AN INVESTIGATION REGARDING THE ADVANTAGES OF CRUSHED STONE FINES AS ENGINEERED BACKFILL

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ABSTRACT

The objective of this study was to investigate the advantages of crushed stone fines as engineered backfill for mechanically stabilized retaining walls. Three materials; a granite screenings, a limestone pond fines and a natural pit run sand; were tested for strength, permeability and chemical properties. Measured properties were compared with requirements in several backfill specifications. A 12 ft. high geotextile fabric reinforced wall was designed using measured properties of the three materials. A design with crushed stone fines required less fabric reinforcement, i.e., fewer and shorter strips. In addition to the obvious economic and construction advantages of less reinforcement, shorter strips can mean less required excavation, less required backfill, and more useable space in congested urban environments.

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INTRODUCTION

In recent years, mechanically stabilized earth (MSE) or reinforced earth has become a primary way to design and quickly build stable cost effective retaining walls or slopes. Incorporating reinforcing elements into a soil mass across potential failure planes creates tensile resistance so that the soil mass acts as a rigid unit much like a gravity wall system.

Classical reinforced earth utilizes metal strips to reinforce the backfill thereby reducing the earth pressure against the wall face. Geosynthetics as primary reinforcing elements perform essentially the same function as metal strips. Geosynthetics include geotextiles, geomembranes, and geogrids. Metal reinforcement is susceptible to corrosion, whereas geosynthetics are resistant to corrosion. However, durability and creep become an issue for concern when using geosynthetics.

The soil or backfill is the second component in a mechanically stabilized or reinforced earth structure. Most designs of reinforced earth specify a free draining granular backfill material. Current practice recommends sands or material which has sand size particles and is free draining. Natural materials may not be available or cost effective. An alternative to natural material, particularly in urban areas, is crushed stone fines.

Crushed stone fines, in general, will perform better than natural soils in terms of strength and stability. This can be significant when horizontal clearance is limited requiring maximization of slopes and minimization of reinforcing element lengths. Crushed stone or manufactured material will have neglible organic matter which should reduce corrosivity. Also, properties of the crushed stone can be controlled in the manufacturing process, so that the designer might select the properties needed and have them produced, rather than having to locate and use less suitable local natural materials. This may result in properties of manufactured materials that have lower variability than natural material, thus, allowing the use of lower factors of safety or, conversly, higher confidence level designs.

OBJECTIVE

The purpose of this research is to investigate the feasibility and advantages of crushed stone fines as engineered backfill for mechanically stabilized retaining walls.

SCOPE

Crushed stone fines can be used as backfill material for mechanically stabilized retaining walls and earth slopes, if the soil strength properties, the chemical properties, and the permeability properties are determined to be satisfactory. Three different materials, 810 granite screenings, limestone pond fines, and natural pit run sand, were selected and tested. Triaxial testing was used to determine soil strength properties. Pullout tests were conducted to estimate shear transfer between backfill and geotextile reinforcement. Chemical composition testing was conducted. Constant head permeability tests were used to determine hydraulic conductivity. Design examples to show the advantages of crushed stone fines versus natural sand backfill are developed and presented.

DESIGN METHODS

There are several procedures for designing reinforced earth retaining walls (Koerner, 1990; AASHTO, 1991; Mirafi, 1989). Three of the more widely used procedures are the rigid block method, equivalent homogeneous soil, and the failure wedge method (Bell, 1992). The failure wedge method is based on conventional earth pressure theory and is the most popular for extensible reinforcing. Typically the design progresses in parts with the first being the internal stability, second the external stability, and thirdly to check any construction details such as wall facing connections (Koerner, 1990). External stability of mechanically stabilized earth walls is essentially the same as for other retaining walls.

The failure wedge method will be used in design examples to demonstrate the advantages of crush stone. Analysis of internal stability and construction details will show that the amount of reinforcement (length and number of reinforcing elements) and amount of backfill may be reduced. This reduction will be primarily due to higher strength. However, other potential advantages from improved drainability, constructibility and uniformity cannot be quantified with current design methods. External stability is not a function of backfill or reinforcement and will not be included in examples.

Design details of a mechanically stabilized wall are shown in Figure 1. The backfill is assumed to fail according to the Rankine earth pressure theory. Pressure on the wall face is computed with the Rankine active earth pressure coefficient

$$K_a = \tan^2 (45 - \phi/2)$$

where

ϕ = the backfill angle of internal friction

Failure occurs along a failure plane oriented at an angle of 45 + ϕ/2 with the horizontal as shown in Figure 1.

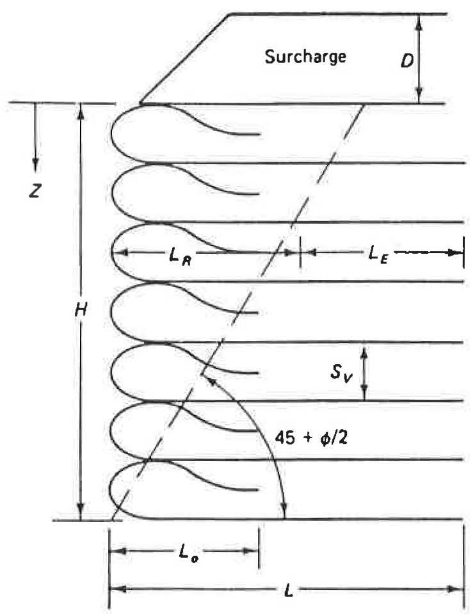

Figure 1 Cross Section of Design Details

Internal stability analysis involves computation of reinforcement length and spacing. Reinforcement element length from the wall face is computed with the equation

$$L = L_E + L_R$$

where

L_E = the length past the failure plane
= active reinforcement length

and

L_R = the length from the wall face to the failure plane
= nonactive reinforcement length

The total length of reinforcement elements is the sum of L and L_O where

L_O = the overlap length.

Minimum active reinforcement and overlap lengths of 3 ft. are recommended. Reinforcement element spacing is designated S_v and H the total wall height.

To develop relationships for reinforcement element lengths and spacings horizontal stress is computed with the equation

$$\sigma_h = K_a \gamma z + K_a q$$

where

γ = backfill unit weight
z = depth
q = surcharge loading.

Vertical reinforcing strip spacing is computed with the equation

$$S_v = T_a /(\sigma_h)(FS)$$

where

T_a = allowable reinforcement tensile strength
FS = factor of safety.

Non-active reinforcement length is a function of depth and failure angle orientation and is computed with the equation

$$L_R = (H\text{-}z) \tan (45 - \phi/2).$$

Active reinforcement length is computed with the equation

$$L_E = \frac{S_v \ \sigma_h \ FS}{2 \ (\gamma \ z \ \tan \ \delta)}$$

Where

δ = friction angle between the backfill and reinforcement.

Overlap lengths are computed with the equation

$$L_O = \frac{S_v \ \sigma_h \ FS}{4 \ (\gamma \ z \ \tan \ \delta)}$$

Although reinforcement lengths and spacings shown in Figure 1 are constant with depth, application of the above equations will give lengths and spacings that decrease with depth. During design, the equations and rules of rounding are applied to give only two or three sets of element lengths and spacing for ease of construction.

STRENGTH PARAMETERS

Soil backfill parameters used in the design of a reinforced earth wall include unit weight (γ) which is directly related to the horizontal stress, angle of internal friction (ϕ) which is used to determine the failure surface and lateral earth pressure coefficient, angle of friction between backfill and reinforcement (δ) which is a function of the backfill friction angle and reinforcement roughness, and cohesion (c) which is taken as zero if the backfill is granular. The embedment length is a function of the shear strength of the backfill. Granular backfill shear strength is a function of angle of internal friction (ϕ) and the normal stress, which is a function of the depth (z) and unit weight (γ) of the backfill material. The pullout resistance is a product of the overburden pressure and the coefficient of friction between the backfill and the reinforcing element which is taken as one half to two-thirds the angle of internal friction of the backfill. Therefore, L_E decreases as ϕ and z increases. The total reinforcement length will also decrease as ϕ and z increase since the overlap length, L_O, is normally constant.

Generally designs with a factor of safety of 1.3 to 1.5 can be achieved with reasonably sized reinforcing elements if the unit weight of the backfill and the internal angle of friction are approximately 100-120 pounds per cubic foot and 30°, respectively. Designs with manufactured materials having less variability may be acceptable with somewhat lower factors of safety.

OTHER FACTORS EFFECTING BACKFILL PERFORMANCE

Other considerations that must be examined when determining backfill for a reinforced slope or wall are drainage, creep if using a geosynthetic, and corrosion resistance. Drainage of the backfill material is essential to prevent pore pressure build up behind the structure. The selected backfill material should not retain water and should drain freely. Most specifications do not contain minimum limits for coefficient of permeability, but rely on gradation requirements to insure free drainage. Maximum percentages passing the #200 sieve are common in many specifications.

When using metallic reinforcement, corrosion resistance must be taken into consideration. Corrosion of buried metals depends on dissolved salts, pH, porosity, and degree of saturation. Highest corrosion rates are produced by a high dissolved salts content, a high chloride concentration, a high sulfate content, and acidic or alkaline pH conditions in the soil. Corrosion rate predictions are difficult and uncertain. Corrosion rates are based on correlations between electrochemical cells, and measurements taken of actual structures. One method of determining corrosive rates is the Box Test (Christopher, 1990). In the Box Test, specimens of metal are placed into soils with known properties, and metal weight loss is determined as a function of time.

CURRENT BACKFILL SPECIFICATIONS

Specifications for backfill materials may contain requirements for some or all of the following properties:

Gradation,
Strength,
Soundness and Durability,
Resistivity, and
Chemical Properties.

For comparative purposes, Georgia Department of Transportation (DOT) and Federal Highway Administration (FHWA) recommendations are summarized below.

GEORGIA DEPARTMENT OF TRANSPORTATION SPECIFICATIONS

Georgia DOT specifications call for a backfill material which is free of organic or other deleterious material. Listed below are specific requirements for special embankment backfill .

1. Soundness (AASHTO T-104): The loss in weight when subjected to five cycles of the Magnesium Sulfate Soundness test shall not be more than 15%.

2. Percent Wear (AASHTO T-96): Maximum Loss 65% ("A" Grading)

3. Grading:

Sieve Size	Percent Passing
4 inches	100
2 inches	80 - 100
No. 10	20 - 90
No. 200	0 - 12

4. Fine Aggregate: Fine aggregate for backfill material shall consist of natural sand, manufactured sand, or a blend of each, having strong,

hard, durable particles free of organic matter, they shall be non-plastic.

5. Additionally, all material shall conform to the following requirements.
   A. pH 6.0 to 9.5
   B. Resistivity > 10,000 ohms/cm
   C. Chlorides < 20 ppm
   D. Sulfates < 15 ppm
   E. $CaCO_3$ Acidity < 15 ppm

FEDERAL HIGHWAY ADMINISTRATION SPECIFICATIONS

FHWA recommendations for select granular backfill for mechanically stabilized earth are listed below (FHWA, 1988):

1. Gradation:

Sieve Size	Percent Passing
4 in.	100
No.40	0 - 60
No.200	0 - 15

2. Angle of Internal Friction > 34°. No strength testing is required for backfill material where 80% of particles are greater than 3/4 in.

3. Soundness: The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30% after four cycles.

4. Other Requirements:
   A. Resistivity > 3,000 ohms/cms
   B. pH 5 to 10
   C. Chlorides < 50 ppm
   D. Sulfates < 500 ppm

FHWA backfill specifications also suggests that the plasticity index not exceed 6, and that the maximum percent passing the No. 200 sieve may be increased to 25 when all other criteria are satisfied. All test methods should be in accordance with AASHTO or ASTM requirements.

CORROSIVITY REQUIREMENTS (Christopher 1989)

Listed below are ranges for maximum corrosive material to minimum corrosive material.

Resistivity The following qualitative relationship is generally accepted:

Aggressiveness	Resistivity ohm/cm
very corrosive	< 700
corrosive	700 - 2,000
moderately corrosive	2,000 - 5,000
mildly corrosive	5,000 - 10,000
noncorrosive	> 10,000

pH Soils with pH < 4.5 (acidic) or with pH > 9 (alkaline) are generally associated with high corrosion rates in carbon steel. Also, galvanization is strongly attacked in highly acidic or alkaline soils.

Water Content Maximum corrosion rates generally occur at saturations of 60 to 75 percent.

Soluble Salts Chloride and sulfate accelerate corrosion by disrupting the formation of protective layers.

The following list is the recommended electrochemical properties for suitable reinforced soil backfill.

Property	Criteria	Test Method
resistivity	> 3,000 ohm/cm	California DOT No. 643
pH	> 5 and < 10	California DOT No. 643
chlorides	< 200 ppm	California DOT No. 422
sulfates	< 1000 ppm	California DOT No. 417

California test procedures were recommended because AASHTO and ASTM methods were not considered to provide adequate sensitivity or repeatability.

TESTING PROGRAM

Three types of materials were evaluated for potential as engineered backfill. These were granite screenings (ALDOT 810 gradation) from Vulcan Materials' LaGrange, Georgia quarry, dolomitic limestone pond fines from Dravo's Auburn, Alabama quarry, and a natural sand from a Lee County, Alabama borrow pit. Gradations for the materials are shown in Figure 2.

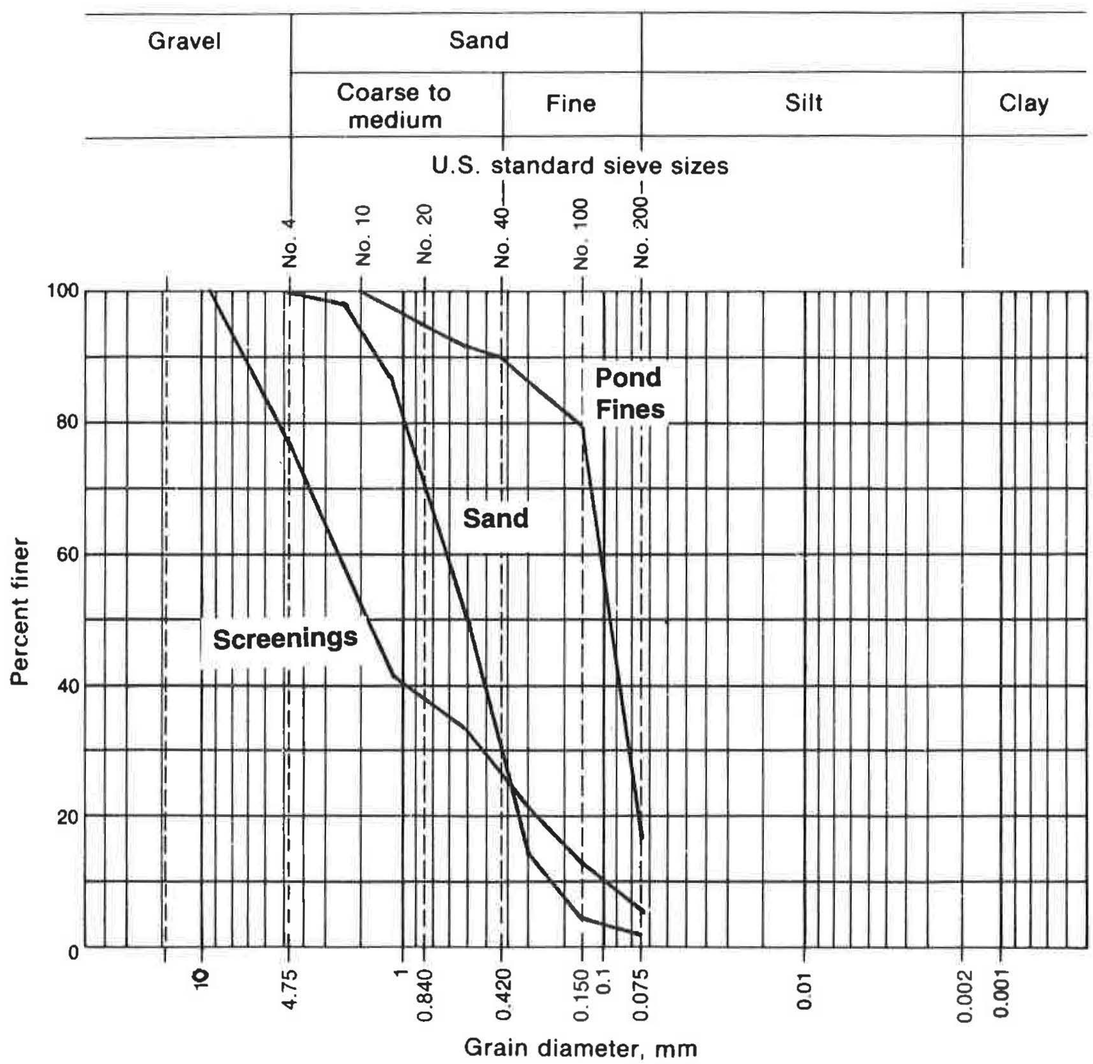

Figure 2. Gradations of Materials Tested

To characterize the materials for mechanically stabilized earth walls and slopes, four types of testing were used. Triaxial testing was used to determine the strength parameters, c and ϕ. Permeability tests were performed to determine Darcy's coefficient of permeability. Pullout tests were used to determine the interactive properties between a geosynthetic and the fill materials. Chemical testing was employed to determine the materials' corrosive potential.

TRIAXIAL TESTING

Due to its versatility, the triaxial test was chosen to determine strength properties. Control of the sample's drainage and of the stress paths to failure are some advantages with triaxial testing. The consolidated drained (CD) test was used to measure effective stress properties of the materials.

Samples for triaxial testing were prepared using a minimum and maximum relative density approach, with American Society of Testing and Materials D4253 and D4254 as a reference guide. The triaxial base and a split mold were used for preparing samples. A vibrating table was used to compact samples. Each sample was prepared as close as possible to the same relative density.

After compaction of the sample, the triaxial chamber was assembled. To keep the sample steady and minimize disturbances a vacuum was applied until the water was added to the chamber. To evacuate as much air as possible from the sample, de-aired water was slowly introduced at the bottom cap of the sample where the vacuum had been released. When the water had displaced most of the air throughout the sample and air bubbles could not be observed leaving the sample through the top cap, seepage saturation was discontinued. Back pressure saturation was then applied in increments of 5 psi to complete saturation. A pressure difference of 2 psi was employed between the chamber and the sample.

The CD triaxial tests were preformed on samples consolidated to effective confining stresses of 5, 10, and 15 psi. Drainage lines were kept open during shearing and the volume change of the sample measured. Deflection and load on the sample were also recorded during shear. From these measurements the axial strain, the volumetric strain, and the principal stresses were determined. These data are shown as Mohr-Coulomb diagrams, p-q plots, stress-strain plots, and axial strain - volumetric strain plots in Appendix A.

Results from triaxial tests are summarized in Table 1. As expected, the 810 granite screenings had the highest angle of internal friction due to their coarser gradation and angular particle shape. Even though the pond fines had the finest gradation with 16.4% passing the #200, the friction angle was 37 ^{o} and there was no discernible cohesion. This is indicative of the angular and nonplastic nature of the crushed particles. The natural sand had the lowest angle of internal friction and reflects its more rounded particle shape.

The last column in Table 1 shows the axial strain at maximum deviator stress, i.e., failure. These values range from 4 - 6% for the 810 screenings to approximately 15% for the natural sand. Although limit design procedures for reinforced walls and embankments do not consider movement, the strain values suggest structures constructed with the natural sand would likely deform more than structures constructed with the crushed materials.

Table 1 Strength Characteristics from Triaxial Tests

Material	D_r	ϕ	c	Axial Strain at Failure
810 Screenings	76%	40.5 °	4.7psi	4 - 6%
Pond Fines	89%	37 °	0	7 - 8%
Natural Sand	99%	35 °	0	≈15%

Plots in Appendix A indicate consistent relative density for the screenings at an average of 76%. Volumetric strains indicate dilation during shear. This is reflective of the moderate (compared to the pond fines and natural sand) compaction achieved and the rough textured and angular particle. Samples of pond fines were compacted to consistent average relative densities of 89%. Dilation was observed during shear indicating high compaction and angular particle shape. The natural sand was compacted to the highest, average of 99%, relative density. However, no dilation was observed during shear reflecting the natural sands more rounded particles with smoother surface texture.

PERMEABILITY TESTING

ASTM test method D-5084 was used for determining permeability coefficients. Hydraulic gradients of approximately one were used so that lab measurements might represent field conditions.

Average coefficients of permeability for the three materials are shown in Table 2. There is an order of magnitude difference between the permeability of the 810 screenings and the pond fines, with the natural sand between. This reflects the influence of gradation, particularly the difference in the percent passing the #200 sieve. It should be noted that density of the samples also had some effect on the measured permeability, but probably not enough to change their relative drainability.

Table 2 Summary of Permeability Testing

Material	Coefficient of Permeability
810 Screenings	1.2×10^{-3} cm/sec
Natural Sand	1.5×10^{-3} cm/sec
Pond Fines	3.4×10^{-4} cm/sec

The 810 screening have relatively good drainage characteristics with about the same magnitude as the natural sand. Pond fines are a very fine silty material with poorer drainage characteristics. However, no cohesion was measured during the triaxial testing of pond fines which indicates they are inert and that water would not adversly affect strength or cause expansive pressure.

With the addition of some larger sizes in the mix of this material, drainage characteristics could likely be improved.

PULLOUT TESTING

Pullout tests were used to characterize the geosynthetic-soil interaction. The test device is shown in Figure 3. The reinforced plywood box, into which soil and geotextile strips were placed, is 3 ft. long, 2 ft. wide and 1 1/2 ft. deep. It was supported on steel I beams with screw jacks for leveling. An airbag confined by a lid on top of the box was inflated to vary the normal stress on the geosynthetic. A geosynthetic strip was pulled through a slot in the front of the box. Special clamps grab the geosynthetic and the load is applied with a mechanical loading machine..

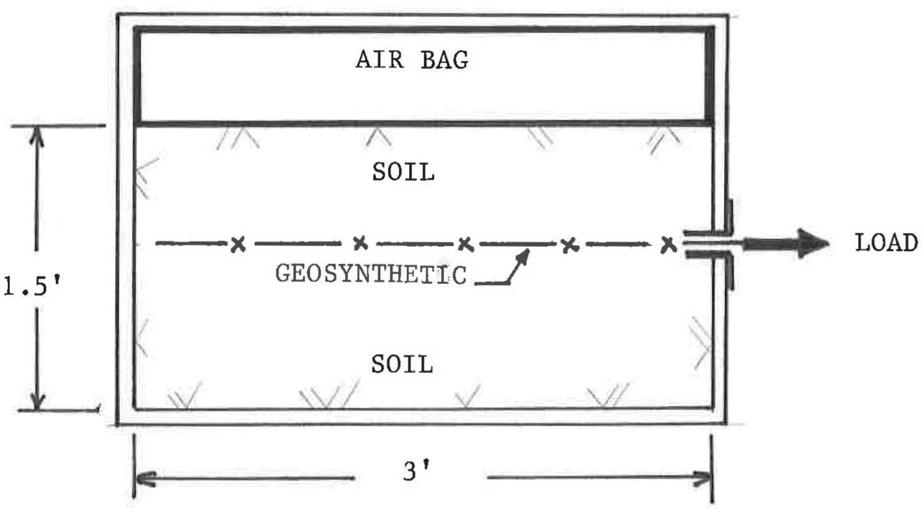

Figure 3 Cross Section of Soil-Geosynthetic Interaction Box

A geosynthetic sample was cut approximately 4 inches wide by 39 inches long. Three inches of the sample was enclosed in the clamps, so 36 inches extend into the box. Soil was placed in the box and compacted in 2 inch increments until the level of the soil reached the opening. The geosynthetic strip was then placed. More soil was added and compacted until the level reached approximately 2 to 3 inches from the top of the box. A 2 foot by 3 foot piece of sheet metal was placed on the soil providing a flat uniform surface for the airbag. A deflated airbag was placed on the sheet metal and the lid secured. Air was supplied to the air bag creating a normal pressure on the soil. This pressure was maintained constant throughout the test. The testing machine crosshead speed was set at 1 inch per minute and a maximum of 10 inches displacement set as the test end point.

Loads and displacement were monitored as the geosynthetic strips were pulled from the box. These were plotted and are shown in Appendix B. Figures B1 - B4 show plots for normal pressures of 0.5, 1.5, 2.5, and 3.5 psi.

At approximately 6 inches displacement, the load-displacement curves indicate a decrease in load. At this displacement, the clamp exits the pull-out box and is no longer supported. For deflections larger than 5 or 6 inches, the measured forces are suspect because of the influence of the unsupported clamp, possible binding as the fabric exits the box and deformation of the fabric. Because of these potential inaccuracies the analysis and use of results from the pullout tests will be limited to deflections less than 5 inches.

Figure B.5 through B.7 show replotted graphs of the mobilized shear stress up to 5 inches displacement versus displacement. The mobilized shear stress is calculated using the applied loads and corrected contact areas between soil and fabric. Figure 4 shows the maximum mobilized shear stress versus the normal stress. Linear regression performed on each set of data indicates the best line fit for each material, and provides an indication of the adhesion (shear stress at zero normal stress) and coefficient of friction (slope of line) between the soil and fabric. Adhesions and friction angles are summarized in Table 3.

Table 3 Summary of Pull-Out Testing

Material	Adhesion	Friction Angle
810 Screenings	0.7 psi	7^{o}
Natural Sand	0.5 psi	9^{o}
Pond Fires	1.1 psi	2^{o}

The measured interface friction angles do not compare well with the often used assumption, $\delta = 2/3\phi$. The pullout test does not appear to provide a reliable indication of δ for the following reasons: (1) Low normal stresses may not fully represent field conditions. Average normal stress in typical walls 30 to 40 feet range from 11.4 to 15.3 psi. Normal stress in the pullout test ranged from .5 to 3.5 psi. (2) Elongation and necking of the geosynthetic may have resulted in measured deflections larger than actual relative movements between fabric and soil. (3) Progressive failure, where shear stresses are

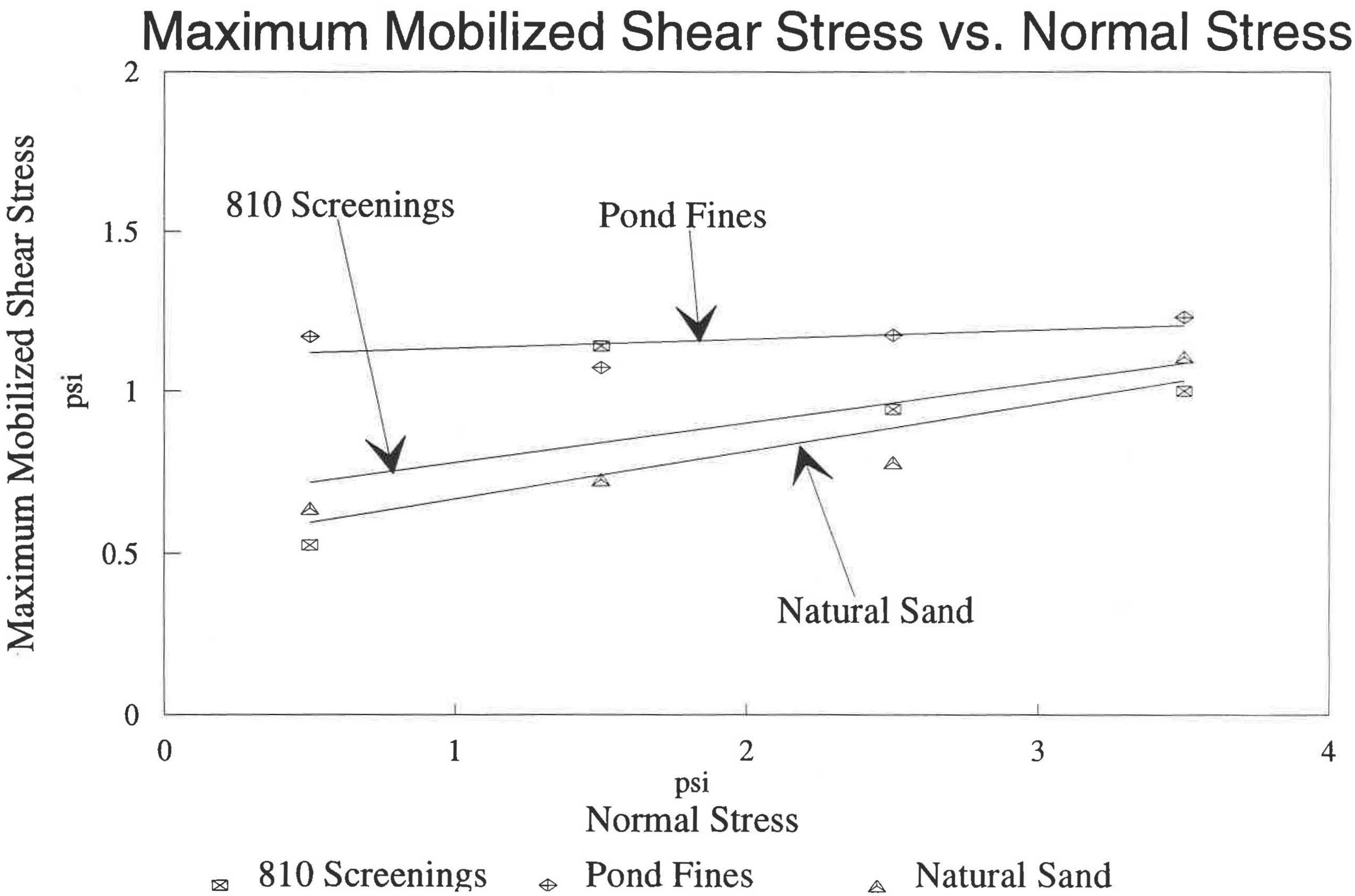

Figure 4 Maximum Mobilized Shear Stress vs. Normal Stress

mobilized step by step rather than simultaneously along the length of the fabric probably occurred. The load at the clamp represents the average shear stress over the sample. (4) Density of the soil was not measured or controlled during the placement and may not have been comparable to densities in the triaxial tests. Because of the unexplained disparities in measured and normally used values, friction angles from the triaxial tests will be used in design examples rather than measured interface friction values.

A detailed examination of the load-displacement plots in Appendix B reveals the following:

The 810 screenings and pond fines did not achieve maximum shear transfer at 10 in. displacement.

The natural sand was nearing peak shear transfer at 5 in. displacement and had reached or exceeded the peak value at 10 in. displacement.

The inability to reach peak shear transfer at relatively large displacement is thought to be due to elongation of the fabric and development of progressive failure along the length of the fabric strip.

The 810 screenings and pond fines demonstrated a greater ability to lock the fabric into the soil and, therefore, developed larger shear transfer than the natural sand. This is attributable to the differences in particle shape and texture.

Only the natural sand demonstrated the expected increase in shear transfer with increasing normal stress. The 810 screenings and pond fines response was erratic.

CHEMICAL TESTING

Chemical testing was performed on all the materials by the Agriculture Soils Testing Laboratory at Auburn University. The materials were tested for pH, electrical resistance and sulfur content. These results are summarized in Table 4. Most specifications contain maximum chloride and sulfates criteria but the Agricultural Testing laboratory could not perform the required test.

Table 4 Chemical Test Results

Sample Material	pH	Resistivity	Sulfur Content
		(ohms/cm)	(%)
810 Screenings	9.12	8333	.0040
Pond Fines	9.15	8333	.0088
Natural Sand	4.28	20,000	.0048

The pH of the 810 screenings (9.12) and pond fines (9.15) meets all three backfill specifications listed previously, but is near the alkaline limit. The pH of the sand (4.28) is lower than the acid limit for all three specifications. The resistivities of the 810 screenings and pond fines (8333 ohms/cm) are less than the minimum of 10,000 ohms/cm in the Georgia DOT specification, but greater than the minimum of 3000 ohms/cm in the other two specifications. The resistivity of the natural sand (20,000 ohms/cm) meets all three specifications.

DESIGN EXAMPLES

A 12 foot high wall with a surcharge load of 200 lb./ft.^{2} will be designed using properties of the 810 screenings, pond fines, and natural sand. A factor of safety, FS = 1.3, will be used in all three designs, although some differences might be warranted because of differences in chemical properties and stiffness. Two reinforcement lengths will be selected for each design with vertical spacings varied in 6 inch increments. Only internal stability will be considered since externally stability is not greatly influenced by backfill properties. A fabric with ultimate wide width tensile strength of 250 lb./in., will be used for all three examples. This will be reduced to an allowable value, T_a = 40 lb/in., by applying factors of safety of 1.5 for installation damage, 3.0 for creep, 1.25 for chemical degradation and 1.1 for biological degradation. Any tensile force supplied by cohesion is assumed zero and cohesion is ignored as a strength factor. The friction angle between the fabric and soil is assumed equal to 2/3ϕ. A unit weight of 110 lb./ft.^{2} will be used for all materials.

WALL DESIGN WITH 810 SCREENINGS

A backfill angle of internal friction, $\phi = 40.5\ ^{o}$, and a backfill reinforcement friction angle, $\delta = 2/3\phi = 26.6\ ^{o}$, will be used for the 810 screenings. Applying this value ϕ and appropriate design values of γ and q gives the following equation for horizontal stress as a function of depth:

$$\sigma_h = 24.2\ (z) + 44$$

Inserting the above equation for σ_h and design values for T_a and FS into the equation for vertical strip spacing gives the following:

$$S_v = \frac{373}{[24.2\ (z) + 44]}$$

Substituting values of into the above equation for S_v and rounding to the nearest .05 ft. gives the following spacing scheme:

Depth (ft.)	Spacing (ft.)
0 - 5	2.0
6 - 8	1.5
9 - 12	1.0

Active and non-active reinforcement lengths are computed by substituting design values into appropriate equations as follows:

$$L_E = \frac{S_v\ \sigma_h\ FS}{2\ (\gamma\ z\ \tan\ \delta)}$$

$$= \frac{S_v\ [32(z) + 57]}{110(z)}$$

$$L_R = (H\text{-}z)\ \tan\ (45\text{-}\phi/2)$$
$$= 5.5 - 0.46(z)$$

Spacings and lengths are summarized in Table 5. Lengths are rounded up to give two strip lengths for ease in construction.

Table 5 Reinforcement Requirements for 810 MSE Wall Example

Layer no.	Depth z (ft.)	Spacing S_v (ft.)	L_E (ft.)	L_E min (ft.)	L_R (ft.)	L (ft.)	Use L (ft.)
8 top	3.0	2.0	0.9	3.0	4.1	7.1	8
7	5.0	2.0	0.8	3.0	3.2	6.2	8
6	6.5	1.5	0.6	3.0	2.5	5.5	8
5	8.0	1.5	0.5	3.0	1.8	4.8	5
4	9.0	1.0	0.3	3.0	1.4	4.4	5
3	10.0	1.0	0.3	3.0	0.9	3.9	5
2	11.0	1.0	0.3	3.0	0.4	3.4	5
1	12.0	1.0	0.3	3.0	0	3.0	5

The final step in design is to check overlap lengths. This is accomplished by substituting design values into the equation for overlap length as follows:

$$L_o = \frac{S_v\ \sigma_h\ FS}{4\ (\gamma\ z\ \tan\ \delta)}$$

$$= \frac{S_v\ [32(z) + 57]}{220(z)}$$

A recommended minimum overlap length is 3 ft. and a maximum overlap length will be computed with the above equation for the top strip where z = 3 ft. and S_v = 2 ft. For the top strip, L_o = 0.5 ft. Therefore, the minimum overlap length of 3 ft. will be required for all strips.

The final design for the wall is eight (8) layers with the lower four (4) layers at 1- ft. centers, the middle two (2) layers at 1.5- ft. centers and the upper two (2) layer at 2- ft. The top three reinforcing strips are 11 ft. long (8 ft. + 3.0 ft.) and the lower five reinforcing strips are 8 ft. long (5 ft. + 3.0 ft.). Figure 5 illustrates the final design.

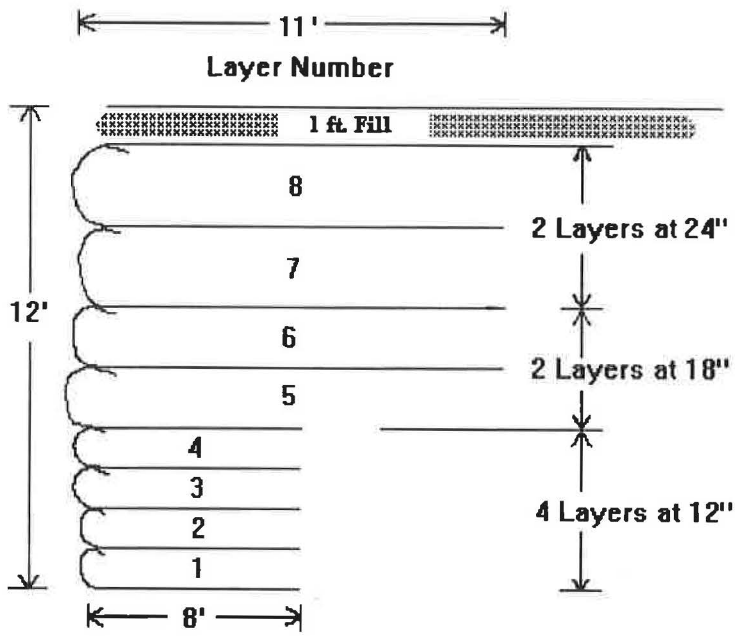

Figure 5 Final Design with 810 Screenings

WALL DESIGN WITH POND FINES

A backfill angle of internal friction, $\phi = 37^{\circ}$, and a backfill reinforcement friction angle, $\delta = 24.6^{\circ}$, will be used for the pond fines. Following the procedure outlined above for the 810 screenings wall, the strip lengths shown in Table 6 are computed. A minimum overlap length of 3 ft. is required for all strips.

Table 6 Reinforcement Requirements for Pond Fines MSE Wall Example

Layer no.	Depth z (ft.)	Spacing S_v (ft.)	L_E (ft.)	L_E min (ft.)	L_R (ft.)	L (ft.)	Use L (ft.)
9 top	2.0	2.0	1.4	3.0	5.0	8.0	8
8	4.0	2.0	1.0	3.0	4.0	7.0	8
7	5.5	1.5	0.7	3.0	3.2	6.2	8
6	7.0	1.5	0.7	3.0	2.5	5.5	8
5	8.0	1.0	0.4	3.0	2.0	5.0	5
4	9.0	1.0	0.4	3.0	1.5	4.5	5
3	10.0	1.0	0.4	3.0	1.0	4.0	5
2	11.0	1.0	0.4	3.0	0.5	3.5	5
1	12.0	1.0	0.4	3.0	0	3.0	5

The final design for the wall is nine (9) layers with the lower five (5) layers at 1- ft. centers, the middle two (2) layers at 1.5- ft. centers and the upper two (2) layers at 2- ft. The top four (4) reinforcing strips are 11 ft. long (8 ft. + 3.0 ft.) and the lower five (5) reinforcing strips are 8 ft. long (5 ft. + 3.0 ft.). Figure 6 illustrates the final design.

WALL DESIGN WITH NATURAL SAND

A backfill angle of internal friction $\phi = 35^{\circ}$ and a backfill reinforcement friction angle, $\delta = 23.3^{\circ}$, will be used for the natural sand. Strip lengths are shown in Table 7. Again a minimum overlap length of 3 ft. is required for all strips.

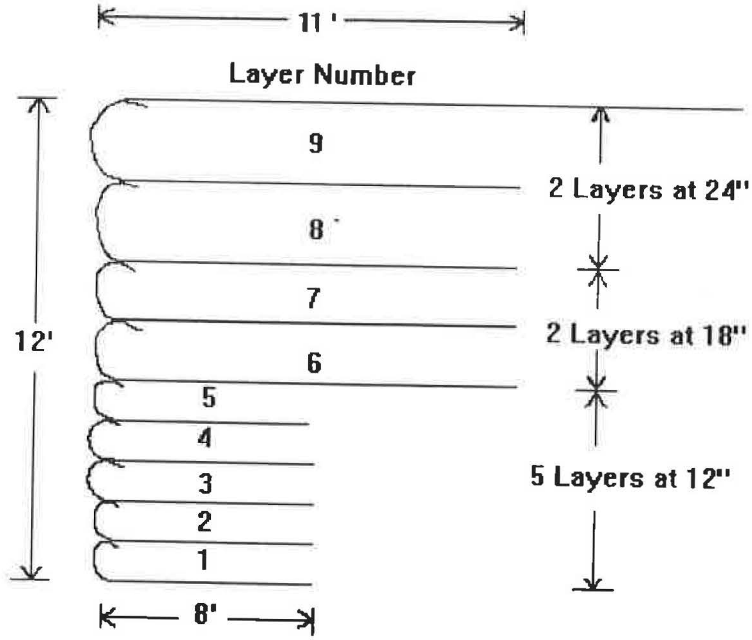

Figure 6 Final Design with the Pond Fines

Table 7 Reinforcement Requirements for Natural Sand MSE Wall

Layer no.	Depth z (ft.)	Spacing S_v (ft.)	L_E (ft.)	L_E min (ft.)	L_R (ft.)	L (ft.)	Use L (ft.)
11 top	1.5	1.5	1.4	3.0	5.5	8.5	9
10	3.0	1.5	1.0	3.0	4.7	7.7	9
9	4.5	1.5	0.9	3.0	3.0	6.9	9
8	6.0	1.0	0.5	3.0	3.1	6.1	9
7	7.0	1.0	0.5	3.0	2.6	5.6	9
6	8.0	1.0	0.5	3.0	2.1	5.1	9
5	9.0	1.0	0.5	3.0	1.6	4.6	5
4	10.0	1.0	0.5	3.0	1.1	4.1	5
3	11.0	1.0	0.5	3.0	0.5	3.5	5
2	11.5	0.5	0.2	3.0	0.3	3.3	5
1	12.0	0.5	0.2	3.0	0	3.0	5

The final design for the wall is eleven (11) layers with the lower two (2) layers at 0.5- ft. centers, the middle six (6) layers at 1- ft. centers and the upper three (3) layers at 1.5- ft. centers. The top six (6) reinforcing strips are 12 ft. long (9 ft. + 3.0 ft.) and the lower five (5) reinforcing strips are 8 ft. long (5 ft. + 3.0 ft.). Figure 7 illustrates the final design.

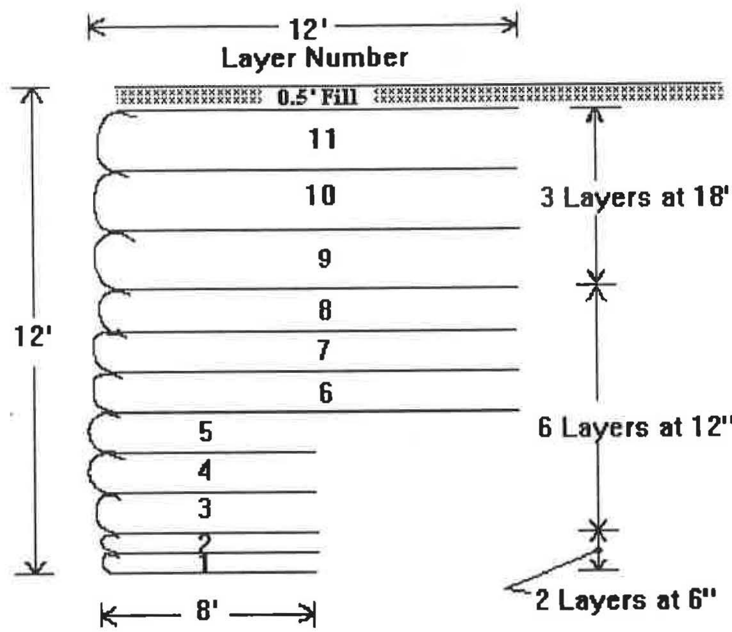

Figure 7 Final Design with the Natural Sand Material

SUMMARY OF EXAMPLE WALL DESIGNS

A summary of the three wall designs is given by Table 8. The design using the 810 screening requires the smallest number of layers. The design for the natural sand requires the most layers and longer top layers. Only eight layers are required for the 810 screenings compared to the eleven layers for the natural sand. The wall with pond fines requires nine layers. Per unit width the wall with 810 screenings requires 73 feet of fabric, the wall with pond fines 84 feet of fabric and the wall with natural sand 112 feet of fabric. Although the maximum strip length for the natural sand wall is only 1 ft. longer than the maximum lengths for the 810 screenings and pond fines, this theoretically means more clear space between wall face and natural ground will be required. This translates to less useable space outside the wall in confined conditions and larger quantities of excavation and backfill.

Table 8 Summary of Wall Designs

Layers	810 Screenings			Pond Fines			Natural Sand		
	z (ft.)	S_v (ft.)	Total Length*	z (ft.)	S_v (ft.)	Total Length*	z (ft.)	S_v (ft.)	Total Length*
1	3.0	2.0	11	2.0	2.0	11	1.5	1.5	12
2	5.0	2.0	11	4.0	2.0	11	3.0	1.5	12
3	6.5	1.5	11	5.5	1.5	11	4.5	1.5	12
4	8.0	1.5	8	7.0	1.5	11	6.0	1.0	12
5	9.0	1.0	8	8.0	1.0	8	7.0	1.0	12
6	10.0	1.0	8	9.0	1.0	8	8.0	1.0	12
7	11.0	1.0	8	10.0	1.0	8	9.0	1.0	8
8	12.0	1.0	8	11.0	1.0	8	10.0	1.0	8
9	--	--	--	12.0	1.0	8	11.0	1.0	8
10	--	--	--	--	--	--	11.5	0.5	8
11	--	--	--	--	--	--	12.0	0.5	8

*Total Length includes overlap length.

CONCLUSIONS AND RECOMMENDATIONS

Crushed stone fines can be used effectively as backfill material for mechanically stabilized earth walls and slopes. Material tests indicated that pond fines and 810 screenings are acceptable alternatives to natural sand materials as engineered backfill material. Crushed stone fines had higher friction angles than natural sand and, therefore, produce lower active earth pressures. Pullout tests indicate that quarry fines effectively develop greater frictional resistance against pullout than natural sands. Thus, designs with a greater factor of safety may be obtained with crushed stone fines using the same amount of reinforcement or conversely smaller amounts of reinforcement may be used with crushed stone fines to achieve comparable factors of safety.

The 810 screenings were the best for engineered backfill material. The pond fines had good strength properties, but more testing is recommended concerning the permeability. Low permeability of crushed material with large amounts of fines could probably be remedied by adding small amounts of larger grain sizes. Chemical properties of crushed stone fines should also be studied in greater detail to determine their characteristics and their affects. These test should concentrate on evaluating and comparing the consistancy and predictability of the chemical properties of manufactured and natural materials.

Expected ranges of various properties should be developed for by-product fines materials, such as the pond fines so that reliable and predictable designs using these materials may be effected.

As performed in this research, the pull-out test did not adequately measure soil-fabric interface behavior. More effective clamping techniques, load and deformation measureing techniques, and soil/fabric placement procedures should be developed. In addition equipment capable of applying larger and, therefore, more realistic normal stresses at the soil fabric interface should be developed.

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APPENDIX A

RESULTS FROM TRIAXIAL TEST

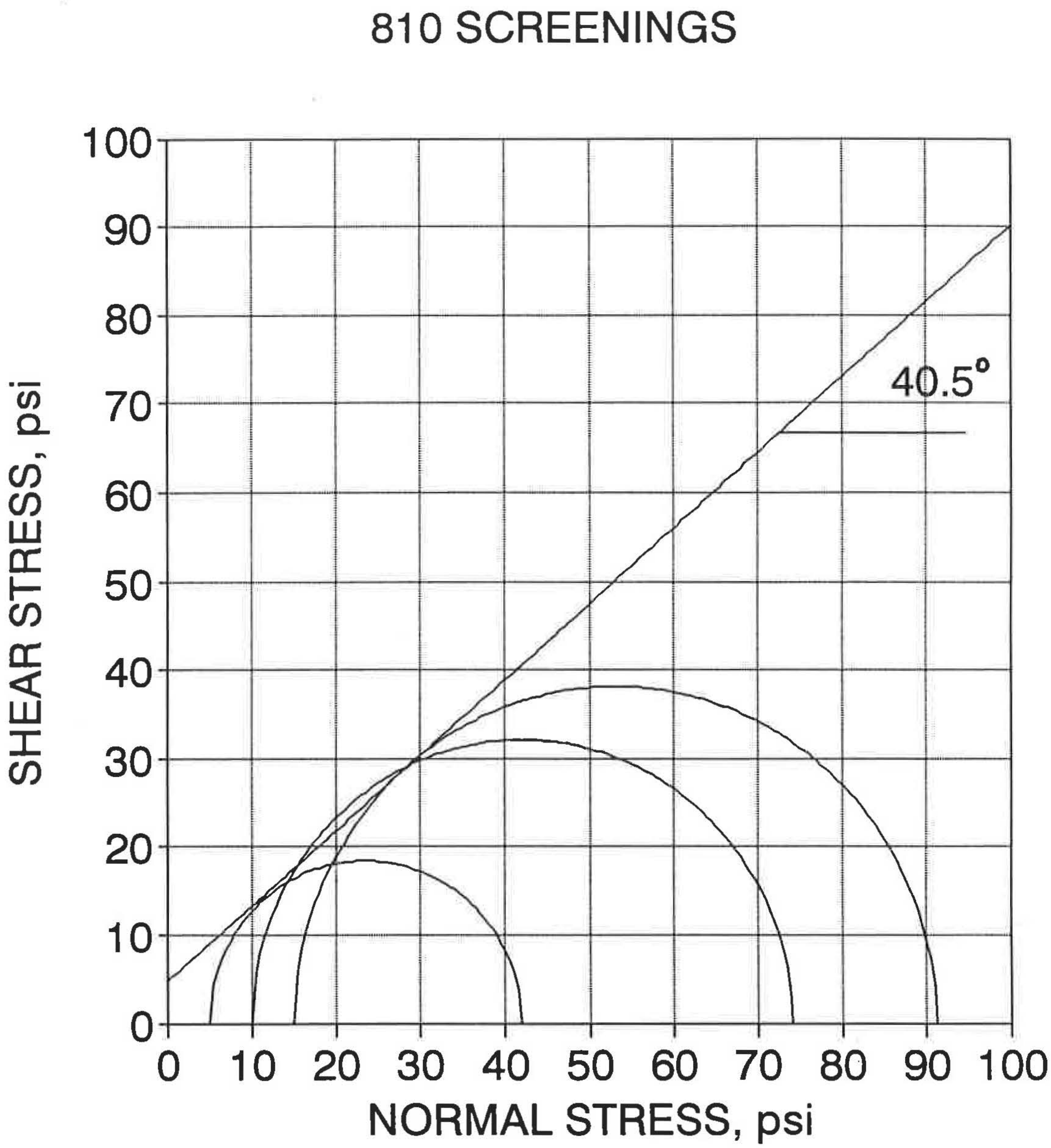

Figure A.1 Mohr-Coulomb Failure Surface of the 810 Screenings

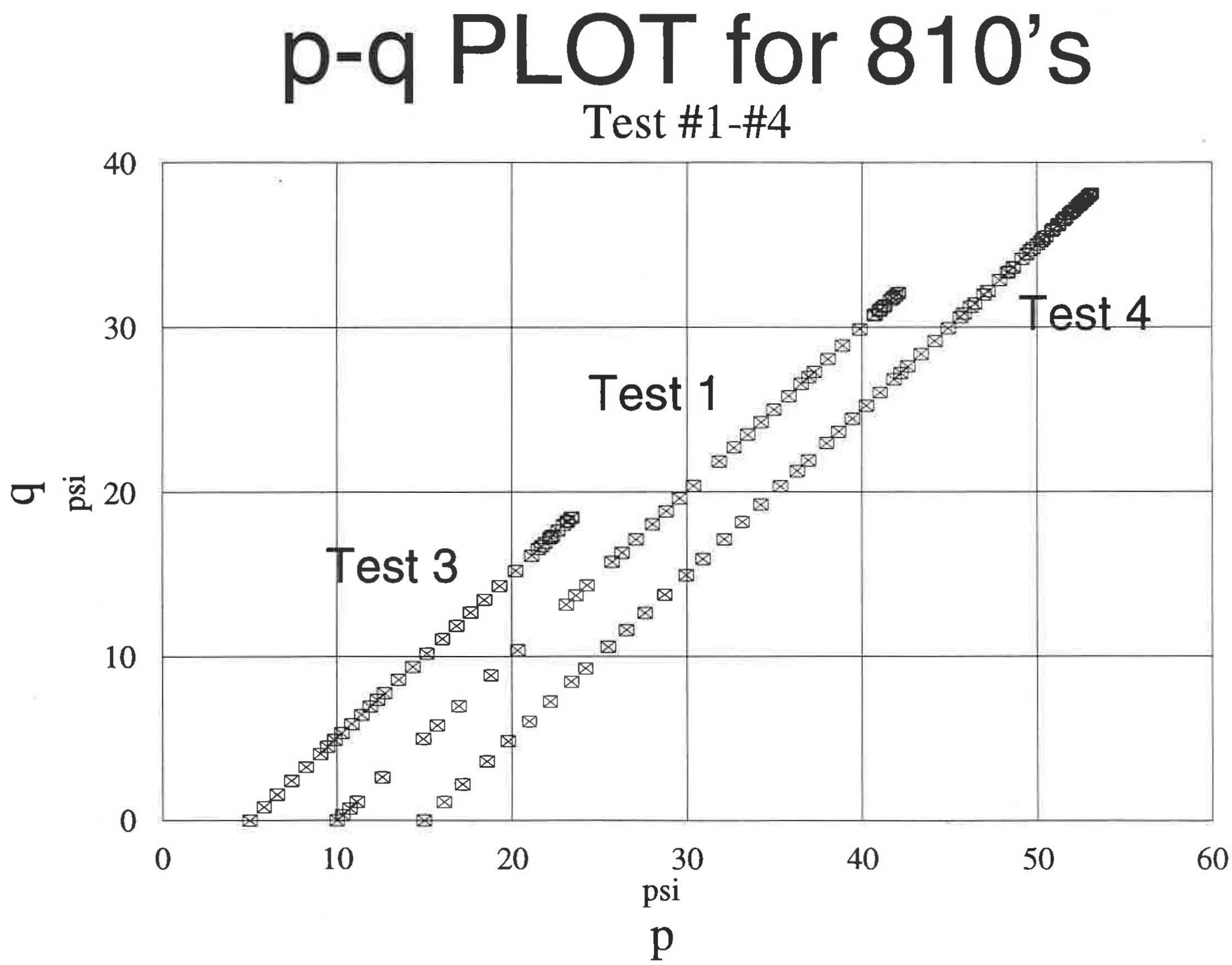

Figure A.2 p-q Plot of 810 Screenings

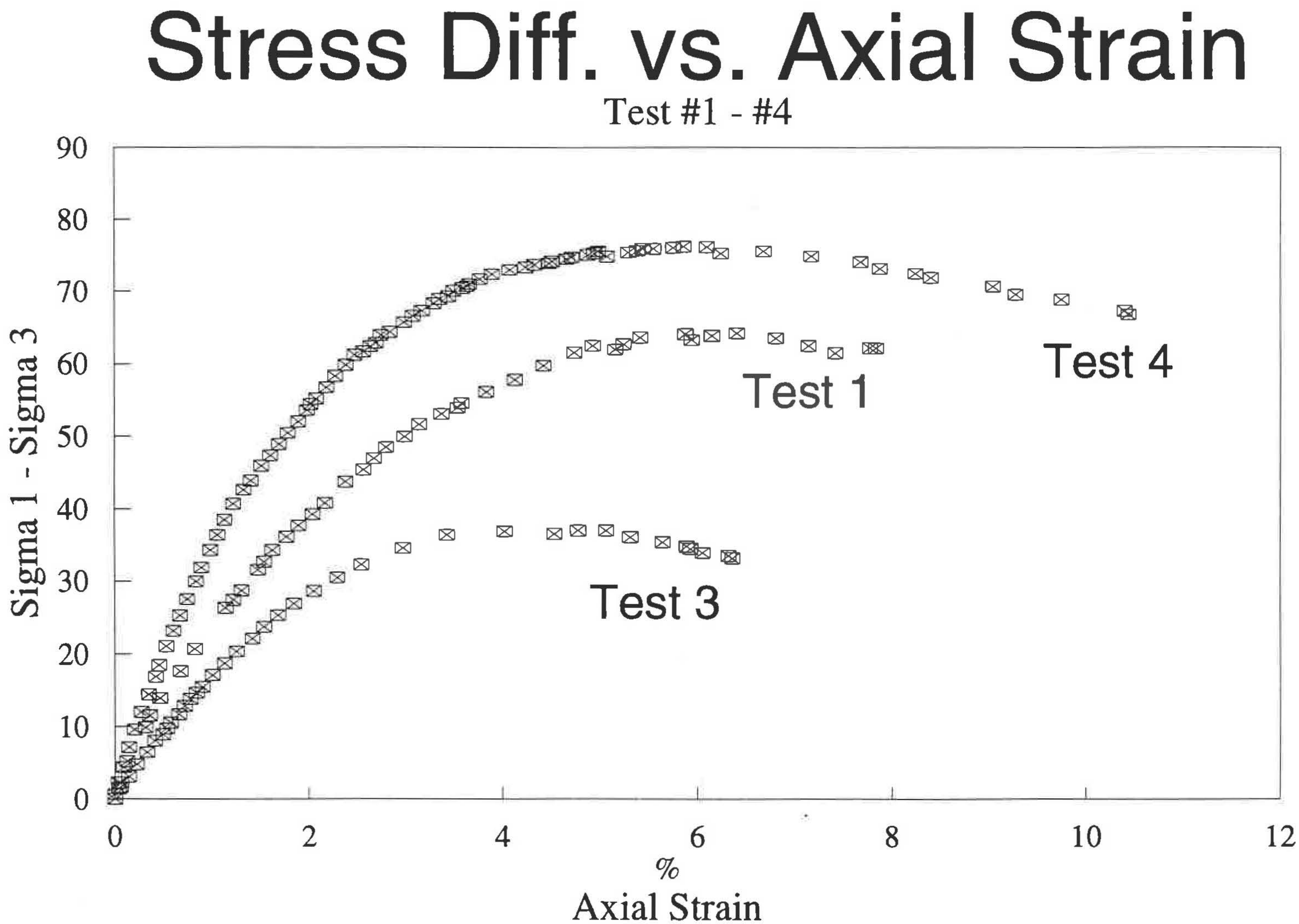

Figure A.3 Stress Difference vs. Axial Strain of the 810 Screenings

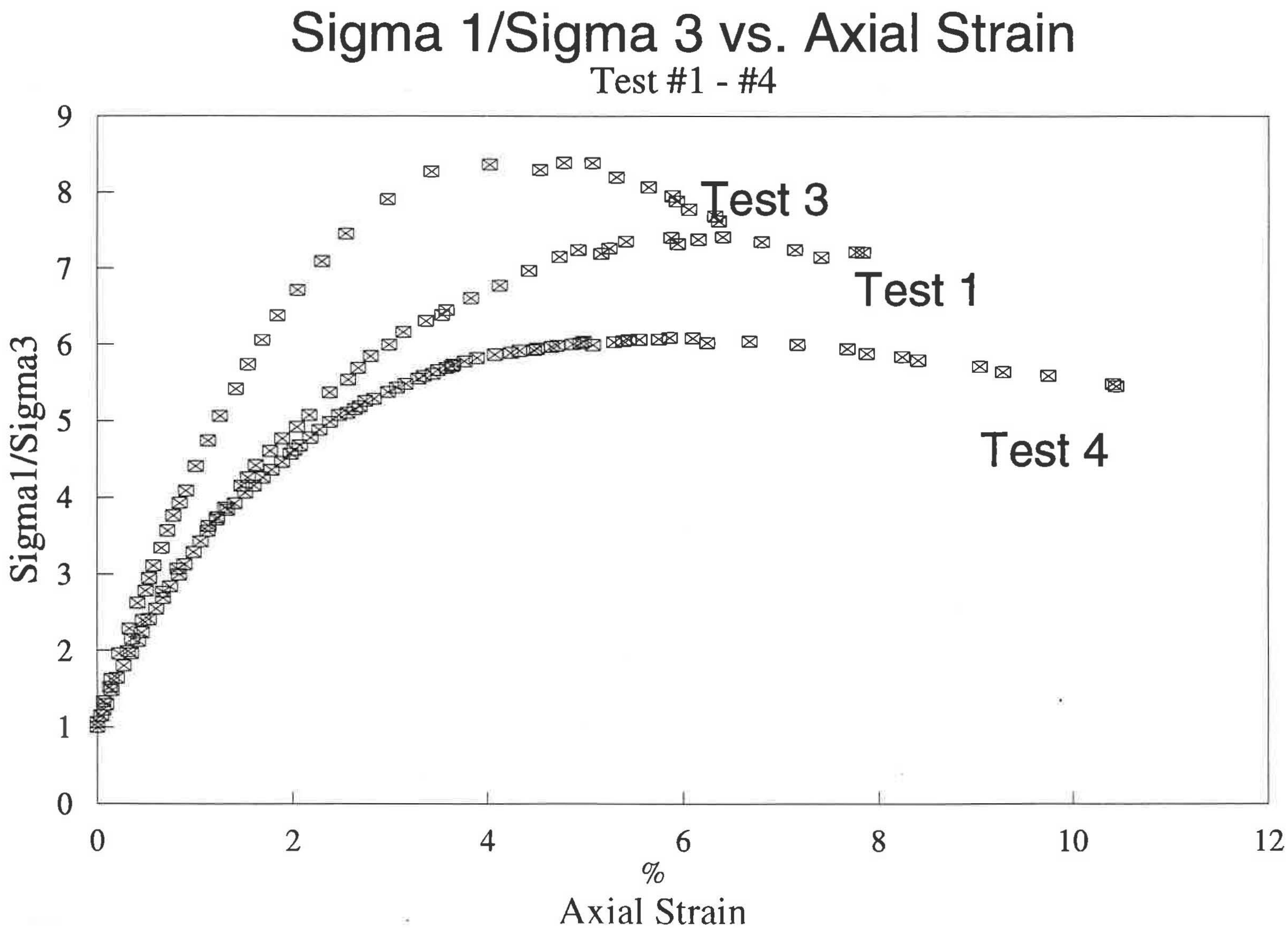

Figure A.4 Stress Ratio vs. Axial Strain of the 810 Screenings

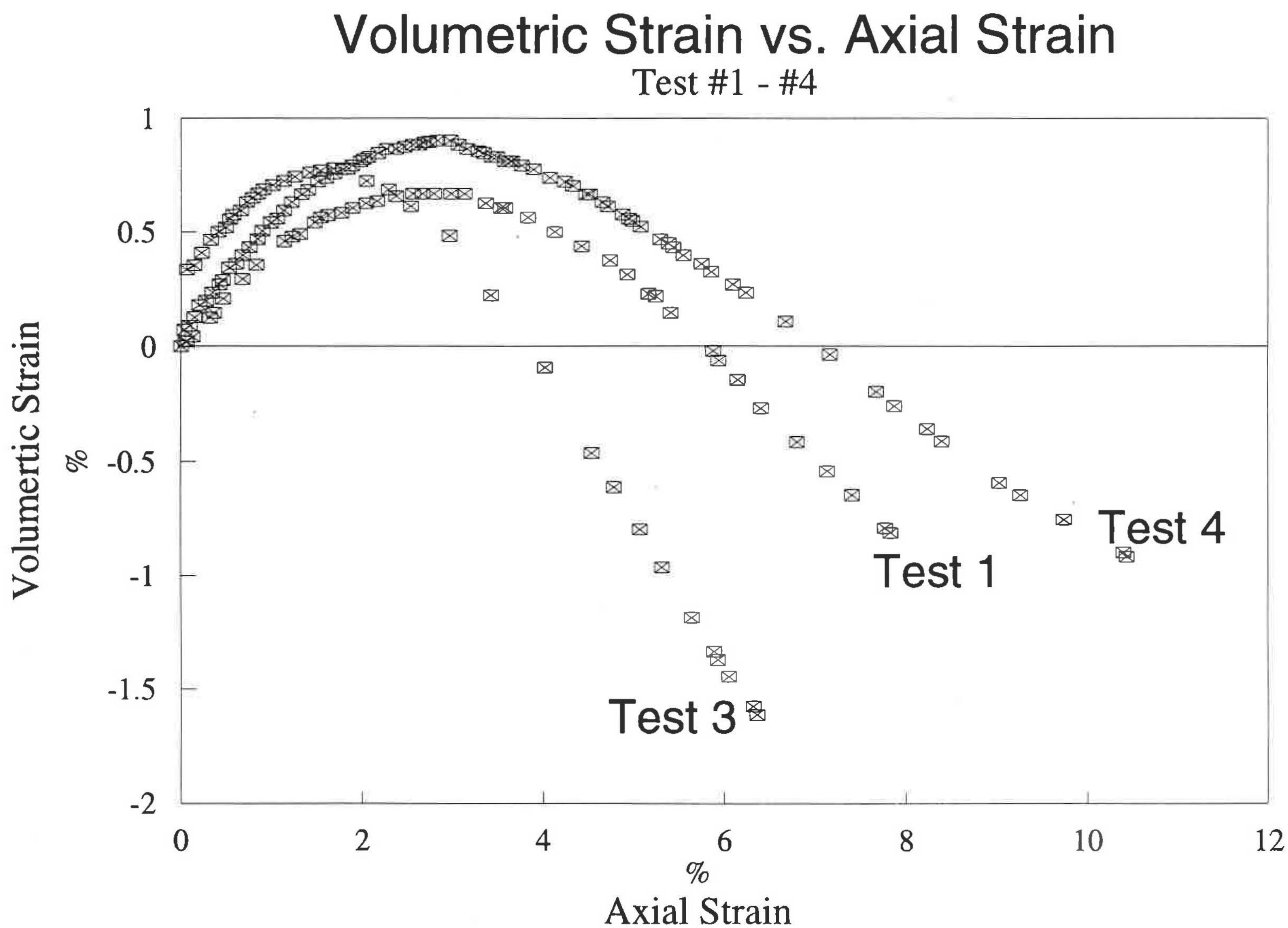

Figure A.5 Volumetric Strain vs. Axial Strain of the 810 Screenings

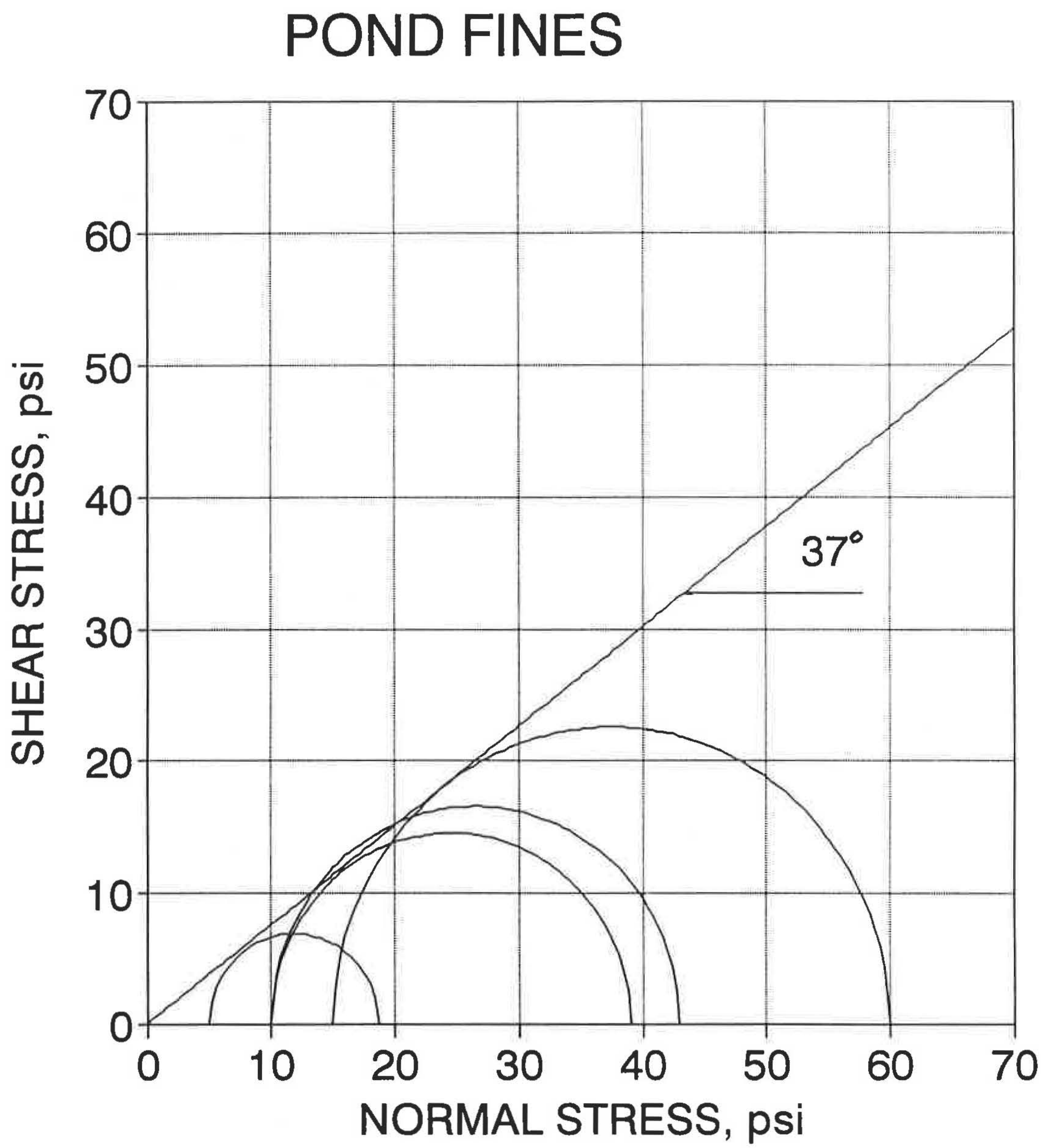

Figure A.6 Mohr-Coulomb Failure Surface of the Pond Fines

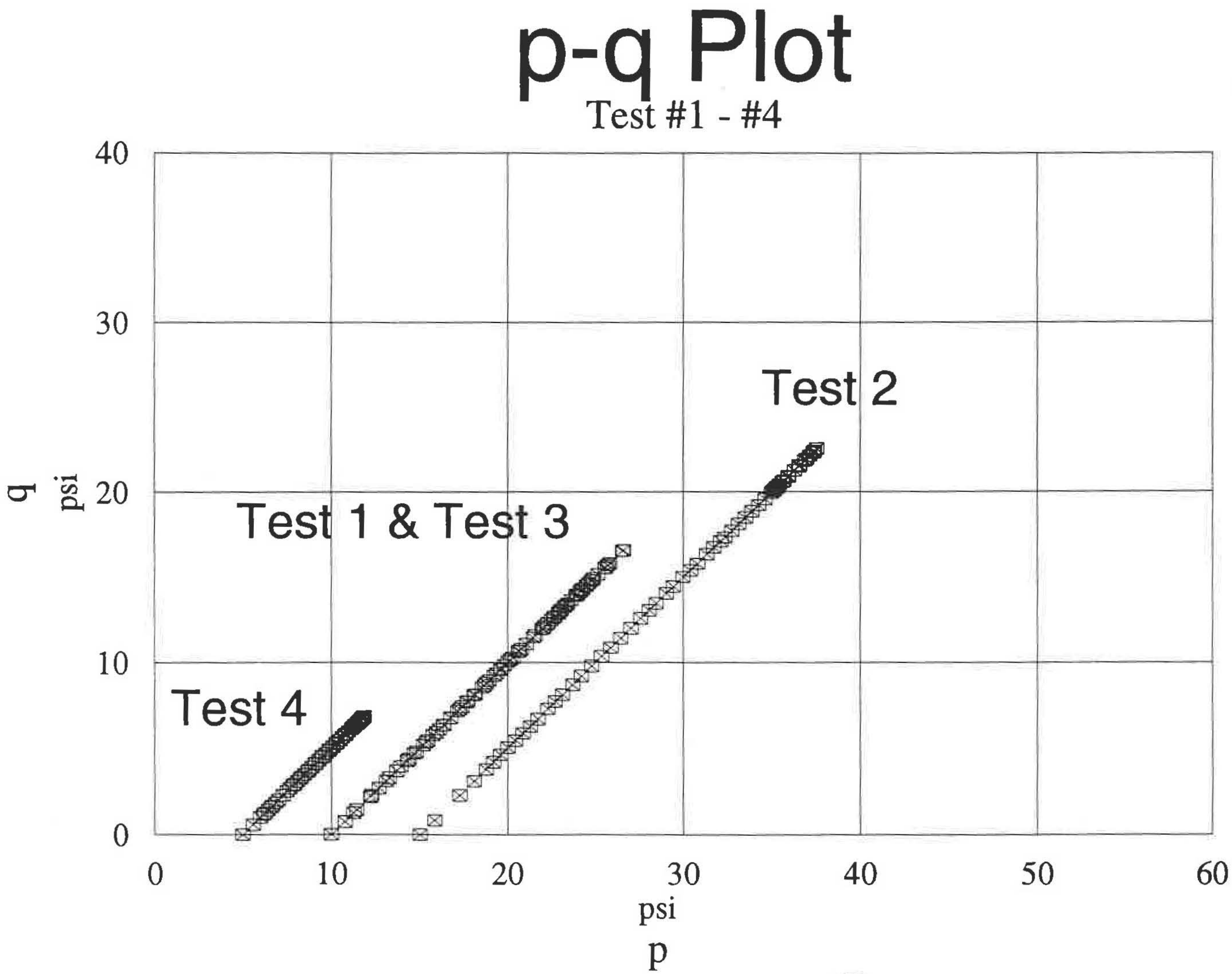

Figure A.7 p-q Plot of Pond Fines

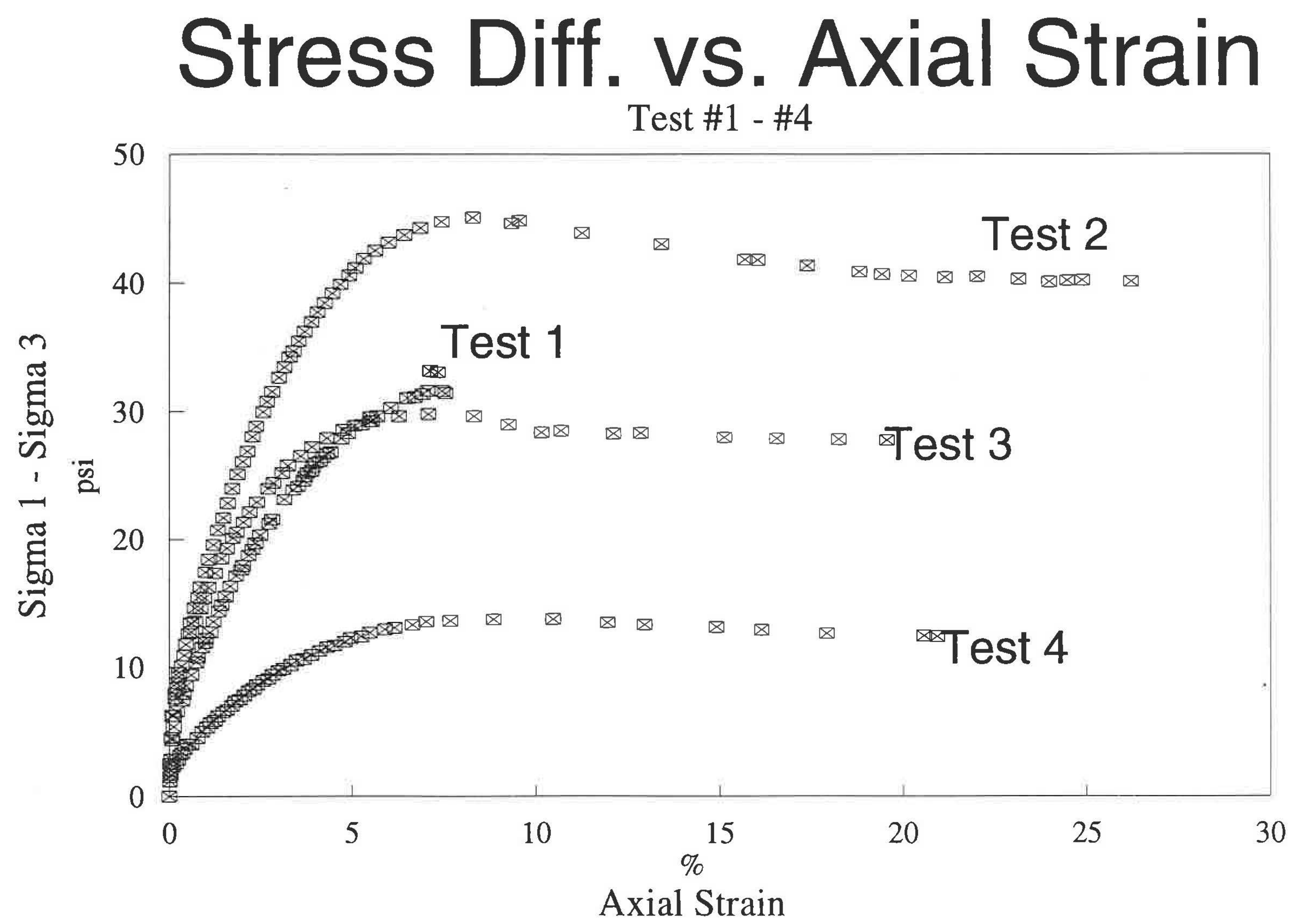

Figure A.8 Stress Difference vs. Axial Strain of the Pond Fines

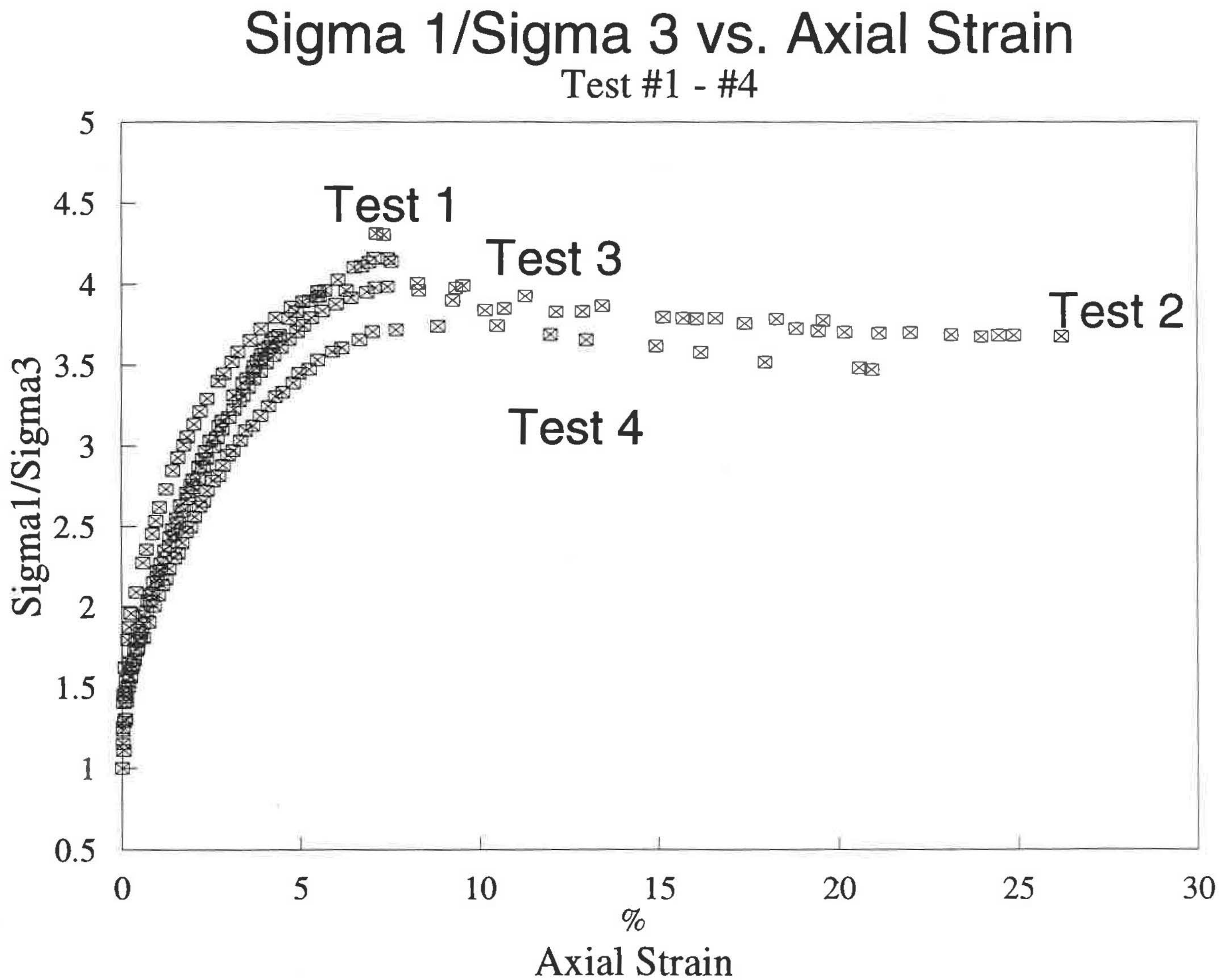

Figure A.9 Stress Ratio vs. Axial Strain of the Pond Fines

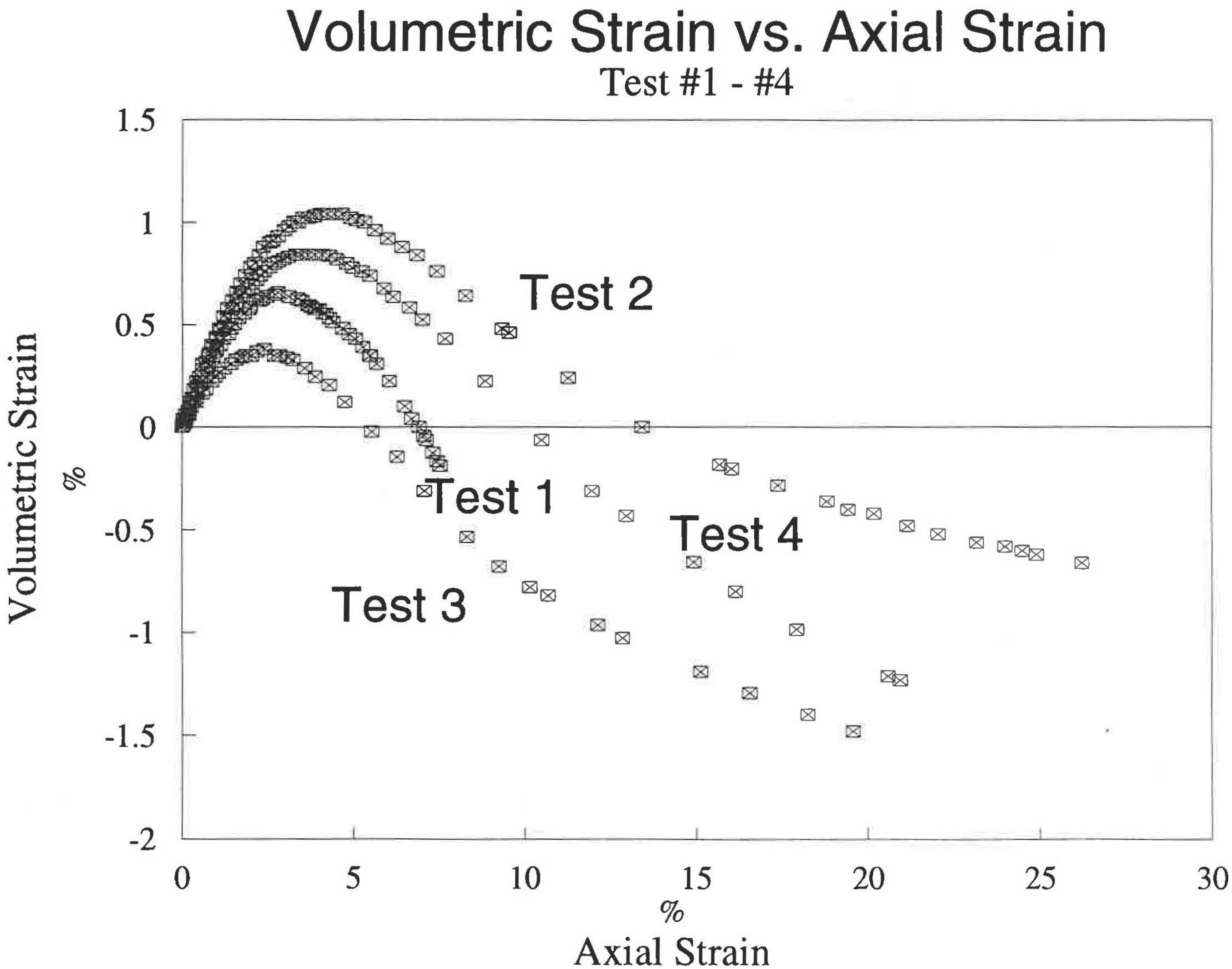

Figure A.10 Volumetric Strain vs. Axial Strain of the Pond Fines

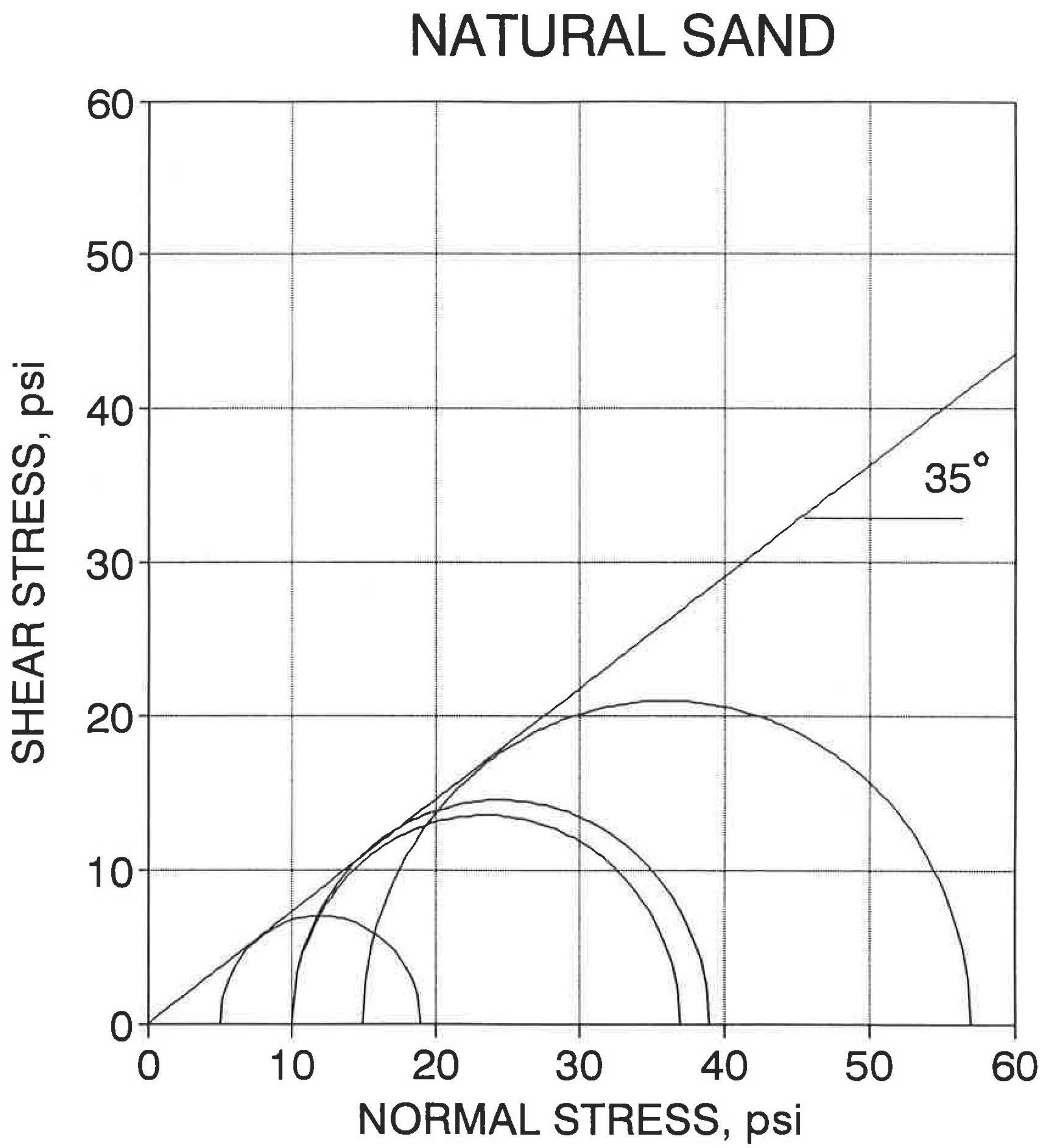

Figure A.11 Mohr-Coulomb Failure Surface of the Natural Sand

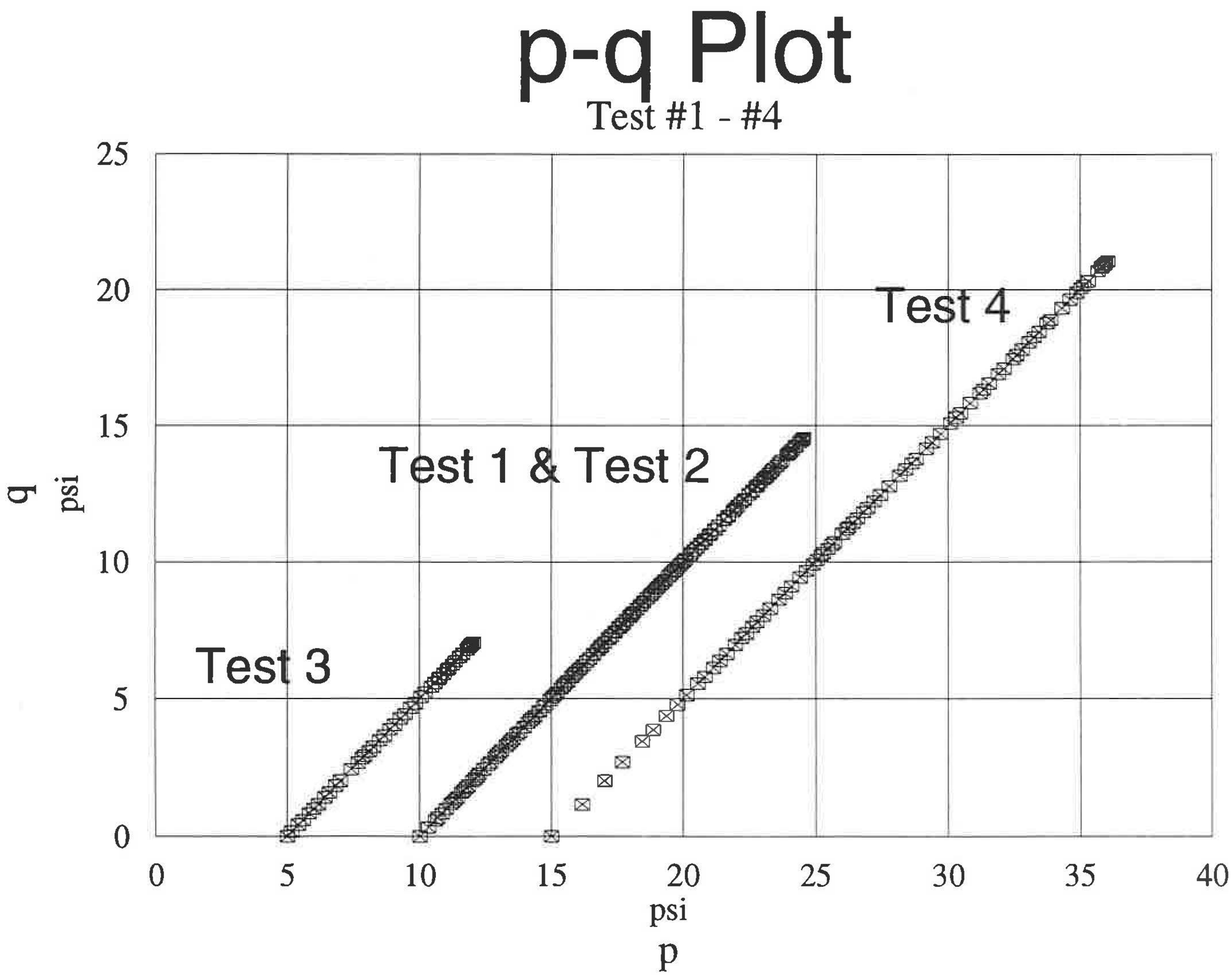

Figure A.12 p-q Plot of Natural Sand

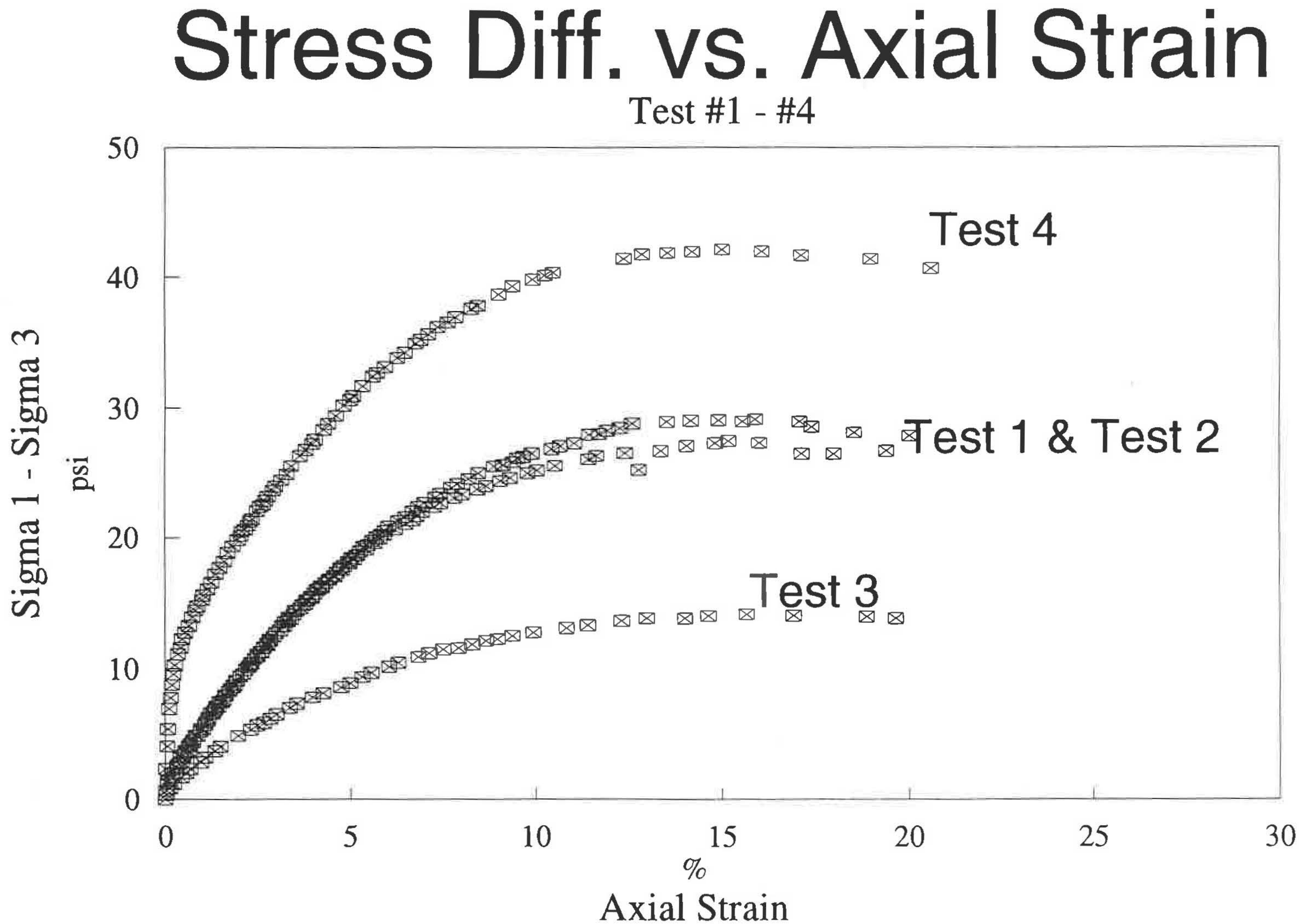

Figure A.13 Stress Difference vs. Axial Strain of the Natural Sand

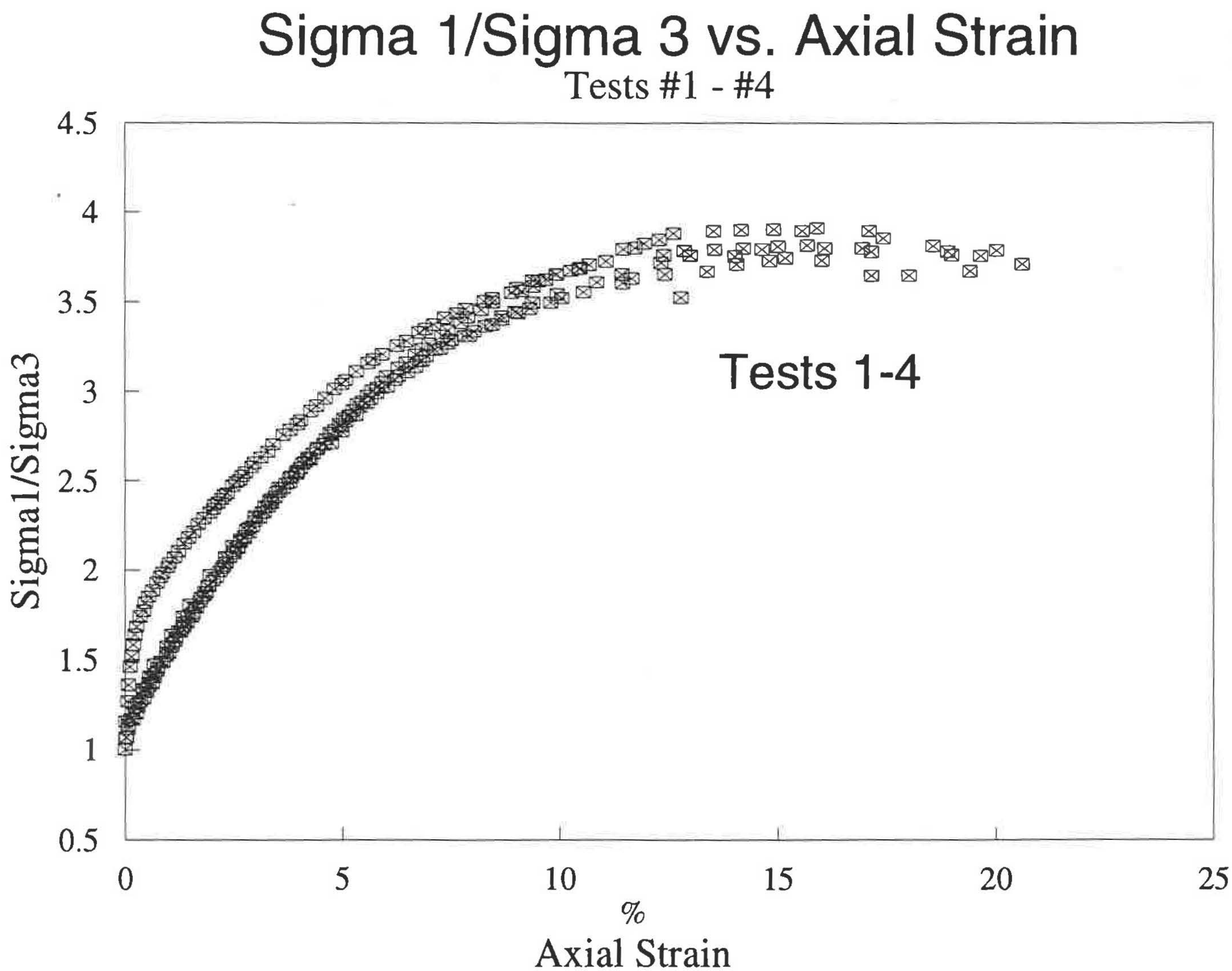

Figure A.14 Stress Ratio vs. Axial Strain of the Natural Sand

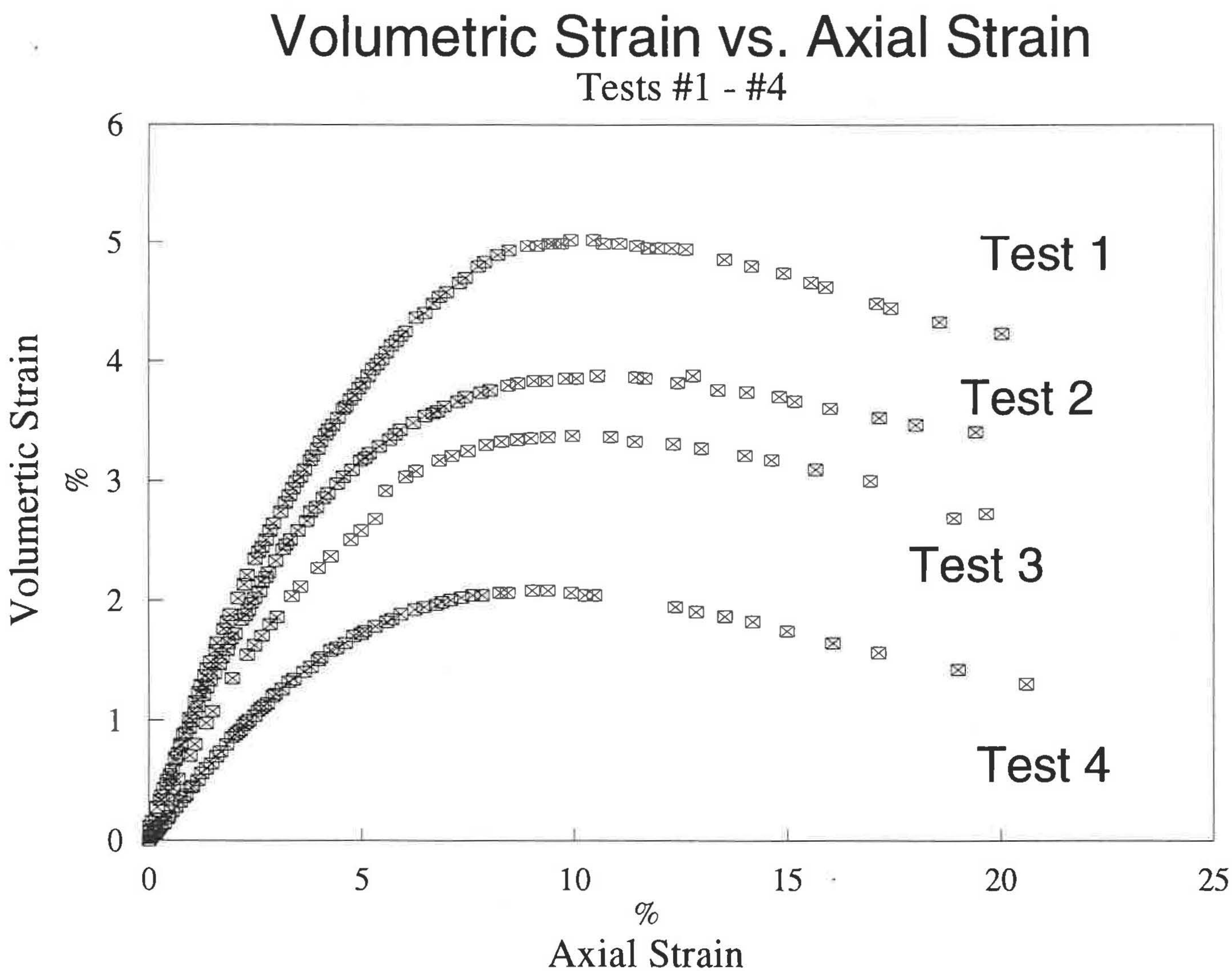

Figure A.15 Volumetric Strain vs. Axial Strain of the Natural Sand

APPENDIX B

RESULTS FROM PULL-OUT TEST

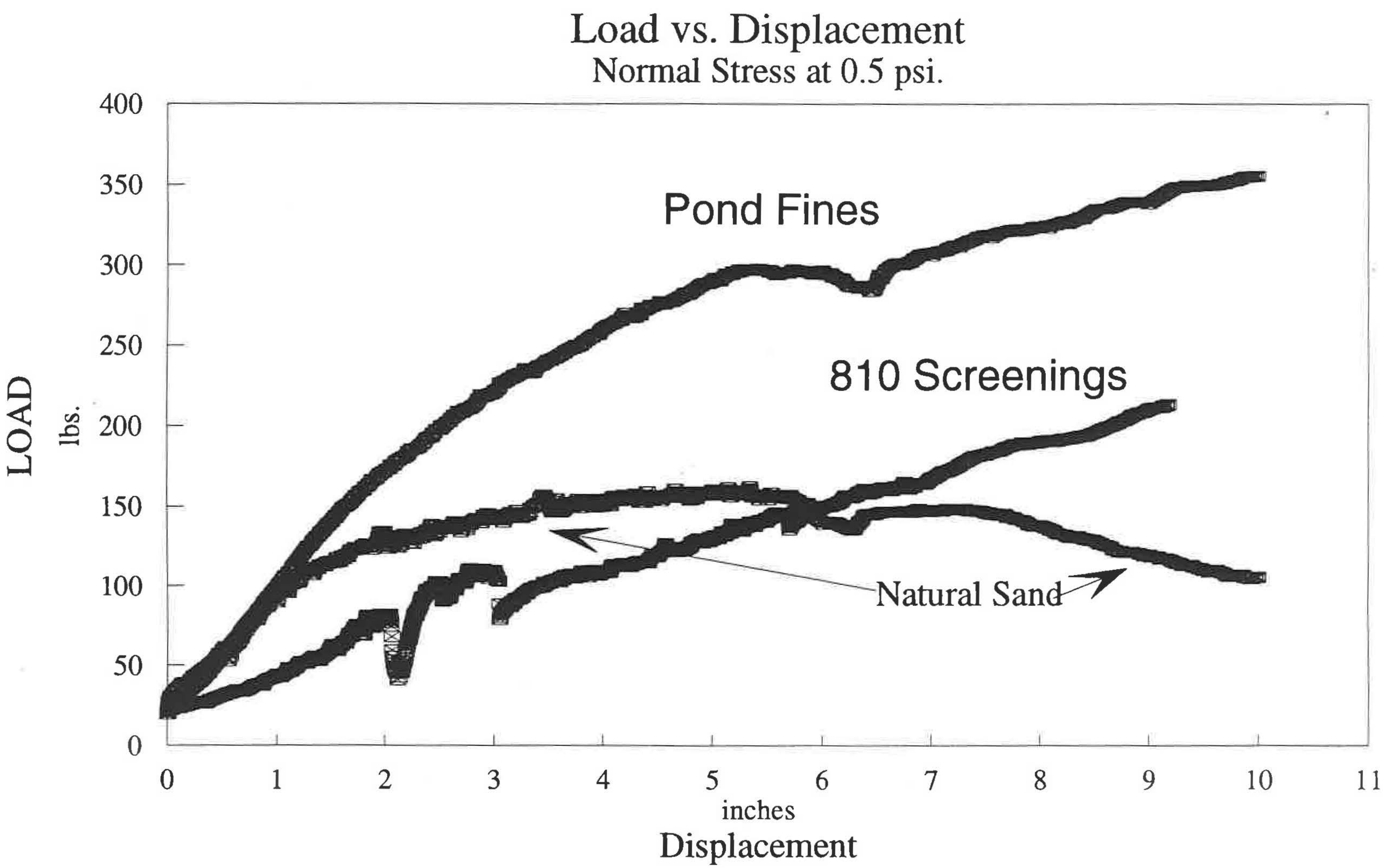

Figure B.1 Pull-out Test at a Normal Stress of 0.5 psi

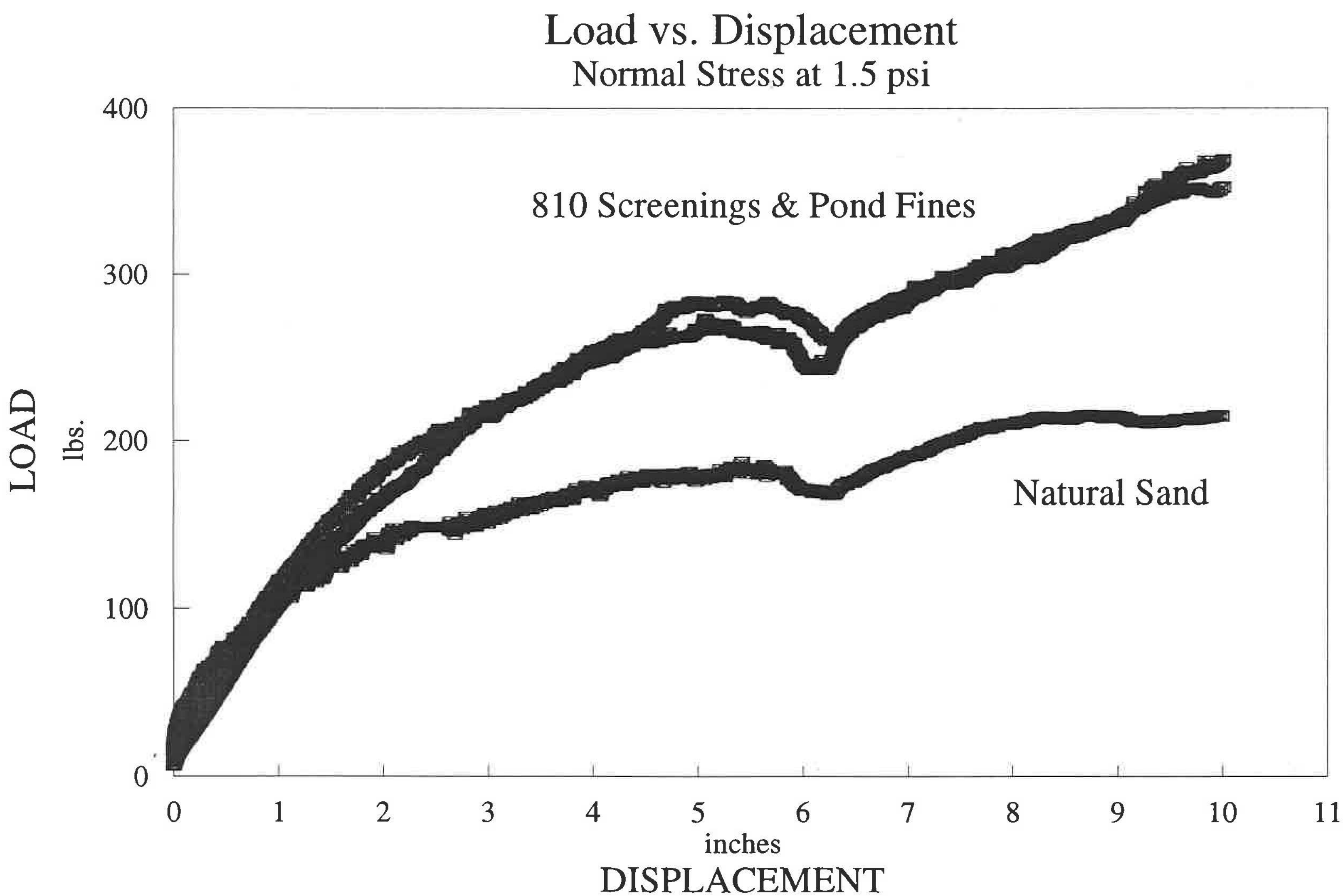

Figure B.2 Pull-out Test at a Normal Stress of 1.5 psi

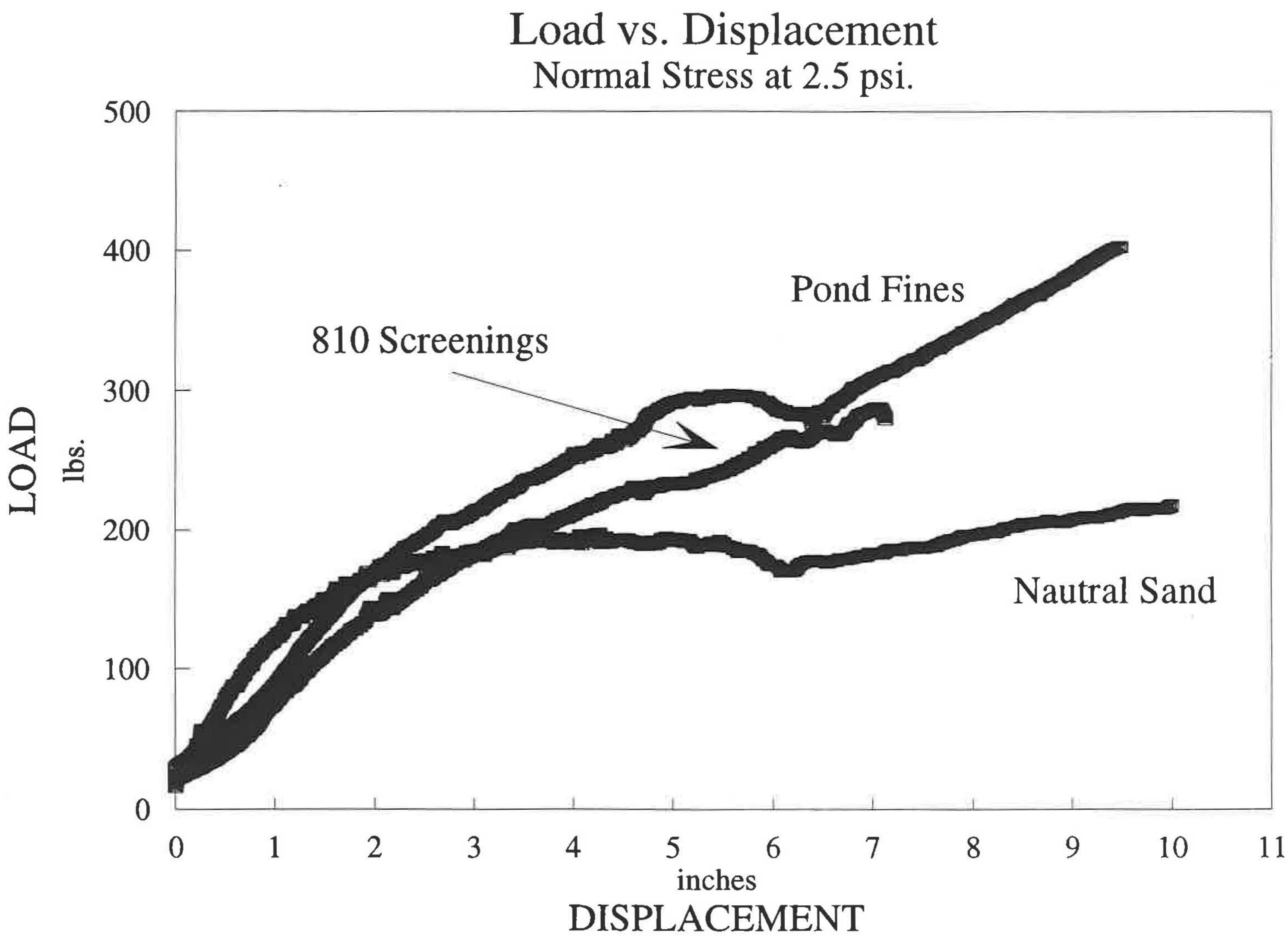

Figure B.3 Pull-out Test at a Normal Stress of 2.5 psi

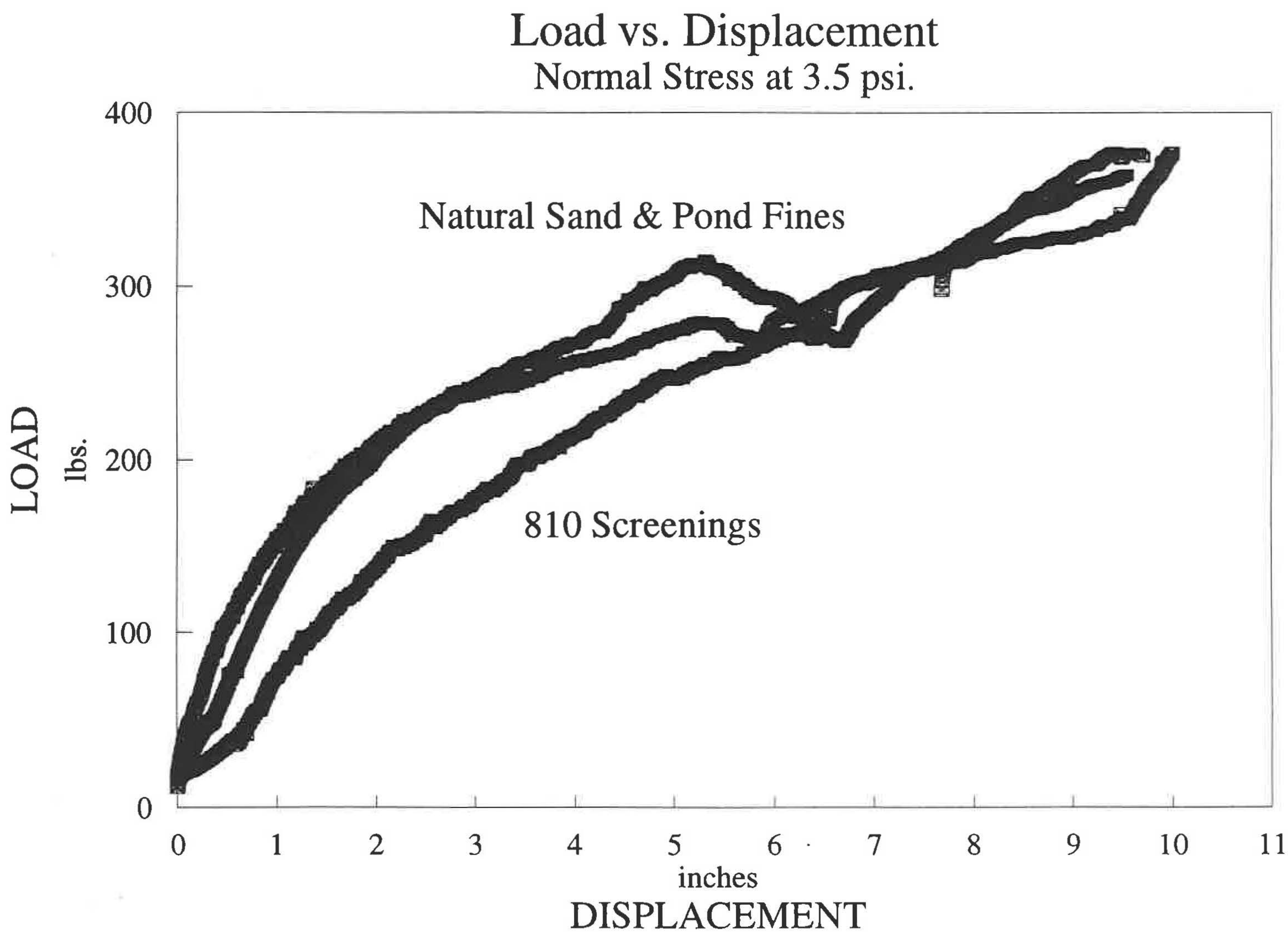

Figure B.4 Pull-out Test at a Normal Stress of 3.5 psi

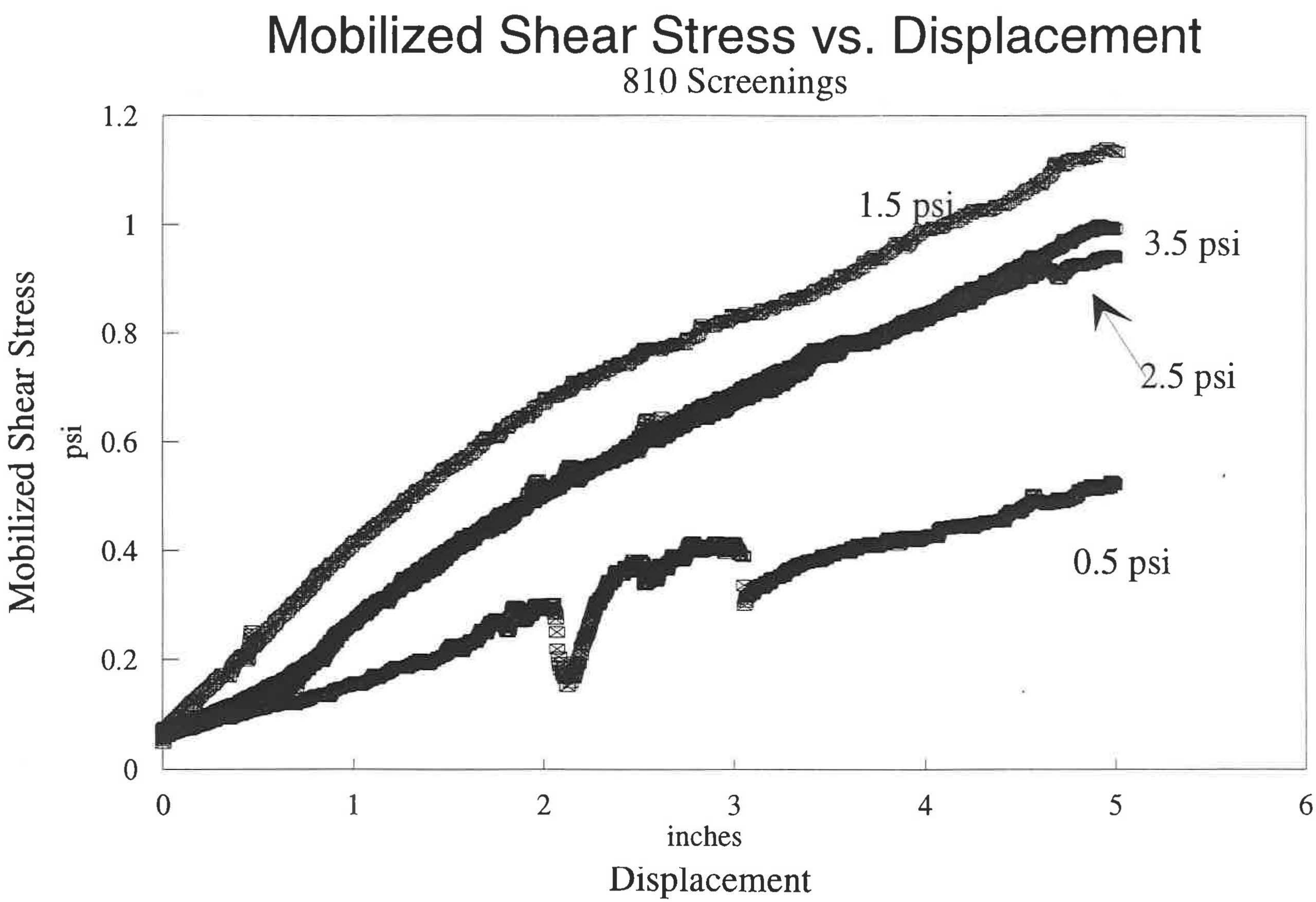

Figure B.5 Mobilized Shear Stress vs. Displacement for the 810 Screenings

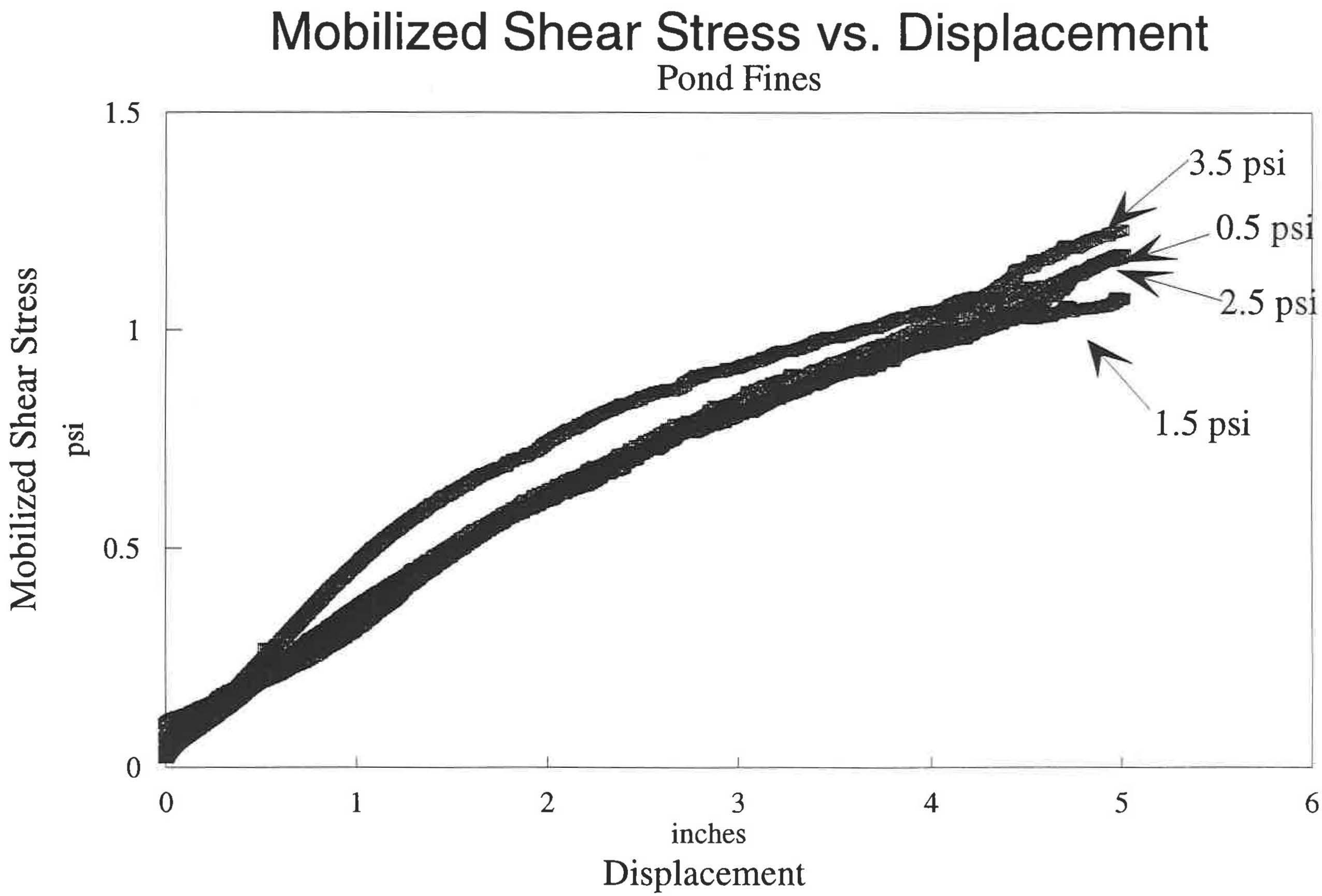

Figure B.6 Mobilized Shear Stress vs. Displacement for the Pond Fines

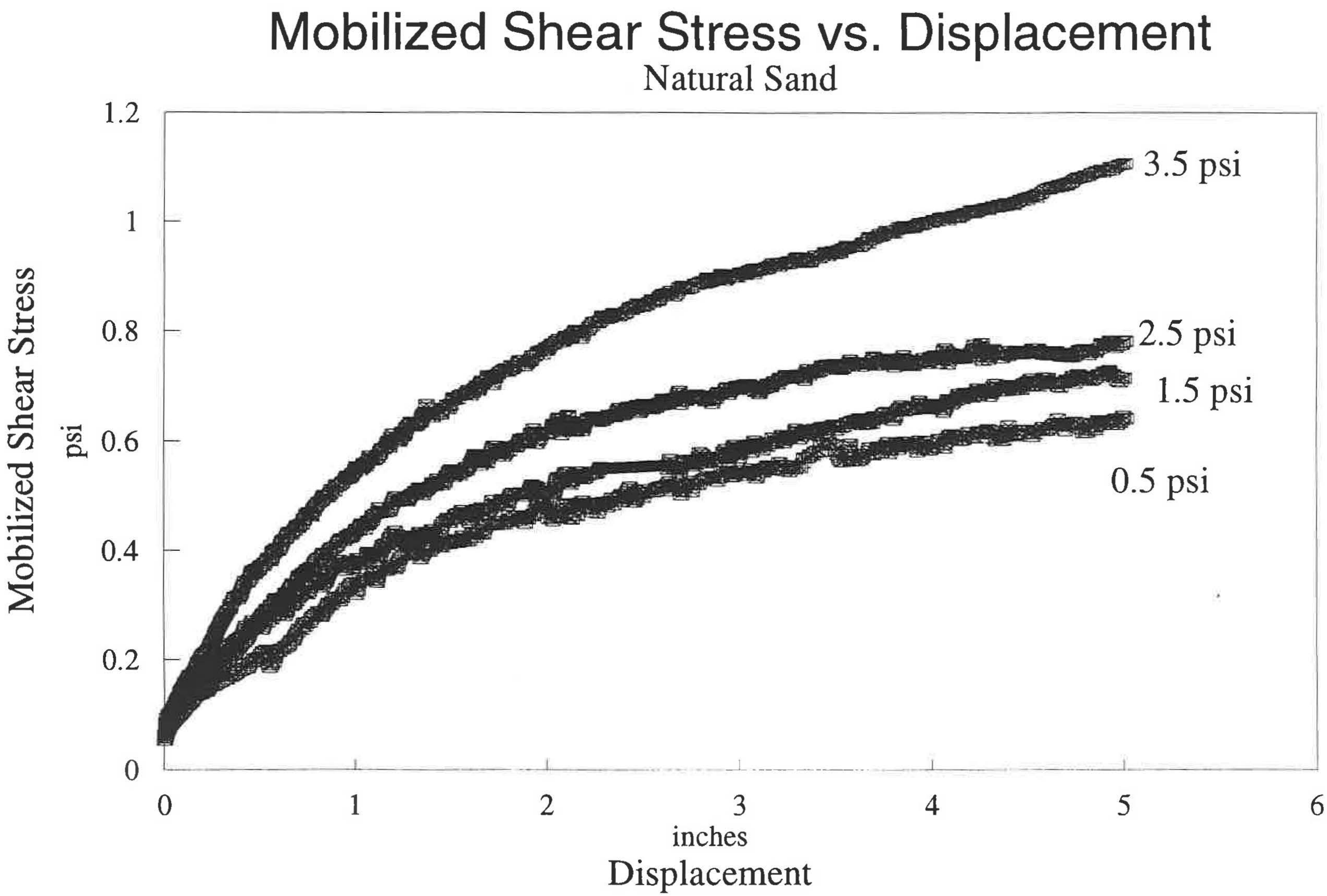

Figure B.7 Mobilized Shear Stress vs. Displacement for the Natural Sand