2.5	1209	6	5.33	6,530	337	6,193
2/09/83	0.25	3	2.5	1814	6	5.25	10,795	499	10,296
2/09/83	0.25	3	2.5	2418	6	5.17	14,635	655	13,980
2/09/83	0.25	3	2.5	3023	6	5.08	17,030	804	16,226*
2/21/83	0.25	3	2.5	605	6	5.33	6,405	169	6,236
2/21/83	0.25	3	2.5	1209	6	5.25	7,910	332	7,578
2/21/83	0.25	3	2.5	1814	6	5.17	11,115	491	10,624
2/21/83	0.25	3	2.5	2418	6	5.08	13,705	643	13,062
2/21/83	0.25	3	2.5	3023	6	5.00	14,630	791	13,839*
2/05/83	0.375	6	2.5	453	6	5.33	8,070	190	7,880
2/05/83	0.375	6	2.5	907	6	5.25	12,760	374	12,386
2/05/83	0.375	5	2.5	1360	6	5.00	15,630	534	15,096
2/05/83	0.375	5	2.5	1814	6	4.92	17,895	701	17,194
2/05/83	0.375	5	2.5	2267	6	4.83	20,950	860	20,090
2/17/83	0.375	6	2.5	605	6	5.33	9,245	253	8,992
2/17/83	0.375	6	2.5	1209	6	5.25	15,305	499	14,806
2/15/83	0.375	4	2.5	605	6	5.33	8,450	253	8,197
2/15/83	0.375	4	2.5	1209	6	5.25	13,370	499	12,871
2/15/83	0.375	4	2.5	1814	6	5.17	18,795	736	18,059
2/15/83	0.375	4	2.5	2418	6	5.08	25,000	965	24,035*
1/29/83	0.375	3	2.5	605	6	5.33	5,775	253	5,522
2/03/83	0.375	3	2.5	2418	6	5.25	12,605	997	11,608

Date	d (in)	N	W (ft)	σ_v (psf)	M	L (ft)	F_T (lbs)	F_F (lbs)	F_P (lbs)
2/03/83	0.375	3	2.5	3627	6	5.17	20,700	1473	19,227
2/03/83	0.375	3	2.5	4836	6	5.08	23,900	1929	21,971*
6/08/83	0.15	10	2.5	605	16	5.33	10,435	270	10,165
6/08/83	0.15	10	2.5	1209	16	5.25	16,880	532	16,348*
6/08/83	0.177	10	2.5	605	16	5.33	9,055	319	8,736
6/08/83	0.177	10	2.5	1209	16	5.25	17,305	627	16,678
6/08/83	0.177	10	2.5	1814	16	5.17	24,100	927	23,173*

Pea Gravel Soil

Date	d (in)	N	W (ft)	σ_v (psf)	M	L (ft)	F_T (lbs)	F_F (lbs)	F_P (lbs)
3/01/83	0.25	5	2.5	605	6	5.00	11,710	237	11,473
3/01/83	0.25	5	2.5	1209	6	4.92	16,700	467	16,233*
4/26/83	0.25	5	2.5	605	6	5.00	6,660	237	6,423
4/26/83	0.25	5	2.5	1209	6	4.92	14,240	467	13,773
4/26/83	0.25	5	2.5	1814	6	4.83	16,700	688	16,012*
2/26/83	0.25	4	2.5	605	6	5.33	10,610	253	10,357
2/26/83	0.25	4	2.5	1209	6	5.25	17,665	499	17,166
2/26/83	0.25	4	2.5	1814	6	5.17	19,230	736	18,494*
2/24/83	0.25	3	2.5	605	6	5.33	9,000	253	8,747*
2/24/83	0.25	3	2.5	1209	6	5.25	10,480	499	9,981
5/17/83	0.25	3	2.5	605	6	5.33	3,685	253	3,432
5/17/83	0.25	3	2.5	1209	6	5.25	7,405	499	6,906
5/11/83	0.375	5	2.5	605	6	5.00	12,495	356	12,139
5/11/83	0.375	5	2.5	1209	6	4.92	22,100	701	21,399
5/11/83	0.375	5	2.5	1814	6	4.83	35,300	1032	34,268*
5/10/83	0.375	4	2.5	605	6	5.33	8,095	380	7,715
5/10/83	0.375	4	2.5	1209	6	5.25	15,320	748	14,572
5/10/83	0.375	4	2.5	1814	6	5.17	23,750	1105	22,645
5/10/83	0.375	4	2.5	2418	6	5.08	32,700	1447	31,253
5/10/83	0.375	4	2.5	3627	6	5.00	40,400	2136	38,264*
5/03/83	0.375	3	2.5	605	6	5.33	5,390	380	5,010
5/03/83	0.375	3	2.5	1209	6	5.25	9,735	748	8,987
5/03/83	0.375	3	2.5	1814	6	5.17	16,810	1105	15,705

Date	d (in)	N	W (ft)	σ_v (psf)	M	L (ft)	F_T (lbs)	F_E (lbs)	F_P (lbs)
5/03/83	0.375	3	2.5	2418	6	5.08	21,400	1447	19,953*
5/24/83	0.15	10	2.5	605	16	5.33	10,725	405	10,320
5/24/83	0.15	10	2.5	1209	16	5.25	19,290	798	18,492*
5/28/83	0.177	10	2.5	605	16	5.33	13,150	478	12,672
5/28/83	0.177	10	2.5	1209	16	5.25	24,700	941	23,759*

Washed Sand Soil

6/06/83	0.25	5	2.5	605	6	5.00	7,885	237	7,648
6/06/83	0.25	5	2.5	1209	6	4.92	14,355	467	13,888
6/06/83	0.25	5	2.5	1814	6	4.83	16,200	688	15,512*
6/04/83	0.25	4	2.5	605	6	5.33	5,970	253	5,717
6/04/83	0.25	4	2.5	1209	6	5.25	10,675	499	10,176
6/04/83	0.25	4	2.5	1814	6	5.17	15,860	736	15,124
6/04/83	0.25	4	2.5	2418	6	5.08	18,120	965	17,155*
6/03/83	0.25	3	2.5	605	6	5.33	3,865	253	3,612
6/03/83	0.25	3	2.5	1209	6	5.25	5,595	499	5,096
5/30/83	0.375	5	2.5	605	6	5.00	9,080	356	8,724
5/30/83	0.375	5	2.5	1209	6	4.92	20,500	701	19,799
5/30/83	0.375	5	2.5	1814	6	4.83	29,700	1032	28,668
5/30/83	0.375	5	2.5	2418	6	4.75	33,200	1353	31,847*
6/01/83	0/375	4	2.5	605	6	5.33	9,055	380	8,675
6/01/83	0.375	4	2.5	1209	6	5.25	18,050	748	17,302
6/01/83	0.375	4	2.5	1814	6	5.17	26,050	1105	24,945
6/01/83	0.375	4	2.5	2418	6	5.08	33,000	1447	31,553
6/30/83	0.375	4	2.5	3627	6	5.00	43,000	2136	40,864
5/30/83	0.375	3	2.5	605	6	5.33	8,155	380	7,775
5/30/83	0.375	3	2.5	1209	6	5.25	16,065	748	15,317
5/30/83	0.375	3	2.5	1814	6	5.17	18,030	1105	17,195
5/30/83	0.375	3	2.5	2418	6	5.08	20,750	1447	19,303
6/14/83	0.15	10	2.5	605	16	5.33	10,065	405	9,660
6/14/83	0.15	10	2.5	1209	16	5.25	18,030	798	17,232*

Date	d (in)	N	W (ft)	σ_v (psf)	M	L (ft)	F_T (lbs)	F_F (lbs)	F_P (lbs)
6/14/83	0.177	10	2.5	605	16	5.33	9,365	478	8,887
6/14/83	0.177	10	2.5	1209	16	5.25	18,405	941	17,464
6/14/83	0.177	10	2.5	1814	16	5.17	25,000	1390	23,610*

* Mat broke in tension before reaching displacemnt of 0.75 inches.