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Sponsor: The Tensar Corporation

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Title: A Field Study of the Performance of a Tensar Reinforced Haul Road

ADMINISTRATIVE DATA

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FINAL REPORT

A FIELD STUDY OF THE PERFORMANCE OF A TENSAR REINFORCED HAUL ROAD

PREPARED FOR

THE TENSAR CORPORATION
BOX 986
1210 CITIZENS PKWY
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OCTOBER 29, 1985

GEORGIA INSTITUTE OF TECHNOLOGY
A UNIT OF THE UNIVERSITY SYSTEM OF GEORGIA
SCHOOL OF CIVIL ENGINEERING
ATLANTA, GEORGIA 30332

Contract Research
GTRI Research Project E20-674

FINAL REPORT

A FIELD STUDY OF THE PERFORMANCE OF A TENSAR REINFORCED HAUL ROAD

by

Richard D. Barksdale
Professor of Civil Engineering

and

Joseph E. Prendergast
Capt., U.S. Air Force

Prepared for

THE TENSAR CORPORATION

October, 1985

ACKNOWLEDGEMENTS

Sincere appreciation is extended to Mr. Harold Higgerson and Mr. Don Higgerson of Higgerson-Buchanan Contractors for their fine cooperation throughout this study. The extensive assistance given by Higgerson-Buchanan contractors made this research project possible. Appreciation is also extended to Dr. Rudy Bonaparte of the Tensar Corporation for his help throughout the study.

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ABSTRACT

This report presents the results of full-scale tests conducted to analyze the relative benefits of a geogrid compared to selected geotextiles for use as reinforcement in haul roads constructed over soft or very soft subgrades. Properties of the subgrade, fill, geotextiles, and geogrid are evaluated. Methods and equipment used to construct the test sections are discussed. Test data are compiled and analyzed, and conclusions are drawn regarding the performance of the geogrid and geotextiles.

CHAPTER I

INTRODUCTION

1.1 Background

Geogrids and geotextiles are used in a wide variety of Civil Engineering applications. An important use of geogrids is soil reinforcement and stabilization although this unique system has many other applications including drainage and erosion control. New applications for geogrids are still being developed on a regular basis.

Geogrids are manufactured using polymers stretched into a grid pattern having various weights and configurations including aperture shapes and sizes. The high stiffness and strength of geogrids is an important advantage over geotextiles. Further, since geogrids are non-corrosive they have an important advantage over steel bar mesh and strip reinforcement systems, and geogrid systems can be used in most environments.

Some of the applications of geogrids include:

1. Haul Roads - Geogrids are laid on very soft subgrades over which the fill embankment for the haul road is placed.

2. Railways - Geogrids are placed below the ballast of railways to increase stability of the system.

3. Retaining Walls - Geogrids may be attached to the face of the retaining wall and extended horizontally into the soil behind the wall to stabilize the earth mass. The structural face of the wall can, if desired, be omitted by folding the geogrid up and back into the fill.

4. Gabions - Large geogrid boxes are filled with granular materials and tied together. Applications include gravity retaining walls and erosion control.

5. Embankments and Slopes - Geogrids are used for stabilizing new embankments constructed at steep slopes or to correct landslide problems within existing slopes. Geogrids are placed in several horizontal layers within a slope across potential slip planes to reinforce the slope against a stability failure. Finally, an embankment constructed over a soft foundation can be stabilized by placing a geogrid mattress in the bottom of the embankment. The geogrid boxes forming the mattress are filled with granular material and tied together.

Geogrids and geotextiles are both frequently used to reinforce haul roads constructed over soft subgrade soils. Throughout this report, reinforcement refers to the use of either geogrids or geotextiles in haul road construction. Several potential advantages may be derived when soil reinforcement is used in the construction of haul roads. Soil reinforcement allows construction operations to proceed on very soft subgrades and/or in inclement weather. In many cases proper haul road reinforcement can prevent heavy construction equipment from becoming bogged down with resulting loss of productivity.

Reinforcement may also reduce the thickness of fill required to support the expected traffic. The use of less fill can result in monetary savings, especially in areas where high-quality aggregate fill materials are not readily available. Once a haul road with reinforcement is constructed and in service, savings in maintenance costs may also be realized since permanent subgrade deformations are generally less severe compared to those occurring in haul roads without reinforcement.

Unfortunately, very few full-scale field tests have been conducted to demonstrate the actual behavior mechanisms for reinforced haul roads in general and geogrid reinforced roads in specific.

1.2 Scope

The purpose of this research is to report and briefly analyze the results of full-scale test sections constructed over a very soft subgrade. Geogrids and geotextiles were both used as reinforcement in the test sections. The tests were conducted in South Carolina on a dredge spoil area adjacent to the Savannah River, just north of Savannah, Georgia. The in-situ soil at the site is a dredge spoil consisting of a very soft to soft, organic silty clay. The U.S. Army Corps of Engineers has an on-going project at this location to construct holding ponds to contain dredge spoil from the nearby Savannah River.

The specific objectives of the research project are as follows:

1. Determine, for specific loading conditions, the required thickness of a sand fill when reinforced with Tensar SS-1 geogrid.

2. Compare the behavior of Tensar SS-1 geogrid reinforced test sections with similar sections reinforced with geotextiles.

3. Compare the behavior of a crushed stone fill to a sand fill when both are reinforced with a geogrid. This objective was added during the last phase of the study.

4. Evaluate techniques for the field construction of Tensar SS-1 reinforced haul roads placed over very soft subgrades.

5. Evaluate allowable rut depths for various types of hauling equipment.

CHAPTER II

LITERATURE REVIEW

2.1 Mechanisms of Geogrid and Geotextile Reinforcement

The use of geogrid and geotextile reinforcement has been reported by several researchers (Bender and Barenberg, 1978; Ramalho-Ortigao and Palmeira, 1982; Giroud, Ah-Line, and Bonaparte, 1984) to provide the following beneficial effects in applications where a fill is placed over a soft subgrade in the construction of haul roads: (1) a reduction in the required thickness of fill necessary for a given number of equivalent 18 kip axle load repetitions, (2) less severe and less localized rutting and elastic deformation, and (3) separation of the fill and subgrade which prevents the intrusion of fill particles into the subgrade.

Bender and Barenberg (1978) and Giroud, Ah-Line and Bonaparte (1984) have identified the following four principle mechanisms of improvement by a geogrid or geotextile placed between a fill and a soft subgrade: (1) confinement and reinforcement of the fill layer, (2) confinement of the subgrade, (3) separation of the subgrade and fill layer, and (4) prevention of fill layer contamination by fine particles. In addition, Giroud et al (1984) pointed out that an interlocking mechanism exists between a geogrid and the aggregate in the fill layer; such a mechanism would not exist when a geotextile is used.

According to Bender and Barenberg (1978), the reinforcement of the fill layer is generally considered to be due primarily to the high tensile modulus of the reinforcing element.

Sufficient shear stress is considered to develop between the reinforcing element and the fill to allow the reinforcing element to resist tensile stresses in the fill layer and decrease horizontal strains.

Pressure cell measurements made in model tests (Barksdale et al, 1983) indicated a reduction in vertical stress of about 20 percent existed beneath the load at the top of the subgrade for fabric reinforced systems compared to similiar systems without fabrics (Figure 2.1). A nonlinear finite element study (Barksdale et al, 1983) indicated the presence of a geotextile would reduce the stress between about 8 and 14 percent compared with Boussinesq stress distribution theory. Permanent vertical subgrade strains were also found to be reduced. In apparent contast to these findings, Rowe et al (1984) found from field tests that the introduction of a single layer of even a strong geotextile was not sufficient to prevent large shear deformations in the subgrade.

According to Bender and Barenberg (1978) and Giroud et al (1984), for relatively large movements, a reinforcing element confines the subgrade by restraining the upheaval generally associated with a shear failure. This confinement effect, frequently referred to as the tensioned membrane effect, increases the bearing capacity of the soil as illustrated in Figure 2.2.

Separation. The separation of the soil and aggregate layers by a reinforcing element has been reported to increase strength in two ways (Bender and Barenberg, 1978; Giroud, et al, 1984). First, geogrids and geotextiles prevent the infiltration of fines from the subgrade into the fill. Giroud et al (1984) state that geogrids prevent infiltration of fines by limiting lateral displacements of fill

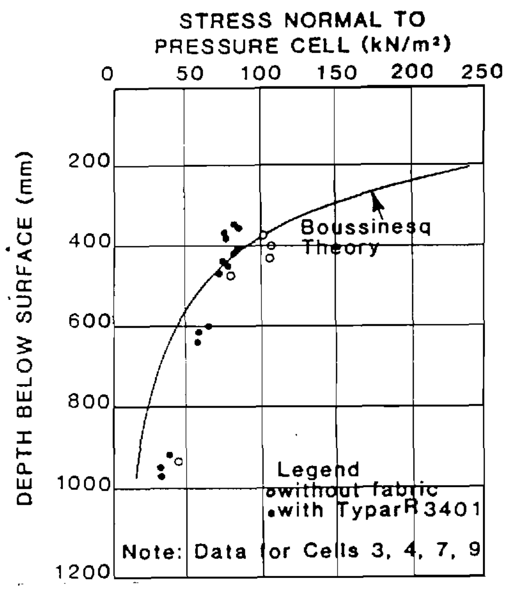


Figure 2.1. Measured vertical stress beneath the centerline of a load for fill/subgrade test sections with and without geotextile reinforcement (after Barksdale, et al., 1983).

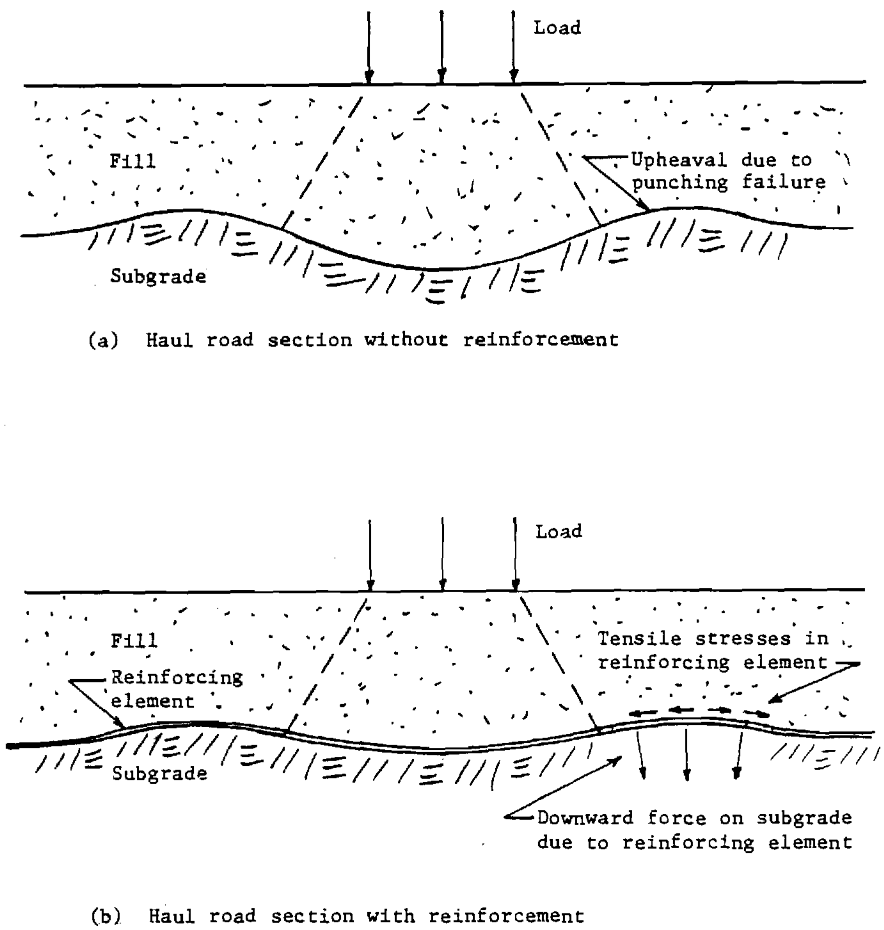


Figure 2.2. Effect of reinforcement on behavior of a subgrade subjected to vertical stress (Bender and Barenberg, 1978).

which would create openings into which fines could migrate. By limiting infiltration of fines, reinforcing elements help to maintain free drainage in the fill, which increases the stability of the fill, especially under saturated conditions. Secondly, reinforcing elements prevent the loss of fill material into the subgrade. This maintains the effective depth of the fill layer, thus preventing an increase of stress on the subgrade due to a reduction in thickness. Giroud et al (1984) report that a geogrid performs well in this respect, especially if the fill is well-graded and the maximum particle size is greater than one-half the size of the geogrid's aperture opening.

The mechanism of interlocking between fill aggregate and the subgrade displayed by geogrids has several advantages (Giroud et al, 1984). The interlocking action limits the magnitude of lateral movement of the fill. This has been reported to help prevent aggregate breakdown, especially in a weak aggregate. Whether reduction in aggregate breakdown actually occurs in the field is questionable. Interlocking also contributes to maintaining the effective thickness of the fill layer, which is important in limiting the magnitude of vertical stress reaching the subgrade due to traffic loading.

2.1.2 Filtering Medium and Drainage. A geogrid and geotextile acts as a filtering medium by allowing pore water to escape from the subgrade into a granular fill while preventing migration of fines into the fill. This allows the subgrade to consolidate under the weight of the fill and gain strength. It also prevents degradation of the fill due to infiltration of fine particles.

Giroud et al (1984) report that geogrids work effectively as a filtering medium by interlocking with the aggregate at the bottom of the fill layer. This prevents lateral movements which would otherwise create openings into which fine particles could migrate.

Raymond (1984) for railroad rehabilitation applications found that certain geotextiles effectively drain water laterally when the fill has a low permeability. Non-woven, needle punched geotextiles performed most effectively in this respect, with lateral permeabilities being measured of at least 25 times the permeability of the subgrade. Woven geotextiles were ineffective in transmitting water laterally, and over a period of time became clogged with slime which prevented vertical drainage. Raymond concluded for railroad applications the best performing geotextiles were heat or resin bonded fabrics which prevented particle penetration with time into the geotextile. Resin bonded fabrics out-performed heat bonded fabrics which would with time become fouled. Particle penetration increased the equivalent opening size of the fabric. For railroad track rehabilitation work, geotextiles having at least 80 needle penetrations per square inch should be used. The Equivalent Opening Size (EOS) should be less than 38 or the No. 400 sieve for 5 percent passing. To ensure low EOS, Raymond concluded that a fiber with a low mass per unit length is best (6 deneir or 0.67 tex or less).

2.2 Geotextile Field Test Results

The haul road reinforcement problem is quite complex because of the presence of heavy loads and soft soils, and the difficulties associated with construction on a soft foundation. Further, the interaction between the reinforcing element and surrounding materials is complicated, especially considering the overlying material is usually granular and not capable of withstanding large tensile stresses. Because of these many uncertainties, the review of past studies will be limited to full-scale field tests which are felt to be of considerably more practical value than small-scale model test results.

2.2.1 Access Road Near Rio de Janeiro, Brazil. Ramalho-Ortigao and Palmeira (1982) conducted tests on an access road in Brazil constructed over a very soft clay (CH) having a mean shear strength of about 210 psf. Four test sections were constructed using a combination of two different geotextiles and four installation methods. Two additional control test sections were constructed without a geotextile. The properties of the fill material, subgrade, and geotextiles used in the study are summarized in Table 2.1.

The geotextile was placed between the uncompacted clayey silt (ML) fill embankment and the subgrade in all reinforced test sections. The fill was placed using a light bulldozer with no additional compaction. The method of geotextile installation was varied to include extending the edges of the geotextile to (1) a point within the embankment shoulders, (2) the outside edge of the embankment shoulders, (3) the outside edge of the embankment shoulders and then folding the geotextile back over and (4) the embankment shoulders and then anchored by driving wooden stakes at three meter intervals. Figure 2.3 illustrates these installation methods.

Some interesting findings were obtained from instrumentation installed in the test sections. After 200 16 kip axle passes,

Table 2.1. Properties of Fill Material, Subgrade, and Geotextile Used in Tests on an Access Road Near Rio de Janeiro.

Fill: 39 in. thick residual clayey silt (ML) of gneissic origin.

LL = 49%

PL = 31%

γ_w = 131 pcf

w = 20 ± 4%

Grain size: 67% passing #200 sieve

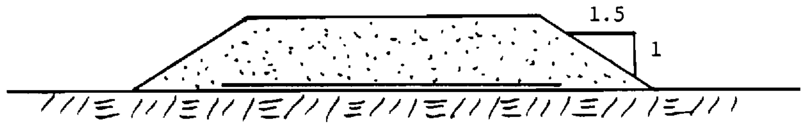
Subgrade: 36 ft. thick layer of very soft clay (CH) overlying sandy layers.

LL: 150% near top of layer; 90% near bottom of layer

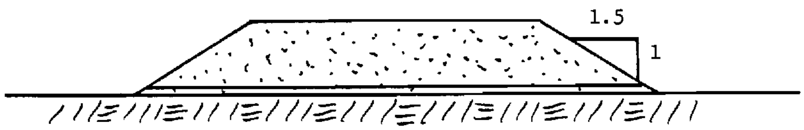
PI = 80

c_u: 100 - 313 psf; mean value = 210 psf

Geotextile: Bidim	OP-30	OP-40
Weight (oz./yd.^{2})	12.6	16.9
Thickness (in.)	0.14	0.15
Monodirection tensile strength (lb./in.)	91	120
Elongation at failure (%)	50-70	50-70
Bidirection tensile strength (lb./in.)	131	177
Elongation at failure (%)(2-D test)	27-30	27-30

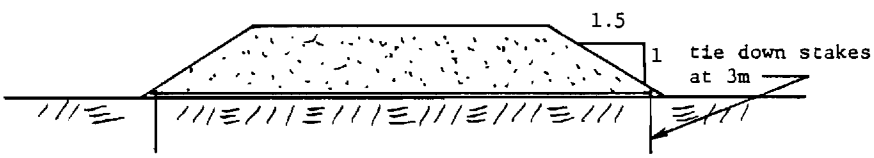


(a) Geotextile not extended to embankment shoulders



(b) Geotextile extended to embankment shoulders

(c) Geotextile extended to embankment shoulders and folded over



(d) Geotextile extended to embankment shoulders and anchored with wooden stakes

Figure 2.3. Methods of geotextile installation for access road near Rio de Janeiro, Brazil (Ramalho-Ortigao and Palmeira, 1982)

horizontal movement of both the geotextile and the fill material was nearly equal, indicating that sufficient friction was developing between the geotextile and the fill to prevent slippage. Maximum rut depths of 5.9 to 7.8 in. and maximum strains in the geotextiles of about 10 percent generally occurred directly beneath the wheel track in the test section without geotextile reinforcement, and in the test section where the geotextile was not completely extended to the embankment shoulders. The other test sections performed well, with rut depths generally measuring one-half those reported in the unreinforced section. The section where the geotextile was anchored with wooden stakes displayed the most uniformly distributed tensile strains in the geotextile.

Ramalho-Ortigao and Palmeira concluded that a primary benefit of using a geotextile is to prevent local yielding in the subgrade. They also found that the best performance was obtained by laterally anchoring the geotextile using wooden stakes. Finally, using a geotextile was found to save 10 to 24 percent by volume on the amount of fill required. A minimum 30 percent reduction in fill quantity was, however, required to be cost-effective at 1981 prices.

2.2.2. Full-Scale Experiment by Ruddock, Potter, and McAvoy. Ruddock, Potter, and McAvoy (1982) conducted a comprehensive full-scale experiment on granular unsurfaced sections and also on bituminous-surfaced pavements to study the effects of placing a geotextileat the granular fill/subgrade interface. In an earlier experiment (Potter and Currer, 1981), including a geotextile in the test sections was found to reduce transient horizontal strains in the subgrade significantly, but transient vertical strains remained the same.

For the experiment a road was constructed on a clay subgrade with CBR values ranging from 0.7 at the surface to 3.5 (about 420 to 2100 psf shear strength) at a depth of 12 in. The clay had a liquid limit of 51 percent, plastic limit of 23 percent, and a natural moisture content of 28 percent. The fill was a crushed granite placed in thicknesses of 11.8 in. and 16.7 in. The 11.8 in. section was placed in a single lift. The 16.7 in. section was constructed by placing over the initial 11.8 in. thick lift an additional 4.9 in. lift. Each lift was compacted using a tandem vibrating roller.

Geotextiles used in the test included three woven, one non-woven, and one impermeable coated geotextile. Test sections were constructed so that each geotextile was placed in an 11.8 in. section and a 16.7 in. section. Control sections were also included for each fill layer thickness.

The sections were loaded using two-axle trucks with dual wheels on the rear axle. Axle loads were increased incrementally up to 22 kips. The line of travel was randomly varied over a lateral distance of 31.5 in. When stresses and strains were measured, however, the wheel was driven directly over the gauges. Since axle loads during the test varied, all axle loads were related to an equivalent 22 kip value. The equivalence factor was the ratio of transient vertical strain measured at a depth of 5.9 in. below a given axle load to the transient vertical strain at the same depth caused by a 22 kip axle.

Data obtained from the test indicated that the presence of a geotextile did not decrease transient vertical stresses and strains compared to the control sections. However, transient horizontal strains decreased significantly. Permanent horizontal and vertical strains were both reduced at the granular fill/subgrade interface and at 2 in. below it. In sections containing a weak geotextile that tore during loading, this reduction did not occur. Similar behavior was measured at a depth of 6 in. below the granular fill/subgrade interface.

After completion of loading, trenches were dug across the section to investigate the condition of the geotextile and relate it to the thickness of the granular fill and to the magnitude of rut depths measured in the surface. It was found that for the 15.7 in. thick granular fills, the geotextile generally tore if rut depths exceeded 3.5 in. For 15.7 to 18.5 in. fill thicknesses, the geotextile tore when rut depths exceeded 4.3 in.

The performance of the geotextiles depended not only on their properties, but also on the thickness of the granular fill and the strength of the subgrade. For instance, in one section a strong geotextile failed due to a thin granular fill layer and a weak subgrade. In sections that did not fail, those containing a geotextile displayed less permanent deformation than those without a geotextile.

When loading was completed, vane shear tests were conducted on the surface of the subgrade under the impermeable coated geotextile. The shear strength of the subgrade in these sections decreased sharply during the test. In contrast, vane shear measurements taken under the permeable geotextiles showed that the shear strength had doubled during the test. The variation in observed behavior can be attributed to the difference in drainage characteristics between the two geotextiles. The permeable geotextile allowed pore pressure to escape, but the impermeable geotextile did not, which resulted in a decrease in strength.

The geotextiles effectively maintained separation between the granular fill and the subgrade. In the control section, the granular fill and subgrade were intermixed to a depth of about 2 in.

The trench studies indicated that the geotextile was stretched taut under the wheel paths, but was wrinkled between and outside of the paths. The wrinkles were oriented parallel to the direction of travel. Deflection of the granular fill/subgrade interface was sufficient to cause strains in the geotextile of only 1 to 2 percent, and permanent strains measured were less than 2 percent. However, strains of 10 percent were necessary to cause the geotextile to tear.

Ruddock, et al indicate the tears observed in the geotextile after testing may be caused by movements between individual particles within the granular fill that may be larger than the average strain in the fill. These individual particles may therefore grip the geotextile and cause enough movement for strain to exceed 10 percent and hence cause rupture. Once the geotextile tears, it can no longer limit horizontal strains in the granular fill, and so it continues to tear.

They also reported that portions of the granular fill bounded by cracks appeared to be acting as rigid blocks, rocking on the subgrade. It was suggested that the rocking action may have pulled the fabric out from under the wheel paths causing the observed longitudinal wrinkles.

Ruddock et al concluded that geotextiles did reduce transient horizontal stresses and strains and permanent vertical and horizontal strains. Geotextiles did not reduce transient vertical stresses and strains. The geotextiles were effective in separating the granular fill layer and the subgrade. Provided the geotextile was not torn, the rate of surface deformation was smaller in fabric reinforced test sections compared to the control sections.

2.2.3. Waterways Experiment Station Field Tests. Webster and Watkins (1977) performed a field test at the U.S. Army Waterways Experiment Station in which two different geotextiles were placed between 14 in. of crushed limestone fill and a heavy clay subgrade with a CBR of 0.7 to 1.0, which corresponds to a shear strength of about 400 to 600 psf. One section was constructed with Bidim C-38 geotextile and another section used T-16 geotextile. A control section without a geotextile was also included. All sections were tested using dual wheel, tandem axle trucks, with total loads of 25,000, 35,000, 47,000, and 55,000 lbs. The unreinforced control section accommodated 70 equivalent 18 kip axle loads before developing a three inch rut. On the other hand, the Bidim C-38 and T-16 reinforced sections required 250 and 10,000 repetitions, respectively, before reaching the same level of rutting. These results illustrate the potential beneficial effects of using a reinforced fill over a soft subgrade when subjected to heavy loads.

2.2.4. Forest Service Field Tests. Jones (1982) described two field tests conducted by the U.S. Forest Service. In the first test, a crushed stone fill 2 to 3 ft. thick was placed over a fine-grained, organic subgrade with the water table near the surface. The CBR was in places as low as 0.1, which corresponds to a shear strength of about 60 psf. Various non-woven geotextiles with weights ranging from 4.0 to 12.4 ounces per square yard were used as reinforcement. Strain gauges were attached to the geotextiles. The exact depth of fill was selected to give an estimated vertical stress on the subgrade of 3.3 times the cohesion of the soil i.e. $\sigma_z = 3.3c$. After two years and

1700 equivalent 18 kip single axle passes, maximum rut depth was only two inches and strain in the fabric was less than ten percent. Furthermore, the Forest Service found that the subgrade had gained strength due to consolidation caused by the weight of the fill. This strength gain was sufficient to allow the use of 50 percent less fill and still keep σ_z on the subgrade equal to 3.3c.

In the second test reported by Jones, 3 to 6 in. of crushed stone and a layer of Bidim C-22 geotextile was placed over a subgrade with CBR values ranging from less than 1.0 to greater than 3.0, with corresponding shear strengths of 580 to 1700 psf. The water table was near the surface. After 80 passes of logging trucks with tandem axle weights of approximately 45 kips (equivalent to roughly 290 passes of an 18 kip single axle) the rut depths ranged from 2 to 8 in. Since logging operations were nearly complete, the haul road represented an economical, adequately performing design. Jones calculated σ_z/c values and found they ranged from 12.5 to 20, depending on the stress distribution charts used. Since σ_z/c values of 3 to 6 are usually used for design by the Forest Service (Steward and Mohney, 1982), the road performed better than expected. Using other design methods, Jones calculated required fill thicknesses ranging from 7 to 13.5 in. compared to the 3 to 6 in. actually used. The unexpectedly good performance of the haul road was attributed to using the lowest values of soil strength determined in the field. An important increase in shear strength with time may have also contributed to the apparent good performance of this haul road (and decrease the actual σ_z/c values). Also, the thickness of the section may have actually been greater than reported.

2.3 Geogrid Field Test Results - Falkland Islands Experience

After Great Britain regained the Falkland Islands, the need developed to rapidly extend the existing runway over a soft subgrade consisting mainly of peat and sand. The water table was very high with standing water visible in places. CBR values were in the range of 1 to 3, which corresponds to shear strengths of approximately 560 to 1700 psf. A base layer with a CBR of 7 to 14 was required, but local materials were not readily available of the necessary quality.

Preliminary field tests were first conducted in Great Britain to determine if the required CBR could be obtained by cement stabilization and/or through the use of geogrid reinforcement in combination with a permeable membrane to prevent the migration of fines into the base.

Four test sections were constructed in Great Britain over a peat and sand subgrade with a CBR of 1.5 (shear strength of about 860 psf). In all four sections, a layer of Terram 1000 permeable membrane was first laid over the subgrade. A layer of Netlon CE 121 geogrid was then placed on the Terram 1000 in all but the fourth section. A 17.7 in. thick sand fill was then constructed. A geogrid layer was placed at mid-depth within the fill in the second section. In the third and fourth sections, the upper 5.9 in. of fill was stabilized with cement. An impermeable membrane was placed on top of the fill, followed by steel mats used for the runway extension. These sections were then loaded using heavy aircraft and the resulting settlements measured. Both sections without the cement stabilized surface layer failed, although the section with geogrid at mid-depth was significantly stronger. The cement-stabilized sections showed no distress, and there was no notable difference in performance between the section with geogrid on the subgrade and the section without. The geogrid was found to provide a stiffening effect in the unstabilized fill.

Because of the poor site and environmental conditions found to exist at the Falklands airfield site, the decision was made to use the geogrid and permeable membrane design. The final design of the runway extension is illustrated in Figure 2.4. Since cement stabilization was considered impractical in the cold climate, a 5.9 in. thick layer of crushed stone was constructed over the sand fill. Tensar SS-1 geogrid was placed between the crushed stone and the sand fill to act as reinforcement and prevent the stone from sinking into the sand.

During construction, the Tensar geogrid went down easily, although it had a tendency to open at the joints. This problem was solved by using plastic ties at 24 in. intervals to close the joints. To avoid damaging the geogrid, it was necessary to have at least 7.9 in. of loose material over the geogrid during grading operations.

The geogrids proved valuable in providing a stable working platform for placement of sand fill and crushed stone, and performed very well as a separation layer to prevent the loss of valuable crushed stone.

Geogrids were also used in the construction of roads throughout the islands. Tensar SS-1 was chosen over Netlon CE 121 because it was lighter, cheaper, and performed equally as well.

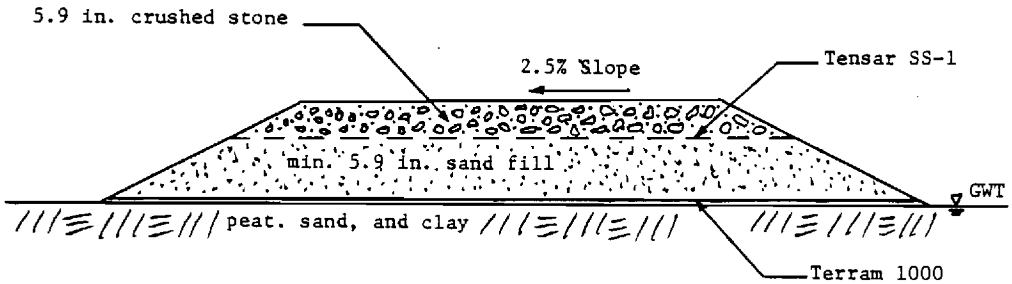


Figure 2.4. Final design of Falkland Islands runway extension - typical section.

2.4 Selected Design Methods

Several methods have been proposed for the design of haul roads with and without geosynthetic reinforcement. The methods vary widely in degree of complexity from the simple theory of Bender and Barenberg (1978) to the more complex method of Giroud et al (1984). This section briefly reviews selected theories of unsurfaced haul road design.

2.4.1 Simplified Bearing Capacity Theory.

Bender and Barenberg (1978) developed a simple approach for haul road design which is easy to apply. Their method is a variation of classical bearing capacity theory for computing required structural fill thicknesses above the subgrade based on the cohesive strength of the soil:

$$q = cN_c^* \qquad (1)$$

where q is the maximum allowable vertical stress on the top of the subgrade, c is the undrained cohesion of the subgrade, and N_c^* is a bearing capacity factor. Steward and Mohney (1982) give the following values of N_c^*: 2.8 for haul roads without a geotextile, and 5.0 for haul roads reinforced with a geotextile and subjected to more than 1000 18 kip axle loads when less than 3 in. of rutting are desired. When less than 1000 18 kip axle loads and more than 3 inches of rutting are expected, N_c^* can be increased to 3.3 and 6.0, respectively following the recommendations of Steward and Mohney.

Vertical stress under the centerline of the loaded area is determined using Boussinesq stress distribution theory.

The required thickness of cover is selected so that the calculated change in vertical stress on top of the subgrade due to the applied wheel loading equals cN_c^*. This is a very practical and easy to apply method for design since the only measured variable needed is the cohesive strength of the soil, which can easily be determined using a field vane shear device or estimated from cone penetrometer results. A better definition is needed of the appropriate values of N_c^* required for design.

2.4.2 <u>Complex Bearing Capacity Theory</u>. More recently, Giroud, Ah-Line, and Bonaparte (1984) have developed a design method based on the bearing capacity of the subgrade for both unreinforced and geogrid reinforced haul roads. For an unreinforced haul road the solution is developed from the basic premise that

$$p_e = \pi c + \gamma h \tag{2}$$

where p_e is the maximum allowable pressure on the top of the subgrade at the elastic limit of the subgrade, c is the undrained shear strength of the subgrade, γ is the unit weight of the fill, and h is the height of the fill. For a reinforced haul road, the basic equation is

$$p_{lim} = (\pi + 2)c + \gamma h \tag{3}$$

where p_{lim} is the pressure on the top of the subgrade at the plastic limit of the soil. From these basic relations, Giroud et al developed comprehensive but complex solutions for each of the above cases that gives directly the required thickness of fill.

These equations were then nicely summarized in the form of a design chart, which is shown in Figure 2.5. To use the chart, enter the right half of the chart with the undrained shear strength, c_u of the subgrade,

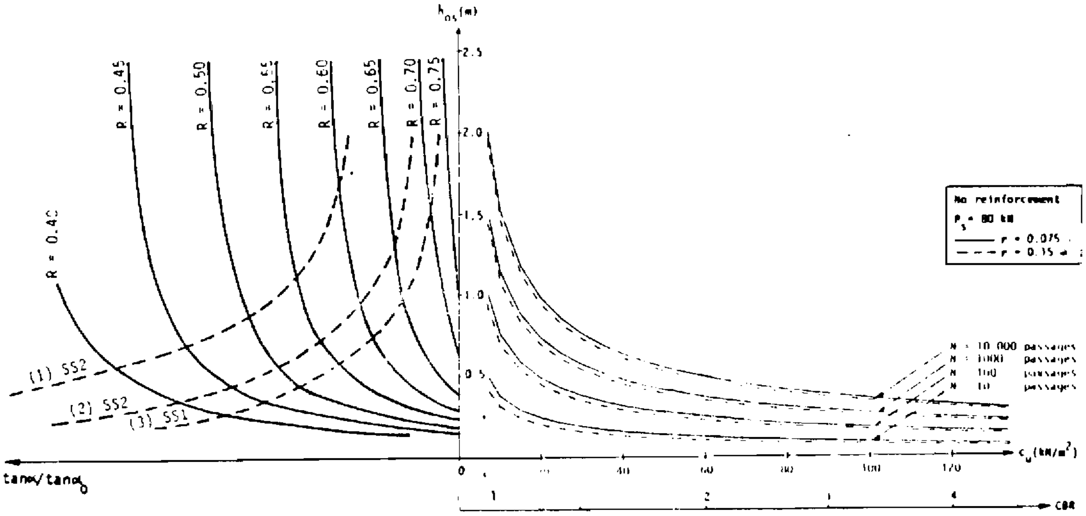


Figure 2.5. Design chart for Tensar geogrid-reinforced haul roads developed by Giroud et al (1984).

number of passages N, and allowable rut depth r, and then determine the required thickness of unreinforced fill layer h_{os}. Using the left half of the chart, find the intersection of h_{os} and the dashed curve for the type of Tensar geogrid to be used, and by interpolation find a value for the thickness ratio R. The thickness of the Tensar reinforced fill layer h is determined as follows:

$h = Rh_{os}$ for rut depth less than 6 in. and/or where traffic is not channelized;

$h = 0.9Rh_{os}$ for rut depth greater than 6 in. and where traffic is channelized (the 0.9 factor accounts for tensioned membrane effect).

2.4.3 Corps of Engineers Method. Hammitt (1970) developed a thickness design equation for unsurfaced roads by modifying the conventional Corps of Engineers method for flexible pavement thickness design. This modification was based on the results of full-scale field tests performed at the Waterways Experiment Station. Fills were used in the field study having CBR strengths varying from 9 to 17 (shear strengths from 5400 to 10,000 psf) and thicknesses varying from 6 to 24 in. Substrate thickness varied from 24 to 39 in. with CBR strengths of 2 to 4 (shear strengths of approximately 1200 to 2400 psf).

Since geotextiles were not included, the design equation developed is valid only for unreinforced roads, but does serve as a valuable guide for the upper bounds on the thickness that would be required for reinforced-type construction. Also of practical significance is the fact that the Corps of Engineers method is the only one developed for haul roads using full-scale field test results. Single wheel axle loads of varying magnitude were applied to the sections until failure occurred. Failure was defined as (1) rut depths greater than 3 in., (2) elastic deflections greater than 1.5 in., or (3) overall subsidence greater than 4 in.

The Corps of Engineers general flexible pavement design equation is

$$t = f\sqrt{\frac{P}{8.1\ (CBR)} - \frac{A}{\pi}} \tag{4}$$

where t = required structural section thickness (in.)

P = equivalent single wheel load (lb.)

A = tire contact area ($in.^2$)

f = a correlation factor discussed below

To develop a design relation for unsurfaced, unreinforced roads, data from each field test were inserted into equation (4) to give a revised value for f which was denoted as f'. The factors f' and c for each test were plotted on a semi-log graph and the following curve was fitted to the data using the method of least squares:

$$f' = 0.176 \log c + 0.120 \tag{5}$$

where c = number of vehicle coverages

The thickness required when equation (5) is used in equation (4) is only about 75 percent of that given by the flexible pavement design equation. This difference is the result of the failure criteria for unsurfaced roads being 3 in. of permanent deformation or 1.5 in. of elastic deformation. On the other hand, for flexible pavements, failure is defined as any very small movement of the subgrade which would cause cracking of the pavement. The Corps of Engineers equation could of course be modified for unsurfaced roads with reinforcement by following a similar procedure to that of Hammitt.

2.5 Summary

Relatively few full scale field tests have been conducted to verify the specific mechanisms which account for the observed improvement in performance of reinforced haul type roads. Ramalho-Ortigao and Palmeira (1982) found for a geotextile reinforced haul road on a very soft subgrade that approximately 10 to 24 percent less fill was required when reinforcement was used. Webster and Watkins (1977) found for a firm clay subgrade that reinforcement with Bidim C-38 increased the required repetitions to failure from 70 to 250 equivalent 18 kip axle loads, and with T-16 geotextile to 10,000 repetitions.

Ramalho-Ortigao et al concluded that a geotextile prevents local yielding of the subgrade. Also, their results indicate sliding probably does not occur at the geotextile/fill interface. In the Ramalho-Ortigao and Palmeira study, maximum strains in the geotextile of about 10 percent occurred directly beneath the load at rut depths of about 6 to 8 in. The Forest Service (Jones,1982) reported non-woven geotextile strains also of 10 percent, but they occurred at rut depths of only 3 in. In contrast, Ruddock et al (1982) observed fabric strains of only 1 to 2 percent and longitudinal wrinkling of the geotextile, indicating the geotextile was not in tension between the wheel paths. Ramalho-Ortigao et al (1982) found that anchoring a geotextile with wooden stakes more uniformly distributed tensile strains.

The subgrade has been found to increase in strength by as much as 100 percent with time due to consolidation under the weight of the added fill load (Ruddock et al, 1982; Steward and Mohney, 1982; Jones,1982). As a result, insuring good drainage of water from the subgrade is important (Ruddock et al, 1982 and Raymond, 1984). Ruddock et al (1982) found when an impermeable geotextile was used, the subgrade strength decreased to one-half its original value, which is in agreement with the more recent findings of Raymond (1984). In contrast, the strength doubled when a permeable geotextile was used. Raymond (1984) found that lateral drainage in the plane of a needle punched geotextile can result in improved performance where a relatively impermeable fill overlies the geotextile.

To avoid damage during grading, approximately 8 in. of cover was found to required over the geogrid (Kennedy, 1984). To prevent damage during tamping, an 8 in. minimum cover was also found by Raymond (1984) to be required when geotextiles were used beneath ballast.

The theoretical and model studies reported by Barksdale et al (1983) suggests that the elastic vertical stress and strain in reinforced haul roads may be slightly less than those without reinforcement, and slightly less than Boussinesq stress distribution theory. However, the inclusion of a geotextile apparently has relatively little effect on vertical stress or resilient strain distribution as indicated by full scale field tests (Ruddock et al, 1982). These findings give at least some support to the simplified design approach which uses Boussinesq stress distribution theory and estimates failure as a constant times the undrained shear strength of the subgrade.

CHAPTER III

MATERIAL PROPERTIES

3.1 Introduction

Samples of the sand and stone fill and the in-situ subgrade soil were taken at the test road and brought to the laboratory to evaluate the basic material properties. Testing was conducted using standard procedures as described by Bowles (1978) for most tests. These procedures follow closely the standard test methods given by the American Society of Testing and Materials (ASTM), and by the American Association of State Highway and Transportation Officials (AASHTO).

3.2 Sand and Crushed Stone Fill Properties

The following tests were performed on the sand and crushed stone fill:

1. Grain Size
2. Relative Density simulating poor construction
3. Compaction
4. Califormia Bearing Ratio
5. Resilient Modulus

Disturbed samples of sand were obtained and tested from two areas. Samples taken from the fill where traffic testing was conducted are referred to as "Test Fill Sand". Samples taken from the retention dike contractor's main haul road located adjacent to the test site are designated as "Haul Road Sand". Similarly, samples taken from the crushed stone fill test section are designated as "Crushed Stone Fill". All sand and crushed stone samples were hand excavated from near the surface and placed in bags or containers.

3.2.1 Grain Size. The sand samples were first washed through a No. 200 sieve to determine the percentage of fines, and then oven-dried. They were then shaken through a sieve stack consisting of Nos. 4, 10, 20, 40, 60, 100, 140, and 200 U.S. Standard sieves. The Crushed Stone Fill sample was passed through a sieve stack consisting of 25 mm, 12.5 mm, 6.3 mm, No. 10, No. 20, and No. 100 sieve sizes. For each material, the grain size distribution curve was plotted, and values for the coefficient of uniformity, C_u and the coefficient of concavity, C_c were computed. All samples were then classified according to the Unified Soil Classification System.

The grain size distribution curves for the Test Fill Sand are shown in Figures 3.1 and 3.2. The Test Fill Sand was found to have 95 percent or more passing the No. 4 sieve and 12 to 19 percent passing the No. 200 sieve (washed). This sand is described as a tan, subangular, well-graded, silty fine subangular sand (SM). Atterberg Limits conducted on the minus No. 40 fraction indicated the material to be non-plastic.

The grain size distribution curve for the haul road sand sample is shown in Figure 3.3. The Haul Road Sand had about 99 percent passing the No. 4 sieve and 9 percent passing the No. 200 sieve (washed). The minus No. 40 fraction was non-plastic. Based on the test results, the sample was classified as a tan, well-graded, medium subangular sand (SW-SM).

The grain size distribution curve for the Crushed Stone Fill sample is shown in Figure 3.4. The crushed stone had a top size of about 1.5 in. with about 2 percent passing the No. 200 sieve. This well-graded crushed stone has a Unified Soil Classification System designation of GW. Very likely the crushed stone underwent segregation during construction; probably the gradation of the crushed stone as delivered had more than 2 percent fines.

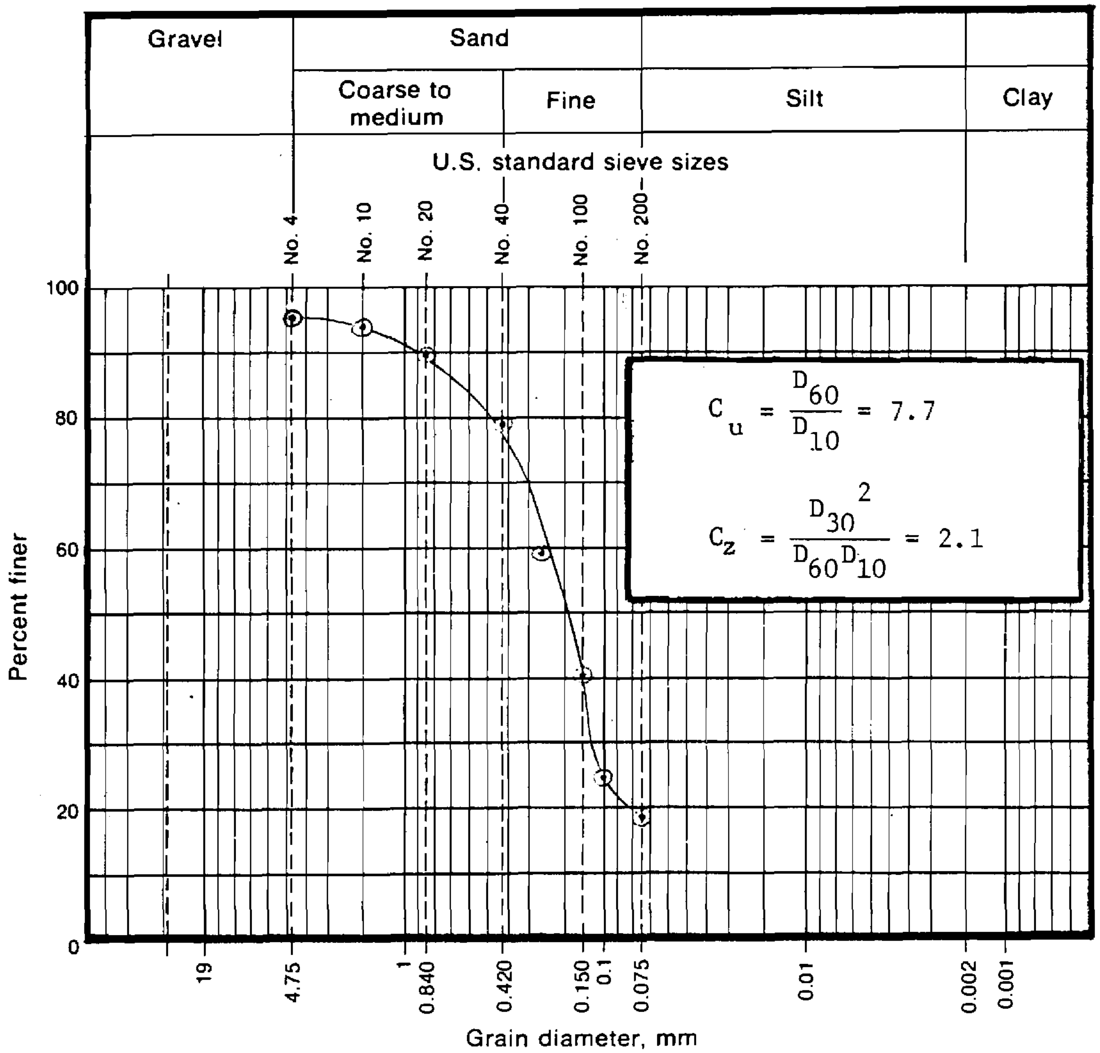


Figure 3.1. Grain size distribution curve for Test Fill Sand-1.

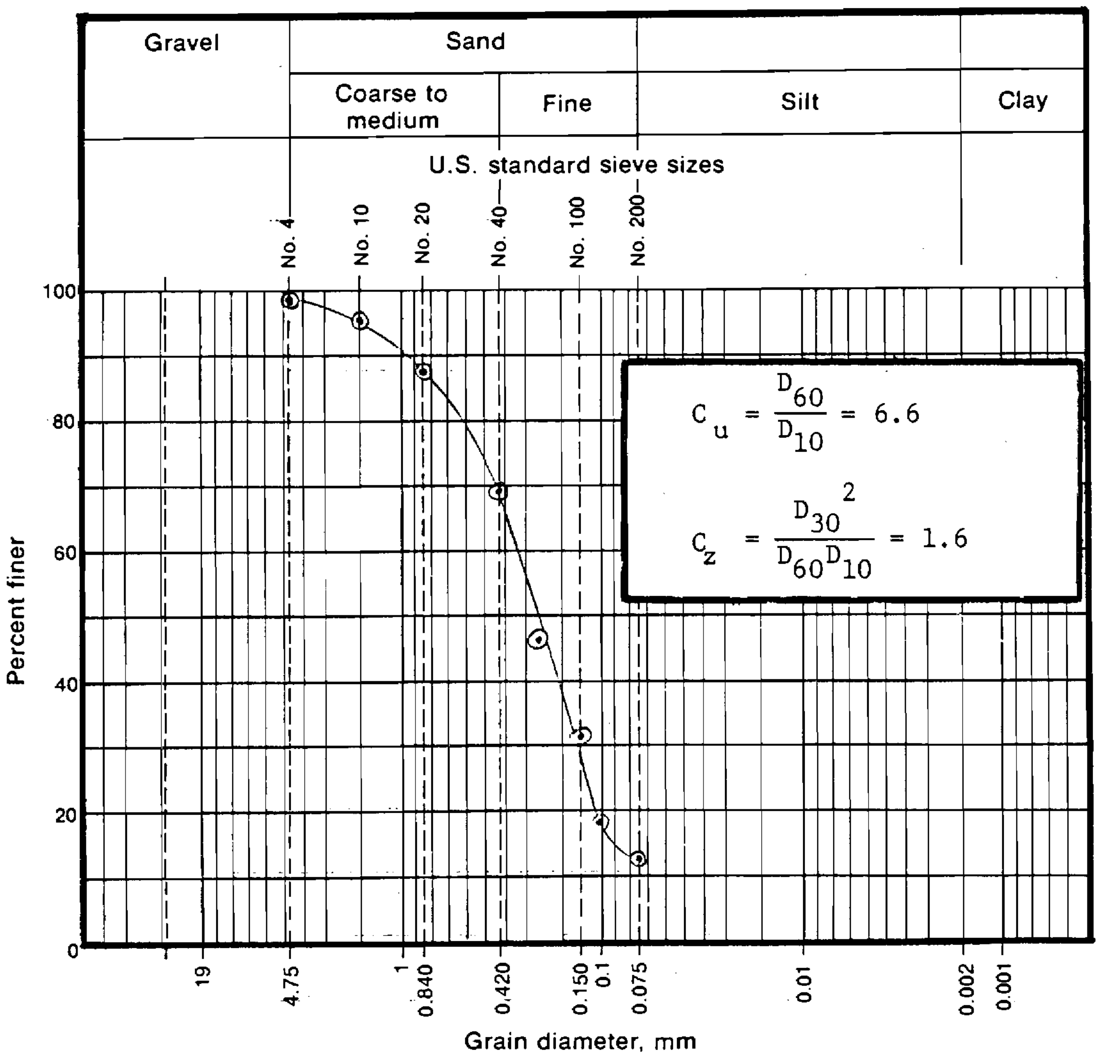


Figure 3.2. Grain size distribution curve for Test Fill Sand-2.

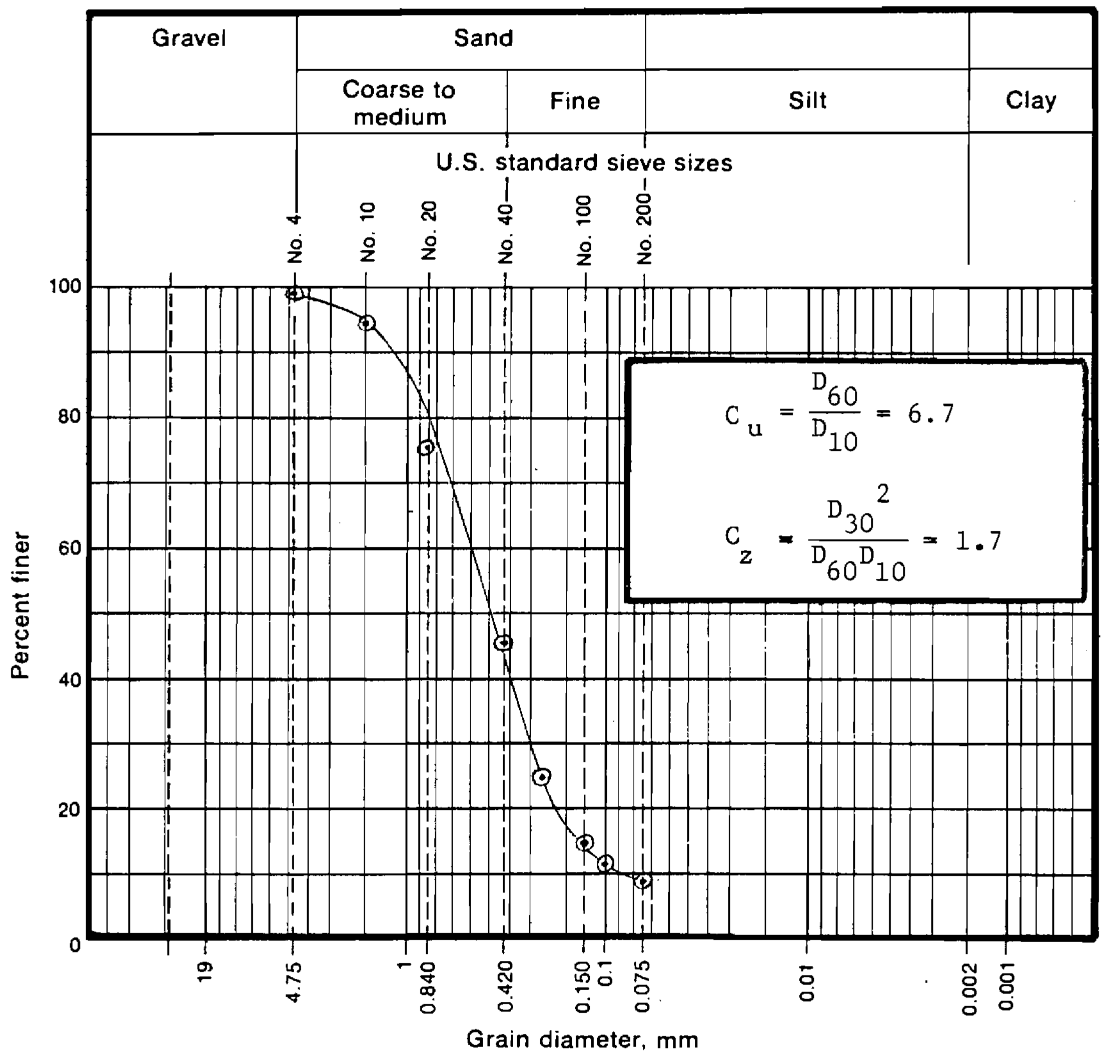


Figure 3.3. Grain size distribution curve for Haul Road Sand.

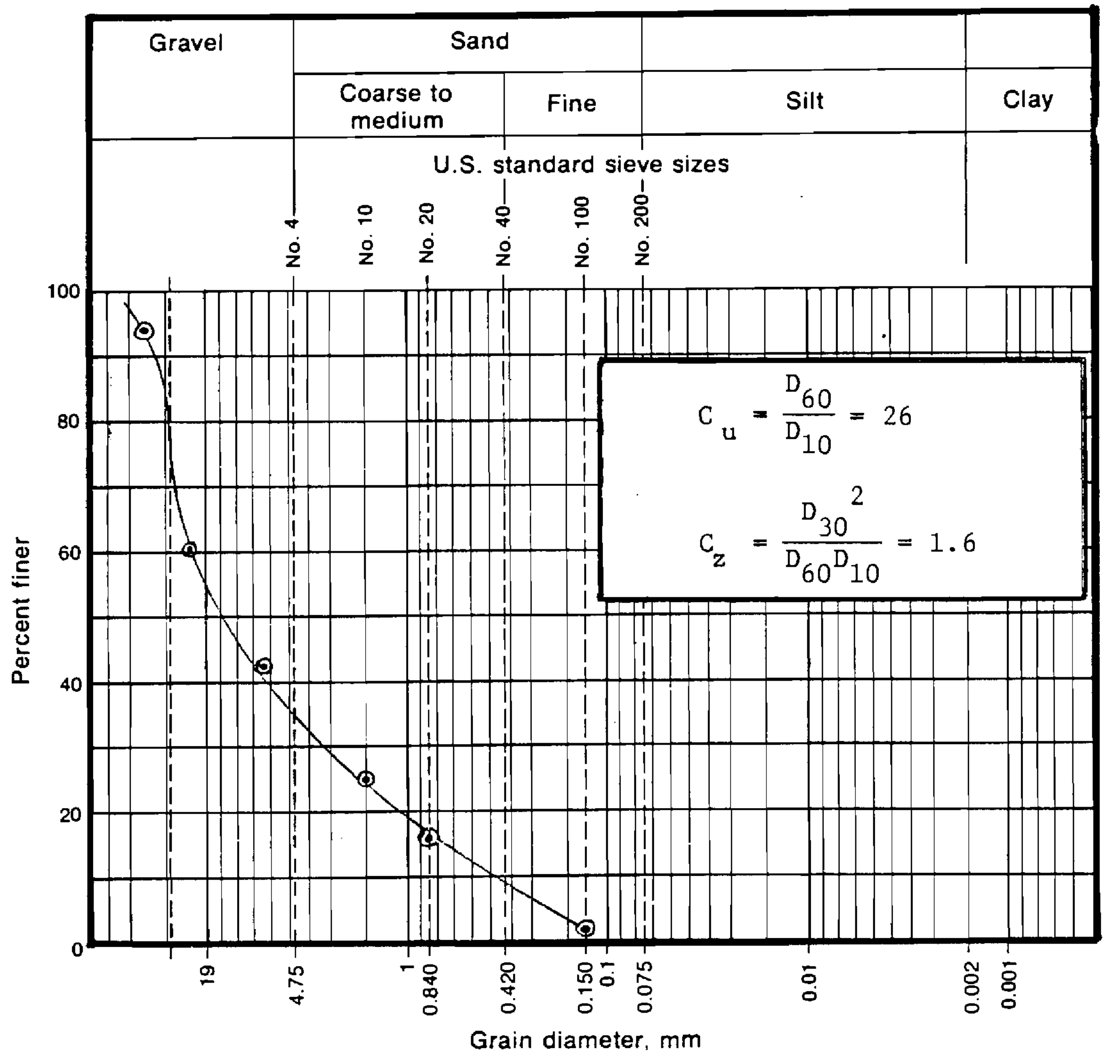


Figure 3.4. Grain size distribution curve for Crushed Stone Fill.

3.2.2. Relative Density. A crude maximum density was found by placing the sample in a 1/30 cubic foot mold in five layers and vibrating each layer by striking the mold with a rubber mallet 25 times while applying 12 kilograms of confinement to the layer. This procedure was performed three times for each sample, and the largest of the three densities obtained is presented as the maximum density.

Minimum density was determined by placing the soil into the mold by pouring in a circular motion from an inverted flask held within one inch from the previously placed soil, and striking excess soil from the top of the mold before weighing. This procedure was repeated twice for each sample,and the smallest of the three values was taken as the minimum density.

The results of the relative density tests for all samples are summarized in Table 3.1. The Test Fill Sand had the lowest minimum and maximum densities, while the Crushed Stone Fill had the highest. The range of densities indicated by these tests are indicative of the spread in density that might be achieved in the field using a bulldozer to place the fill and hauling equipment to achieve additional compaction.

3.2.3. Compaction Tests. Standard Proctor moisture-density compaction tests were performed on all samples following ASTM D698. Each air-dry sample was placed in a 1/30 cubic foot mold in three equal layers with each layer receiving 25 blows of a 5.5 lb. hammer dropped from a height of 12 in. Values of maximum dry density and corresponding optimum moisture content for each soil are summarized in Table 3.1. As expected, the Crushed Stone Fill had the highest maximum density (141.3 pcf) at the lowest

Table 3.1. Preliminary Values of Density for Sand and Crushed Stone Fill from Relative Density and Standard Proctor Compaction Tests

SOIL DESCRIPTION	RELATIVE DENSITY		STANDARD PROCTOR COMPACTION	
	Minimum Density (pcf)	Maximum[1] Density (pcf)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
Test Fill Sand-1	82.8	96.6	112.7	15.2
Test Fill Sand-2	85.8	100.5	116.5	13.6
Haul Road Sand	93.3	110.7	119.7	12.3
Crushed Stone Fill	96.0	123.6	141.3	7.9

Note (1): This density would be about the maximum density that could be achieved by filling with a bulldozer in the field.

optimum moisture content (7.9 percent). The Test Fill Sand, which was the finest-grained soil, had the lowest maximum density (113 to 117 pcf) at the highest optimum moisture content (14 to 15 percent).

3.2.4. California Bearing Ratio (CBR). Soaked CBR tests were conducted on all samples except the Crushed Stone Fill, on which an unsoaked test was performed. An unsoaked test was also performed on the Test Fill Sand. All samples were prepared at optimum moisture content in general accordance with ASTM D698. To approximate densities existing in the field, however, a 5.5 lb. hammer was dropped 25 times on each layer instead of 56 times as specified by ASTM D698. Samples for soaked CBR tests were then immersed in water for 96 hours, and swell measurements were taken. The largest swell measurement was recorded in the Test Fill Sand, with a reading of 0.22 percent. Therefore, the soils were not sensitive to swelling.

After soaking 96 hours, the samples were allowed to drain for 15 minutes, and then were placed in a compression testing machine and loaded using a 3 square inch piston moving at a rate of 0.05 inches per minute. A load-penetration curve was plotted and the zero point of the curve was adjusted as necessary. Values of load at penetrations of 0.1 and 0.2 in. were used to calculate a CBR value. In all but one test, the CBR at a penetration of 0.2 in. was higher than the CBR at 0.1 in.

CBR values determined from these tests are shown in Table 3.2. The numbers in parentheses are the lower of the values calculated at penetrations of 0.1 and 0.2 in. The CBR values of the Test Fill Sand varied from 13 to 15. Soaking had only a minor influence on the CBR

Table 3.2. CBR Values for Sand and Crushed Stone Fill

SOIL DESCRIPTION	UNSOAKED	SOAKED
Test Fill Sand-1	14.1 (10.7)[(1)]	13.2 (13.1)
Test Fill Sand-2	-	15.3 (12.0)
Haul Road Sand	-	32.7 (29.0)
Crushed Stone Fill	86.7 (66.7)	-

Note (1): Numbers in parentheses are the lower of the CBR values determined at penetrations of 0.1 and 0.2 in.

of this material. Due to its smaller amount of fines and large grain size, the CBR of the Haul Road Sand was 33, considerably higher than the CBR of 12 to 15 for the test fill sand. In general, CBR values increased with increasing grain size and decreasing fines content.

3.2.5. Resilient Modulus. Resilient modulus tests were conducted in accordance with the guidelines described in Transportation Research Board Special Report 162 by Barksdale et al (1975). Dynamic tests were performed on all samples except the Crushed Stone Fill.

The samples were tested in a triaxial cell placed within a special load frame. Load pulses were applied by an air actuated Bellofram placed on top of the triaxial cell load piston. Load pulse duration and frequency were set by means of an electronic timer at 0.2 seconds and 45 pulses per minute, respectively. A constant confining cell pressure was maintained using air throughout the test. Three Linear Variable Differential Transformers (LVDTs) mounted to the load frame measured sample deflection by recording the movement of a horizintal plate attached to the load piston.

The magnitude of load on the sample was measured using a load cell mounted in the sample base. Voltage readings from the LVDTs and the load cell were recorded on a strip chart recorder. The LVDTs and load cell were calibrated in order to accurately measure sample deflection and magnitude of load directly from the strip chart recorder. Drainage for the sample was provided through the bottom of the triaxial cell.

Samples were prepared by placing a rubber membrane over the sample base and assembling a split mold around the membrane. The top of the membrane was then stretched over the rim of the mold.

A vacuum was applied to press the membrane smoothly to the mold, and a predetermined amount of soil poured into the mold. The soil was placed in three equal layers, with each layer tamped so that the sample dry density would approximate densities measured in the field.

Sample dimensions were approximately 2.8 in. in diameter and 5.8 in. in height. After the sample was prepared, vacuum was applied to the sample, the split mold was removed, and exact sample dimensions were measured using calipers. The vacuum was then removed while simultaneously applying confining pressure to the sample. Finally, the sample was checked for air leaks, and a series of conditioning stresses applied to limit initial loading effects. The conditioning sequence is outlined in Table 3.3.

The sample was then tested at constant confining pressures of 1, 3, 5, 7, and 10 psi. At each confining stress, 25 repetitions each of deviator stress equal to 2, 3, and 4 times the confining stress were applied. Readings of resilient deflection were obtained from the strip chart recorder and used to compute the resilient modulus, E_r. Resilient moduli values were plotted against the sum of the principal stresses, σ_θ on a log-log graph. For each sample, this plot gave data points with an approximately linear relation, except for the data points obtained for a confining stress of 1 psi. A possible explanation for this deviation at low confining stress is that friction effects within the test equipment may have been more severe at the very low pressures. The results obtained at a confining pressure of 1 psi were not considered in further data analysis. For the remaining data points, the equation for the best fit curve was determined by the method of least squares.

Table 3.3. Conditioning Sequence for Resilient Modulus Testing of Sand Samples

σ_3 (psi)	σ_3 (psi)	S.R.	N
30	10	2	25
40	10	3	25
35	7	4	25
30	5	5	25
21	3	6	25

Definition of Variables:

σ_1 = axial stress on the sample

σ_3 = confining stress on the sample

S.R. = Stress Ratio = $\frac{\sigma_1 - \sigma_3}{\sigma_3}$

N = number of load pulses applied

For a linear curve fit on a log-log plot, the resilient modulus E_r can be expressed as

$$E_r = K\sigma_\theta^N \quad (1)$$

where K and N are constants evaluated from the tests, and σ_θ is the sum of the principle stresses, i.e, $\sigma_\theta = \sigma_1 + \sigma_2 + \sigma_3$.

Two tests were performed on each sample; one with air-dried soil, and one with the soil at or near optimum moisture. A dry density was used approximately equal to the value measured in the field for each material. Plots of resilient modulus versus sum of the principal stresses are shown in Figures 3.5 through 3.7. By estimating the approximate average vertical and horizontal stresses in the Test Fill Sand using Boussinesq's stress distribution theory, calculated values for resilient modulus were found to range from about 13,000 to 15,000 psi for air-dried samples and from about 8400 to 11,000 psi for samples prepared at optimum moisture content. For the Haul Road Sand, resilient modulus values for average Boussinesq stress within the fill range from about 11,000 psi for samples at optimum moisture content to about 20,000 psi for air-dried samples.

3.3 Subgrade Properties

The following tests were performed on samples from the silty clay dredge spoil subgrade:

1. Grain Size
2. Atterberg Limits
3. Resilient Modulus

Undisturbed samples of the clay were obtained by pushing thin-wall Shelby tubes to a depth of approximately 36 in. The tubes were immediately sealed with paraffin to retain the samples' moisture.

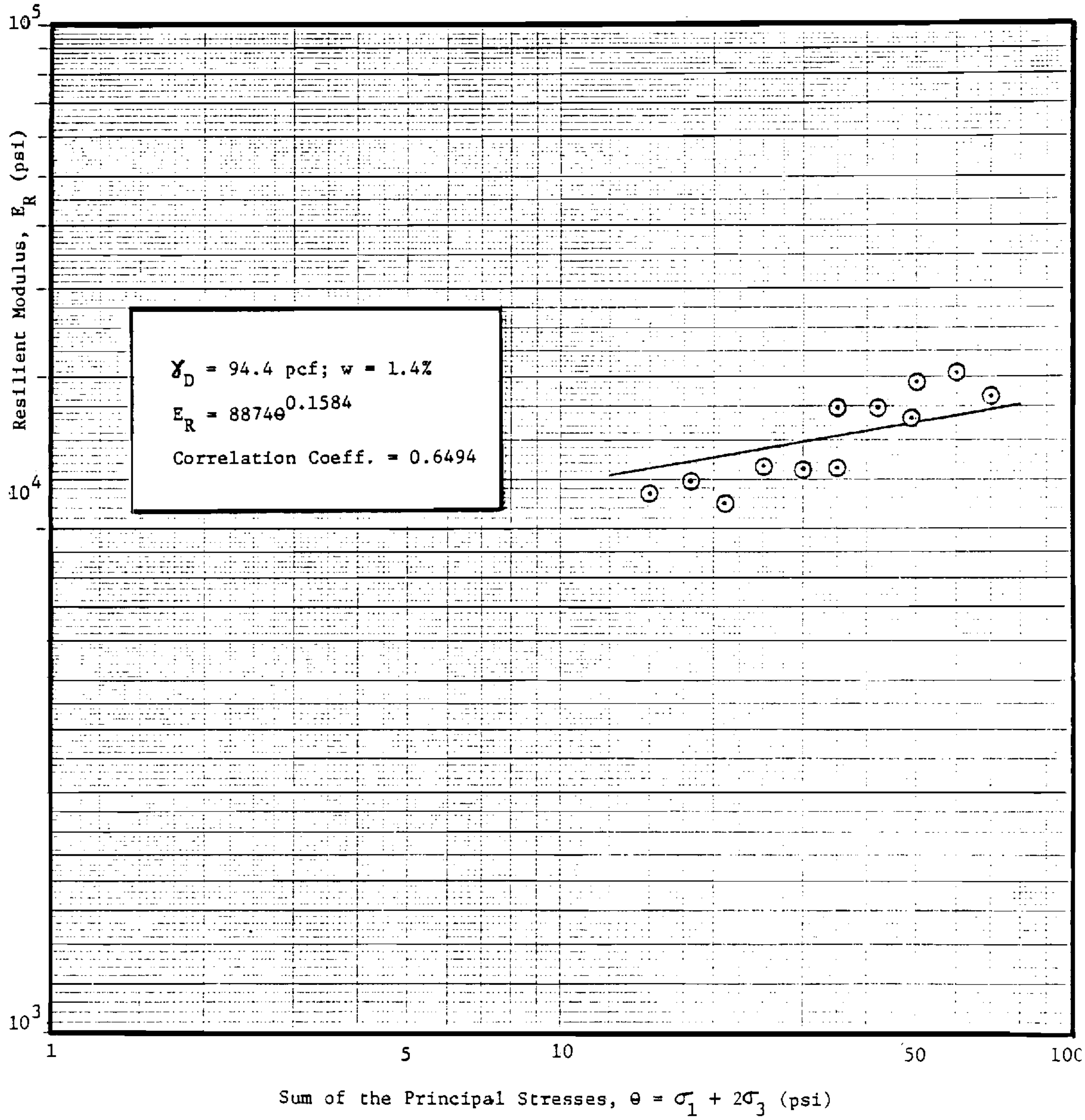


Figure 3.5(a). Resilient modulus test results for Test Fill Sand-1 (air-dried).

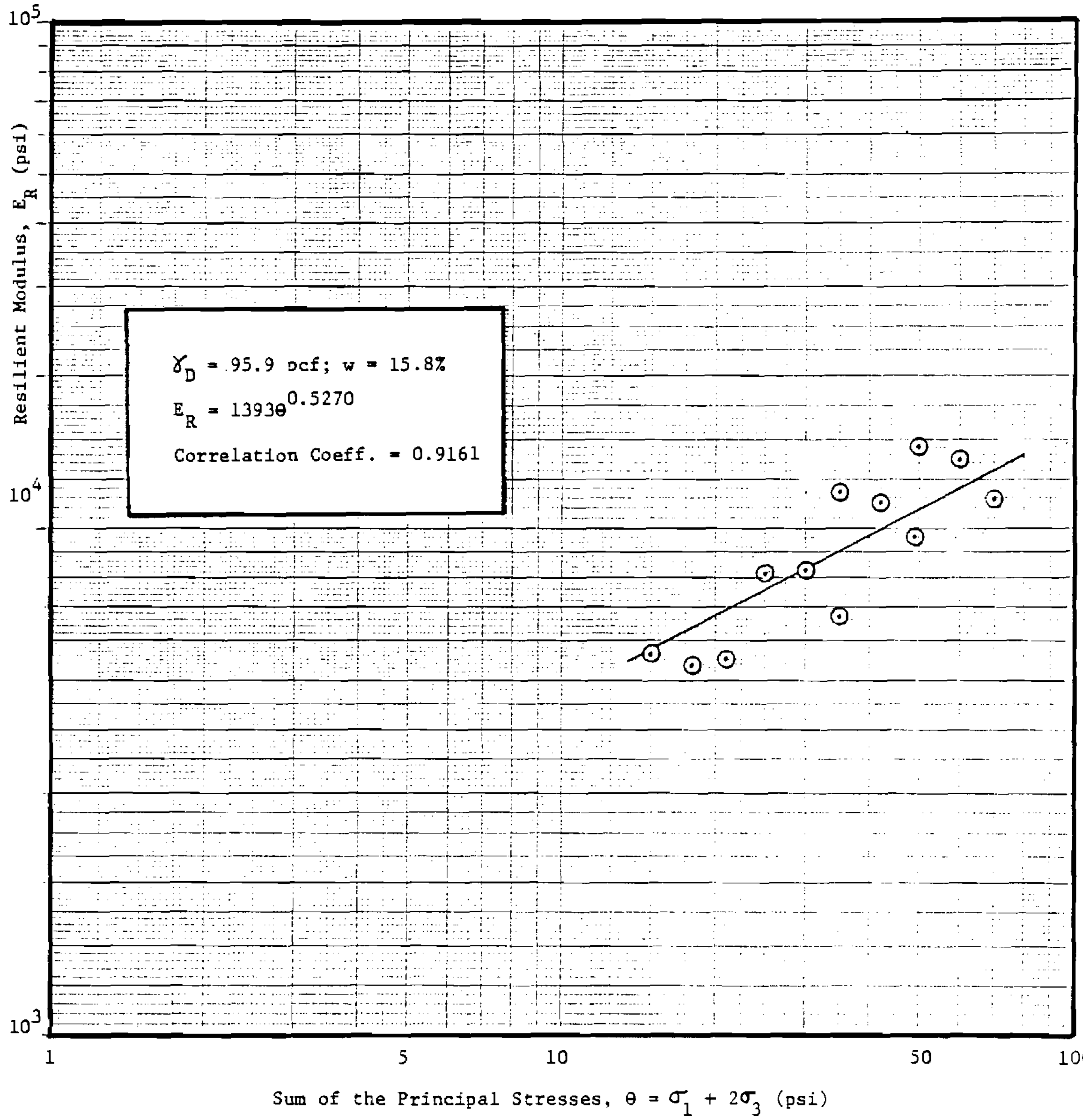


Figure 3.5(b). Resilient modulus test results for Test Fill Sand-1 (at optimum moisture content).

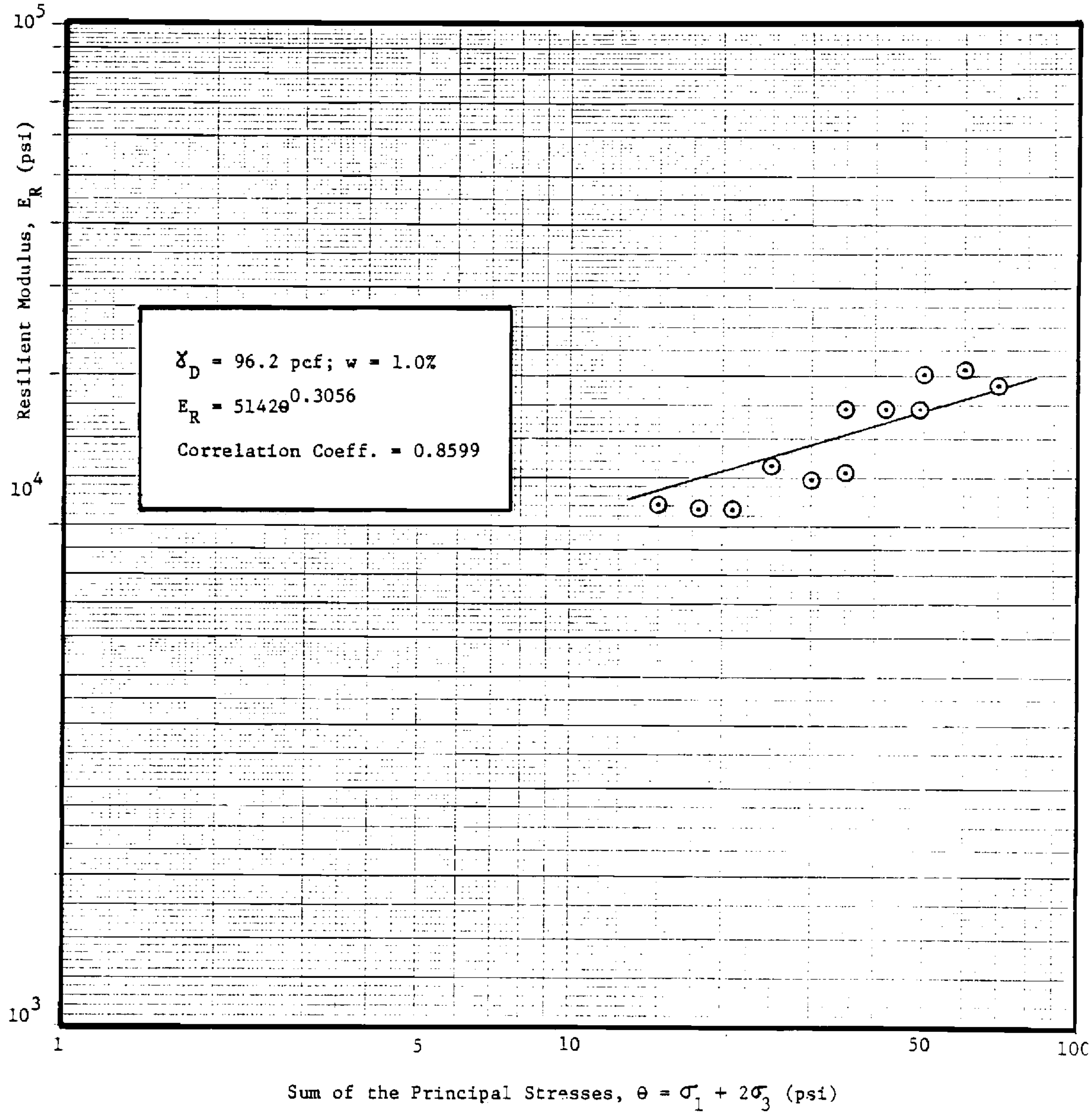


Figure 3.6(a). Resilient modulus test results for Test Fill Sand-2 (air-dried).

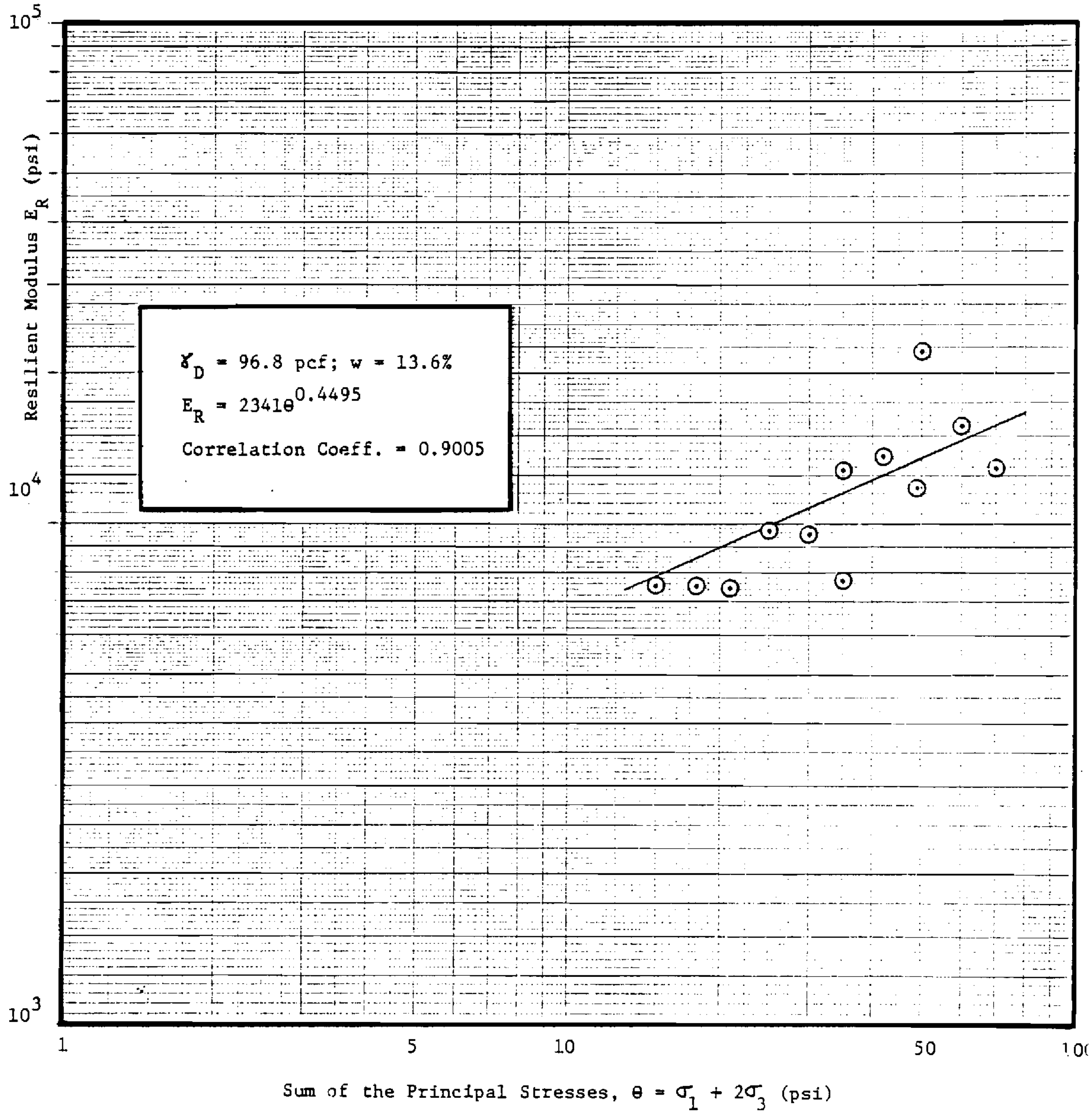


Figure 3.6(b). Resilient modulus test results for Test Fill Sand-2 (at optimum moisture content).

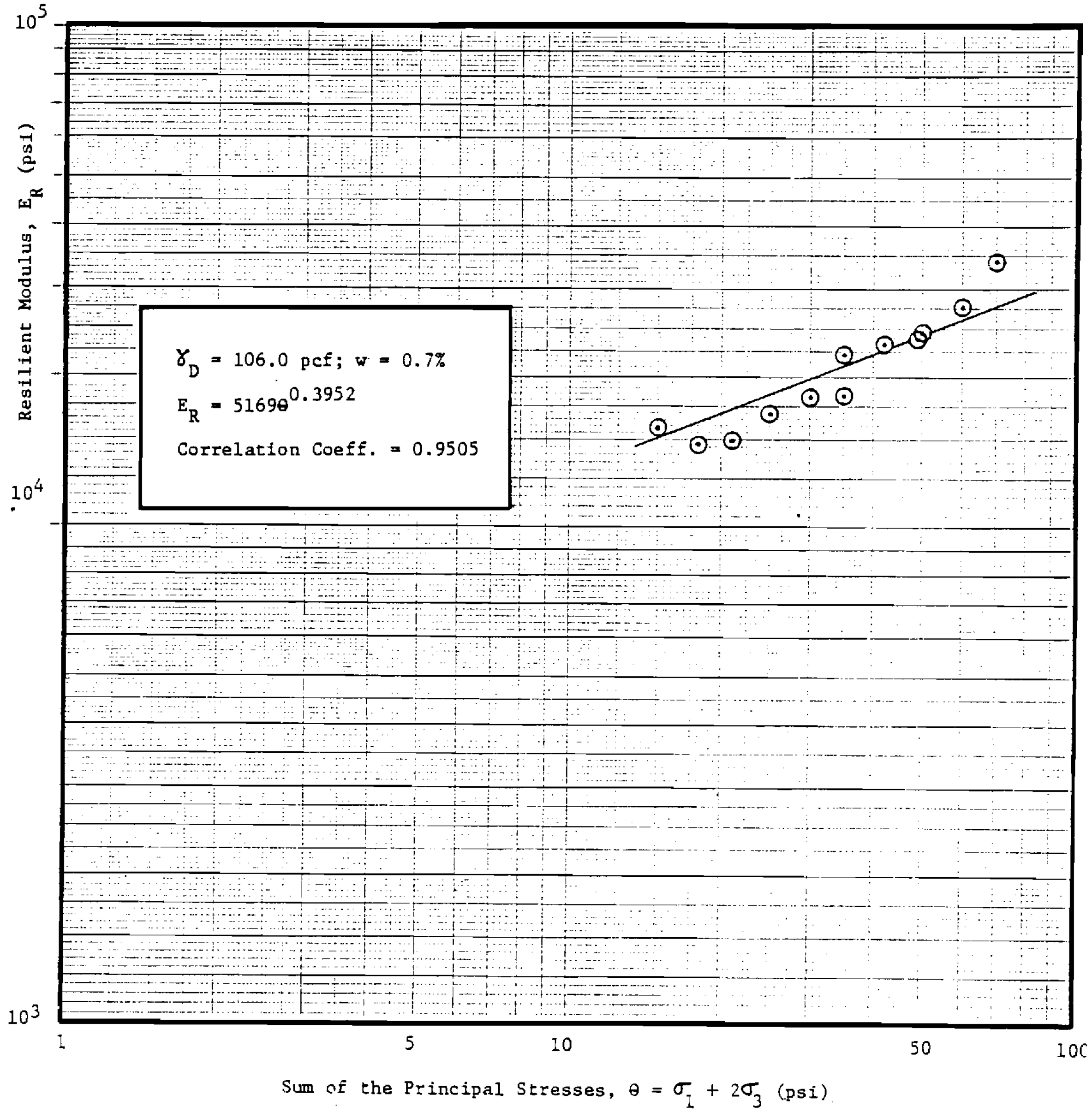


Figure 3.7(a). Resilient modulus test results for Haul Road Sand (air-dried).

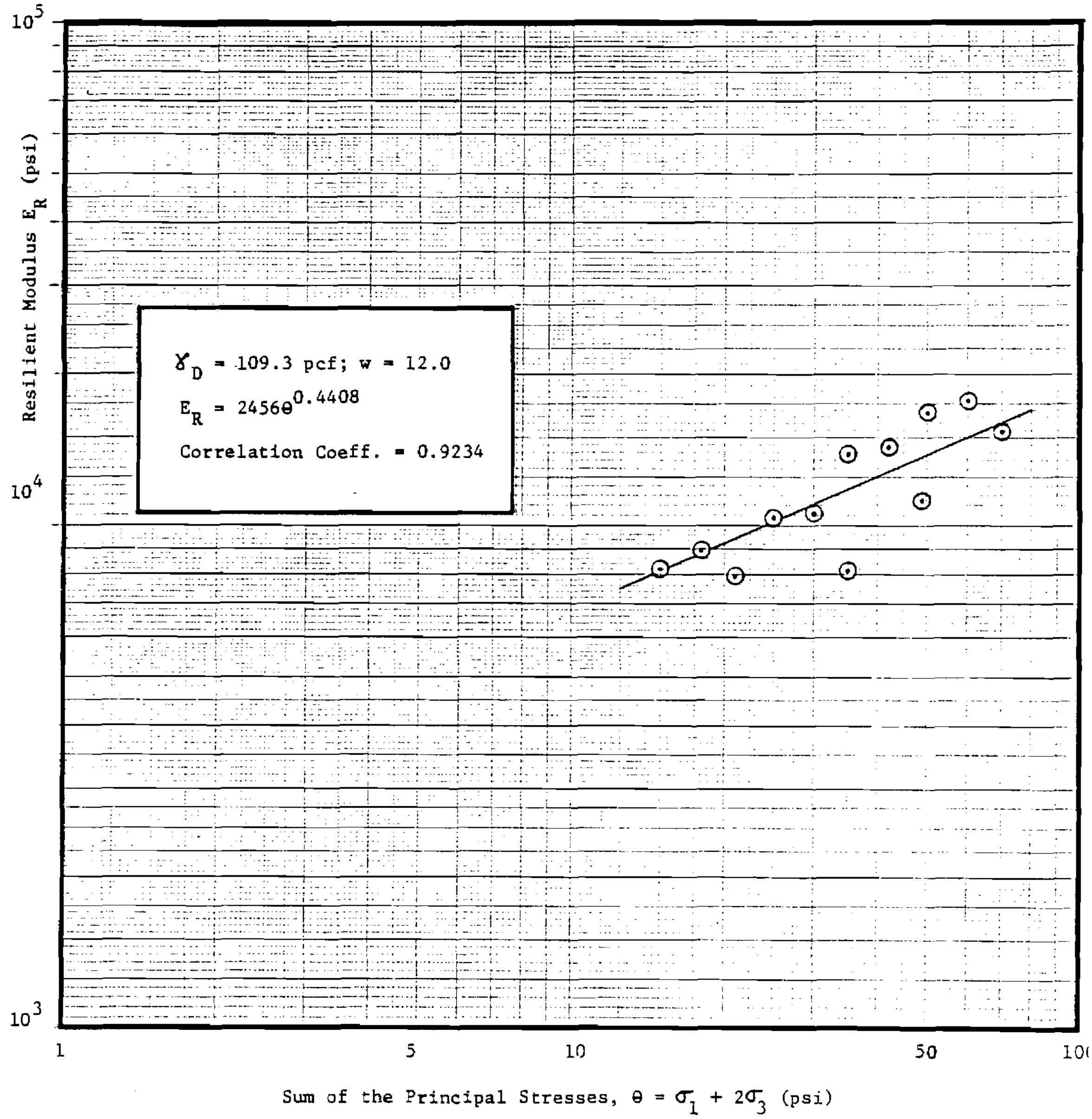


Figure 3.7(b). Resilient modulus test results for Haul Road Sand (at optimum moisture content).

3.3.1. Grain Size. Grain size tests were performed using the hydrometer method on a brownish-grey dredge spoil soil obtained from a depth of about 6 in. and on a black dredge spoil obtained from a depth of about 36 in. Exactly 50 grams of each sample were mixed with 125 milliliters of 4 percent sodium metaphosphate solution and mixed in a malt-mixer cup after soaking for 16 hours. The mixture was transferred to a sedimentation cylinder, and agitated thoroughly for one minute. A hydrometer was then inserted periodically into the mixture, and readings from the hydrometer were taken over the next 70 hours. The readings were used to compute grain sizes and percent passing.

The grain size curves for each sample are shown in Figures 3.8 and 3.9. Each sample contained approximately 30 percent clay-sized particles and had essentially 100 percent passing the No. 200 sieve.

3.3.2. Atterberg Limits. Atterberg Limits tests were performed on samples taken from the same tubes as used in grain size testing described above. Liquid limits were determined using a standard ASTM liquid limit device following standard ASTM procedure.

The liquid limit of the brownish-grey sample was 190 percent, and the plastic limit was 66 percent. For the black sample, the liquid limit was 193 percent, and the plastic limit was 70 percent. The plasticity index was therefore about 124 for both soils. Using the Unified Soil Classification System's plasticity chart, both samples plot very close to the "A-line", which separates MH/OH and CH soils. Based on these results, both samples can be described as dark brown and black, very soft to soft, highly compressible, organic silty clays (CH).

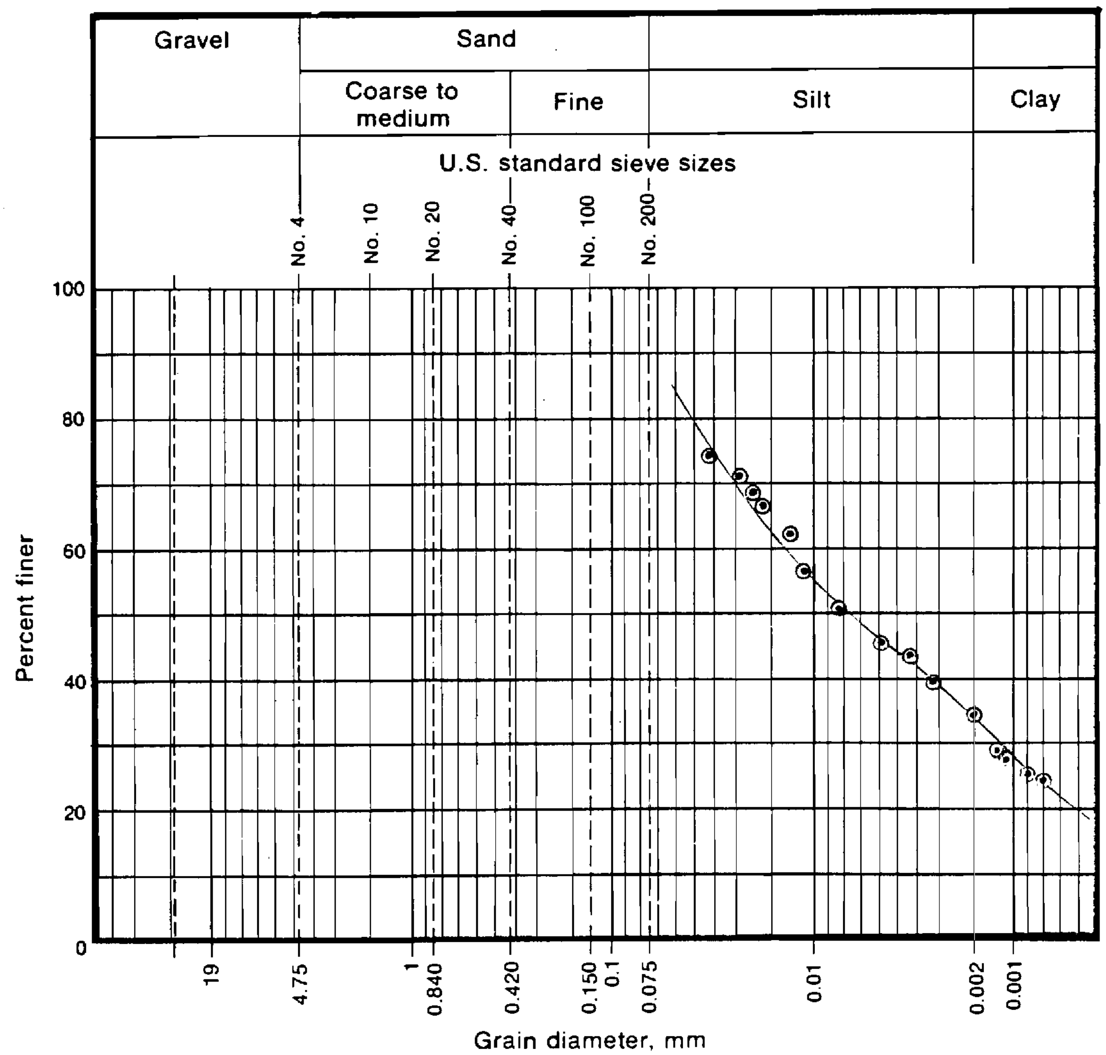


Figure 3.8. Grain size distribution curve for Brownish-grey Clay Subgrade.

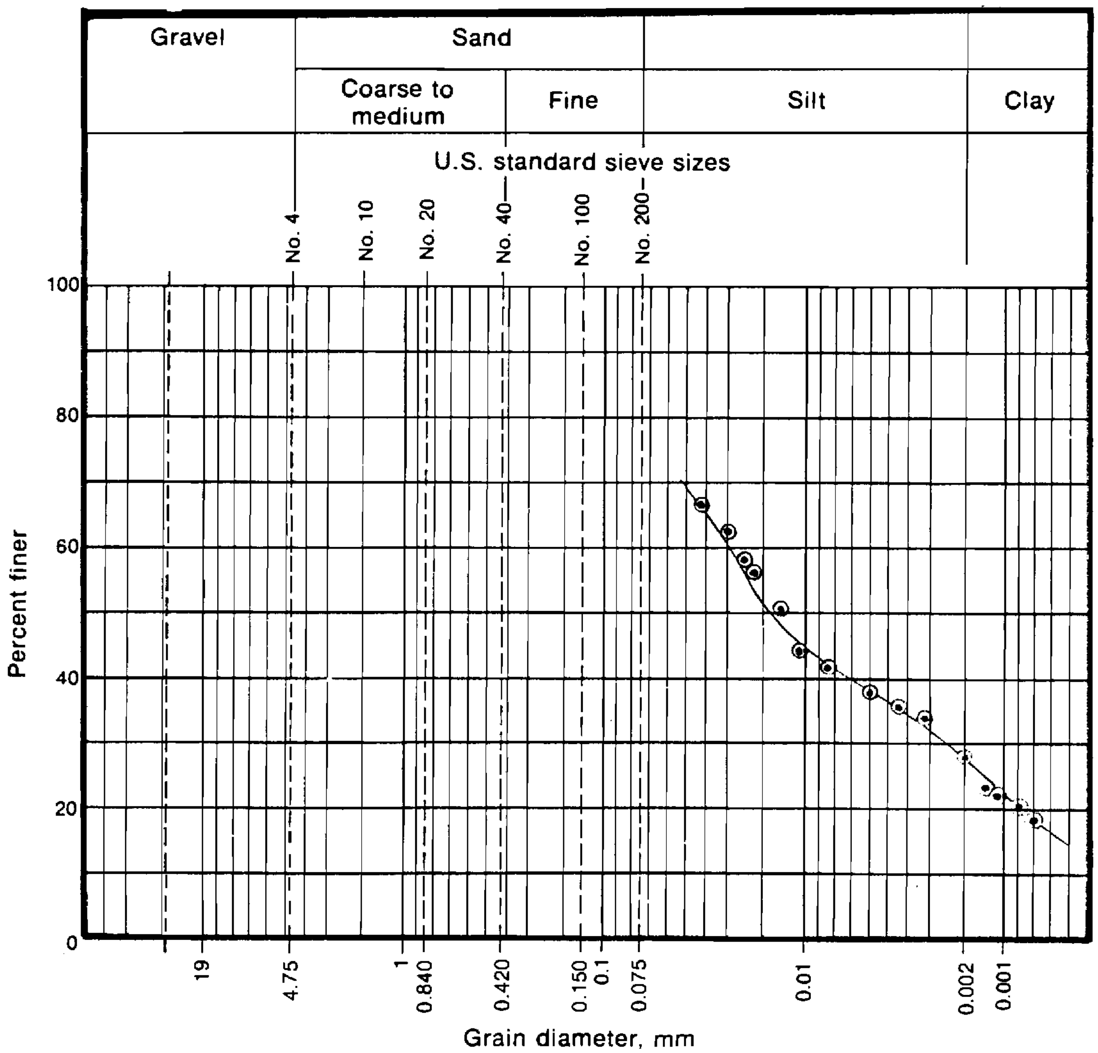


Figure 3.9, Grain size distribution curve for Black Clay Subgrade.

3.3.3. Resilient Modulus. Resilient modulus testing was conducted on samples of the clay subgrade using the same equipment and general procedures as previously described for the sand samples. Deviations from the previously described testing procedure are as follows. First, instead of preparing samples using a split mold, the silty clay samples were extruded from the Shelby tubes and placed upright on the triaxial cell base. A rubber membrane was then placed around the sample using a membrane stretcher. Second, drainage was provided at the top of the sample as well as the bottom. Because of the short time to failure, the samples were, however, essentially undrained. Third, the conditioning sequence was omitted to help prevent premature failure of the very soft samples. The samples failed very early in the tests, which prevented gathering a sufficient number of data points to plot the relationship of resilient modulus versus sum of the principal stresses.

The subgrade samples were tested at a confining stress of 3 psi. A deviator stress of 6, 7.5, and 8 psi was applied 25 times each. All of the six samples tested failed within this range of deviator stresses. Resilient moduli values varied from about 100 psi for a more disturbed sample to about 450 psi for the best quality sample at the smallest deviator stress. Most resilient modulus values were between 200 and 250 psi. Dry densities obtained from the samples ranged from 31 to 35 pcf, with natural moisture contents varying from 128 to 158 percent.

3.4 Geogrid and Geotextile Properties

Tensar SS-1 geogrid and Perma-Tex 200 geotextile materials were used as reinforcement in the formal test sections. An Amoco 2006 geotextile was, however, used on fill constructed by Higgerson-Buchanan

Contractors, which abutted against the north side of the test sections. Material properties for all three were obtained from the respective manufacturers. Material properties presented for the Tensar SS-1 geogrid are typical values determined from material tests performed by the Tensar Corporation. Material properties were also obtained for the Perma-Tex 200 and Amoco 2006 from tests conducted by the Georgia Department of Transportation.

3.4.1 Tensar SS-1. Tensar SS-1 was used in the geogrid reinforced sections. The Tensar SS-1 high density polypropylene geogrids used for haul road stabilization are manufactured by bi-axially stretching the polymer in both directions. Orientation of long chain polymer molecules results in a stiffer and stronger polymer for a given weight of material than if stretching is not employed. Typical characteristics of Tensar SS-1 geogrids are summarized in Table 3.4. The tensile strength of the geogrid across the roll width is about 1430 lb./ft. and along the roll length is 860 lbs./ft. The SS-1 geogrid is resistant to all naturally occurring alkaline and acidic soil conditions, and also resistant to attack by bacteria and fungi.

Table 3.4. Manufacturer's Test Data for Tensar SS-1 Geogrid.

MECHANICAL PROPERTIES:

Characteristic tensile strength (lb/ft)
Across Roll Width: 1430
Along Roll Length: 860

(Note: Samples, 3 junctions long and 1 rib wide were extended at a constant rate of 2 in/min, at a temperature of 68± 2°F)

STRUCTURAL CHARACTERISTICS:

Roll length (ft)	164.0
Roll width (ft)	9.8
Roll weight (lb)	67.0
Color	Black
Rib Spacing	
Across roll width	1.5 in.
Along roll length	1.1 in.

RAW MATERIAL-PHYSICAL & CHEMICAL PROPERTIES:

Polymer	Polypropylene
Shore hardness D (Din 53505)(°F)	74
Vicat Softening Point (Din 53460) (°F)	298
Impact strength (Din 53453) (lb/ft)	308
Abrasion Resistance (Din 53754E) (in^3/100 revs)	$8.5x10^{-4}$
Chemical Resistance:	Resistant to all natural occurring alkaline and acidic soil conditions
Biological Resistance:	Resistant to attack by bacteria and fungi
Sunlight Resistance:	Stabilized for long periods of exposure to U.V.

Table 3.5. Manufacturer's Test Data for Permatex 200 Geotextile.

FABRIC PROPERTY	VALUE
Weight (ASTM D-3776 - 79)	4 oz.
Grab Tensile (ASTM D-751)	200 lbs.
Mullen Burst (ASTM D-751)	375 psi
Trapezoid Tear (ASTM D-1117)	75 lbs.
Grab Elongation (ASTM D-1682)	20%
Thickness (ASTM D-1117)	18 mils
Equivalent Opening Size (EOS)	40-60
Roll Width	12.5 ft.
Roll Length	400 ft.
Roll Weight	150 lbs.

3.4.2. Perma-Tex 200. Perma-Tex 200 is a geotextile woven from monofilaments of high strength polypropylene. The material properties supplied by the manufacturer are shown in Table 3.5. The Georgia DOT performed ultimate tensile strength tests on the seams of the geotextile and also on the geotextile in the warp and fill directions. Tests on the seams were conducted in the fill direction on an 8 in. wide specimen having 5 stitches per inch. The results of tests performed on the Perma-Tex 200 by the Georgia DOT are summarized in Table 3.6.

3.4.3. Amoco 2006. Amoco 2006 is a geotextile woven from polypropylene weighing approximately 6.5 ounces per square yard. Material properties obtained from the manufacturer are shown in Table 3.7. The Georgia DOT conducted strength tests on the fabric in both the warp and fill directions and recorded the tensile strength at 10 percent strain and the ultimate tensile strength. The results of these tests are given in Table 3.8.

Table 3.6. Georgia DOT Strength and Mullen Burst Test Results for Perma-Tex 200 Geotextile

TEST CONDITION	Tensile Strength (lb./in.)	
	10% Strain	Ultimate
SEAM STRENGTH - FILL DIRECTION		
Test 1	-	100
Test 2	-	97.5
Test 3	-	106
FABRIC - WARP DIRECTION		
Test 1	67	110
Test 2	57	114
Test 3	66	106
FABRIC - FILL DIRECTION		
Test 1	102	142
Test 2	102	146
Test 3	101	145
MULLEN BURST	390 $lb./in.^2$	

Table 3.7. Manufacturer's Test Data for Amoco 2006 Geotextile

FABRIC PROPERTY	VALUE
Tensile Strength	275 lbs.
Elongation	20%
Mullen Burst Strength	600 psi
Puncture Strength	120 lbs.
Accelerated Weathering (Strength retained)	70%
Equivalent Opening Size (EOS)	70/30

Table 3.8. Georgia DOT Tensile Strength Test Results for Amoco 2006 Geotextile

TEST DIRECTION	Tensile Strength (lb./in.)	
	10% Strain	Ultimate
WARP DIRECTION		
Test 1	131	229
Test 2	118	210
Test 3	115	207
Test 4	114	218
FILL DIRECTION		
Test 1	204	240
Test 2	226	275
Test 3	244	288
Test 4	213	256

CHAPTER IV

TEST SECTION LAYOUT AND CONSTRUCTION

4.1 General Site Conditions

The test site was located in South Carolina adjacent to the Savannah River directly across from the city of Savannah, Georgia. The site is a dredge spoil disposal area actively being used by the Savannah District of the U.S. Army Corps of Engineers. The ground surface within the containment dikes is relatively flat. Hexagonal dessication cracks are visible on the surface during at least most of the year. The insitu soil at the site to a depth in excess of 36 ft. is very soft to soft, organic, silty clay dredged from the Savnanah River. Properties of the soil determined by laboratory testing are described in Chapter III; subgrade vane shear strengths are given in Section 4.5 of this chapter.

The groundwater table is located near the surface. During early inspection of the site in May and June, 1984, the groundwater was approximately 1.5 ft. below the surface. In July and August, the groundwater level rose about 1 to 1.5 ft. to just below the ground surface due to several heavy rains. At the time of haul road construction at the test site, the groundwater table was within 0.5 ft. of the surface.

During the time the haul road tests were conducted, Higgerson-Buchanon Contractors were working within the dredge spoil area on a Corps of Engineers contract to construct two new dikes and related facilities to contain additional dredge spoil. The carry out this contract, Higgerson-Buchanon widened an existing dike which passed along the south and east sides of the test sections; the enlarged dike served as a main haul road. The main haul road embankment material was a well-graded silty sand having the properties given in Chapter III.

4.2 Test Section Layout

Eight test sections were constructed for Phase I of the testing program using a silty sand as fill material. The test sections were arranged to give four traffic lanes (Figure 4.1). Each lane contained one section reinforced with Tensar SS-1 geogrid and another section reinforced with Perma-Tex 200 geotextile. Properties of the Tensar SS-1, Perma-Tex 200, and the silty sand fill are summarized in Chapter III.

Figure 4.1 shows the general layout of the Phase I test sections. Initial sand fill thicknesses varied from about 42 to 49 in. as indicated in Figure 4.1. Thinner fill thicknesses were originally planned. However, the fill was placed to predetermined elevations based on the original ground surface elevation. As a result of a large amount of immediate and long-term surface settlement, the actual thickness of sand fill placed was greater than planned. Fill thickness in Lanes 3 and 4 were subsequently reduced in Phase IA of the testing program.

Two additional test sections (Lane 5) were later constructed during Phase II of the study. A general plan of the Lane 5 test sections is shown in Figure 4.2. Section 5U was an unreinforced section consisting of sand fill placed directly upon the subgrade. The fill was the same silty sand used in Phase I. Section 5T used crushed stone for the fill and Tensar SS-1 geogrid for reinforcement. Properties of the sand and crushed stone used in these sections were given in Chapter III. The fill thicknesses of each section was originally 2 ft. (Figure 4.2). After testing the 2 ft. thick sections, 1 ft. of additional crushed stone was placed over the existing

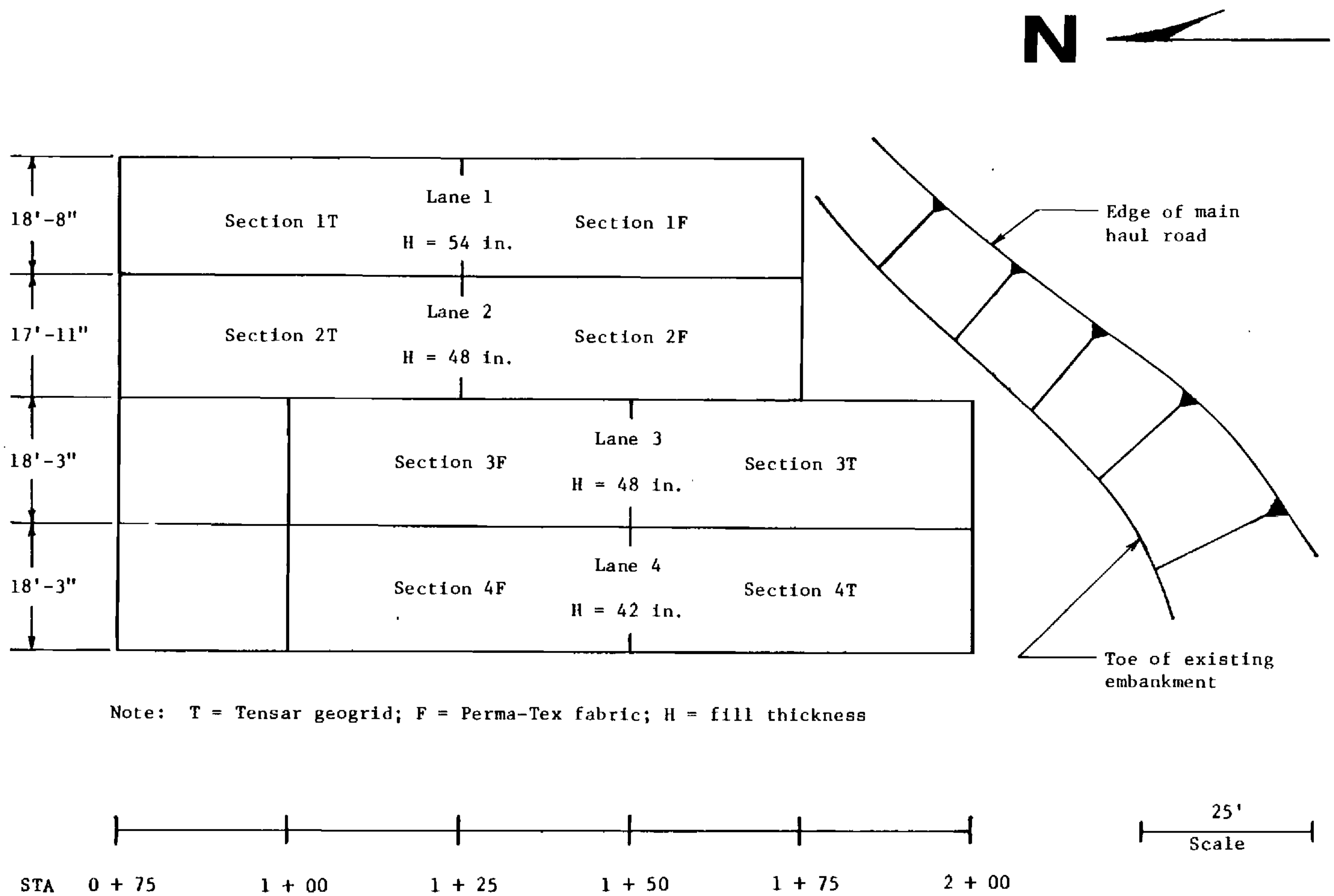


Figure 4.1. Phase I test section layout - sand fill.

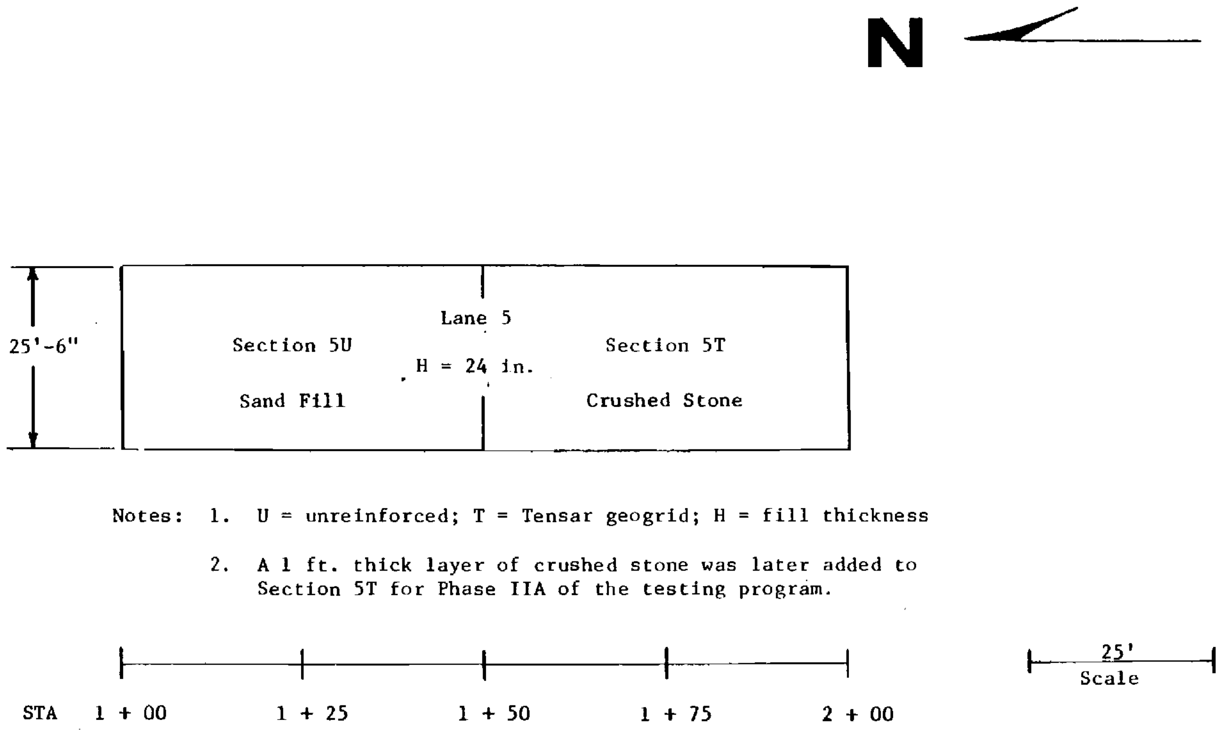


Figure 4.2. Phase II test section layout.

stone and the subgrade allowed to consolidate. This section was then tested in Phase IIA.

4.3 Strain Coils

Bison type strain coil groups consisting of 0.5 and 1.0 in. diameter coils were attached to both the Tensar SS-1 geogrid and the Perma-Tex 200 geotextile before construction of the sections to monitor strains in these materials. The locations of these groups within the Phase I test sections are shown in Figure 4.3. The configuration of each strain coil group is illustrated in Figure 4.4. The axes of the strain coil groups were placed perpendicular to the lane direction. In Phase II, a strain coil group was also installed on the geogrid in Section 5T. The location and configuration of the coil group is shown in Figures 4.5 and 4.6, respectively.

To attach the Bison coils to the Tensar SS-1, the upper face of each Bison coil was covered with epoxy glue and then placed on the underside of the geogrid at a grid intersection. The coils were then tied to the geogrid with electrical twine and a small wooden dowel to temporarily hold them in place (Figure 4.7) until the glue could harden. The coils and grid intersections were then covered with Magnabond 55-3 two-part epoxy glue to form an encapsulation-type connection.

Bison coils were fastened to the Perma-Tex 200 geotextile using a slightly different procedure. A special insert shaped like a machine screw except without threads was machined from micarta. The insert was made to fit tightly into the center hole in the Bison coil. To make the connection in the field, epoxy glue was placed on the bottom of the Bison coil and inside face of the insert.

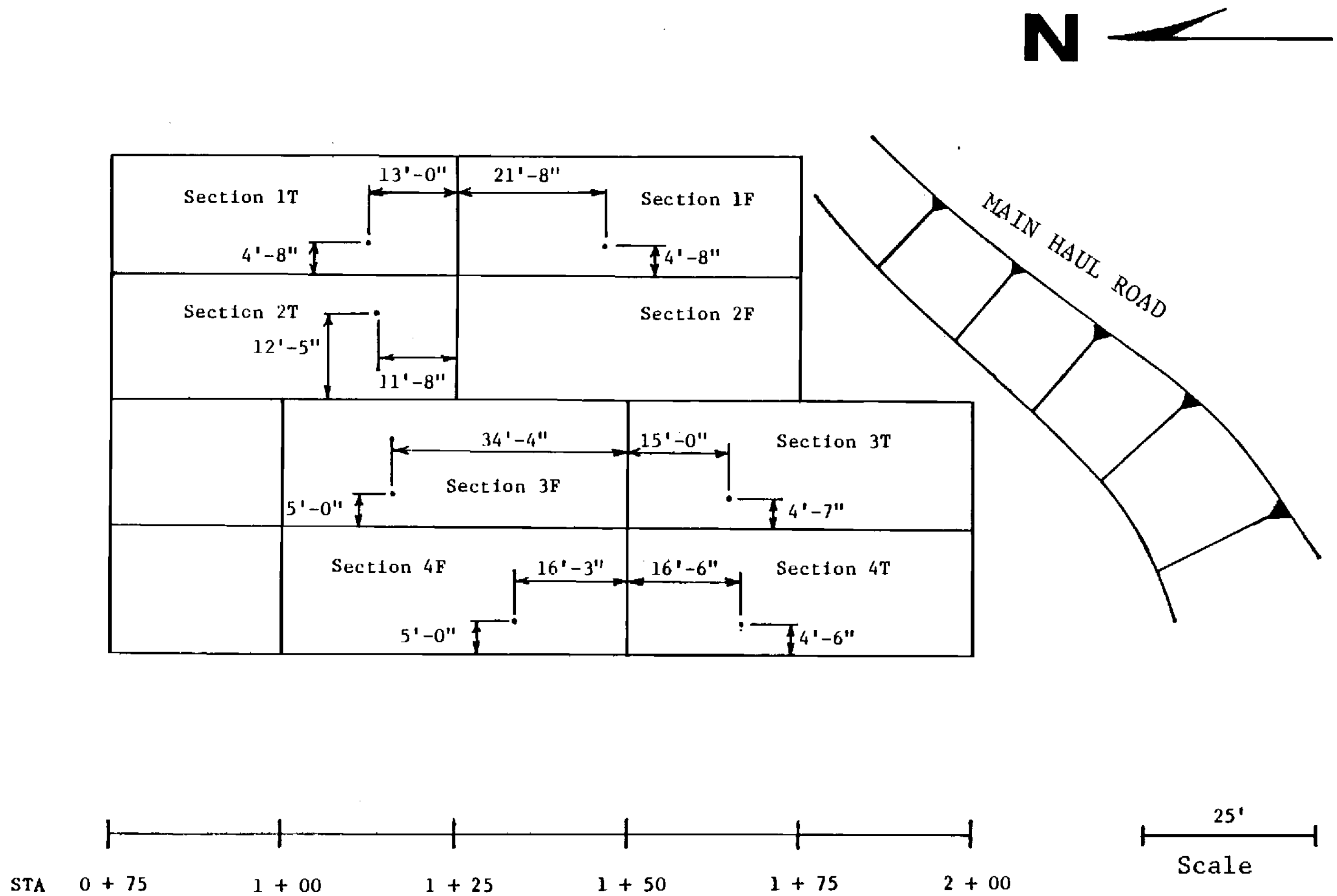


Figure 4.3. Phase I Bison coil location plan.

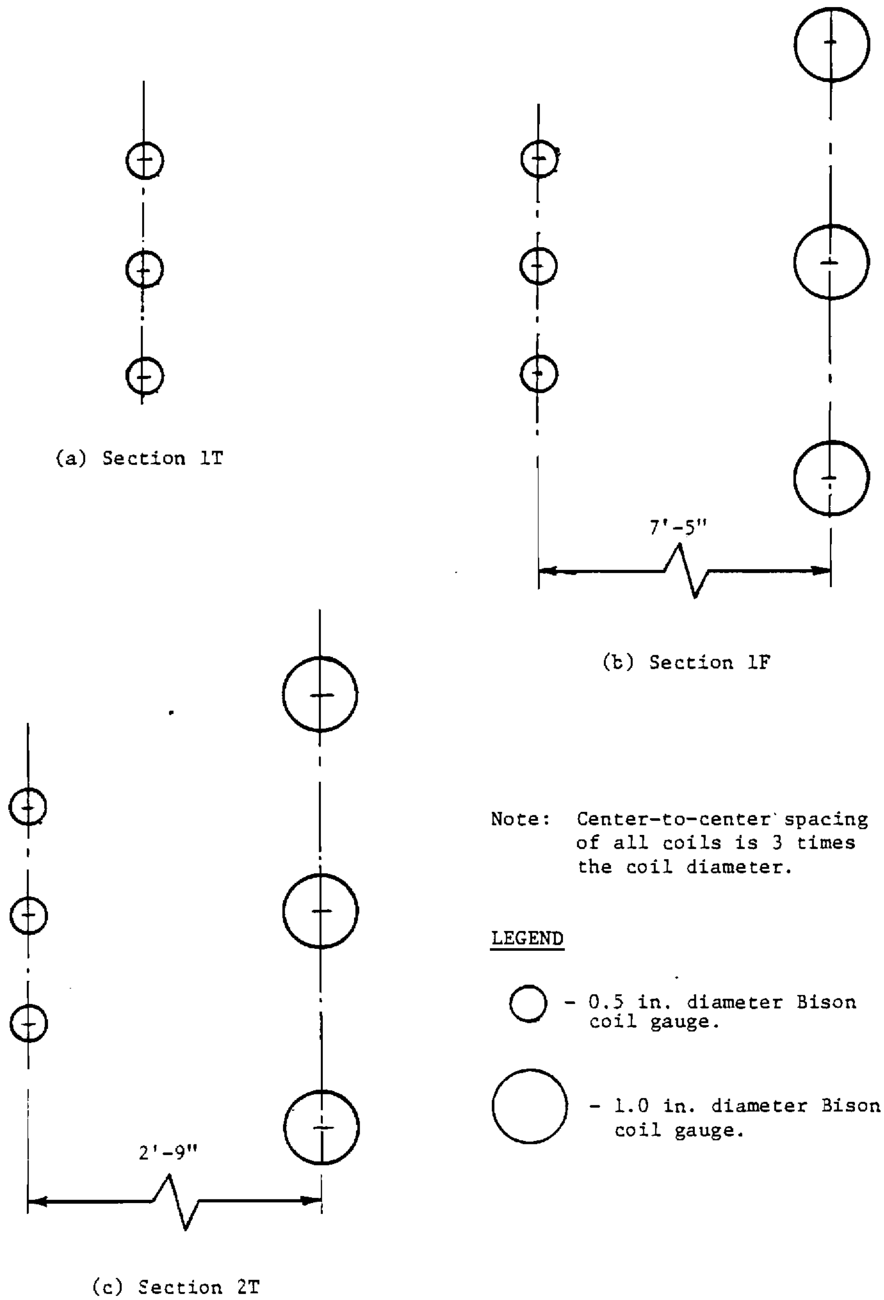
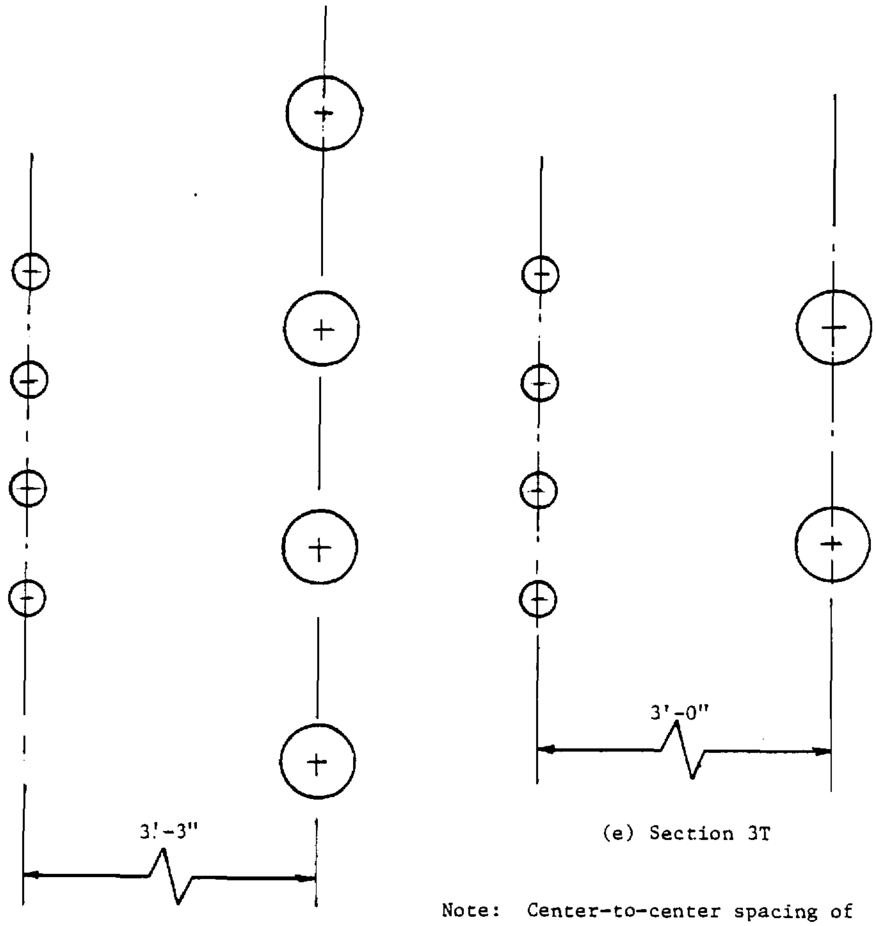


Figure 4.4. Phase I configuration of Bison coil groups.



(d) Section 3F

(e) Section 3T

Note: Center-to-center spacing of coils is 3 times the coil diameter.

LEGEND

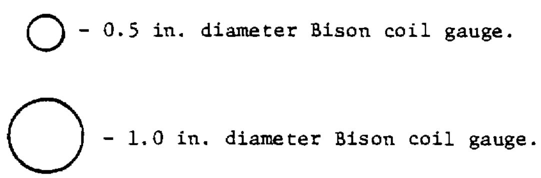
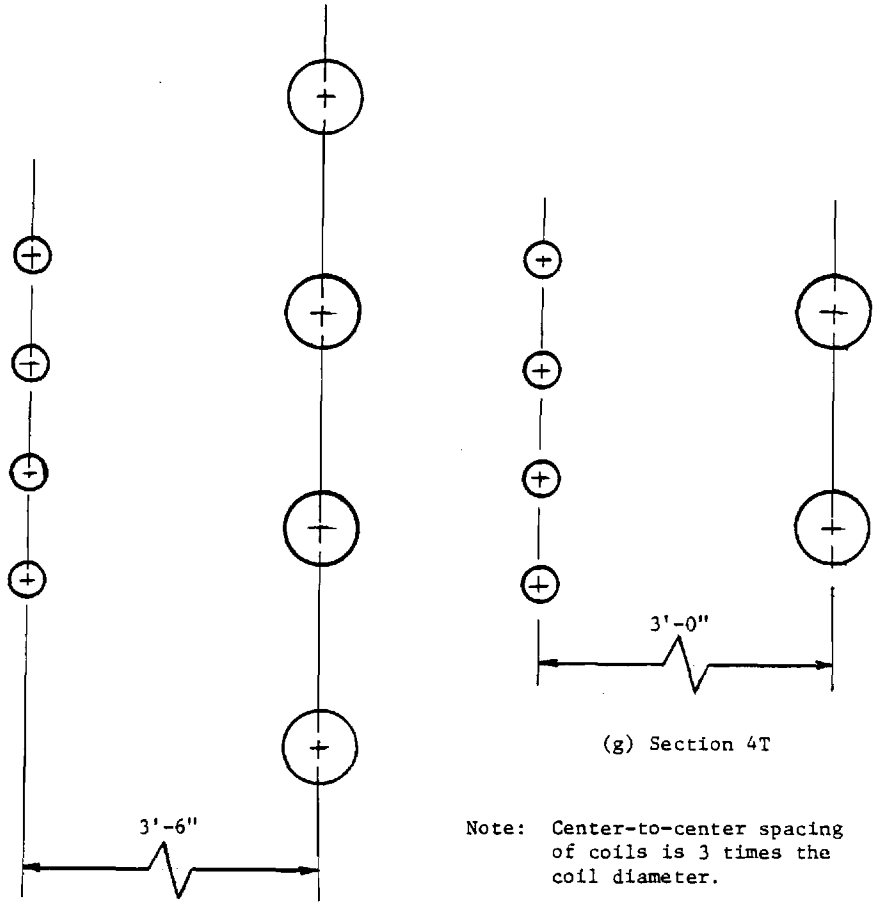


Figure 4.4. (cont.)



(g) Section 4T

Note: Center-to-center spacing of coils is 3 times the coil diameter.

(f) Section 4F

<u>LEGEND</u>

 - 0.5 in. diameter Bison coil gauge.

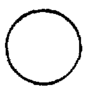
 - 1.0 in. diameter Bison coil gauge.

Figure 4.4. (cont.)

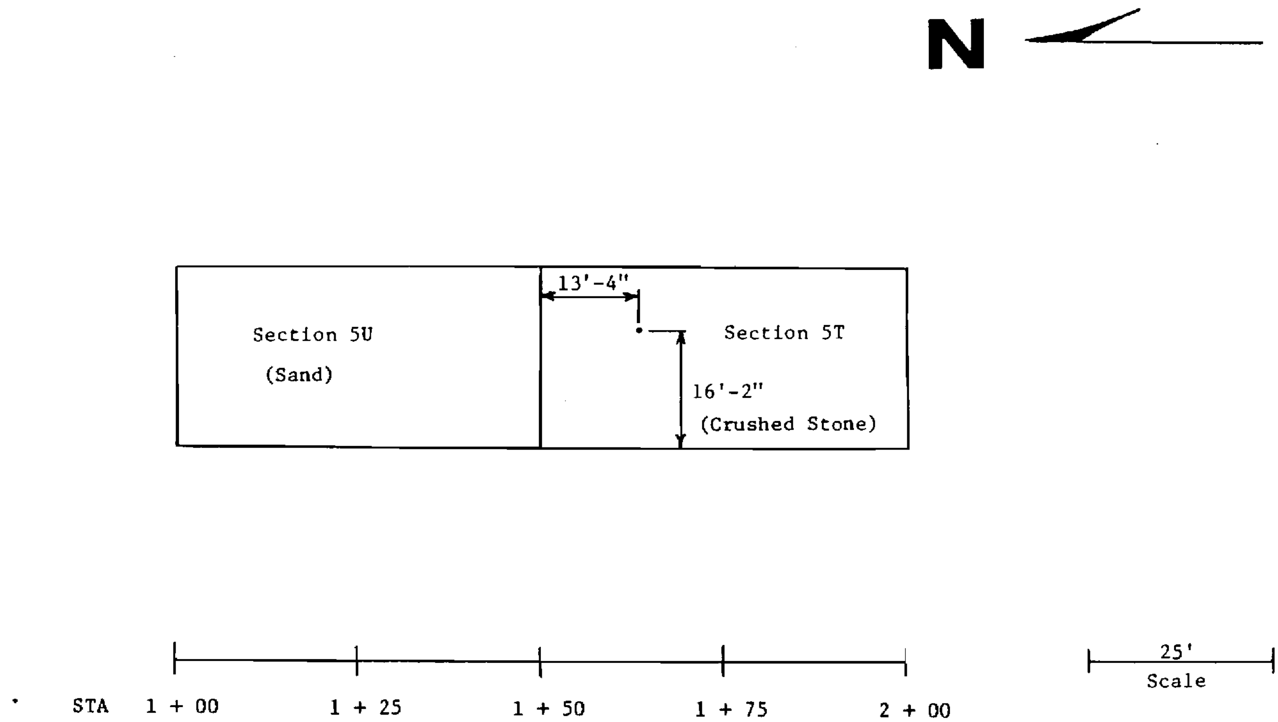


Figure 4.5. Phase II Bison coil location plan.

Section 5T

Note: Center-to-center spacing of coils is 3 times the coil diameter.

LEGEND

 - 0.5 in. diameter Bison coil gauge.

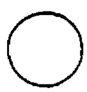
 - 1.0 in. diameter Bison coil gauge.

Figure 4.6. Phase II configuration of Bison coil group.

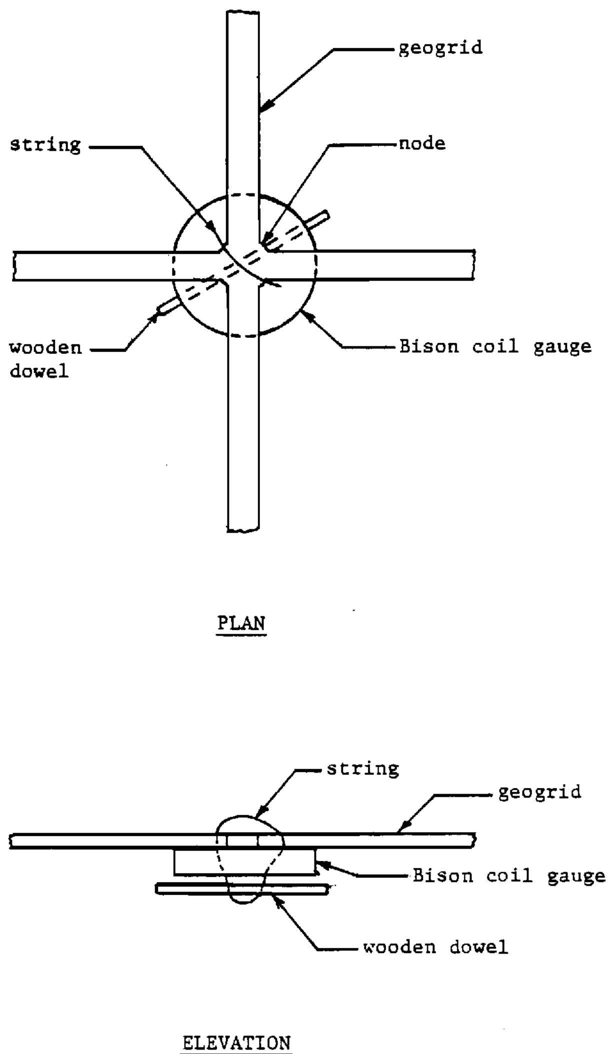


Figure 4.7. Attachment of Bison coil gauges to Tensar geogrid.

A small hole was made in the geotextile by pushing the fibers apart. The insert was then pushed from below through the hole in the geotextile. The Bison coil was then placed on top of the geotextile and mated firmly with the insert. The techniques used to attach the coils to the geogrid and geotextile were later found to work extremely well under adverse field conditions which included submergence below water. To protect the Bison coils during fill placement, they were covered beforehand with a pile of sand. Extra slack was left in the cables leading from the coils to allow movement of the reinforcing element during fill placement without causing damage to the coils or wire.

4.4 Test Section Construction

4.4.1. Reinforcement Fastening and Placement. After removing all small bushes and weeds, the Tensar SS-1 and Perma-Tex 200 was rolled out over the site. Two rolls of the Tensar SS-1 geogrid (or Perma-Tex 200 geotextile) were used to form each lane. To prevent possible slippage, the reinforcement material was securely fastened together in the longitudinal direction down the central portion of the lane. The Perma-Tex 200 was fastened together using a hand-held sewing machine powered by a portable electric generator. This device sewed 4 to 5 stitches per inch using a heavy thread. Using a crew of 5 men, the method was fast, with 100 ft. lengths being sewed together in approximately 20 minutes.

Tying the rolls of Tensar together was more tedious and much slower. The Tensar was tied by overlapping the sides of the two rolls approximately 6 in. The overlapping grids were lined up, and polypropylene brading was hand woven through each opening and tied.

Because the geogrid openings varied in size, it was usually necessary to slightly offset one of the rolls after tying a length of about 2 to 3 ft. so that the openings would again match up. Waviness of the Tensar SS-1 geogrid was visible after it was tied together. This waviness was caused by aligning and tying together grid openings of varying size.

The roughness and sharpness of the Tensar and the brading caused unprotected worker's hands to become sore and blistered; the use of work gloves tended to be too cumbersome for tying the small grid openings and thus slowed the pace of tying. On the average, approximately the equivalent of 12 to 14 man-hours was required for tying a 150 ft. length of Tensar SS-1 geogrid together.

When fastening was completed, the Tensar and Perma-Tex were positioned in the appropriate test sections and stretched by hand as tightly as possible to take out slack. During placement of the reinforcement for Phase I test section construction, the observation was made that the Tensar was hard to stretch out straight because of the existing waves caused by tying. On the other hand, the 4 oz. Perma-Tex 200 was easy to stretch out but did not have sufficient resistance to bending to prevent the geotextile from settling down into the many depressions existing on the hummocky surface, particularly after it was walked upon by workers. As a result, waviness existed in the Tensar SS-1 and slack generally existed in the Perma-Tex 200 before fill placement. During construction of the Tensar SS-1 Section 5T in Phase II, the Tensar was stretched out by four men as tightly as possible and the edges staked using 4 ft. long straight 0.5 in. diameter pipe and straight No. 4 or No. 5 steel rebars placed 8 ft. apart along the edges.

A moderate to strong breeze was found to lift the Perma-Tex 200 off the ground like a sail. Such behavior is often referred to as "kiting". The kiting problem was controlled by placing piles of soil about every 10 ft. along the edges of the Perma-Tex. A moderate to strong breeze had no effect on the "porous" Tensar SS-1 geogrid, and it stayed nicely in place before fill placement.

At the longitudinal and transverse joints between the test sections in Phase I, the Tensar geogrid and Perma-Tex 200 geotextile were overlapped slightly but were not fastened together. Figure 4.8 shows the amount of overlap that existed at each joint.

A very long section of Amoco 2002 6 oz. woven geotextile was placed by Higgerson-Buchanon Contractors immediately to the north of the formal test sections. The Amoco 2002 geotextile reinforced area was informally included in the study where it abutted up to the formal test lanes. Initially, the same thickness of sand fill was placed over the Amoco 2002 as used over the test sections. The area covered by the Amoco geotextile formed the beginning of one segment of a new addition to the dredge spoil dikes. Therefore additional sand and/or dried silty clay dredge spoil was later added to bring the dike to the specified grade.

The Amoco geotextile was placed at right angles to the formal test sections (i.e., in an east-west direction). The total length of the Amoco geotextile in this direction was 150 ft. with the width of an individual geotextile strip being about 18 ft. after sewing. All rolls of Amoco were sewed together on each side using the same portable sewing machine as employed for the Perma-Tex 200 test sections.

To stretch/pretension the Amoco geotextile, fingers of sand fill were pushed out along the east and west sides of the fabric perpendicular to the long direction of the roll. Typically fill was pushed out 40 to 100 ft.

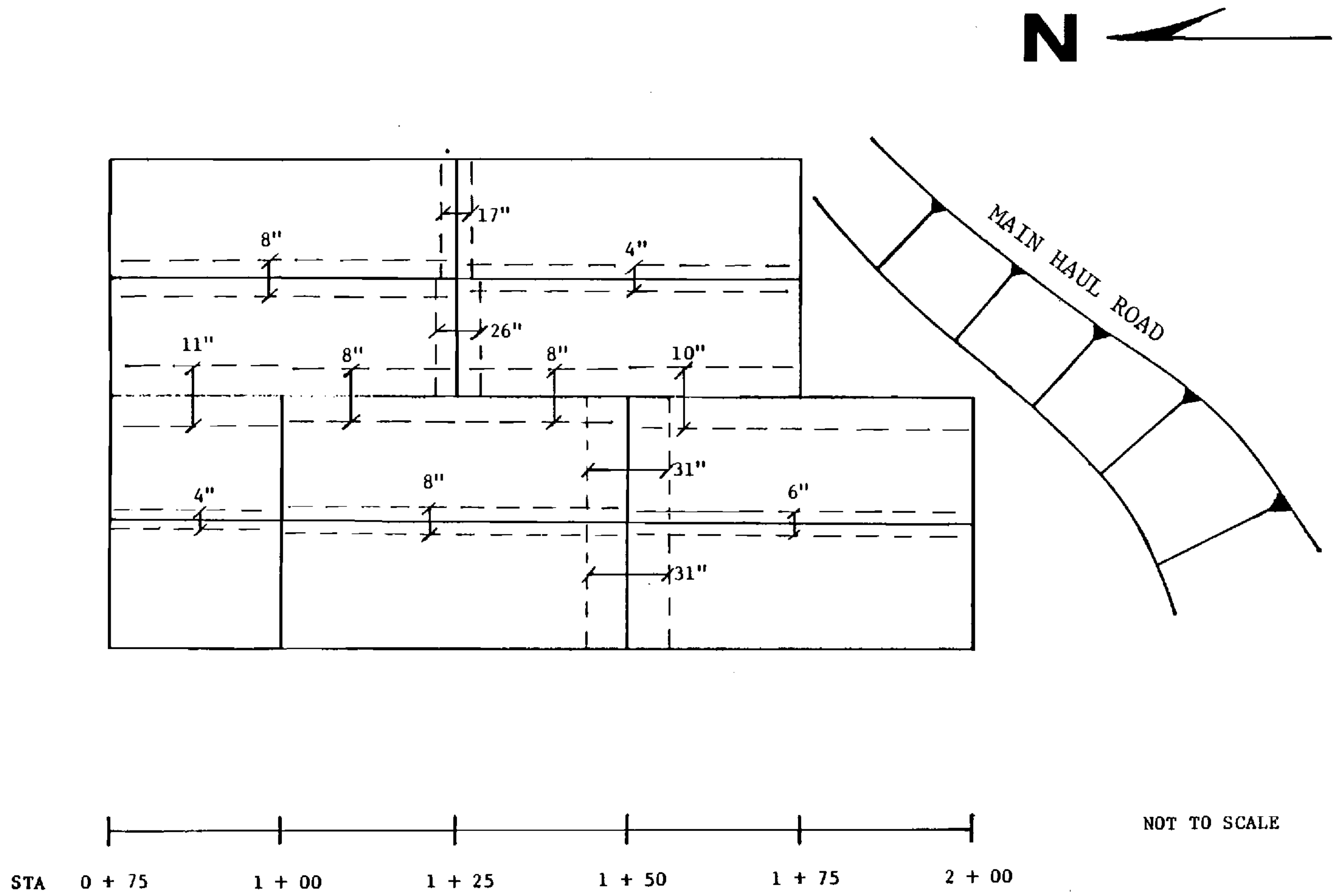


Figure 4.8. Phase I fabric overlap plan.

ahead of the main area of fill placement between the fingers. The fingers of fill pushed out were about 20 to 30 ft. in width. These fingers of fill served to anchor the two ends of the geotextile, and appeared to be reasonably effective in reducing or eliminating slack in the geotextile. Because of problems with kiting, the Amoco 2002 was not placed on windy days.

4.4.2 Sand Fill Placement. The next step in construction was to place the sand fill over the test sections and adjoining dike area. To build the haul road in the test area, the fill was end-dumped from 35 ton DJB off-highway trucks down the side of the existing haul road located along the south end of the test sections (Fig. 4.3). From there, the fill was pushed out over the test sections in an approximately 1 ft. thick initial lift using a light bulldozer. The remaining fill was pushed out using lift thicknesses of about 4 to 8 in. Lifts were generally pushed northward approximately parallel to the lane directions. Some material, however, was pushed onto the sections from the segment of the haul road running along the east side of the test site. This construction is considered representative of how a typical contractor would operate on a big project utilizing large, off-highway equipment with flotation-type tires.

The D5B bulldozer used to construct the Phase I test sections had 33 in. wide "swamp tracks", and pushed approximately an 11 ft. width of soil ahead of it. It performed well on the very soft subgrade when operating over about 1 ft. of sand fill underlain by reinforcing. A D6 bulldozer was used to construct the Phase II test sections. The narrow 22 in. wide tracks on the D6 caused the ground to roll, and this larger dozer did not perform as well as the lighter D5B with wider tracks.

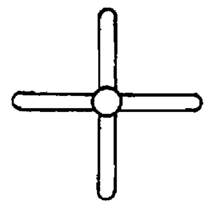
Because the fill was pushed primarily from the south edge of the test sections northward, the southern test sections received more compaction from the D5B bulldozer than the northern sections located further away from the point of dumping. This difference in compaction effort was later evident in the test results.

During fill placement, the Tensar SS-1 was observed to undergo a waving action just ahead of the fill placement. Waving was considered to be moderate, with crest heights up to 12 in. above the surface.

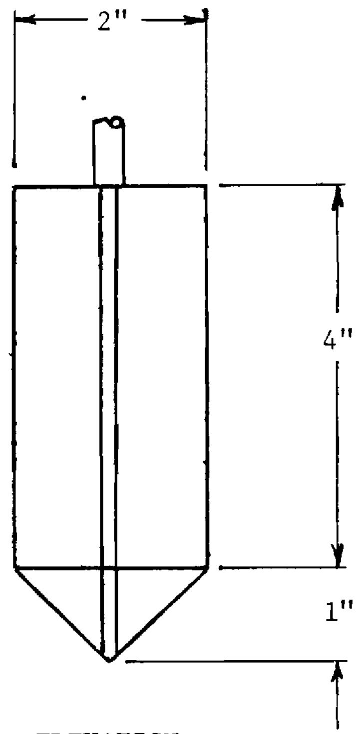
After the fill was placed, the bulldozer was used to establish finish grade for each lane. Fill thickness was determined by using a level to measure the difference in elevation at various points before and after fill placement. This technique did not, however, account for settlement of the subgrade due to the weight of the fill. Measurements indicate that subgrade settlement was generally 25 to 40 percent of total fill thickness. Thus, measured fill thicknesses were larger than originally planned.

4.5 Subgrade Shear Strength

The undrained shear strength of the in-situ soil was measured before fill placement using a 2 in. dia. field vane shear device. The vane shear apparatus consisted of a long, 0.5 in. diameter steel rod with a vane attached at the bottom end. Dimensions of the vane are shown in Figure 4.9. A "Snap-On" brand torque wrench reading to 150 in.-lb. or a "Craftsman" torque wrench reading to 600 in.-lb. was attached to the top end. Measurements were made by inserting the vane into the ground to the desired depth, rotating the vane clockwise by slowly turning the torque wrench, and reading the maximum value of torque. After reaching the peak value, clockwise movement was continued for one revolution at which time a remoulded shear strength reading was taken. The torque readings were converted



PLAN



ELEVATION

Figure 4.9. Details of the field vane shear apparatus.

to undrained shear strength using the following equation which corrects for friction on the shaft:

$$c = (5.04 - 0.0194D)T$$

where T = field torque (in.-lb.)

D = depth of vane (in.)

c = undrained shear strength (psf)

Vane shear measurements were taken in each test section and are summarized in Tables 4.1 through 4.10. The data obtained for each section were then averaged and plotted to show the typical variation of undrained vane shear strength with depth for the entire test area (Figure 4.10). The average shear strength was almost constant for the first 3 ft., varying from about 185 to 200 psf. Below 3 ft., the average strength increased significantly to 375 psf at 4 ft. and 500 psf at a depth of 5 ft. The lower 25th percentile shear strength averaged about 140 psf from a depth of 1 to 3 ft. Below 3 ft., the lower 25th percentile shear strength increased to about 300 psf at a depth between 4 and 5 ft. Remoulded shear strengths were also measured and generally ranged from 20 to 50 percent of the peak shear strength, with an average value of 35 percent of the shear strength.

A Corps of Engineers boring located approximately 250 ft. south of the southern end of the test sections indicated that the in-situ soil was a 0 blow material to a depth of at least 36 ft. The drill rod sank to this depth either by its own weight or by the weight of the hammer plus the rod. The field classification of the soil at the boring location was a very soft, grey, high plasticity clay (CH).

Table 4.1. Measured Values of Shear Strength Versus Depth for Test Section 1T.

SHEAR STRENGTH, c (psf)										
	LOCATION									
Depth (ft.)	#1	#2	#3	#4	#5	#6	Mean	Stan. Dev.	Var.	Lower 25%
1	130	120	288	120	96	132	148	69.9	4075	101
2	149	309	174	217	114	149	185	69.5	4026	138
3	185	185	467	195	169	182	230	116	11,245	152
4	401	329	288	403	298	318	340	50.5	2127	306
5	339	233	426	233	213	911	392	267	59,321	212
Mean	241	235	329	234	178	338	259			
Stan. Dev.	122	86.7	118	104	81.4	328				
Variation	11,825	6008	11,168	8673	5305	86,244				

Table 4.2. Measured Values of Shear Strength Versus Depth for Test Section 1F.

SHEAR STRENGTH, c (psf)										
	LOCATION									
Depth (ft.)	#1	#2	#3	#4	#5	#6	Mean	Stan. Dev.	Var.	Lower 25%
1	96	-	130	132	115	212	137	44.3	1573	107
2	156	-	149	149	183	183	164	17.6	247	152
3	152	-	185	182	195	139	171	23.9	455	155
4	214	-	401	318	630	164	345	184	27,002	221
5	775	-	339	911	473	403	580	249	49,689	412
Mean	279	-	241	338	319	220	279			
Stan. Dev.	281	-	122	328	221	106				
Variation	62,997	-	11,825	86,244	39,185	8924				

Table 4.3. Measured Values of Shear Strength Versus Depth for Test Section 2T.

SHEAR STRENGTH, c (psf)										
	LOCATION									
Depth (ft.)	#1	#2	#3	#4	#5	#6	Mean	Stan. Dev.	Var.	Lower 25%
1	144	–	–	288	120	130	170	78.9	4675	117
2	126	80	194	174	309	149	172	77.9	5061	119
3	174	113	174	467	185	185	216	126	13,177	131
4	493	164	493	288	329	401	361	128	13,602	275
5	213	572	378	426	233	339	360	133	14,634	270
Mean	230	232	310	329	235	241	260			
Stan. Dev.	151	229	153	118	86.7	122				
Variation	18,161	39,372	17,516	11,168	6008	11,825				

Table 4.4. Measured Values of Shear Strength Versus Depth for Test Section 2F.

SHEAR STRENGTH, c (psf)										
	LOCATION									
Depth (ft.)	#1	#2	#3	#4	#5	#6	Mean	Stan. Dev.	Var.	Lower 25%
1	240	168	144	130	-	96	156	53.9	2324	120
2	192	297	126	149	-	156	184	67.5	3641	138
3	-	208	174	185	-	152	180	23.3	407	164
4	298	339	493	401	-	214	349	105	8873	278
5	300	705	213	339	-	775	466	255	52,059	294
Mean	258	343	230	241	-	279	267			
Stan. Dev.	51.8	213	151	122	-	281				
Variation	2011	36,405	18,161	11,825	-	62,997				

Table 4.5. Measured Values of Shear Strength Versus Depth for Test Section 3F.

SHEAR STRENGTH, c (psf)										
	LOCATION									
Depth (ft.)	#1	#2	#3	#4	#5	#6	Mean	Stan. Dev.	Var.	Lower 25%
1	168	-	-	-	144	168	160	13.9	128	151
2	240	-	-	80	126	297	186	100	7518	119
3	206	-	-	113	174	208	175	44.3	1474	145
4	288	-	-	164	493	339	321	136	13,912	229
5	873	-	-	572	213	705	591	280	58,941	402
Mean	355	-	-	232	230	343	287			
Stan. Dev.	293	-	-	229	151	213				
Variation	68,642	-	-	39,372	18,161	36,405				

Table 4.6. Measured Values of Shear Strength Versus Depth for Test Section 3T.

SHEAR STRENGTH, c (psf)										
	LOCATION									
Depth (ft.)	#1	#2	#3	#4	#5	#6	Mean	Stan. Dev.	Var.	Lower 25%
1	192	96	168	168	240	433	216	116	11,208	138
2	170	120	240	297	192	332	225	80.0	5333	171
3	174	174	206	208	-	260	204	35.2	991	180
4	530	260	288	339	298	360	346	97.1	7861	281
5	580	291	873	705	300	698	574	235	46,190	415
Mean	329	188	355	343	258	417	313			
Stan. Dev.	207	85.3	293	213	51.8	169				
Variation	34,295	5815	68,642	36,405	2011	22,868				

Table 4.7. Measured Values of Shear Strength Versus Depth for Test Section 4F.

SHEAR STRENGTH, c (psf)										
	LOCATION									
Depth (ft.)	#1	#2	#3	#4	#5	#6	Mean	Stan. Dev.	Var.	Lower 25%
1	154	173	202	-	-	168	174	20.2	305	160
2	233	375	297	-	-	240	286	65.7	3242	242
3	191	221	156	-	-	206	194	27.8	581	175
4	390	575	447	-	-	288	425	120	10,744	344
5	872	271	163	-	-	873	545	381	108,878	288
Mean	368	323	253	-	-	355	325			
Stan. Dev.	296	160	122	-	-	293				
Variation	69,970	20,363	11,936	-	-	68,642				

Table 4.8. Measured Values of Shear Strength Versus Depth for Test Section 4T.

SHEAR STRENGTH, c (psf)										
	LOCATION									
Depth (ft.)	#1	#2	#3	#4	#5	#6	Mean	Stan. Dev.	Var.	Lower 25%
1	308	207	154	168	96	192	188	70.4	4133	141
2	183	503	233	240	120	170	242	135	15,292	151
3	174	152	191	206	174	174	178	18.3	279	166
4	275	616	390	288	260	530	393	149	18,524	292
5	581	252	872	873	291	580	575	269	60,343	394
Mean	304	346	368	355	188	329	315			
Stan. Dev.	165	202	296	293	85.3	207				
Variation	21,825	32,668	69,970	68,642	5815	34,295				

Table 4.9. Measured Values of Shear Strength Versus Depth for Test Section 5U.

SHEAR STRENGTH, c (psf)							
	LOCATION						
Depth (ft.)	#1	#2	#3	Mean	Stan. Dev.	Var.	Lower 25%
1	144	240	144	176	55.4	2048	139
2	229	183	160	191	35.1	823	167
3	174	152	130	152	22.0	323	137
4	370	575	329	425	132	11,580	336
5	194	271	969	478	427	121,529	190
Mean	222	284	346	284			
Stan. Dev.	88.2	169	357				
Variation	6225	22,882	102,095				

Table 4.10. Measured Values of Shear Strength Versus Depth for Test Section 5T.

SHEAR STRENGTH, c (psf)							
	LOCATION						
Depth (ft.)	#1	#2	#3	Mean	Stan. Dev.	Var.	Lower 25%
1	144	144	240	176	55.4	2048	139
2	160	229	137	175	47.9	1528	143
3	130	174	109	138	33.2	734	116
4	329	740	350	473	231	35,718	317
5	969	233	310	504	405	109,101	231
Mean	346	304	229	293			
Stan. Dev.	357	247	105				
Variation	102,095	48,652	8837				

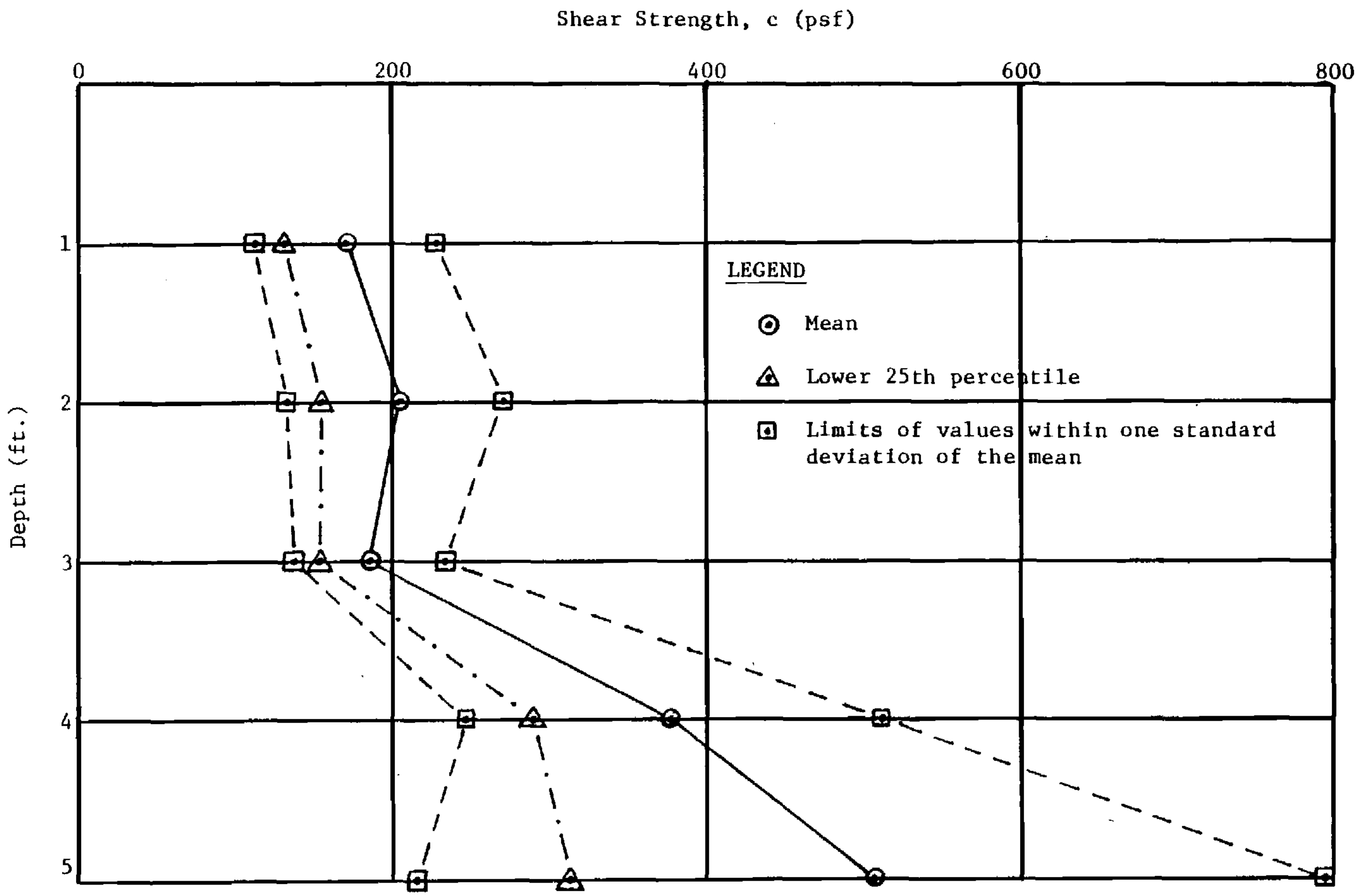


Figure 4.10. Shear strength as a function of depth - average for all test sections.

CHAPTER V

FIELD TEST RESULTS

5.1 General

Traffic tests were conducted by applying repeated truck loads until the test section failed by either excessive surface rutting or in two instances by a general bearing capacity failure of the subgrade. A rutting failure was defined as rut depths of sufficient severity to cause the frame of the vehicle to hang up (or nearly hang up) on the haul road surface. Allowable levels of rutting were determined during this study and are discussed subsequently. For this condition a bulldozer was required to free the vehicle. Frequently, the sections were regraded using a bulldozer after failure to allow for further testing.

Various field tests and measurements were performed in some or all of the test sections to evaluate the following parameters:

1. Rut depth
2. Fill strength
3. Fill density
4. Geogrid and geotextile residual strain
5. Subgrade undrained shear strength

Rut depths were measured to quantify the performance of the test sections. Measurements were made by laying a 4 ft. straightedge across the top of the rut and measuring the vertical distance from the straightedge to the deepest part of the rut using a ruler.

Fill strength was measured using a hand-held, Corps of Engineers type static cone penetrometer having a cone area of 0.5 square inches. Measurements were made by pushing the penetrometer into the fill to specified depths, and reading the deflection of a dial gauge mounted within a proving ring at the top of the apparatus. This reading can be converted to average cone pressure as follows:

$$P = \frac{F}{A} \tag{5-1}$$

where P = cone pressure (psi)

F = total force on the cone when pushed into the soil; dial gage reading is converted into force using a calibration relationship.

A = cross-sectional area of the cone tip (in.^{2})

The purpose of the cone penetrometer readings was to correlate the values obtained to the ability of the sand fill to carry the applied loads, particularly in regard to surface rutting. In some test series, cone penetrometer readings were also taken after traffic testing (usually in the ruts) to determine the effects of repeated wheel loads on the fill strength.

Fill density measurements were made using the sand cone method to determine the relative quality of fill construction. The measured values were also correlated with cone penetrometer readings in an effort to develop a relationship between them. Density measurements were made using the equipment and procedure specified by ASTM.

Residual strain in the geogrid and geotextile was measured to determine the behavior of the material under a fill layer subjected to load. Residual strain is the strain left in the reinforcing element after traffic loading has been removed. Strain was measured by attaching Bison coils, which are basically small spools wound with electrical wire, to the geogrid or geotextile and measuring the amplitude of an induced electrical current. This measurement was converted to the actual distance between the coils using a previously developed calibration curve. By comparing initial distances between pairs of coils to the distances after loads were applied, the residual strain in the geogrid or geotextile was determined.

Undrained shear strength of the subgrade was measured using the equipment and procedure described in Chapter IV. Measurements of the subgrade strength were made before the fill was placed. This information is presented in Chapter IV. Measurements were also taken in the last test section to determine the effect of fill placement and traffic loadings on the subgrade strength.

Traffic lanes 1 through 4 were constructed on June 27, 1984. Phase I testing began on the following day. On August 11, 1984, Lane 5, which included a crushed stone section, was constructed, and Phase II testing began. At the same time, Lane 4 was regraded and Phase IA of the study conducted. On October 13, 1984, an additional 1 ft. thick lift of crushed stone was placed on Section 5T and allowed to remain until Phase IIA testing was begun on November 17, 1984.

A weak area of fill was observed to exist in the northern portions of Test Sections 1T, 2T, 3F, and 4F. The weak fill area was probably due to a higher silt and clay content of the sand; the subgrade might also have been slightly weaker, although this is not readily apparent from the field vane shear data. To minimize the effects of fill and subgrade variability, comparisons of the performance of the two sections within a lane should in general be made using the test results obtained adjacent to the transverse joints separating the Tensar test sections from the Perma-Tex test sections.

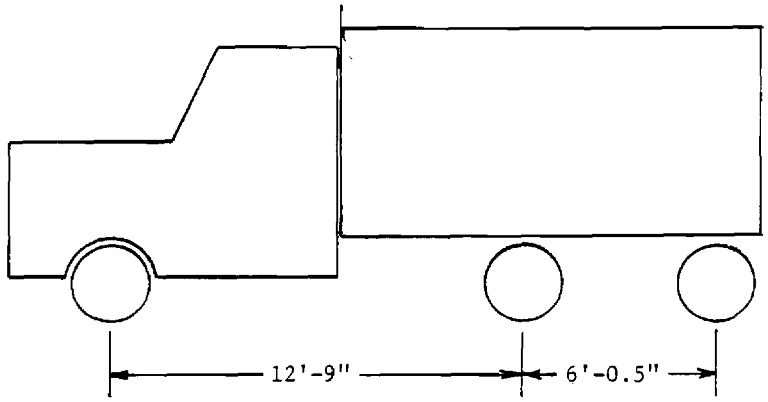
5.2 Phase I Testing

Phase I testing was conducted in Lanes 1 through 4 (Figure 4.1). Each lane consisted of one section reinforced with Tensar SS-1 geogrid, and one section reinforced with Perma-Tex 200 geotextile. Supplementary tests were performed in an area immediately east of the formal test sections. This area was reinforced solely with Tensar SS-1 geogrid.

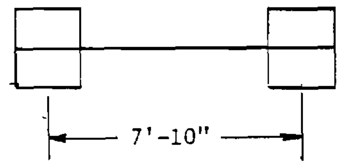
Loads were applied using a DJB 35 ton, tandem rear axle, off-highway dump truck. Figure 5.1 gives the axle configuration, dimensions, and weight distribution, and the tire characteristics. The ground clearance of the undercarriage was sufficient to allow rut depths to reach about 18 to 24 in. before the underside of the truck would hang up on the ground surface.

As discussed in Chapter IV, the southern portion of the Phase I test area received more compaction during test section construction due to the sequence of fill placement used. The resulting variation in density influenced the test results somewhat, and must be considered in analyzing the data.

5.2.1. Test Series No. I-1 - Lane 1. Test series No. I-1 was conducted on June 28, 1984, the next day after the test sections were constructed. The test was performed on Lane 1, which consisted of Test Section 1T and 1F (Figure 4.1). Section 1T was reinforced with Tensar SS-1 geogrid (indicated by a "T"), and Section 1F was reinforced with Perma-Tex 200 geotextile (indicated by an "F"). Traffic loads were applied on both sections and measurements taken of rut depth and transverse strain developed in the Tensar and Perma-Tex. All strains measured in the Tensar and Perma-Tex were residual strains that existed after the transient dump truck loading



SIDE VIEW



END VIEW

AXLE WEIGHT DISTRIBUTION (tons)		
	UNLOADED	LOADED (30T)
FRONT AXLE	11.6	19.3
CENTER AXLE	6.2	17.4
REAR AXLE	5.8	16.9

TIRE DATA

Type: Michelin 26.5 - 25XR

Tire Pressure: 45 psi

Approximate Contact Area: 24 in. X 24 in.

Figure 5.1. Tire configuration and axle loads of DJB dump truck used in Phase I.

was removed. When the truck was on the fill above the Bison gauges, the strains would have been greater than the values reported.

Average fill thickness of these sections was about 54 in. Initial cone penetrometer readings taken down the centerline of Lane 1 are given in Table 5.1.

After seven truck passes, maximum observed rut depths were 9 to 12 in. in Section 1T and 7 to 8 in. in Section 1F; Bison coil readings indicated no residual strain in the Perma-Tex. However, the Tensar SS-1 geogrid showed a residual strain of 0.5 percent. Two more truck passes were applied to the test sections, and maximum rut depths increased to 24 in. in Section 1T and 10 in. in Section 1F. At this point, the truck became stuck. Lane 1 was then regraded with a bulldozer, and one additional pass was made. Maximum rut depths of 12 in. were measured in Section 1T and 7 in. in Section 1F. A 7 in. rut depth was also recorded in the informal Amoco 2006 test area located immediately to the north of Section 1T. The truck again became stuck, so the test was terminated.

After completing Test Series No. I-1, a test pit was dug in Section 1T at the location of the 12 in. rut to expose the Tensar. Upon close inspection, it was evident that the geogrid was stretched and taking load in the transverse direction.

The greater amount of rutting in Section 1T was probably due to a lower density of the fill than in 1F, since Section 1T was located in the northern portion of the test area. The initial cone penetrometer readings for Lane 1 (Table 5.1) indicate an increase in fill strength toward the south end of the lane.

Table 5.1. Initial Cone Penetrometer Readings Along the Centerline of Lane 1.

STATION NO.	PENETROMETER READING (1/1000 in.)		
	DEPTH (in.)		
	1.5	3.5	7
0 + 75	5	22	36
1 + 00	12	16	42
1 + 25	30	45	38
1 + 50	16	25	58
1 + 75	15	49	70

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

5.2.2. Test Series No. I-2 - Lane 2. Test Series No. I-2 was conducted June 29, 1984 on Test Sections 2T and 2F of Lane 2. Test Section 2T was reinforced with Tensar, while Test Section 2F was reinforced with Perma-Tex. Average fill thickness was approximately 48 in. in these sections. Cone penetrometer readings taken along the centerline before traffic was applied are given in Table 5.2. Fully loaded DJB truck traffic was then applied to the test sections, and measurements were taken of rut depth and residual transverse strain in the Tensar and Perma-Tex.

After one pass of the dump truck, maximum rut depths measured near the joint between Sections 2T and 2F were 8 in. in the Tensar section and 9.5 to 12 in. in the Perma-Tex section. After the second pass, these rut depths became 8 to 9 in. in the Tensar section and 9 to 14 in. in the Perma-Tex section. Based on these measurements, the Tensar appeared to perform a little better than the Perma-Tex. Rut depths at the south end of Section 2F after two passes were 3 to 7 in.

Upon the second truck pass, the loaded DJB got stuck in the weak area at the north end of Section 2T at a rut depth of 18 in. The difference in performance between the two extreme ends of 2T and 2F were probably due to the higher compaction energy applied to the south end during test section construction, although this is not readily apparent from the cone penetrometer readings.

Both sections were then regraded and three additional truck passes were made. Measurements of maximum rut depths were again taken at the joint between the Tensar section and the Perma-Tex section to minimize the effect of fill strength variability. Rut depths were

Table 5.2. Initial Cone Penetrometer Readings Along the Centerline of Lane 2.

STATION NO.	PENETROMETER READING (1/1000 in.)		
	DEPTH (in.)		
	1.5	3.5	7
0 + 75	23	33	48
1 + 00	12	24	56
1 + 25	17	25	37
1 + 50	15	24	20
1 + 75	17	18	23

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

16 to 17 in. in the Tensar section and 15 to 20 in. in the Perma-Tex section. Thus, for this sequence, the Tensar again appeared to perhaps perform slightly better than the Perma-Tex.

Bison coil readings were taken at this point in the Tensar section. They indicate that the Tensar was sustaining a residual strain of 4 to 5 percent. Therefore, the Tensar was carrying lateral load at the bottom of the fill, and thereby performing properly as a reinforcement element. Bison coils were not located in the Perma-Tex section in this lane.

Rut depths measured in the Perma-Tex section at the south end of Lane 2 were 10 to 14 in. In the Tensar section at the north end of Lane 2, rut depths measured 19 to 20 in. causing the DJB to become stuck. This difference in performance is again most likely due to higher density of the fill toward the south end of the test sections. Test Series No. I-2 was terminated at this point because of the deep ruts on the north end of the lane. Slight to moderate surface rolling was observed in Lanes 1 and 2 under the loaded DJB trucks.

5.2.3. Test Series No. I-3 - Lane 3. Test Series No. I-3 was performed on Monday, July 2, 1984, five days after the test sections were constructed. Rain occurred over the weekend, and the fill surface was moist at the time of the test. In Lane 3, the positions of the sections were reversed from that used for Lanes 1 and 2 (Figure 4.1). Tensar section 3T was located over the southern half of the lane, while the Perma-Tex section 3F was located in the northern half. The sections were reversed to minimize the influence of possible variation along the lane of subgrade strength and also fill composition and density.

Fill thickness of Lane 3 was about 48 in. Cone penetrometer measurements were made on June 29 along the centerline of Lane 3 before traffic testing began. This data is presented in Table 5.3. Two cone penetrometer readings taken on July 2 suggest a significant increase in strength near the surface of the sand fill from the June 29 readings. At least some of this increase is probably due to allowing the fill to sit for a few days, during which time a rain occurred. Fully loaded DJBs were applied to the sections, and measurements were made of rut depth and residual transverse strain in the Tensar and Perma-Tex.

After three passes of the loaded DJB, rut depths near the transverse joint between the two test sections ranged from 9 to 14 in. (11.5 in.)[1] in the Perma-Tex section. Rut depths in the Tensar section were only about 6 in. The ruts measured in the Perma-Tex, however, were located adjacent to a large, soft area where excessive rutting was a problem. This soft area probably accounted for at least most of the observed difference in performance. Rut depths at the north end of the Perma-Tex section varied from 3 to 8 in. (5.8 in.). These ruts were outside of the soft area. Rut depths at the south end of the Tensar section varied from 3 to 8.5 in. (5.6 in.). From these measurements, it appears the Tensar and Perma-Tex reinforced sections demonstrated about the same performance.

After applying an additional truck pass, rut depths in the Tensar section ranged from 5 in. near the center of the section to 10 in. near the joint with the Perma-Tex section. However, the truck became stuck in the soft area of the Perma-Tex section at a rut depth of 24 in.

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1. Numbers given in parentheses are the average rut depth in inches.

Table 5.3. Initial Cone Penetrometer Readings Along the Centerline of Lane 3.

STATION NO.	PENETROMETER READING (1/1000 in.)		
	DEPTH (in.)		
	1.5	3.5	7
1 + 00	26 (27)	35 (39)	44 (54)
1 + 25	19	35	40
1 + 50	17	28	40
1 + 75	21 (19)	32 (39)	42 (65)
2 + 00	38	44	53

Notes:
1. Values in parentheses were measured three days later, just before the start of testing.
2. Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

Strain coil readings taken at this time indicate that the Tensar had a residual strain of up to 0.2 percent, while the Perma-Tex had strains from 0.8 to 2.6 percent. The higher residual strain in the Perma-Tex was probably due to the large amounts of rutting observed in that section. This test series was discontinued after a total of 4 passes due to the excessive rut depths in the Perma-Tex section.

Overall, the Tensar section performed better than the Perma-Tex. This difference is probably at least partly due to the higher strength of the fill in the Tensar section, as demonstrated by the cone penetrometer data in Table 5.3.

5.2.4. Test Series No. I-4A - Lane 4. Test Series I-4A was also conducted on July 2, 1984. The test was performed in Lane 4, which had an average fill thickness of about 42 in. Section 4T, reinforced with Tensar, was located in the southern half of Lane 4 (Figure 4.1.). Section 4F was located in the northern half of Lane 4 and was reinforced with Perma-Tex.

Initial cone penetrometer readings taken on June 29, 1984 along the centerline of Lane 4 are shown in Table 5.4. Two additional readings taken on July 2, 1984 indicate that the strength near the surface of the fill apparently increased substantially over the 3 day period, which included some rainfall.

In this test series, the first three truck passes were applied with empty DJB trucks, followed by 8 passes of fully loaded trucks. As usual, measurements were taken of rut depth and residual transverse strain in the Tensar and Perma-Tex. Additionally, cone penetrometer readings were taken in the ruts of both sections after a total of eleven passes (empty and full) had been applied.

Table 5.4. Initial Cone Penetrometer Readings Along the Centerline of Lane 4 - Test Series No. I-4A.

STATION NO.	PENETROMETER READING (1/1000 in.)		
	DEPTH (in.)		
	1.5	3.5	7
1 + 00	7 (31)	12 (40)	22 (58)
1 + 25	11	15	18
1 + 50	16	22	29
1 + 75	25 (43)	34 (43)	45 (52)
2 + 00	27	36	56

Notes: 1. Values in parentheses were measured three days after initial cone tests, just before the start of traffic loading.

2. Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

After three passes of the empty trucks, rut depth measurements were taken on each side of the transverse joint separating the two sections. Rut depths in the Tensar section varied from 0.5 to 5 in. (2.0 in.), while rut depths ranged from 1.5 to 6 in. (2.6 in.) in the Perma-Tex section. At the north end of the Perma-Tex section, rut depths ranged from 1 to 5 in. (2.3 in.). At the south end of the Tensar section, rut depths ranged from 1.5 to 4.5 in. (2.3 in.). The Tensar reinforced section (4T) appeared to perform slightly better than the Perma-Tex section (4F) in the area adjacent to the transverse joint, although the difference was not great. The Perma-Tex section performed very well in the northern portion of Section 4F, considering the cone penetrometer readings (Table 5.4) in this area were much lower compared to the readings obtained in the southern portion of Section 4T.

After five passes of the dump trucks fully loaded with wet sand, rut depths in the Tensar section near the transverse joint varied from 1.5 to 4 in. (2.5 in.). Rut depths in the Perma-Tex section ranged from 2 to 4 in. (3.3 in.) near the joint and from 1.5 to 3.5 in. (2.5 in.) at the north end. Rut depths at the south end of the Tensar section varied from 2.5 to 5 in. (3.6 in.). Therefore, at the transverse joint, where the fill strength of the two sections should be similar, the Tensar reinforced section exhibited slightly lower rut depths. The Perma-Tex reinforced section at the north end of Lane 4 had slightly lower rut depths than the Tensar section at the south end (2.5 in. compared to 3.6 in. rut in the Tensar section). On the other hand, cone penetrometer readings suggest that the fill is much weaker at the north end where the Perma-Tex was located, compared to the south end (refer to Table 5.4).

Rut depth measurements were discontinued at this point. However, cone penetrometer readings were taken in the rut paths of both sections after eleven total truck passes had been made. These data are presented in Table 5.5. The cone readings are significantly higher than values measured immediately after the fill was constructed. The cone readings, however, are only about equal to or slightly higher than the readings taken just prior to commencing the traffic tests.

The fully loaded DJB trucks were also driven over the informal test area immediately to the north of Section 4F (i.e., DJB traffic continued to move northward onto the Amoco reinforced area after leaving Lane 4). The Amoco reinforced fill supported the loads quite well, with only a slight amount of rolling observed under the truck tires. Rut depths ranged from 1 to 8 in. (2.8 in.). The approximate fill thickness of 42 in. appeared to be about the minimum required for the loads applied, assuming adequate compaction of the fill.

5.2.5. Test Series No. I-4B - Lane 4. Test Series I-4B was conducted in Lane 4 on July 4, 1984. Test Sections 4T and 4F were both regraded to give an average fill thickness of 38.5 in. Otherwise, the test format was essentially the same as Test Series I-4A. Initial cone penetrometer readings were taken on the regraded fill just before the test was started (Table 5.6). The data suggests that the fill strength was fairly uniform along the entire length of Lane 4.

For this test, measurements were taken of rut depth, fill strength, and residual strain in the Tensar and Perma-Tex sections after six passes of fully loaded DJB trucks. In addition, test pits were dug to examine both the Tensar and the Perma-Tex.

Table 5.5. Cone Penetrometer Readings Taken in the Rut Paths of Lane 4 After Eleven Truck Passes - Test Series No. I-4A.

STATION NO.	PENETROMETER READING (1/1000 in.)		
	DEPTH (in.)		
	1.5	3.5	7
1 + 00	42	56	64
1 + 25	30	35	41
1 + 50	24	40	52
1 + 75	33	43	49
2 + 00	35	52	58

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

Table 5.6. Initial Cone Penetrometer Readings Along the Centerline of Lane 4 - Test Series No. I-4A.

STATION NO.	PENETROMETER READING (1/1000 in.)		
	DEPTH (in.)		
	1.5	3.5	7
1 + 00	24	39	51
1 + 25	23	32	34
1 + 50	25	50	61
1 + 75	22	40	53
2 + 00	19	48	58

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

After 3 to 4 truck passes, the fill began to roll. Rut depths adjacent to the transverse joint between Section 4T and 4F ranged from 3 to 6 in. (4.9 in.) in the Tensar section after 3 passes. Corresponding rut depths in the Perma-Tex section varied from 2.5 to 6 in. (4.6 in.). In the northern portion of the Perma-Tex section, rut depths after 3 passes ranged from 2.5 to 8 in. (5.4 in.), while in the southern portion of the Tensar section, rut depths of 4.5 to 9 in. (6.6 in.) were observed. Differences in rut depth measurements between the two sections were not great, and the Tensar and Perma-Tex were considered to have performed equally well.

At this point, traffic testing was terminated, and penetrometer readings were taken in the rut paths of both sections (Table 5.7). In comparing this data with the readings given in Table 5.6, it appears the fill strength may have increased slightly under the action of traffic.

Two test pits were hand excavated at selected rut locations to expose the Tensar geogrid. Both pits contained water, indicating that the groundwater was draining into the fill due to subgrade consolidation by the weight of the fill. Plastic ties had been attached to the Tensar at specified intervals prior to fill placement. These ties were uncovered, and the distances between them measured to determine if residual strain existed in the Tensar. The permanent deflection basin of the subgrade was also measured in the bottom of the pit.

In the first pit, the Tensar was "tight", but measurements of the plastic ties indicated no permanent strain had occurred. Maximum measured differential deflection of the subgrade surface was 2.5 in. over a horizontal distance of 24 in. In the second pit, the Tensar

Table 5.7. Cone Penetrometer Readings Taken in the Rut Path of Lane 4 After Six Truck Passes - Test Series No. I-4B.

STATION NO.	PENETROMETER READING (1/1000 in.)		
	DEPTH (in.)		
	1.5	3.5	7
1 + 00	32	52	68
1 + 25	23	33	39
1 + 50	27	55	64
1 + 75	31	48	58
2 + 00	30	50	63

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

could be easily lifted up 2 to 3 in. by hand. Rut depth above this area was 8 in. Therefore, no residual strain was present. Maximum measured differential surface deflection in this pit was 4 in. over a horizontal distance of 42 in. The two subgrade deflection profiles are given in Appendix A. Fill thickness measured in both pits was 38.5 in.

Gage lines were sewn, using white thread, into the Perma-Tex at specified intervals for the purpose of measuring residual strain. These gage lines were uncovered in a third test pit excavated at a rut location in the Perma-Tex section. From measuring distances between marks, it was determined that no residual strain was present although the fabric was tight in this area. Maximum measured differential deflection of the subgrade was 4 in. over a distance of 36 in. The subgrade deflection profile is shown in Appendix A.

Six fully loaded DJB truck passes were also applied to the informal test area immediately to the north of Section 4F. This area was underlain by Amoco 2006 geotextile reinforcement. The fill thickness was approximately 38.5 in., about the same as the other two sections. The fill material appeared in some areas to be cleaner sand than used in the formal test section and was believed to have performed better, although the rut depths measured did not indicate this. The Amoco section started to roll under the action of traffic after about 4 truck passes. Rut depths in the fill area having the cleaner sand varied from 3 to 8 in. (5 in.); maximum rut depth outside the good area was 11 in. Maximum rut depths were 10 to 12 in. in the Tensar reinforced section and 14 in. in the fabric section. Considering all factors, the Amoco area generally performed about the same as the Tensar and Perma-Tex sections.

The 38.5 in. fill thickness used in this test series appears to be the minimum that would be necessary to accomodate, with surface regrading, on the order of 25 rapidly applied passes of loaded DJB equipment. The Amoco and Perma-Tex sections were observed to be rolling under loading somewhat more than the Tensar section; surface rolling movement of the Tensar section was considered to be slight under the six applied load applications. The fill for this test series was allowed to sit for 5 days before traffic was applied.

5.2.6. Test Series No. I-4C - Lane 4. Lane 4 was again regraded to an average thickness of 34 in. for Test Series I-4C. This series of tests was conducted on July 5, 1984, one day after Test Series I-4B. Initial cone penetrometer readings along the wheelpaths of Test Sections 4T and 4F are given in Table 5.8. Fill strength along Lane 4 was generally uniform except in the southern portion of Section 4T, where the strength was significantly higher. As with previous tests, measurements were taken of rut depth, fill strength, and residual strain in the Tensar and Perma-Tex. Also, test pits were dug to examine the Tensar and the Perma-Tex.

A total of six passes were made over the test sections using fully loaded DJB trucks. After the first two passes, rolling of the fill under the loaded DJB trucks was greater than observed in Test Section I-4B. This is probably due to (1) only a one day rest period between loading and (2) the thinner fill thickness used in the test, which increased the vertical stress reaching the very soft subgrade. Rolling in the Tensar section was slight to moderate. Rolling was moderate in the Perma-Tex section. The Tensar section thus appeared to slightly outperform the Perma-Tex section with regard to overall surface stability.

Table 5.8. Initial Cone Penetrometer Readings Along the Wheelpaths of Lane 4 - Test Series No. I-4C.

STATION NO.	PENETROMETER READING (1/1000 in.)		
	DEPTH (in.)		
	1.5	3.5	7
1 + 00			
Wheelpath 1	12	36	50
Wheelpath 2	11	32	48
1 + 25			
Wheelpath 1	18	30	34
Wheelpath 2	19	22	44
1 + 50			
Wheelpath 1	25	50	50
Wheelpath 2	25	56	55
1 + 75			
Wheelpath 1	15	31	35
Wheelpath 2	19	48	58
2 + 00			
Wheelpath 1	35	60	75
Wheelpath 2	34	66	70

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

Ground movement tended to increase with each successive truck pass. After six passes, rut depths near the transverse joint between Sections 4T and 4F ranged from 5 to 7 in. (6.1 in.) in the Tensar section. In the Perma-Tex section, rut depths varied from 5 to 8 in. (6.1 in.). Rut depths in the northern portion of the Perma-Tex section varied from 4.5 to 8.5 in. (6.3 in.). In the southern portion of the Tensar section, rut depths ranged from 6 to 7 in. (6.6 in.). Based on these results, the Tensar and the Perma-Tex appear to be approximately equally effective in limiting the severity of surface rutting.

Traffic testing was terminated because of surface rolling, and cone penetrometer measurements were again made in the wheelpaths of both sections (Table 5.9). The effect of repeated applications of heavy DJB truck loadings on the strength of the fill as defined by the cone penetrometer readings was not readily apparent.

Bison coil readings were taken to determine whether residual strain had been induced in either the Tensar or Perma-Tex by the loads applied in Test Series I-4A, I-4B, and I-4C. Strain readings in the Tensar were 2.5 and 5.8 percent. In the Perma-Tex, residual strain readings ranged from 0.9 to 4.7 percent, with an average value of 2.3 percent. Therefore, both the Tensar and the Perma-Tex, at least at the locations where the Bison coils were installed, were stretched and carrying residual tensile stresses in the bottom of the fill layer. An inspection pit was hand excavated to expose the Tensar. Plastic ties spaced 1.5 in. apart had been previously attached to the Tensar at this location to permit measuring any change in spacing between them to determine the amount of residual (permanent) strain. However, 1.5 in. of slack was present in the Tensar directly beneath the

Table 5.9. Cone Penetrometer Readings Taken in the Wheelpath of Lane 4 After Six Truck Passes - Test Series No. I-4C.

STATION NO.	PENETROMETER READING (1/1000 in.)		
	DEPTH (in.)		
	1.5	3.5	7
1 + 15 (1)			
Wheelpath 1	17	43	43
Wheelpath 2	27	35	41
1 + 40			
Wheelpath 1	15	37	65
Wheelpath 2	28	36	75
1 + 50			
Wheelpath 1	23	66	67
Wheelpath 2	23	52	52
1 +75			
Wheelpath 1	41	81	87
Wheelpath 2	9	38	50
2 + 00			
Wheelpath 1	24	47	44
Wheelpath 2	16	31	64

Notes: 1. Readings taken at location of 11 in. rut.

2. Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

centerline of the rut, so no permanent strain had occurred. The thickness of the fill was measured to be 34 in., and the water level in the pit was at the bottom of the fill layer. The maximum measured deflection of the subgrade surface was 5 in., which equaled the rut depth at the surface. Thus, the 5 in. of surface rutting was not sufficient to take the slack out of the Tensar geogrid. A subgrade surface deflection profile is shown in Appendix A.

Another inspection pit was dug to uncover a section of the Perma-Tex geotextile. Grid lines had been sewn with white thread into the Perma-Tex to permit measurement of residual strain in the Perma-Tex. Although there was no slack in the Perma-Tex, the distance between the sew marks had remained unchanged, indicating residual strain had not been induced. Maximum measured subgrade surface deflection was only 2.5 in., even though rut depths at the surface were 6.5 to 7.5 in. A subgrade surface deflection profile is given in Appendix A. Also, the measured fill thickness was 31 in., which was 3 in. less than the fill thickness in the Tensar pit. Based on these observations in the test pits, the Perma-Tex may have performed slightly better than the Tensar in limiting vertical stress on the subgrade and hence permanent rutting. A possible explanation may be that the Perma-Tex was able to develop more strain during transient truck loadings since slack did not exist in the Perma-Tex in the unloaded condition.

5.2.7. Supplementary Tests. Supplementary tests were conducted in the fill area immediately to the east of the formal test sections between Higgerson-Buchanon's haul road and Lane 1. The entire area was reinforced with Tensar SS-1 geogrid, and the average fill thickness was approximately 58 in. The supplementary tests were performed on June 28, 1984, one day after the fill was placed.

For the first test, fully loaded DJB trucks were diverted from the haul road through the test area. There was no observable movement of fill and subgrade under the truck loading. A 20 in. deep rut developed after the area had sustained 50 truck passes. The area was then regraded and sustained another 30 passes of the DJB trucks before forming a rut 24 in. deep. This area was retested the next day and supported 60 truck passes with rut depths ranging from 2 to 6 in. The area had compacted well under the traffic and was clearly adequate for the loads being applied.

In a second supplementary test, trucks were diverted through the southeast corner of Section 1F, which was reinforced with Perma-Tex, and onto the Tensar reinforced area immediately to the east of Lane 1. This was done to compare the behavior under load of similar sections containing Tensar and Perma-Tex. After subjecting the test area to eleven passes of fully loaded DJB trucks, rut depths were measured near the joint between the Tensar portion and the Perma-Tex portion. Rut depths were 2 to 3 in. in the Tensar section and 1 to 5 in. in the Perma-Tex section. The Tensar appeared to perform slightly better than the Perma-Tex, although such a conclusion could vary depending upon where the ruts were measured. Cone penetrometer readings in the test area ranged from 5 to 49 at a 1.5 in. depth, 9 to 84 at a 3.5 in. depth, and 28 to refusal at a depth of 7 in. These were significantly higher than readings recorded along the centerline of Lane 1. This accounts for the excellent performance demonstrated by both the Tensar and Perma-Tex portion of the test area.

5.3 Phase IA Testing.

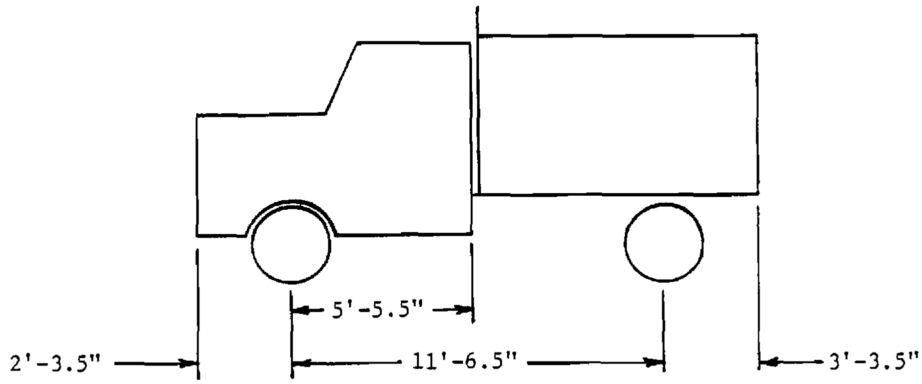
Phase IA testing was conducted on August 11 and 12, 1984, on Lane 4 of the formal test area. At this time, the fill had been in place 45 days. The average fill thickness at the time of the test was approximately 30 in. Lane 4 was reinforced with Tensar SS-1 geogrid in Test Section 4T, and with Perma-Tex 200 geotextile in Test Section 4F.

Loads were applied by driving a loaded Chevrolet C-65 box type dump truck over the sections. The total weight of the truck loaded with 7 cubic yards of wet fill sand was approximately 15 tons. The configuration, dimensions, and properties of the truck's axles and tires aré shown in Figure 5.2. The C-65 truck had sufficient ground clearance to allow rut depths to reach approximately 15 to 18 in. before the underside of the truck hung up on the ground surface.

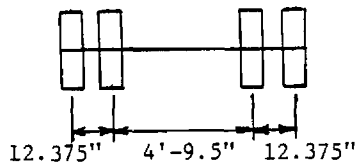
A total of five test series was conducted in Phase IA. Each series was terminated when rut depths became severe enough to prevent further traffic testing. The lane was then regraded and the next test series begun.

Cone penetrometer readings were taken along the centerline of Lane 4 to determine the initial strength of the fill (Table 5.10). The cone penetrometer readings indicate that an increase in fill strength occurred since the last Phase I test series about five weeks earlier (compare Table 5.10 with Table 5.8 and 5.9).

5.3.1. Test Series No. IA-1. A total of 30 passes of the fully loaded Chevrolet C-65 dump truck were applied to Lane 4 in Test Series No. IA-1. Cone penetrometer and rut depth measurements were made after 10, 20, and 30 passes (Tables 5.11 - 5.13).



SIDE VIEW



END VIEW

AXLE WEIGHT DISTRIBUTION (tons)			
	UNLOADED	LOADED (10T)	LOADED (8.5T)
FRONT AXLE	2.8	4.0	3.8
REAR AXLE	1.9	10.7	9.4

TIRE DATA

Type: 900 R 20
LOAD F
12 Pr

Tire Pressure: 100 psi

Approximate Contact Area: 16.5 in. X 8 in.

Figure 5.2. Tire configuration and axle loads of Chevrolet C-65 dump truck used in Phase IA and Phase II.

Table 5.10. Initial Cone Penetrometer Readings Along the Centerline of Lane 4 - Phase IA.

STATION NO.	PENETROMETER READING (1/1000 in.)		
	DEPTH (in.)		
	1.5	3.5	7
1 + 00	12	57	70
1 + 25	25	70	Refusal
1 + 50	8	70	Refusal
1 + 75	8	38	65
2 + 00	24	65	Refusal

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

Table 5.11. Rut Depth and Cone Penetrometer Measurements for Test Series No. IA-1 - 10 Passes.

TEST SECTION NO.	STATION NO.	PENETROMETER READING (1/1000 in.)			RUT DEPTH (in.)		
		DEPTH (in.)					
		1.5	3.5	7	MIN.	MAX.	AVG.
WHEELPATH NO. 1							
4F	1 + 25	18	28	33	6	6.5	6.25
	1 + 50	42	60	Refusal	2	3	2.5
4T	1 + 50	20	55	65	1	3	1.9
	1 + 75	35	41	60	2	3.25	2.6
	2 + 00	26	70	Refusal	1.5	2.5	2
WHEELPATH NO. 2							
4F	1 + 25	42	60	Refusal	1	2.5	1.8
	1 + 50	15	31	50	2	3.5	2.8
4T	1 + 50	25	27	70	2.25	3.5	2.9
	1 + 75	40	75	Refusal	1	2.5	1.8
	2 + 00	29	70	Refusal	1	3	2.2

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

Table 5.12. Rut Depth and Cone Penetrometer Measurements for Test Series No. IA-1 - 20 Passes.

TEST SECTION NO.	STATION NO.	PENETROMETER READING (1/1000 in.) DEPTH (in.)			RUT DEPTH (in.)		
		1.5	3.5	7	MIN.	MAX.	AVG.
WHEELPATH NO. 1							
4F	1 + 25	36	70	Refusal	2	4	2.8
	1 + 50	32	70	Refusal	2	3	2.7
4T	1 + 50	21	39	52	0.5	1.75	1.25
	1 + 75	42	75	Refusal	2	2	2
	2 + 00	30	50	75	-	-	-
WHEELPATH NO. 2							
4F	1 + 25	13	11	46	4	5	4.7
	1 + 50	14	29	38	0.5	4	2.3
4T	1 + 50	14	30	60	2.5	3.5	3.1
	1 + 75	38	50	65	2	3	2.3
	2 + 00	28	60	70	2.5	2.75	2.7

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

Table 5.13. Rut Depth and Cone Penetrometer Measurements for Test Series No. IA-1 - 30 Passes.

TEST SECTION NO.	STATION NO.	PENETROMETER READING (1/1000 in.) DEPTH (in.)			RUT DEPTH (in.)		
		1.5	3.5	7	MIN.	MAX.	AVG.
WHEELPATH NO. 1							
4F	1 + 25	28	65	Refusal	3	3.5	3.3
	1 + 50	14	36	65	2.5	3	2.75
4T	1 + 50	29	57	75	1	1.75	1.25
	1 + 75	19	70	Refusal	2.5	4.5	3.8
	2 + 00	33	60	65	-	-	-
WHEELPATH NO. 2							
4F	1 + 25	8	15	30	4.5	6	5.4
	1 + 50	14	24	48	5	5	5
4T	1 + 50	27	43	55	3.75	4	3.9
	1 + 75	22	36	70	2.5	5.5	3.7
	2 + 00	25	51	75	-	-	-

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

After 10 passes, rut depths in the Tensar reinforced section ranged from 1 to 3.5 in. (2.2 in.). In the portion of the Perma-Tex section adjacent to the Tensar section, rut depths varied from 2 to 3.5 in. (2.6 in.). The northern half of the Perma-Tex section rutted severely after the first 10 passes and was therefore excluded from subsequent testing.

Rut depths remained practically unchanged after 20 passes of the dump truck (Table 5.12). Rut depths in the Tensar section varied from 0.5 to 3.5 in. (2.3 in.), which was about the same as observed after 10 passes. Rut depths in the Perma-Tex area immediately adjacent to the Tensar section ranged from 0.5 to 4 in. (2.5 in.).

Rut depths increased significantly after a total of 30 truck passes were applied to the test sections (Table 5.13). In the Tensar section, rut depths ranged from 1 to 5.5 in. (3.2 in.). Rut depths in the southern half of the Perma-Tex section ranged from 2.5 to 6 in. (4.1 in.).

The magnitude of rutting in the Tensar section for this test series was slightly less than that measured in the Perma-Tex section. The difference was not great, however, and may have been due to slight differences between the two sections in strength of the fill, although this is not readily apparent from initial cone penetrometer readings (Table 5.10). For practical purposes, the performance of the Tensar and the Perma-Tex was nearly equal.

After 10 truck passes, cone penetrometer readings taken at a depth of 1.5 in. increased sharply from initial readings. Readings taken from depths of 3.5 and 7 in. did not change significantly. Readings taken at depths of 1.5, 3.5, and 7 in. after 20 passes and again after 30 passes suggested a slight to moderate decrease in fill strength with increasing truck passes. From this data, it appears that initial loadings densify and strengthen the loose upper surface of the fill. Then, however, progressive rutting acts to disturb the sand fill structure and cause at least a temporary decrease in strength. In every case, however, the fill was observed to perform better over an extended period of time and increasing number of truck passes due to a gradual increase in overall fill and subgrade strength.

After 30 passes, ruts had become deep enough to hinder the operation of the dump truck. For best operation of the Chevrolet C-65 dump truck on the fill, regrading should have been carried out at a rut depth of 4 to 5 in.

After 30 passes, the test was terminated, and both test sections were regraded in preparation for Test Series No. IA-2.

5.3.2. Test Series No. IA-2. In Test Series No. IA-2, 26 additional truck passes were applied to the regraded test sections of Lane 4. Rut depth and cone penetrometer measurements taken afterward are given in Table 5.14.

Rut depths recorded in the Tensar section ranged from 1.5 to 7.5 in. (4.5 in.). In the southern half of the Perma-Tex section, rut depths varied from 1.5 to 9 in. (5.5 in.). At the joint between the Perma-Tex and Tensar sections, rut depths were for practical purposes the same.

Cone penetrometer readings taken at the end of this test series appeared to be slightly higher than readings taken at the end of the previous series (compare the data in Tables 5.13 and 5.14). Rut depths after only 26 passes, however, were significantly higher than those measured in Test Series No. IA-1 after 30 passes. Also, during

Table 5.14. Rut Depth and Cone Penetrometer Measurements for Test Series No. IA-2 - 26 Passes.

TEST SECTION NO.	STATION NO.	PENETROMETER READING (1/1000 in.) DEPTH (in.) 1.5	3.5	7	RUT DEPTH (in.) MIN.	MAX.	AVG.
WHEELPATH NO. 1							
4F	1 + 25	33	53	53	5	6	5.7
	1 + 50	40	65	Refusal	1.5	4	2.8
4T	1 + 50	30	53	63	3	3	3
	1 + 75	28	65	Refusal	1.5	3	2.2
	2 + 00	40	44	70	3	6	4.7
WHEELPATH NO. 2							
4F	1 + 25	10	26	42	7	9	8
	1 + 50	32	40	33	3.5	7	5.7
4T	1 + 50	19	48	65	4	6	5.7
	1 + 75	24	40	65	6	7.5	6.8
	2 + 00	15	65	Refusal	1.5	7	4.5

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

the test it was observed that the fill surface appeared to be rolling more than in the previous test series. This behavior is probably primarily due to a build up in pore pressure and hence weakening of the very soft, high plasticity, silty clay subgrade caused by the repeated application of the high stress level from the dump truck.

Test Series No. IA-2 was terminated at this point. Lane 4 was then regraded for further testing.

5.3.3. Test Series No. IA-3. The fully loaded Chevrolet C-65 dump truck was driven over the regraded test sections of Lane 4 for a total of 18 additional passes in Test Series No. IA-3. The depths of resulting ruts are given in Table 5.15. The central portion of the Perma-Tex section became severely rutted, with rut depths measured up to 15.5 in. In the portion of the Perma-Tex section closest to the Tensar section, rut depths ranged from 1.5 to 6 in. (3.8 in.). Rut depths in the Tensar section varied from 1.5 to 7 in. (4.2 in.). Excluding data from the soft areas of the Perma-Tex section, performance of the Tensar and Perma-Tex was nearly equal.

Only 18 truck passes were required in this test series to reach nearly the same level of rutting obtained in Test Series No. IA-2 with 26 passes. This test series was the first conducted on August 12, 1984; rain fell the previous night and caused a visible softening of the surface and moderately lower cone penetrometer readings (Table 5.15). Undoubtedly, the rainfall affected the performance of this test series. Another possible factor is a reduction in subgrade strength due to the repetitive loadings. Due to severe rutting, the test sections were regraded in preparation for further testing.

Table 5.15. Rut Depth and Cone Penetrometer Measurements for Test Series No. IA-3 - 18 Passes.

TEST SECTION NO.	STATION NO.	PENETROMETER READING (1/1000 in.) DEPTH (in.)			RUT DEPTH (in.)		
		1.5	3.5	7	MIN.	MAX.	AVG.
WHEELPATH NO. 1							
4F	1 + 25	18	50	60	-	-	-
	1 + 50	18	30	60	5.25	6	5.6
4T	1 + 50	15	35	34	7	7	7
	1 + 75	20	60	Refusal	5	5.5	5.2
	2 + 00	12	24	60	4	6	5.2
WHEELPATH NO. 2							
4F	1 + 25	18	22	40	-	-	-
	1 + 50	18	22	40	1.5	2.5	2
4T	1 + 50	12	22	44	1.5	4	2.75
	1 + 75	8	15	60	2	2.5	2.2
	2 + 00	40	60	Refusal	1.5	4.5	2.6

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

5.3.4. Test Series No. IA-4. Before beginning this test series, surface regrading was carried out in Lane 4 on August 12 using the blade of the bulldozer. For the remaining test series on this lane, regrading was achieved by walking the bulldozer over the rutted areas without using the blade. This new technique appeared to work better than using the blade, and the surface looked good after regrading although it was still wet.

The sections were subjected to 12 additional passes of the Chevrolet C-65 dump truck. Rut depths were measured as shown in Table 5.16. In the southern half of the Perma-Tex section, rut depths varied from 3 to 11 in. (6.8 in.). Rut depths in the Tensar section ranged from 3 to 9 in. (5.4 in.). Based on these results, the Tensar appeared to slightly outperform the Perma-Tex section.

Cone penetrometer readings taken at the end of the test suggest a moderate increase in fill strength compared to the previous test series. Regrading the sections permitted additional load repetitions to be applied. However, the sections withstood fewer load applications than in the previous test series, and rutted more severely. The main cause of this declining performance is most likely temporary loss of strength of the fill and subgrade due to repetitive stress; regrading may also adversely affect performance. The test sections were regraded at this point due to severe rutting.

5.3.5. Test Series No. IA-5. In Test Series No. IA-5, 12 additional truck passes were made over the Tensar and Perma-Tex test sections of Lane 4. Rut depths were recorded and are presented in Table 5.17. In the southern half of the Perma-Tex section, rut depths ranged from 1 in. in one wheelpath to 19 in. in the other wheelpath. The

Table 5.16. Rut Depth and Cone Penetrometer Measurements for Test Series No. IA-4 - 12 Passes.

TEST SECTION NO.	STATION NO.	PENETROMETER READING (1/1000 in.)			RUT DEPTH (in.)		
		DEPTH (in.)					
		1.5	3.5	7	MIN.	MAX.	AVG.
WHEELPATH NO. 1							
4F	1 + 25	12	30	25	7.5	11	9.5
	1 + 50	25	42	50	5	9	6.3
4T	1 + 50	18	28	60	7.5	9	8.0
	1 + 75	10	34	60	7	7.5	7.3
	2 + 00	20	50	60	4.5	5	4.8
WHEELPATH NO. 2							
4F	1 + 25	12	30	60	3	11	7.7
	1 + 50	25	38	60	3	4	3.7
4T	1 + 50	18	40	60	3.5	7.5	5.3
	1 + 75	30	70	Refusal	3	4	3.3
	2 + 00	25	58	45	1.5	5	3.5

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

Table 5.17. Rut Depth and Cone Penetrometer Measurements for Test Series No. IA-5 - 12 Passes.

TEST SECTION NO.	STATION NO.	PENETROMETER READING (1/1000 in.)			RUT DEPTH (in.)		
		DEPTH (in.)					
		1.5	3.5	7	MIN.	MAX.	AVG.
WHEELPATH NO. 1							
4F	1 + 25	22	30	35	8	19	12.2
	1 + 50	20	40	40	7	11	8.6
4T	1 + 50	22	50	60	6	12	9.3
	1 + 75	20	20	50	8	13	10
	2 + 00	15	45	60	3.5	7	4.25
WHEELPATH NO. 2							
4F	1 + 25	18	50	60	2	2	2
	1 + 50	25	25	50	1	3	2
4T	1 + 50	12	22	40	1.5	9	5.1
	1 + 75	25	50	60	3.5	5	4
	2 + 00	20	30	20	5	8	6.3

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation [5-1].

average value for the section was 6.2 in. Rut depths ranged from 1.5 to 13 in. (6.5 in.) in the Tensar section. Because of the large 19 in. rut depth in the Perma-Tex section, overall mobility in the Tensar section was considerably better. At the joint between the two sections, average rut depths in the Perma-Tex section were slightly less.

Cone penetrometer readings taken at the conclusion of testing (Table 5.17) were about the same as readings taken in the previous test series. However, rutting was more severe, and fewer truck loads were applied compared to the previous test series. The wet conditions due to the rain the previous night may have caused a build up in pore pressure in the wet fill which would have become greater with successive truck passes. Phase IA testing was terminated at this point because of the large rut depths in the Perma-Tex section.

5.3.6. Phase IA Test Pits. After completing Test Series No. IA-5, a test pit was hand excavated in a rut of each of the Lane 4 test sections to examine the condition of the Tensar and Perma-Tex, and to measure permanent deformation of the surface of the subgrade. The fill thickness was also recorded. Subgrade surface deformation profiles for each of the test pits are given in Appendix A.

In the test pit dug in the Tensar section, the measured fill thickness was 31 in. The Tensar geogrid could be lifted 1.75 in. by hand, indicating no residual stress existed in the Tensar. Maximum measured deflection of the subgrade surface was about 2 in.

The fill thickness measured in the Perma-Tex test pit was 27.5 in. Maximum measured permanent subgrade surface deflection was 4.5 in. From these results, it appears that the Tensar may have been more effective in limiting the magnitude of permanent subgrade rutting, although the Tensar pit also had slightly more fill cover.

5.4 Phase II Testing.

Phase II testing was conducted on August 11 and 12, 1984, on Lane 5 of the formal test area (Figure 4.2). Lane 5 consisted of two test sections. Section 5U contained no reinforcement fill material was the same silty sand used in construction of the Phase I test sections. Section 5T was reinforced with Tensar SS-1 geogrid placed between the fill and the subgrade. The fill material was a well-graded crushed stone with maximum particle size of 1.5 in. (Figure 3.4). Average fill thickness of both sections was about 24 in. Both sections were loaded with the same Chevrolet C-65 dump truck used in Phase IA testing (Figure 5.2). The total weight of the truck loaded with 6 cubic yards of wet soil was approximately 13 tons.

5.4.1. Test Series No. II-1. Test Series No. II-1 was conducted on August 11, 1984, the same day that Lane 5 was constructed. Lane 5 was constructed using a D6 bulldozer with 22 in. wide tracks. Using this equipment, it was not possible to place and compact the unreinforced sand section as well as for Lanes 1 through 4. Also, the sand and Tensar fills seemed to be more unstable during placement than for previous lanes.

Initial cone penetrometer readings were taken along the centerline of both sections; these readings are shown in Table 5.18. As expected, the crushed stone fill had a much higher initial strength than the sand fill.

On the first pass of the Chevrolet C-65 dump truck, a punching shear failure occurred beneath one set of dual wheels just as the truck

Table 5.18. Initial Cone Penetrometer Readings Along the Centerline of Lane 5 - Test Series No. II-1.

TEST SECTION NO.	STATION NO.	PENETROMETER READING (1/1000 in.)		
		DEPTH (in.)		
		1.5	3.5	7
5U	1 + 00	2	15	50
	1 + 25	10	35	54
	1 + 50	8	32	36
5T	1 + 50	6	20	Refusal
	1 + 75	11	75	Refusal
	2 + 00	20	43	65

Note: Cone penetrometer readings given above can be converted to cone bearing pressures by using equation (1).

backed onto the unreinforced section; at this point, one side of the undercarriage of the dump truck was sitting on the surface of the fill. Testing was discontinued at this time on the unreinforced sand section. Also, about 1 yd^3 of sand was dumped from the truck leaving 6 yd^3. After 3 passes over the Tensar reinforced crushed stone fill section, only a slight amount of surface rutting had occurred on the order of 1 to 2 in. Test Series No. II-1 was discontinued for the day when one wheel of the dump truck went off the Tensar reinforced area and punched into the sand fill to a depth of about 12 to 15 in. This test series indicates that a Tensar SS-1 geogrid placed between the fill and subgrade can indeed play an important role in preventing a bearing failure.

5.4.2 Test Series No. II-2. Test Series II-2 was performed in Lane 5 on August 12, 1984. Only the crushed stone section reinforced with Tensar SS-1 was tested.

On the sixth pass, one dual wheel assembly of the Chevrolet C-65 broke the Tensar SS-1 geogrid, punching the fill into the soft subgrade. The rut depth in this area was 11 in. deep. A test pit was excavated at this location to uncover the Tensar and determine the cause of the failure. The Tensar was found to have sheared in a vertical direction along the centerline of one of the transverse webs of the geogrid. The vertical offset distance between the area which punched into the subgrade and that which remained stationary was approximately 8.5 in. The observes shearing failure of the geogrid appears to be due to an inherent weakness of the SS-1 geogrid to this type of shear. This failure mechanism certainly should be given full consideration in evaluating the reinforcement characteristics of a geogrid for haul road type applications.

In other areas of the crushed stone section, rutting was essentially negligible - on the order of 0.5 in. During successive passes, the surface cracked on a hexagonal pattern, with large cracks transverse to the direction of travel. General rolling of the crushed stone surface was considered to be severe at this time.

Vane shear strength measurements taken after the punching failure beneath the rutted area (Table 5.19) indicate that the undrained shear strength for the first 2 ft. was about equal to or even slightly less than the minimum values of shear strength observed in the vicinity of Lane 5 before the section was constructed (compare Table 5.19 with Tables 4.10 and B.1, Appendix B).

Use of 2 ft. of crushed stone fill reinforced with Tensar SS-1 geogrid over the very soft subgrade was not adequate to carry more than about 8 truck passes in the unconsolidated state. Surface rolling was severe, and wide hexagonal tension cracks developed in the surface of the stone. Nevertheless, the Tensar reinforced section was stronger than the unreinforced section which failed on the first wheel application.

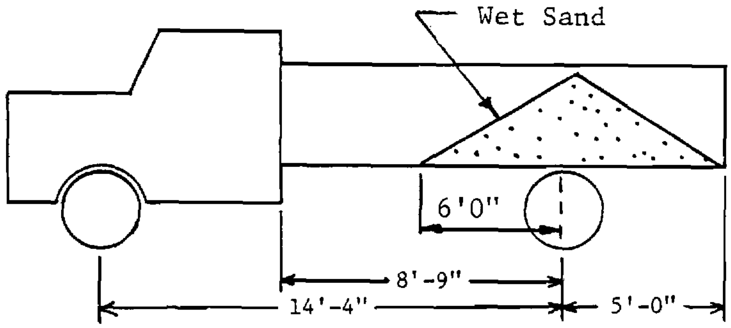
5.5 Phase IIA Testing.

Phase IIA testing was conducted on Lane 5, Section 5T. Section 5T was originally constructed on August 11, 1984 using a 2 ft. thick layer of well-graded crushed stone fill placed over a Tensar SS-1 geogrid. For Phase IIA testing, an additional 1 ft. of crushed stone was added to Section 5T on October 13, 1984. Total fill thickness for Phase IIA testing varied from about 36 in. at the south end of Section 5T to 28 in. at the north end. This section was tested on November 17, 1984.

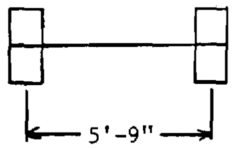
Section 5T was loaded with a Chevrolet C-50 flat bed dump truck. The truck was loaded with about 5.5 cubic yards of wet sand. The gross weight of the truck fully loaded was about 12 tons. Figure 5.3 gives the axle configuration, dimensions, and weight distribution, and the

Table 5.19. Vane Shear Strength of the Subgrade Beneath the Rutted Area of the Crushed Stone Section 5T After 6 Truck Passes - Test Series No. II-2.

DEPTH (ft.)	UNDRAINED SHEAR STRENGTH, c (psf)
0.5	123
1.0	96
1.5	117
2.0	128
2.5	557
3.0	356
3.5	283
4.0	308
4.5	519
AVG.	276



SIDE VIEW



END VIEW

AXLE WEIGHT DISTRIBUTION (tons)		
	UNLOADED	LOADED (7.8T)
FRONT AXLE	2.5	3.0
REAR AXLE	1.6	8.9

TIRE DATA

Type: 8.25 R 20

Tire Pressure: 105 psi

Approximate Contact Area: 14 in. X 8.5 in.

Figure 5.3. Tire configuration and axle loads of Chevrolet C-50 flat bed truck used in Phase IIA.

tire characteristics of this truck.

Phase IIA testing consisted of the continuous application of wheel loadings on Section 5T with a total of 365 truck passes being applied. The truck passes were distributed laterally over a total width of about 15 ft. Rut depths were measured in each wheelpath. An additional test involving 100 passes of the dump truck over the sand fill (Lane 4, Section 4T) adjacent to Section 5T was also conducted to determine the effect of repeated loads on the undrained shear strength of the subgrade.

5.5.1. Section 5T Traffic Testing. Traffic testing on Section 5T consisted of passing the Chevrolet C-50 truck back and forth over the crushed stone fill and measuring rut depths. Observations were also made concerning rolling of the fill under the wheels of the truck.

After 25 truck passes, rutting was negligible, but a slight rolling of the fill was noticeable. After 200 passes, rolling was a little more noticeable. Maximum rut depths were 2.25 in. in one wheelpath and 2.5 in. in the other. Rolling of the fill was significantly greater after 340 passes, and maximum rut depths in each of the wheelpaths was 3.5 and 4 in., respectively. The test was terminated after 365 passes due to excessive surface rolling. Maximum rut depths at this point were 2.5 and 2.75 in. in each of the two wheelpaths. Rolling was more severe in the northern part of the test section where the fill thickness was slightly less than the south end.

The test section performed much better than the the previous Phase II tests conducted on the same section. Several factors help to account for this improvement in performance. First, the truck used for the Phase IIA test had a gross load about one ton lighter than for Phase II. Second, an additional layer of crushed stone was placed to increase the average thickness of the fill from about 24 in. to 32 in. Third, the truck loads were spread laterally over about 15 ft., thereby reducing the total number of load applications at a given point.

Finally, the fill had been left on the site a sufficiently long length of time to cause a very important increase in strength due to consolidation of the subgrade. At the time of testing, the initial 2 ft. thick layer of crushed stone in Section 5T had been in place 98 days, and the additional 1 ft. thick layer was in place 35 days. To determine the increase in shear strength due to subgrade consolidation, vane shear strength values taken before fill placement can be compared with strengths observed beneath the fill at the time of Phase IIA testing (Table 5.20). The average observed gain in shear strength was 66 percent, with the increase at various depths varying from 32 to 114 percent. A statistical analysis indicates that this difference in strength is probably not related to random variation in material properties (i.e. at the 0.05 level of significance). Hence, the strength increase is more than likely due to consolidation of the subgrade.

5.5.2. Effect of Load Repetitions on Subgrade Strength. This supplementary test was conducted to determine the effect of repetitive loading on the shear strength of the in-situ subgrade soil. A loss of undrained shear strength with increasing number of load applications would help to explain the increase in surface rolling observed with increasing numbers of load applications. A test pit was dug in Lane 4 adjacent to the east edge of the crushed stone section. Initial subgrade vane shear strengths in the pit were determined. The test

Table 5.20. Undrained Shear Strength of the Subgrade Before Consolidation and After 98 Days of Consolidation.

DEPTH	UNCONSOLIDATED				CONSOLIDATED (98 days)				% INCREASE
	N	MEAN	STAN. DEV.	VARIATION	N	MEAN	STAN. DEV.	VARIATION	
0.5	8	353	102	10,511	9	465	114	12,980	31.7
1.0	8	179	47.1	2216	9	360	81.3	6605	101
1.5	8	288	111	12,286	8	410	88.7	7866	42.4
2.0	8	220	64.6	4170	9	392	66.1	4367	78.2
2.5	7	193	27.5	758	7	413	126	15,861	114
3.0	8	330	238	56,715	4	419	44.0	1938	26.9
3.5	7	396	166	27,640	-	-	-	-	-
4.0	8	287	78.2	6123	-	-	-	-	-
5.0	7	349	79.3	6283	-	-	-	-	-

Note: N denotes the number of shear strength observations.

pit was then carefully filled and 100 rear axle load applications were applied directly over the pit. The pit was then reopened and new vane shear tests were performed within about 0.5 ft. of the same location. No difference in vane shear strength was observed between the two sets of readings. Perhaps more effect would have been observed if the truck was moved back and forth over a greater distance.

Chapter VI

DISCUSSION

6.1 Joints Between Tensar SS-1 Rolls

Tieing the joint between two 150 ft. long rolls of Tensar SS-1 geogrid with polypropylene braiding requires about 12 to 14 hours or more of hard, concentrated work. Because of the expense and delay of construction, a contractor is not likely to use this procedure, particularly for haul road construction. For applications where the full strength of the geogrid must be developed at the joint, a practical procedure must be devised for connecting two rolls together.

For the very soft subgrade soil conditions present at the Savannah test site, an overlap at the joints transverse to lane direction of 17 to 31 in. was found to be adequate without tying with braiding. An overlap of 6 to 11 in. in the lane direction was found to be sufficient along the edges of the lane. Lane edges, however, in general received only a slight to moderate amount of traffic including some truck passes in a direction at an angle to that of the lane.

A careful layout plan should be developed, and roll widths selected to minimize the number of longitudinal joints, particularly at the center of a lane. Where practical, joints should be located to minimize their effect on performance. Overlapping joints rather than tieing should be done at least until a practical procedure is developed for joint connection. Tentative recommended joint overlaps are given in Table 6.1.

Table 6.1. Tentative Minimum Recommended Overlaps of Tensar SS-1 Geogrid for Haul Road Construction.

Subsurface Condition	Overlap (inches)		
	Transverse	Longitudinal Lane Centerline	Longitudinal Edges
Very soft Subgrade[1] $C<250$ psf	18	30	8
Soft to Firm Subgrade $250 \le C \le 800$ psf	24[2]	36	12

Note: 1. For very deep soft subgrades

2. Refer to Section 6.8.2 for a discussion of reinforcement of haul roads overlying better subgrades than encountered at Savannah; an alternate overlap scheme would be the same as for the very soft subgrade; more experience is needed.

6.2 Waving of Geogrid Reinforcement During Construction

Waving of the geogrid during construction was found to be an important problem, with geogrid waves being up to approximately 12 in. in height. Before fill placement a slight geogrid waving existed due to the tying procedure used to fasten the longitudinal joints together. This initial waviness was however greatly accentuated as the 1 ft. thick initial lift was pushed in by the bulldozer. Mud waving due to fill placement over the very soft subgrade and the relative great resistance to bending of the SS-1 geogrid are the probable causes of the large observed waves in the geogrid.

In an inspection pit dug in Lane 5, Section 5T, the geogrid was found to have apparently completely folded over with a 3 in. space between; a slight possibility even exists that at this location the geogrid might have sheared into. A finer aggregate observed between the two geogrid layers suggests the geogrid probably was just folded over. Folding of the geogrid occurred even though an attempt was made to pretension it. This test lane appeared to undergo more mud waving during fill placement than other areas which contributes to the waving and folding problems. The large mud waving in this area was probably due to (1) a general low shear strength in the area (refer to Table 4.10 and 5.19), and (2) use of a heavier D6 dozer with 22 in. wide tracks rather than the D5B with 33 in. wide tracks used to construct Lanes 1 through 4.

6.2.1 Minimum Cover Over SS-1. On very soft subgrades ($C \lesssim 250$ psf) a minimum cover of 12 in. should be used due to moderate to severe waving during placement. This will make fill placement in lifts possible using bulldozers. On soft to firm subgrades (250 psf $\leq C \leq$ 1000 psf) a minimum cover of 8 in. is tentatively suggested. This last recommendation is consistent with the findings of Kennedy (1984). Finally, the manufacture of a more flexible SS-1 geogrid that is less susceptible to waving would be highly desirable.

In some haul road applications the geogrid may have to be extended a short distance up an existing slope. If fill is to be pushed down the slope toward the geogrid, special attention should be given to preventing the edge of the geogrid from lifting up. Embedding the grid into the slope and covering it with fill is one practical approach.

6.3 Prestretching the Geogrid Reinforcement

An attempt was made in Section 5T to prestretch (i.e., initially take the slack out of the SS-1) by pulling the grid tight by hand and driving stakes around the edges. Because of relatively severe mud waving and rolling of the geogrid, at some locations the geogrid lifted up off the straight 0.5 in. diameter pipe and rebars used for anchors. Waving of the geogrid is believed to be a major cause of the lack of tension in the geogrid observed at several locations before and after loading. Therefore it is felt an attempt should be made where practical to prestretch the geogrid. Vertical stakes if used should have either a bend or cross bar near the top to keep the geogrid from lifting up off the stakes during fill placement, particularly over very soft subgrades. Prestretching the geogrid when placed over wide areas should be accomplished following the construction technique described in Section 4.4.1 used at the Savannah test site for the AMOCO 2002 geotextile.

Prestretching the geogrid when used for one or two lanes would be greatly complicated if a positive connection is not made between longitudinal joints. For lanes just two rolls in width, the joint could be tied with braiding at selected locations. The grid could then be stretched by hand and staked down at about 8 to 10 ft. intervals.

6.4 Haul Road Construction and Surface Rutting Performance

At the Savannah test site a sand fill construction procedure was followed felt to be representative of what a contractor would do on a large project utilizing off-highway dump trucks having low tire pressure tires. A 1 ft. thick lift of fill was initially pushed out on the geogrid, followed by approximately 4 to 6 in. thick lifts up to finish grade (refer to Section 4.4.2).

The D5B bulldozer with 33 in. wide "swamp tracks" was found to be an excellent piece of equipment for constructing haul roads on the very soft subgrade. This dozer or an equivalent one is recommended for future haul road construction on very soft subgrades. The D5B bulldozer performed better than a heavier D6 bulldozer with narrower, 22 in. wide tracks.

Initial compaction of the silty sand fill was at best marginal for efficient operation of the DJB equipment. The DJB dump truck has a gross loaded weight of about 54 tons, and carries a tire pressure of about 45 psi (Fig. 5.1). Initially DJB dump trucks quickly became stuck and required a dozer to push them out; site regrading was required at that time. With time and increasing number of truck passes, the surface became stronger, with rutting relatively rapidly becoming much less of a problem. After about 100 DJB truck passes the sand fill became an excellent haul road, provided the moisture content was maintained in the vicinity of optimum.

Conventional single rear axle dump trucks such as the C-50 and C-65 Chevrolet trucks used in this study have much lighter gross loads, but carry a tire pressure of 100 psi or more (Fig. 5.2 and Fig. 5.3). Because of the high tire pressures it would have been impractical for this equipment to work on the sand fill in its initial state. If conventional trucks are to operate on sand fills constructed over very soft subgrades such as existed at Savannah, it is recommended that the first two feet of sand fill be placed as previously described and the surface densified with a light vibratory roller. The remaining fill should be placed by a bulldozer in 6 in. lifts, and each lift also compacted using the light roller.

6.4.1 Optimum Sand Fill Material. A nonplastic, slightly silty sand having a fines content of about 9 percent was observed to give excellent performance with respect to surface rutting and overall haul road behavior. A clean sugar sand was found to be quite susceptible to rutting. A sand with fines content in the range of 12 to 19 percent (which was the material used at the test site) was moisture susceptible, and did not perform nearly as well as the slightly cleaner sand having about 9 percent fines. These findings indicate a slightly silty sand having a fines content in the range of about 6 to 10 percent would probably make the best sand haul road. The CBR test results for selected sands given in Table 3.2 support this conclusion.

For optimum performance at vehicle speeds of 15 to 25 mph, the surface of the main haul road (which passed beside the test area) generally required regrading about every 50 to 100 passes. This haul road consisted of a silty sand with about 9 percent fines. Curves were observed to be about 3 to 4 times more susceptible to surface rutting than straight segments. Because of the tendency to rut and generate excessive dust when dry, water must be added to a sand haul road during dry periods. The addition of too much water if some fines are present causes a slick surface which can lead to accidents, particularly on curves.

6.4.2 Crushed Stone Fill. The crushed stone fill used for Section 5T was constructed in the same manner as the sand fill. Surface rutting in the crushed stone section was minimal (typically less than 1 to 2 in.) even though the loaded C-65 Chevrolet dump truck having a 100 psi tire pressure was initially used on this section. Thus, a good performing crushed stone haul road can be constructed much more easily than a sand fill haul road. The well-graded crushed stone (Georgia DOT base course material) had a 1.5 in. maximum top size, about 2 to 4 percent fines, and about 35 percent passing the No. 4 sieve. This crushed stone gave excellent performance with respect to surface rutting and is recommended for future haul road construction where crushed stone is used.

6.4.3 Effect of Rut Depth on Vehicle Performance. Little information is available in the literature concerning effect of rut depth on vehicle movement. Table 6.2 gives some guidance concerning equipment mobility as a function of rut depth. The empirical relationships presented were developed from considerable field observations made during this study.

6.5 Subgrade Strength Gain with Time

The elapsed time after haul road construction was found to have a significant effect on the required thickness of fill above the very soft silty clay subgrade. Under the weight of the fill the very soft subgrade consolidated with time, and underwent an important gain in strength. In the stone fill area (Lane 5, Section 5T), the subgrade underwent an average increase in shear strength of 66 percent over the first 3 ft. of depth below the original surface (refer to Table 5.20). Thus, the lower 25 percent quartile shear strength increased from about 140 psf to about

Table 6.2. Suggested Rut Depth Criteria for Selected Earthmoving Equipment.

Type Equipment	Rut Depth, inches		
	Easy[1] Movement	Max. for Normal[2] Slow Operation	Truck[3] Stuck
35 Ton DJB Dump Truck	8	10-12	18-24
Chevrolet Dump Truck - (both C-50 & C-65)	4	6-8	15-18

Notes: 1. Recommended maximum rut depth for speeds of 10 to 15 mph.

2. Recommended maximum rut depth for speeds of 0 to 10 mph.

3. Actual rut depth at which trucks get stuck depends on rut widths and magnitude of rutting in each wheel path.

232 psf within this zone. This gain in strength was caused by an average fill stress of about 280 psf applied for an equivalent effective time of roughly 77 days.

For the most economical design, a haul road should therefore be constructed as long as practical before use. If traffic must be placed on the haul road immediately after construction, the structural thickness of the haul road can later be substantially reduced after an important part of consolidation has occurred. The subgrade beneath most haul roads constructed on very soft soils will usually have a groundwater table near the surface. Where the groundwater is high, the haul road fill should be designed so that drainage can occur of excess pore pressure into the fill. If an impervious fill is to be used, a sand blanket or other type drainage layer should be placed between the fill and the subgrade. Positive drainage will allow the important increase in shear strength to occur with time that was observed in this tudy. If drainage cannot occur, a reduction in strength with time and load repetitions could even result such as that reported by Raymond (1984) for railroad track rehabilitation applications.

6.6 Overall Haul Road Fill Stability and Punching Failures

6.6.1 Overall Stability. At the Savannah test site the overall stability of the combined fill/subgrade was found to generally control the required thickness of the Tensar, PermaTex and Amoco reinforced test sections. Overall fill instability of the fill and underlying very soft subgrade was developed due to excessive general surface movements under the moving loaded vehicles. In the descriptions of test section performance, this was referred to as rolling of the surface. The rolling action observed is somewhat analogous to the motion that would be felt by someone in a medium size boat being subjected to moderate to large waves.

The surface rolling instability mechanism was observed for sections having sufficient structural thickness to withstand a few to a moderate number of vehicle passes (up to 365 in one case) without undergoing a bearing capacity type punching failure. For this level of structural thickness, the surface rolling would be slight to possibly nonperceptable at first, but would gradually increase with number of truck passes to a moderate and then severe level of movement. For example, in the case of the 30 in. thick crushed stone section studied (Section 5.5 of this report), 365 passes were required before reaching a condition described as severe. At this point, further trafficking was considered to be unsafe from the standpoint of risk to equipment and driver safety. At that time, severe hexagonal-type cracking was evident on the crushed stone surface.

The general instability of fill/subgrade observed at the Savannah site is believed to be related to several factors. First, the Savannah site is underlain by a very soft silty clay to in excess of 36 ft. The top 3 ft. was extremely soft clay (CH) having initially an average shear strength of 185 to 200 psf, with the lower 25th percentile shear strength being 140 psf. A Corp of Engineers test boring immediately adjacent to the site showed that down to a depth of 36 ft. the very soft dredge spoil material had an SPT value of zero. The drill rod sank to this depth either under its own weight or due to the combined weight of the hammer and drill rod. This relatively great depth of very soft silty clay is felt to play an important role in the observed general surface instability phenomenon.

Additional factors perhaps related to general surface instability are believed to include: (1) a gradual build-up of pore pressure during loading, and (2) perhaps some loss of strength due to remolding. A supplementary test described in Section 5.5.2 indicated the above two effects might not be important. In these tests, however, the truck was moving back and forth over a very short distance at a slow speed. Also, surface rolling was not observed in this test. Hence, the supplementary test described in Section 5.5.2 is considered to be inconclusive.

6.6.2 Punching Failure. A punching type bearing failure was found to occur when the crushed stone fill reinforced with Tensar SS-1 geogrid was grossly underdesigned. In this case a punching failure occurred where the average measured vane shear strength in the first 2 ft. of depth ($c = 116$ psf) was less than the lowest observed strength in the area. These results suggest if the design is marginal, perhaps the lowest observed shear strength should be used for reasonably uniform subgrade conditions. For more conservative structural designs that would normally be used, the lower 25 percentile shear strength is recommended. If some localized rutting does occur in a few areas for the more conservative design, the ruts can be readily repaired.

A punching failure also occurred on the 24 in. thick, unreinforced sand fill. A fill thickness of 24 in. was considered grossly under-designed for either the reinforced or unreinforced condition. Failure occurred upon the first dual wheel application compared to a total of eight truck passes for the Tensar SS-1 geogrid reinforced crushed stone section. Thus, presence of the geogrid reinforcement improved performance.

Where minimum fill thicknesses are to be used, reinforcement with the Tensar SS-1 geogrid was observed to serve an important function perhaps not given proper credit for in the past. The geogrid reinforcement permitted the stone fill to be placed during initial construction much better than the unreinforced sand fill. Both test sections were constructed using the D6 bulldozer with 22 in. wide tracks, which exerted a considerably higher pressure than the D5B bulldozer with the 33 in. wide swamp tracks.

6.7 Required Structural Thickness

For conditions existing at the Savannah test site, structural fill thickness required for overall fill stability depended upon (1) subgrade strength which is closely related to the fill thickness and elapsed time after placement, (2) number of truck passes and rate of application, and (3) the type of equipment and magnitude of loading.

6.7.1 Loaded DJB Dump Trucks. Immediately after haul road construction (1 or 2 days), a sand fill thickness of about 54 to 56 in. was required to carry relatively large numbers of fully loaded DJB trucks (greater than about 100 truck passes in one or two days). Approximately 48 in. of fill was found initially to be adequate to carry a limited number of truck passes (about 10 to 20 passes in one day). After a 5-day wait, 42 to 48 in. of fill was required for a few to moderate numbers of truck passes. After 8 days, 34 in. to 38 in. of fill could support fully loaded DJB equipment, but was not suited for more than 4 to 6 truck passes applied relatively rapidly.

6.7.2 Loaded Chevrolet C-50 and C-65 Dump Trucks. Forty-five days after fill placement, a 30 in. thick sand fill was found on Lane 4 to be adequate to carry slightly more than 56 passes of a loaded dump truck one day and 30 passes the next day. One section in Lane 4 was reinforced with Tensar SS-1 and the other with PermaTex 200. The 30 in. structural fill thickness used is considered to be the minimum required to carry sustained loadings of this magnitude for the existing subgrade strength. The loaded dump truck was carrying about 7 yd^3 of wet sand. Some rolling of the surface was observed, but it did not become sufficient to terminate this test sequence. The shear strength of the subgrade at the time of testing was roughly 50 percent greater than its initial value (i.e., an average shear strength of about 285 psf for the first 3 ft., with the corresponding lower 25% quartile strength being about 210 psf).

A crushed stone fill 24 in. thick (Section 5T) was grossly inadequate to support loaded dump truck traffic immediately after construction (Section 5.4.2). Three passes of a loaded C-65 Chevrolet dump truck (6 yd^3 of wet sand) one day followed by six passes the next day resulted in a complete punching failure through the Tensar geogrid at one location. At this time another location had a 9.5 in. rut indicating another complete punching failure was probably eminent. This section was under-designed even for a few number of truck loads, and grossly under-designed for large numbers of loadings.

An average of 1 ft. of additional crushed stone was later added to this section, and additional consolidation of the subgrade was allowed to occur. After the addition of crushed stone, this section (5T) was tapered from about 28 to 36 in. in thickness (refer to Section 5.5.1). After an equivalent waiting period of roughly 77 days after construction, the tapered crushed stone section withstood 365 rapidly applied loaded truck passes. The Chevrolet C-50 dump truck used in this test series was loaded with 5.5 yd^3 of wet sand. The strength of the subgrade at this time was found to be about 66 percent greater than the initial strength. The average shear strength within the first 3 ft. of subgrade was therefore about 315 psf, while the corresponding lower 25% quartile shear strength was about 232 psf.

The gross required minimum structural thickness of about 30 in. for this crushed stone section is considered to be about the same as that required for the sand section tested in Series II-1 and II-2, (Section 5.4.1 and 5.4.2). This approximate evaluation roughly considers differences in truck loading, subgrade shear strength, section thickness and number of applied load passes. A thickness of at least 30 in. or slightly more would be required to withstand a moderate number of loaded trucks from the standpoint of overall section stability using either crushed stone or compacted sand fill. Thus, a sand structural fill when compacted is about as good as crushed stone with respect to overall structural stability (and probably stress distribution effects). As discussed previously, however, crushed stone fill is easier to construct and much less susceptible to surface rutting than a sand fill.

6.8 Extrapolation of Findings to Other Sites

6.8.1 Sites Similar to Savannah. The findings of this full-scale field study should be applicable to other sites having similar loadings and subsurface conditions as existed at the Savannah site. The Savannah site was underlain by over 36 ft. of very soft, organic silty clay dredge spoil. For these site conditions and heavy truck loadings, a general instability evidenced by rolling of the fill and subgrade was found to be an important mechanism usually controlling the required structural fill thickness.

Fig. 6.1 shows the variation of maximum vertical stress with depth for the loaded DJB and C-65 dump trucks. Boussinesq type stress distribution is assumed to exist. Using the simplified approach of Bender and Barenberg (1978), the modified bearing capacity factor N_c^* of equation (1) backfigured from the field tests for moderate levels of traffic tentatively appear to be about 5.5 for loaded DJB trucks and 2.6 for a single rear axle, C-65 and C-50 dump truck traffic. Significant differences in the two factors may be related to the observed general instability type failure mechanism rather than rutting of the subgrade, and perhaps also due to differences in stress distributions which actually occur in the field. Finally, the above modified bearing capacity factors are for the approximate shear strength at the time of loading, and not for an initial shear strength which often would be much smaller than the actual value.

6.8.2 Sites with Higher Subgrade Strength. Different mechanisms than observed at Savannah may very likely be found to control failure at sites underlain by higher strength soils, and perhaps where only thin strata of very soft soils are encountered. Hence extrapolation of these results to sites where significantly different subsurface conditions exist should be done with extreme caution, if at all.

The hypothesis is presented that at sites underlain by considerably better soil than encountered at Savannah, reinforced haul roads may perform in relative terms better than observed at Savannah. For such improved subsurface conditions, a thinner structural fill thickness would be required. As a result of less fill, Boussinesq elastic solid theory suggests that a greater tendency would exist for ruts in the subgrade, if they do develop, to have relatively small radii of curvature.

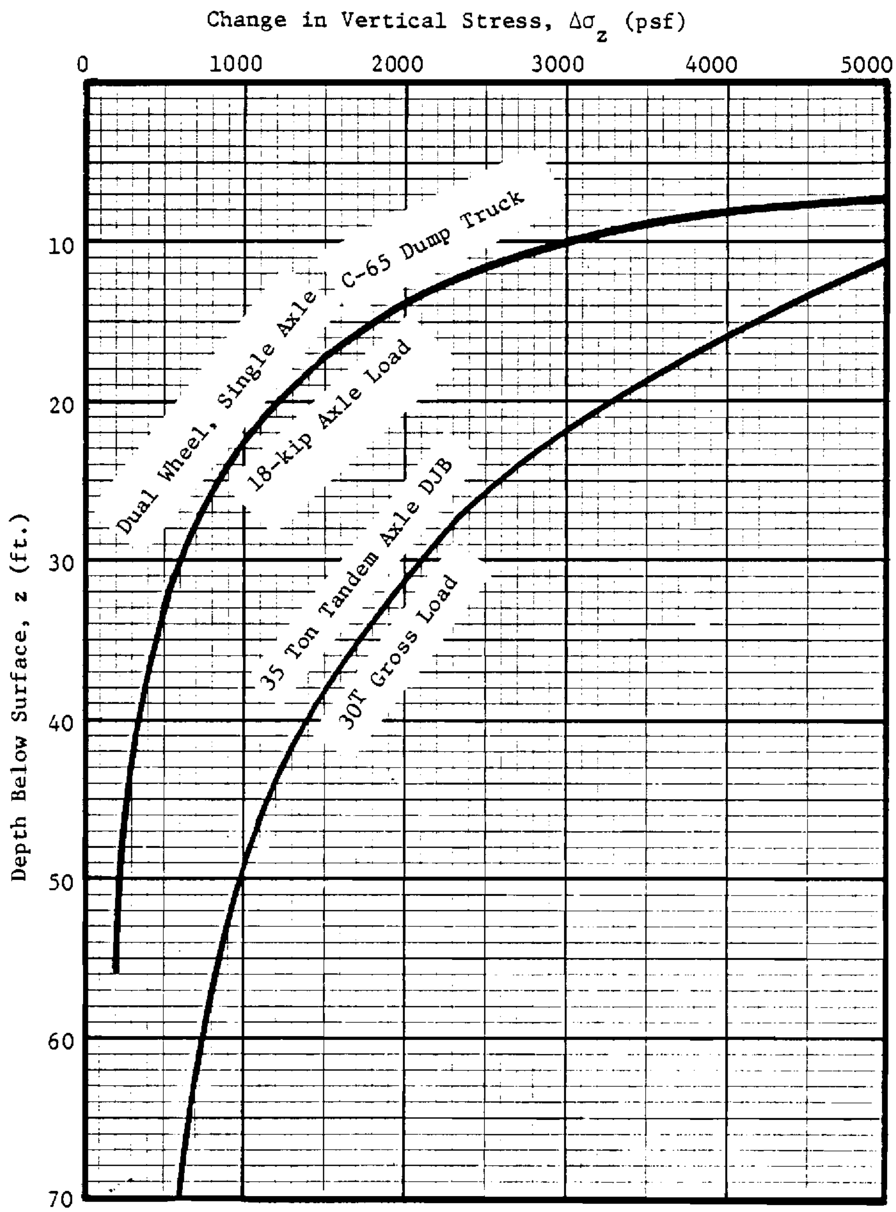


Figure 6.1. Comparison of Vertical Stress with Depth for Two Dump Trucks - Boussinesq Stress Distribution.

A sharp radius of curvature of a rut would better mobilize the forces in the reinforcement. Further, a Tensar SS-1 geogrid would probably perform better than a geotextile, provided the geogrid could be placed in an initially stretched state. Under these conditions the relative performance of the Tensar geogrid compared to the AMOCO or Perma-Tex geotextiles used at Savannah would likely be better than observed in this study.

In contrast to the hypothesized performance at a better site, the permanent deflection basin of the subgrade observed at Savannah was broad and had a quite large radius of curvature (refer to the measured subgrade profile shown in Appendix A). This gentle permanent deflection basin was probably at least partly due in the required thick depth of fill which caused considerable spreading out of stress laterally on the subgrade. To further support this hypothesis, elastic deflection basins and vertical stresses are compared in Fig. 6.2 for fill depths of 14 in. and 42 in.

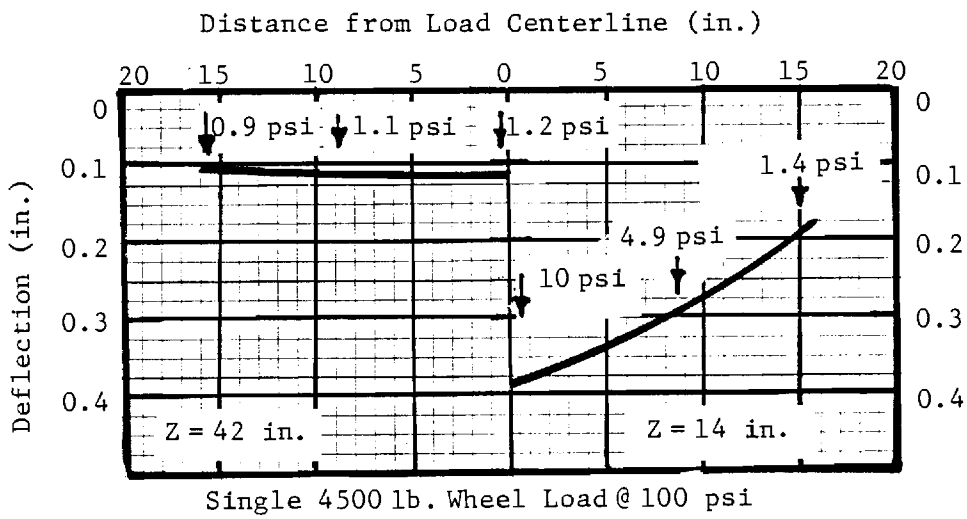


Figure 6.2. Comparison of Elastic Deflection Basin and Vertical Stress at Two Depths Below the Surface - Boussinesq Stress Distribution.

CHAPTER VII

SUMMARY AND CONCLUSIONS

7.1 Summary

The purpose of this study was to evaluate the performance of Tensar SS-1 geogrid and to compare it to selected geotextiles used as reinforcement in haul roads constructed on a very soft, highly compressible subgrade. The full-scale field tests were conducted on a dredge spoil area across the river from Savannah, Georgia. The site was underlain by more than 36 ft. of very soft, organic silty clay (CH). Before fill placement, the average shear strength in the first 3 ft. of depth was about 190 psf, with the lower 25 percent quartile shear strength being about 140 psf.

Five traffic lanes were constructed containing a total of 10 test sections. Four test sections contained Tensar SS-1 geogrid reinforcement placed under a silty sand fill; four other sections incorporated Perma-Tex 200 geotextile under the same silty sand fill; one section contained the silty sand fill without reinforcement; and one section used crushed stone as the fill material, and Tensar SS-1 geogrid for reinforcing. Several informal test areas were reinforced with AMOCO 2002 geotextile. Material properties are given for the reinforcing, subgrade soil and fill materials. Trafficking was accomplished using loaded DJB 35-ton off-highway dump trucks and Chevrolet C-50 and C-65 dump trucks. The data gathered from the tests were analyzed to determine required fill thicknesses, performance of geogrid reinforcement, and various practical aspects of haul road construction on very soft subgrades.

7.2 Conclusions

The following general conclusions were drawn from observations made during this study:

1. "Kiting" due to wind lifting up the geotextiles was a problem during test section construction. As a result, a geotextile could not be laid on windy days. On the other hand, the Tensar SS-1 geogrid did not lift up in moderate to even strong winds, and construction using the geogrid under such conditions presented no unusual problems.

2. A practical procedure is needed to rapidly connect rolls of Tensar SS-1 geogrid together. Tying rolls with polypropylene braiding requires about 12 to 14 hours or more. Until a practical procedure is developed, the joints should be overlapped rather than tied. Also, joints should be located where practical to minimize their effect on performance.

3. Waving of the geogrid during construction was found to be an important problem. Some of this waving was caused by tying the joint; mud waving during fill placement accounted for most of the waving. At one inspection pit the geogrid was found to have completely folded over with a 3 in. spacing between; it was even possible that at this location the geogrid might have sheared into. Geogrid waving resulted in areas where the geogrid had slack in it before, and in several instances after traffic loading. The manufacture of a SS-1 geogrid product less susceptible to waving would be highly desirable for optimum performance.

4. To prevent damage during fill placement due to waving, the geogrid should have a minimum depth of cover of at least 8 in. On very soft subgrades ($c < 250$ psf) a minimum cover of 12 in. should be used due to moderate to severe waving during placement, and to make fill placement possible on the soft subgrade using a bulldozer.

5. To achieve optimum, uniform performance the SS-1 geogrid should be prestretched before fill placement. Additional work needs to be done to develop suitable techniques to achieve a tight condition in the field. when only two rolls are placed together for haul road type construction.

6. The D5B bulldozer with 33 in. wide "swamp tracks" was an excellent piece of equipment for constructing haul roads on the very soft subgrade. This D5B bulldozer performed better than a heavier D6 bulldozer with narrower, 22 in. wide tracks.

7. At the Savannah Site, overall stability of the fill/subgrade which was evidenced as surface rolling was the most important failure mechanism. For this type failure mode, the silty sand fill performed from the standpoint of overall structural stability (i.e., required total fill thickness) about as good as the well-graded 1.5 in. maximum size crushed stone base course material.

   The crushed stone used at Savannah was very easy to place and showed excellent overall performance with respect to surface rutting. Use of crushed stone having a similar gradation is recommended for general haul road construction.

   The equivalency between the silty sand fill and crushed stone used at the Savannah site was that about 1.0 in. (no more than 1.1 in.) of sand was required to replace 1.0 in. of crushed stone. This equivalency does not consider surface rutting problems.

8. A good performing crushed stone haul road can, however, be constructed much more easily than a sand fill haul road. The crushed stone fill was constructed by initially pushing out a 1 ft. thick lift with the dozer, and then bringing the fill to grade by adding about 4 to 6 in. lifts. Resulting surface rutting in the crushed stone was minimal where fill thicknesses were adequate.

   Sand fills constructed in this manner were marginally suited for trafficking by fully loaded DJB equipment having low pressure tires. After a sufficient length of service, the sand hual road gradually improved into an excellent surface, provided the correct range in moisture content was maintained. Initial trafficking with loaded C-50 and C-65 dump trucks having high tire pressures is not practical using the construction technique described above. For this type loading, higher initial densification of a sand fill is required using suitable compaction equipment.

9. A nonplastic, slightly silty sand having a fines content of 9 percent gave excellent performance with respect to surface rutting when in a dense state. A clean "sugar" sand was found to be quite susceptible to rutting especially when it dried out. A sand with fines content in the range of 12 to 19 percent was more moisture susceptible, and did not perform as well as the slightly silty sand.

10. Maximum recommended rut depths for slow equipment operation is 10 to 12 in. for DJB type equipment and 6 to 8 in. for Chevrolet C-50 and C-65 type equipment. (refer to Table 6.2).

11. An important increase in subgrade shear strength on the order of 50 percent occurred over a period of about one to two months. Such change in strength with time should where possible be taken advantage of by

constructing the haul road as long as possible before use. The fill, of course, should be designed to allow drainage of excess pore pressure which is necessary for a strength gain to occur.

12. Because of safety risk to equipment operators and potential damage to equipment, a general punching failure through the fill is highly undesirable when underlain by very soft subgrade soils.

13. Measured residual strain was relatively small (typically 1 to 5%) or even non-existent in both the Tensar SS-1 geogrid and Perma-Tex 200 geotextile after several load applications. In several cases, slack existed in the Tensar and Perma-Tex. Better, more uniform performance would probably result if the geogrid or geotextile was pretightened before placing the fill. In areas where large rutting took place, the geogrid was found to be of important benefit.

14. Considering all factors, the Tensar SS-1 geogrid performed perhaps slightly better than the geotextiles. General surface rolling in sections underlain by the geogrid in several instances was observed to be less than the AMOCO and Perma-Tex geotextiles.

15. Observations from the inspection pits showed that the SS-1 geogrids and geotextile both performed effectively as a separator between the fill and the subgrade.

16. At one location a general punching failure of a dual wheel occurred through the fill and Tensar SS-1 geogrid into the subgrade. An inspection pit showed that the SS-1 geogrid sheared through the center of the ribs transverse to the direction of traffic. This type failure mechanism should be considered in evaluating the strength and overall performance of Tensar SS-1 geogrids in design.

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APPENDIX A

SUBGRADE DEFLECTION PROFILES

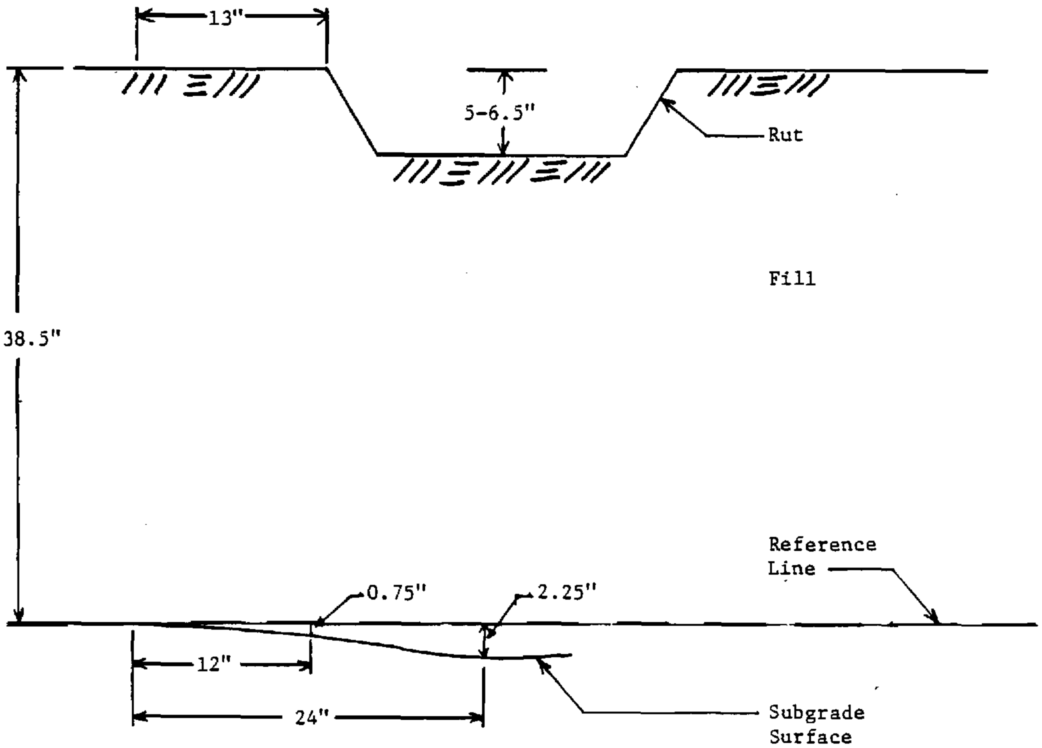

Figure A.1. Subgrade deflection profile of Pit No. 1 containing Tensar SS-1 geogrid - Test Series No. I-4B.

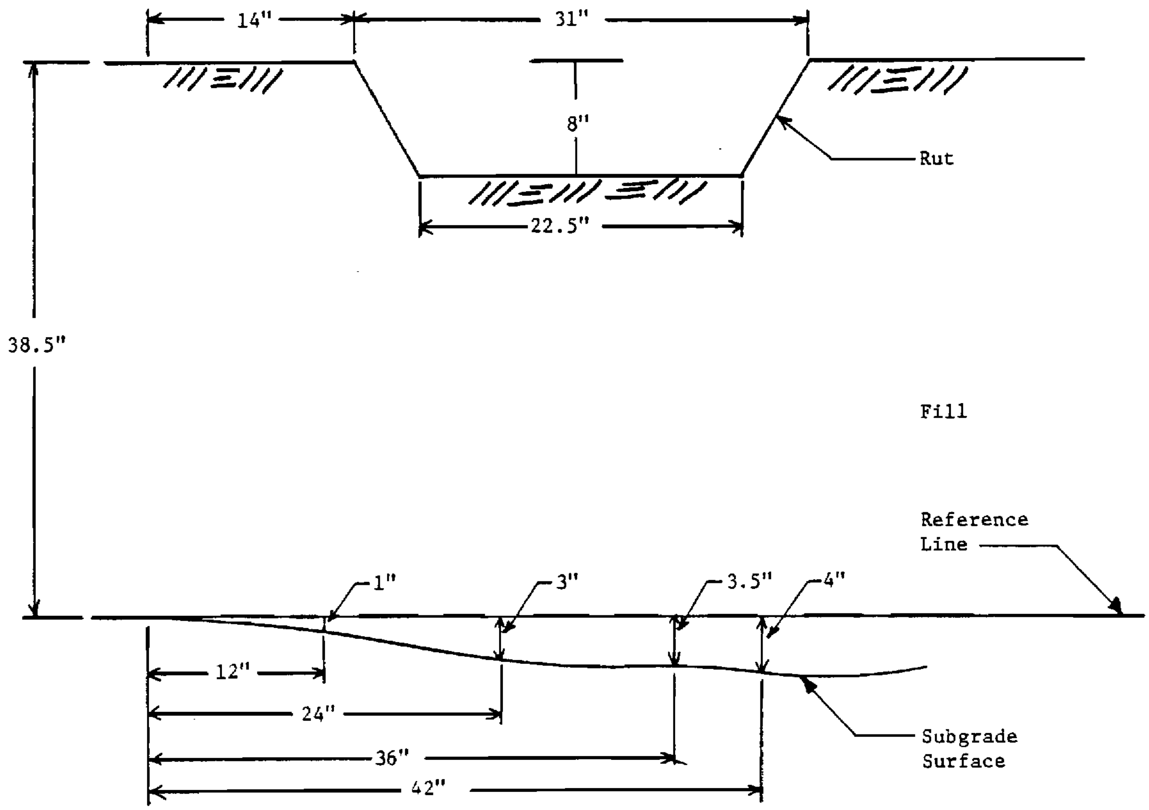

Figure A.2. Subgrade deflection profile of Pit No. 2 containing Tensar SS-1 geogrid - Test Series No. I-4B.

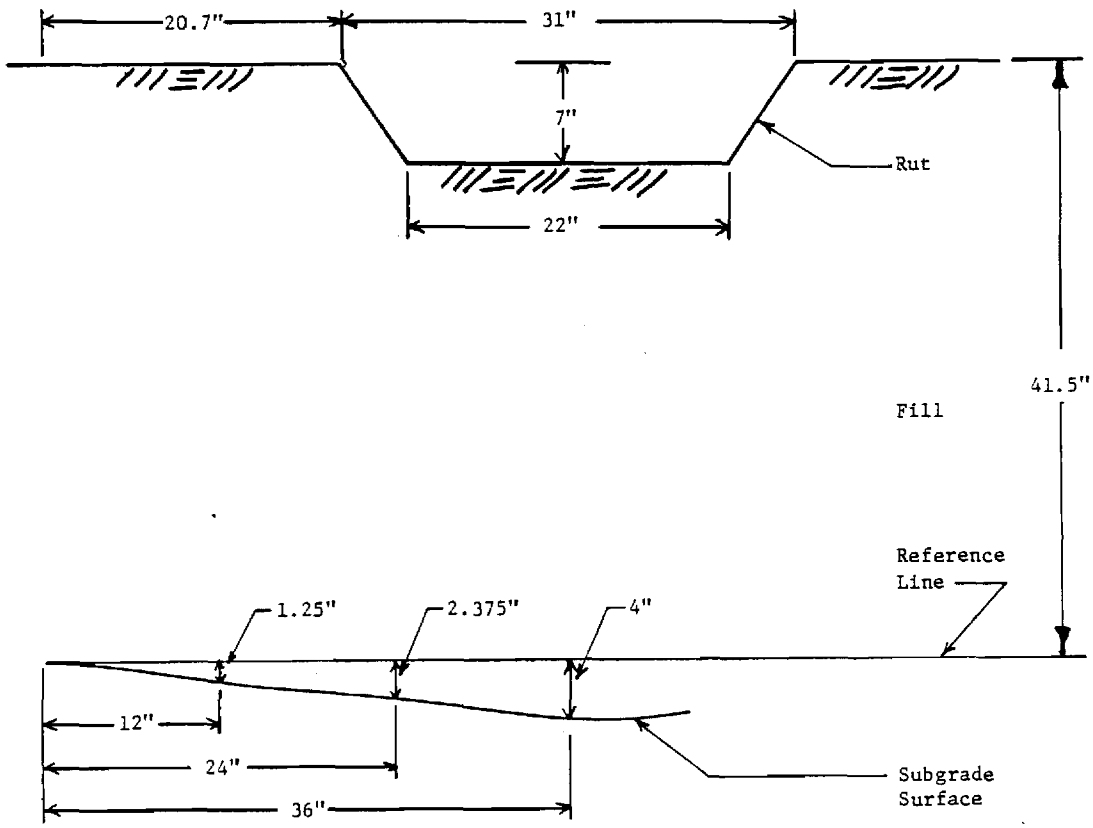

Figure A.3. Subgrade deflection profile of Pit No. 3 containing Perma-Tex 200 geotextile - Test Series No. I-4B.

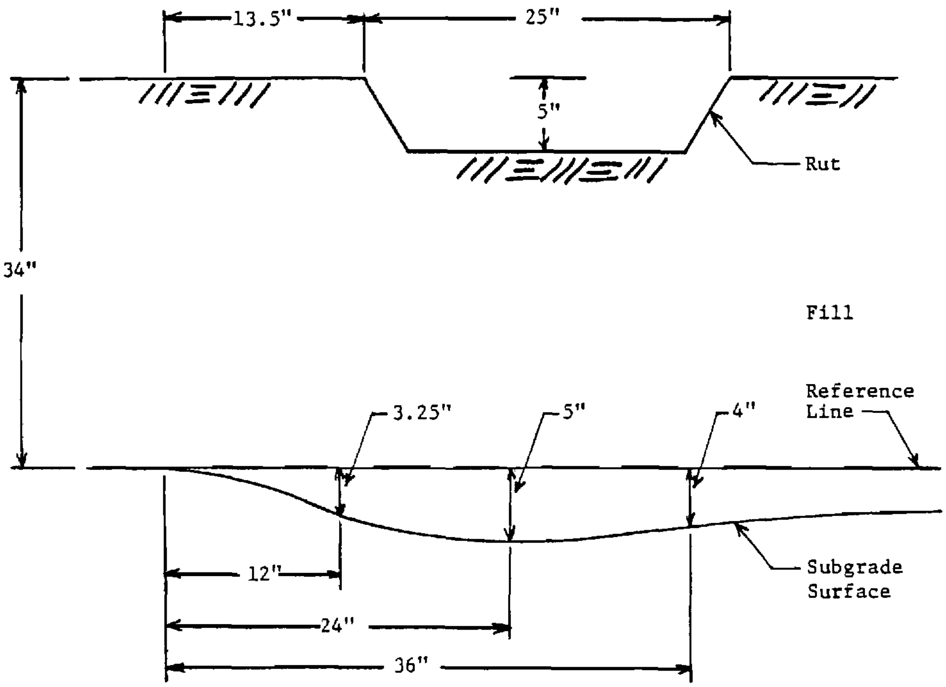

Figure A.4. Subgrade deflection profile of Pit No. 4 containing Tensar SS-1 geogrid - Test Series No. I-4C.

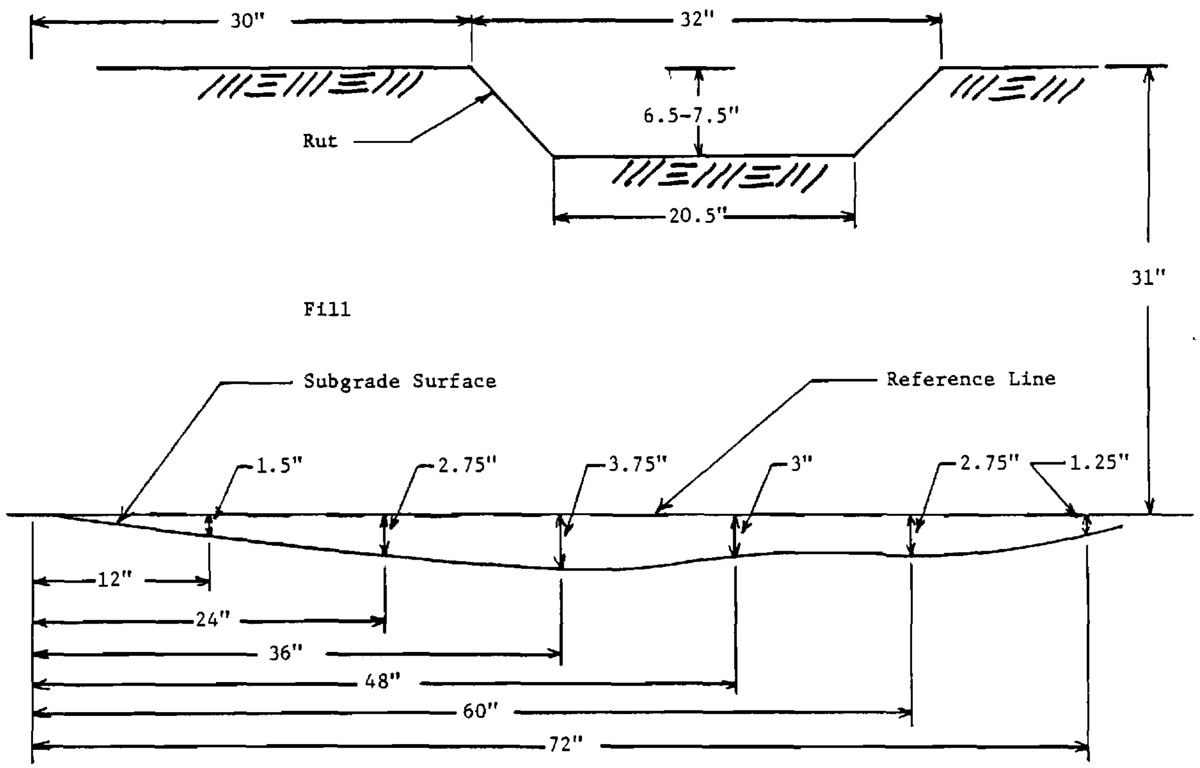

Figure A.5. Subgrade deflection profile of Pit No. 5 containing Perma-Tex 200 geotextile - Test Series No. I-4C.

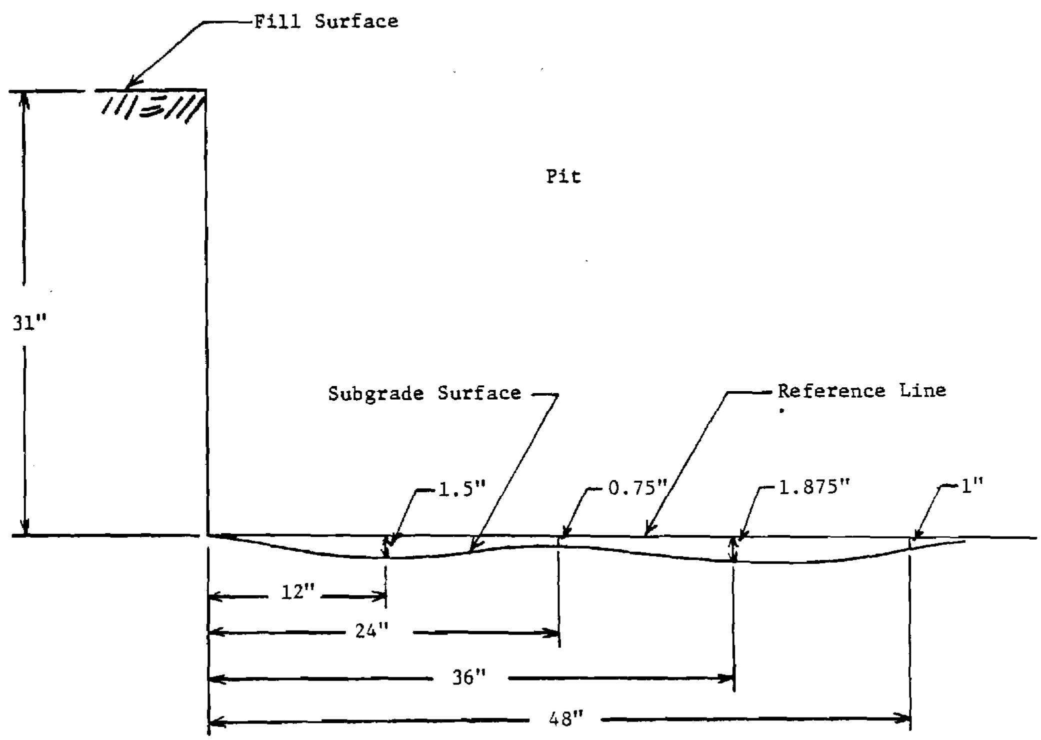

Figure A.6. Subgrade deflection profile of Pit No. 6 containing Tensar SS-1 geogrid - Phase IA.

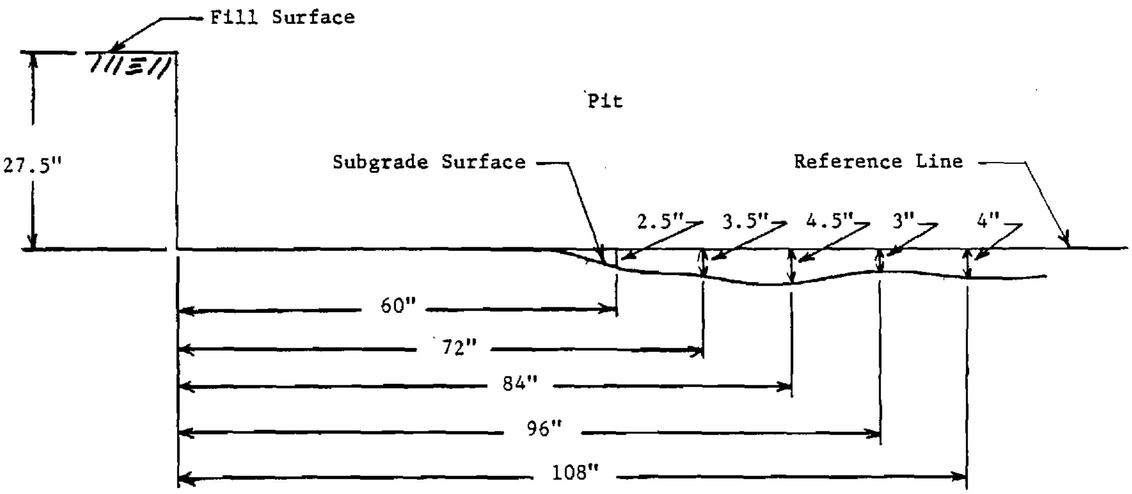

Figure A.7. Subgrade deflection profile of Pit No. 7 containing Perma-Tex 200 geotextile - Phase IA.

APPENDIX B

SUBGRADE UNDRAINED SHEAR STRENGTH DATA

Table B.1. Measured Values of Shear Strength Versus Depth Along the Centerline of Lane 5.

STATION NO.	UNDRAINED SHEAR STRENGTH, c (psf)				
	DEPTH (ft.)				
	1	2	3	4	5
0 + 75	159	316	269	263	267
1 + 00	202	297	156	447	163
1 + 25	173	375	221	575	271
1 + 50	154	233	191	390	872
1 + 75	207	503	152	616	252
2 + 00	308	183	174	275	581