



Determination of geosynthetic constitutive parameters and soil/geosynthetic interaction by in-air and in-soil experiments
by Eli Vincent Cuelho

A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering
Montana State University
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Abstract:

In recent years geogrids and geotextiles have been evaluated for use as reinforcement in the base course layer of flexible pavement structures. Adding reinforcement to the base layer can help reduce the thickness of the base material or increase the life of the roadway. To understand and quantify potential reinforcement benefit, a program of study that relies on both experimental and modeling studies was devised. The increased use of finite element modeling to simulate material and system behavior has also increased the need for more specialized tests performed on the model components.

The focus of this research was to develop test methods and provide data that can be used for the calibration of constitutive models that describe geosynthetic material parameters and soil/geosynthetic interaction parameters. Calibration of geosynthetic material models required that elastic, inelastic, viscous, direction dependant and repetitive load dependant material properties be defined. To define the soil/geosynthetic interaction characteristics, insoil (confined) tests were performed.

A large scale test facility was designed and built for performing both in-air and in-soil experiments on geosynthetics. In-air, displacement-rate controlled (monotonic) and load controlled (cyclic) tests on the geosynthetic material allowed orthotropic elastic and isotropic yield material properties to be obtained. Monotonic pull-out tests were performed at three levels of normal confinement to calibrate soil/geosynthetic interaction parameters. Several common methods of interpreting the data from these tests were examined, but, due to complications associated with these methods, a numerical solution of the governing differential equation, which describes the interaction problem, was formulated.

**DETERMINATION OF GEOSYNTHETIC CONSTITUTIVE
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IN-SOIL EXPERIMENTS**

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A thesis submitted in partial fulfillment
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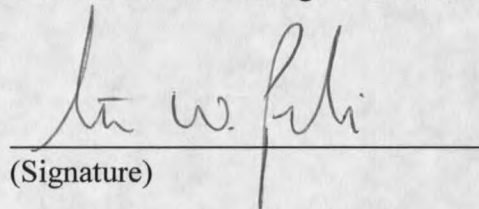
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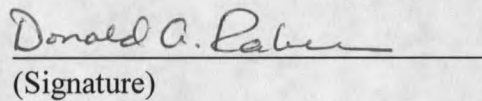
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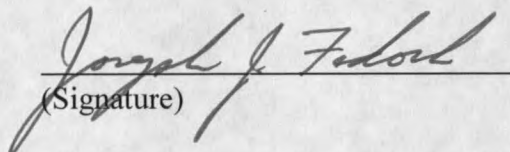
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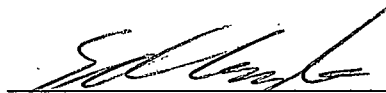
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LIST OF SYMBOLS

ABR TM	Automatic Binary Regulator
AC	Asphalt concrete
b, B	Width of the test specimen
C_1, C_2, C_3	Experimentally determined constants which help describe soil/geosynthetic interaction
d	Thickness of soil/geosynthetic interface layer
D_{85}	Diameter of soil particles at 85% finer
DC	Direct current
D_r	Relative Density
δ	Soil/geosynthetic friction angle
Δx	Length of element
E_m	Elastic modulus for the machine direction
E_x	Elastic modulus for the cross-machine direction
E_z	Elastic modulus for the out-of-plane direction
ϵ	Strain
ϵ_{lat}	Lateral strain
F	Load
F_{peak}	Peak load
F_r	Residual force at L_{eff}

LIST OF SYMBOLS -- Continued

F_{Tmax}	Maximum applied tensile force
gg45	Geogrid, 45 degree direction
ggmd	Geogrid, machine direction
ggxd	Geogrid, cross-machine direction
gtmd	Geotextile, machine direction
gtxd	Geotextile, cross-machine direction
G, G_i	Initial tangent soil/geosynthetic interface shear modulus
G_{mx}	Shear modulus for the machine/cross-machine plane
G_{mz}	Shear modulus for the machine/out-of-plane plane
G_s	Specific gravity of soil solids
G_{xz}	Shear modulus for the cross-machine/out-of-plane plane
H	Length of the test specimen
HDPE	High density polyethylene
HP	Horsepower
i	Node notation
j	Element notation
J	Analytical constant
L	Length of test specimen
L_{eff}	Effective reinforcement length
MD	Machine direction

LIST OF SYMBOLS -- Continued

n	Number of ribs
ν_{mx}, ν_{xm}	Poisson's ratios in the machine/cross-machine plane
ν_{mz}, ν_{zm}	Poisson's ratios in the machine/out-of-plane plane
ν_{xz}, ν_{zx}	Poisson's ratios in the cross-machine/out-of-plane plane
$P_{corrected}$	Corrected load
$P_{measured}$	Measured load
PET	Polyester
ψ_p	Peak soil/geosynthetic interface friction angle
ψ_r	Residual soil/geosynthetic interface friction angle
R_m	Yield stress ratio in the machine direction
R_{mz}	Yield stress ratio in the machine/out-of-plane plane
R_x	Yield stress ratio in the cross-machine direction
R_{xm}	Yield stress ratio in the cross-machine/machine plane
R_{xz}	Yield stress ratio in the cross-machine/out-of-plane plane
R_z	Yield stress ratio in the out-of-plane direction
σ_0	Ultimate reference normal strength
σ_m	Applied stress in the machine direction
$\bar{\sigma}_m$	Ultimate yield strength in the machine direction
σ_{mz}	Applied shear stress in the machine/out-of-plane direction
$\bar{\sigma}_{mz}$	Ultimate shear strength in the machine/out-of-plane plane

LIST OF SYMBOLS -- Continued

σ_n, σ_v	Applied, vertical, normal confinement
σ_x	Applied stress in the cross-machine direction
$\overline{\sigma}_x$	Ultimate yield strength in the cross-machine direction
σ_{xm}	Applied shear stress in the cross-machine/machine direction
$\overline{\sigma}_{xm}$	Ultimate shear strength in the cross-machine/machine plane
σ_{xz}	Applied shear stress in the cross-machine/out-of-plane direction
$\overline{\sigma}_{xz}$	Ultimate shear strength in the cross-machine/out-of-plane plane
σ_z	Applied stress in the out-of-plane direction
$\overline{\sigma}_z$	Ultimate yield strength in the out-of-plane direction
$\tau_{average}$	Average interface shear stress
τ_{res}	Residual interface shear stress
τ_{ult}	Ultimate interface shear stress
τ_0	Ultimate reference shear stress
u	Displacement
$W_{corrected}$	Corrected width
$W_{initial}$	Initial width
x	Position
X	Distance along length of embedded geosynthetic
XD	Cross-machine direction

ABSTRACT

In recent years geogrids and geotextiles have been evaluated for use as reinforcement in the base course layer of flexible pavement structures. Adding reinforcement to the base layer can help reduce the thickness of the base material or increase the life of the roadway. To understand and quantify potential reinforcement benefit, a program of study that relies on both experimental and modeling studies was devised. The increased use of finite element modeling to simulate material and system behavior has also increased the need for more specialized tests performed on the model components.

The focus of this research was to develop test methods and provide data that can be used for the calibration of constitutive models that describe geosynthetic material parameters and soil/geosynthetic interaction parameters. Calibration of geosynthetic material models required that elastic, inelastic, viscous, direction dependant and repetitive load dependant material properties be defined. To define the soil/geosynthetic interaction characteristics, in-soil (confined) tests were performed.

A large scale test facility was designed and built for performing both in-air and in-soil experiments on geosynthetics. In-air, displacement-rate controlled (monotonic) and load controlled (cyclic) tests on the geosynthetic material allowed orthotropic elastic and isotropic yield material properties to be obtained. Monotonic pull-out tests were performed at three levels of normal confinement to calibrate soil/geosynthetic interaction parameters. Several common methods of interpreting the data from these tests were examined, but, due to complications associated with these methods, a numerical solution of the governing differential equation, which describes the interaction problem, was formulated.

CHAPTER ONE

INTRODUCTION

Background

In recent years, geosynthetics have been used as reinforcement contained in the base course layer of flexible pavements. To quantify potential benefit provided by this reinforcement, understanding the mechanisms of interaction between the geosynthetic and the surrounding base material is essential.

Using geosynthetic materials as reinforcement in the base layer of flexible pavement systems may provide savings either through a reduction of the base course layer design thickness or an extension of the life of the road. Studies ranging from outdoor, field-scale tests to indoor, laboratory-scale tests have been conducted to provide insight into the mechanisms of geosynthetic reinforcement and its level of structural contribution. Reinforcement is derived mainly from the interaction between the base course material and the geosynthetic reinforcement. The goal of ongoing research at Montana State University is to understand and quantify potential reinforcement benefit using laboratory experiments and finite element modeling.

A fully instrumented, laboratory-scale test facility has been constructed to examine geosynthetic reinforcement benefit under simulated, truck traffic loading. This test facility

has allowed a geosynthetic reinforced pavement section to be built on a subgrade material within a controlled environment. In these test sections, cyclic loads were applied to the top of the asphalt concrete (AC) layer through a rigid, 305 mm diameter plate. Two types of geosynthetics were used in this study: a biaxial, polypropylene geogrid and a biaxial, woven, polypropylene geotextile. This phase of the research measured stress and strain response in several layers of the pavement section under these loads. Specifically, stress was measured in the base course and subgrade layers and strain was measured in the AC layer, the base course layer, the geosynthetic layer and the subgrade layer. The modeling portion of this study is being developed to match these observed stress and strain responses. Calibrating the model until it accurately predicts the response seen in these tests will allow a parametric study to be formed to predict responses for variables not included in the actual experiments. Using this parametric study, a design methodology for geosynthetic reinforced pavements can be formulated.

To model the reinforced pavement system properly, material parameters describing each component of the system must be defined. Specifically, the intrinsic geosynthetic material properties and soil/geosynthetic interaction properties must be identified. The focus of this thesis is the development and implementation of a testing scheme that provides the appropriate geosynthetic constitutive material parameters to the finite element model.

Scope of Work

Geosynthetic materials exhibit elastic, inelastic, viscous, direction dependent and repetitive load dependent characteristics. The overall purpose of the research contained in

this thesis is to develop test methods and provide data that can be used to obtain appropriate geosynthetic material properties used in the constitutive model, including base course soil/geosynthetic interaction properties. Specifically, the material parameters needed are:

- Geosynthetic, orthotropic, elastic/plastic material properties.
- Geosynthetic, nonlinear, combined isotropic/kinematic hardening material properties.
- Soil/geosynthetic, elastic/plastic, Coulomb type friction coefficients.

To accomplish this, the following objectives were established:

- Design and commission of an apparatus, based on current design practices, to perform both in-air and in-soil tests under monotonic and cyclic loading.
- Use data from in-air tests to calibrate constitutive geosynthetic material parameters in the finite element model.
- Use data from monotonic in-soil tests to formulate an analytical model to establish a method of interpreting the soil/geosynthetic interaction properties to be used in the finite element model.

Experimentation

To define geosynthetic material and interaction properties properly, performing tests on each of the geosynthetic materials used in the plate-load tests was necessary. To do this, two types of tests were preferred: in-air tests and in-soil tests. Each of these tests was conducted under two types of loading: displacement controlled (monotonic) and load controlled (cyclic). The materials studied were biaxial, implying they have different mechanical properties in each of their principle directions. Accordingly, all tests were performed to define the material's anisotropic characteristics.

In-air Tests

The materials' orthotropic, elastic/plastic material properties were defined using monotonic, in-air tests. In this thesis, an in-air test refers to any laboratory experiment performed on the geosynthetic where no confinement is applied in the out-of-plane direction. Since calibration of the constitutive model requires data corresponding to a state of uniaxial tension, all in-air tests were performed as uniaxial tension element tests. Measurements of applied load, axial strain and lateral strains were collected to calculate moduli values for each of the principle directions. Two types of monotonic tests were performed: fast monotonic and slow monotonic. Fast monotonic tests were performed to provide geosynthetic material parameters without creep. Slow monotonic tests were used to calibrate both the elastic and plastic material properties and verify the finite element model's prediction of creep.

Since the elastic/plastic model, calibrated by the monotonic tests, does not necessarily account for material behavior under cyclic loading, a second model is being developed. This second model is being used to define the materials' nonlinear, combined isotropic/kinematic, hardening material properties. To calibrate this model, geosynthetic samples were cyclically loaded, allowing them to experience ratchetting effects. Ratchetting refers to the accumulation of plastic strains within the material under a particular level of repeated load amplitude. Cyclic tests were carried out on the geosynthetic materials using two types of loading: single cycle step loading (Series I) and multiple cycle step loading (Series II). For the Series I tests, the geosynthetic samples were subjected to one cycle at 14 different levels of load amplitude. For the Series II tests, load cycles were repeated at each of these same

levels of load amplitude until it was determined that plastic strains due to ratchetting were fully developed. The Series I tests were used to further investigate the reliability of the material parameters derived from the Series II tests.

In-Soil Tests

To calibrate the elastic/plastic Coulomb friction law describing the interaction between the base course soil and the geosynthetic, in-soil tests were performed. Like in-air tests, in-soil tests were also conducted under two types of loading: monotonic and cyclic. In-soil tests were performed using a pull-out box. This apparatus allowed an embedded sample of geosynthetic to be pulled from the soil while measurements of applied load and displacement along its embedded length were collected. The soil used in these tests was the same as the gravel base course used in the pavement test facility.

Analyzing soil/geosynthetic interaction along the length of the geosynthetic is complicated by the high elongation of the geosynthetic material during pull-out testing. Since engagement of the sample is progressive, shear stress cannot be directly measured between the soil and the geosynthetic. Several related analysis techniques for determining the soil/geosynthetic interaction parameters have been attempted and used in the past. Each of these methods has been investigated and outlined in Chapter 2. Associated problems with several proposed methodologies influenced the author to develop a method of calculating soil/geosynthetic interaction based on numerical techniques.

The interaction parameters were determined from monotonic tests performed on each geosynthetic material in their principle directions. These tests were performed under three

different levels of normal confinement, since the friction law is normal stress dependant. Data from each of these tests was used within the newly developed numerical analysis to calculate interaction parameters for each of the material's principle strength directions. Data from cyclic, in-soil tests will be used to validate and calibrate finite element predicted, interaction parameters under cyclic loads.

Organization of Thesis

A critical review of existing literature for the design, use and interpretation of data obtained from in-air and in-soil testing facilities is contained in Chapter 2. A detailed description of the design and construction of the apparatus used in this research is included in Chapter 3. Chapter 3 also describes the experimental procedure used for preparing and performing these tests, illustrates corrections made to raw data and gives examples of typical results. A discussion of experimental results and an in depth description of the analytical methods used to determine material and interaction parameters from each of these tests is outlined in Chapter 4. Conclusions drawn from this study and recommendations for further research are made in Chapter 5.

CHAPTER TWO

LITERATURE REVIEW

Introduction

The literature review in this chapter consists of two main sections; in-air testing and in-soil testing of geosynthetics. Each section includes a discussion of previous apparatus design, test procedures, and analysis of results. Problems associated with existing test methods and analyses are also discussed.

In-Air Testing

Due to the various factors affecting results obtained from in-air tests, careful consideration must be given to the design and construction of an in-air apparatus. In this review, four areas are considered: 1) the overall geometry of the test specimen, 2) the method of applying the load, 3) the method of gripping the sample and 4) the temperature during testing.

Many geosynthetics experience necking under uniaxial loading. To reduce this effect, sample widths are typically increased so that the experimental setup mimics a plane-strain condition. This condition has been argued to be more closely related to field applications (ASTM, 1995) where geosynthetics in tension are generally very wide. However, calibration

of a constitutive model requires data corresponding to a state of uniaxial tension. Samples pulled under plane-strain conditions may yield different results than samples pulled uniaxially. Samples with a small aspect ratio resemble uniaxial loading since material in the direction of the sample's width is much smaller than the length, therefore reducing its effect on load/strain properties. Choosing a sample size that ensures a state of uniaxial tension over the central region of the sample allows axial and lateral strains to develop freely without influence from the ends. Measured lateral strains allow the load to be corrected for changes in width due to Poisson effects.

Changes in sample geometry have been shown to affect the measured properties of geosynthetics. Myles (1987) showed that strength per unit width decreased in a geotextile when the aspect ratio (B/H) was lowered, where B and H are the width and height (or length) of the test specimen, respectively. Rowe and Ho (1986) showed that initial tangent modulus values for a woven geotextile stayed relatively constant under different aspect ratios. Figure 2.1 illustrates that tangent modulus values from 5-10% axial strain decreased as the aspect ratio became larger.

Both strain-rate controlled and displacement-rate controlled test apparatuses are used to determine the load-strain behavior of geosynthetics. Geosynthetic materials are known to be load-rate sensitive materials. Rowe and Ho (1986) showed that the maximum tensile strength of geotextile materials is quite sensitive to strain rates, where increased strain rate increases the maximum tensile strength, as shown in Figure 2.2. Bathurst and Cai (1994) found that increased loading rate of an HDPE geogrid increased secant and tangent moduli values and its ultimate strength (Figure 2.3). When load magnitudes are applied in a cyclic

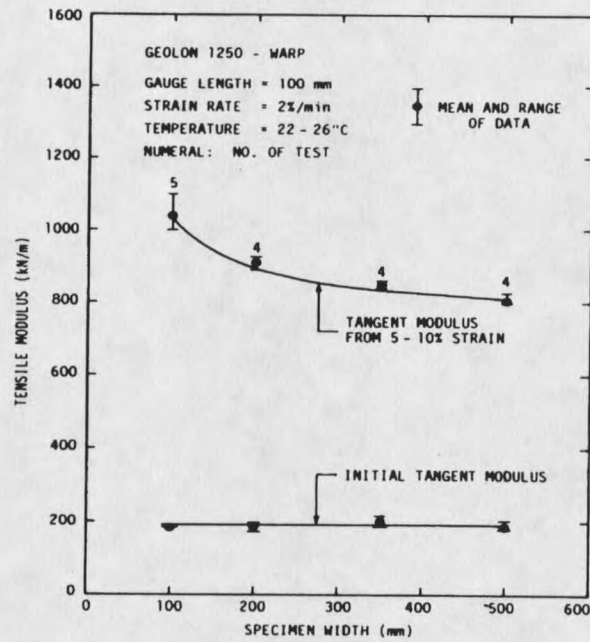


Figure 2.1: Change in tensile modulus for different specimen widths in Geolon 1250 geotextile (Rowe and Ho, 1986).

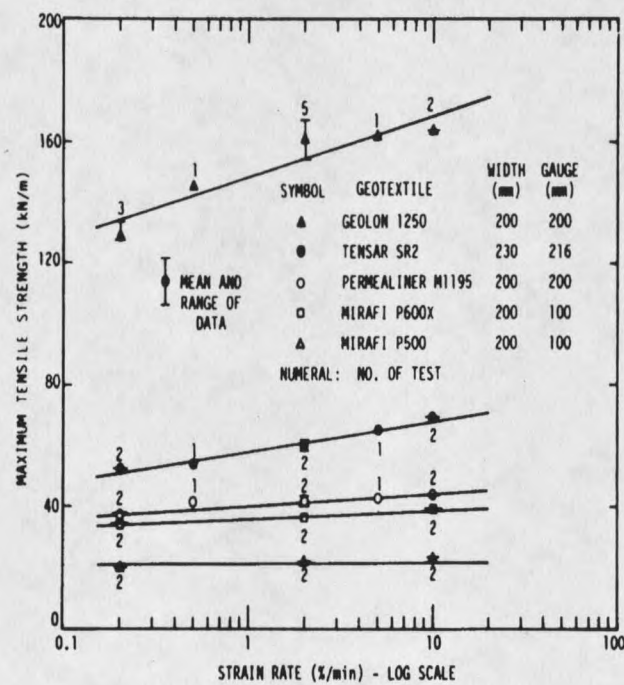


Figure 2.2: Variations of maximum tensile strength under different strain rates (Rowe and Ho, 1986).

manner, such as traffic loads, stresses within the reinforcement also increase and decrease cyclically. Geosynthetic materials exhibit significantly different material properties when loaded cyclically than when loaded monotonically. In particular, Bathurst and Cai (1994) showed that the loading history and the frequency of loading for a HDPE geogrid affected its load-strain properties, as seen in Figure 2.4.

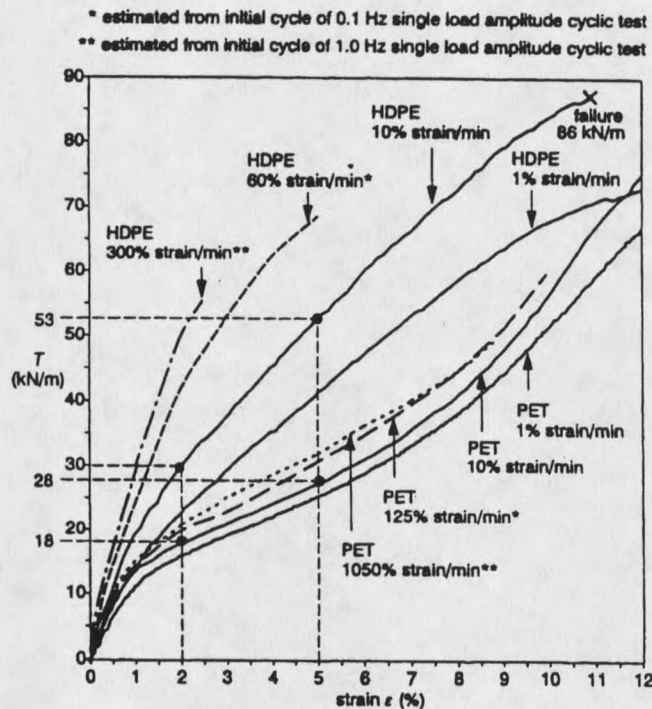


Figure 2.3: Changes in load/strain properties for different loading rates in HDPE and PET geogrids (Bathurst and Cai, 1994).

Gripping the material to transfer load from the loading apparatus into the geosynthetic specimen may influence test results if done improperly. Transferring loads without inducing stress concentrations at the sample/grip interface, is necessary for accurate results. The three most common methods of gripping the sample include soft metal wedges or epoxies, serrated wedges and lateral restraint as seen in Figure 2.5.

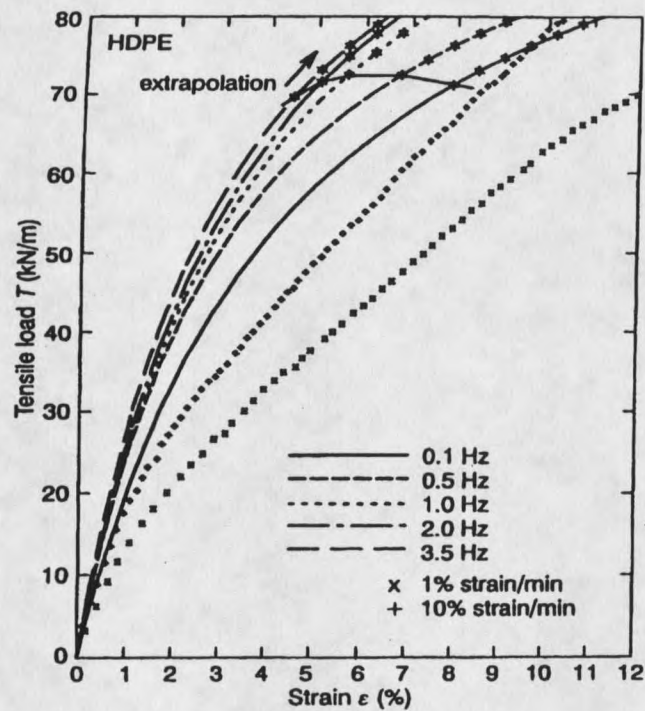


Figure 2.4: Change in load/strain properties in a HDPE geogrid due to changes in frequency for cyclic loading (Bathurst and Cai, 1994).

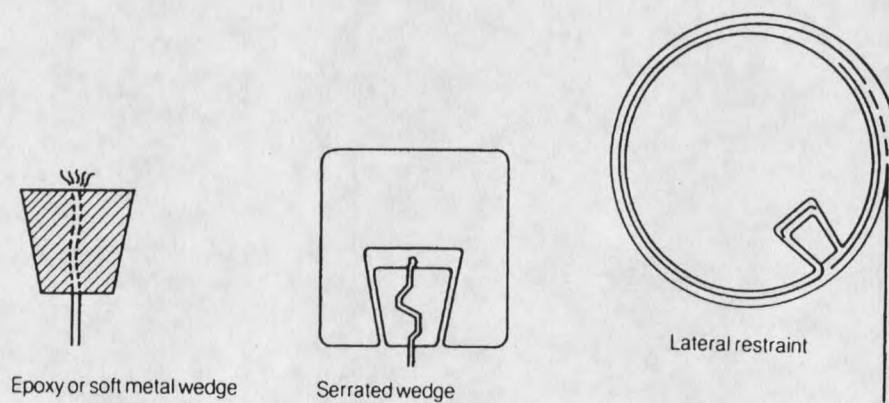


Figure 2.5: Clamping systems (Myles, 1987).

Since geosynthetics are temperature dependent, careful attention regarding their behavior under different laboratory temperatures is necessary. Cyclic loading of HDPE geogrids under two different temperatures, showed that viscoplastic strains increased and elastic moduli decreased with increasing temperatures (Moraci and Montanelli, 1996), as seen in Figure 2.6. The (ASTM, 1995) standard requires that samples be tested at a temperature of $21 \pm 2^\circ\text{C}$ ($70 \pm 4^\circ\text{F}$), to ensure consistent results.

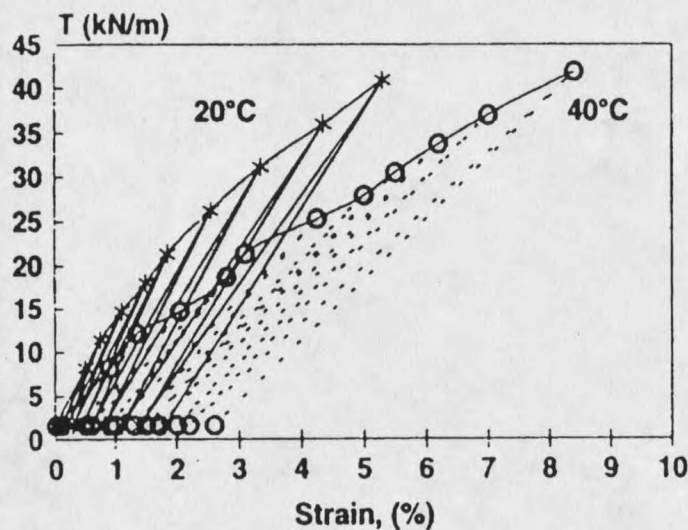


Figure 2.6: Temperature effect on the cyclic load/strain properties of a HDPE geogrid (Moraci and Montanelli, 1996).

In-Soil Testing

Laboratory determination of soil/geosynthetic interaction is influenced by several factors. The most critical factor is the analytical method used for determining the interaction parameters. Of the number of proposed methods and apparatuses available, the two that will be considered for discussion are direct shear devices and pull-out devices. Construction of each apparatus will be described along with associated methods of interpretation. Problems

concerning test methods and interpretive approaches in calculating the soil/geosynthetic interaction parameters will also be discussed.

Direct Shear Devices

Direct shear devices have been used extensively to calculate the interface friction coefficient between geosynthetic materials and soil. Direct shear devices are similar to conventional direct shear devices used for calculating soil friction angles. However, geosynthetic direct shear devices are generally much larger to accommodate large reinforcement samples, thereby reducing boundary effects.

The three most commonly used methods to conduct direct shear tests on geosynthetics are: 1) fixed shear, 2) partially fixed shear and 3) free shear methods (Ingold, 1984). Figure 2.7 shows the typical design of each of these three devices. The fixed shear test is performed by bonding a geosynthetic to a rigid block in the lower half of the box while the soil in the upper half of the box is allowed to slide over it freely. This type of test with geogrids is inadvisable due to its open structure. Restricting the soil particles from bonding through the sample changes its measured frictional characteristics. Therefore, the fixed shear test is generally used for geotextiles since aperture sizes are small.

In the partially fixed, direct shear test, soil is placed above and below the geosynthetic allowing interaction to occur on both sides of the sample. Since soil interacts on both sides and/or through the material, this test is compatible for both geogrid and geotextile materials. In this test, one end of the geosynthetic is fixed to the lower box while the other end is free, allowing the geosynthetic to elongate as the upper box is moved. In the free shear box the

geosynthetic is not fixed on either side and is free to move relative to the soil above or below it. Since the sample is allowed to move freely, at higher confinement levels it may become driven into the soil away from the shear zone. If this occurs, shear may take place in the surrounding soil instead of at the soil/geosynthetic interface (Richards and Scott, 1985).

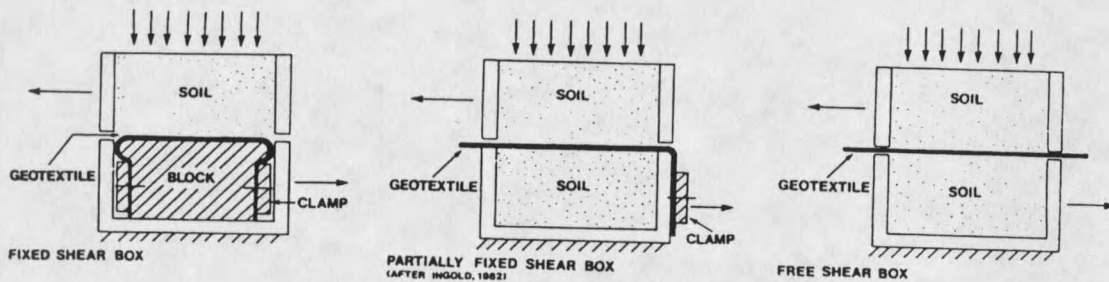


Figure 2.7: Shear box types (Richards and Scott, 1985).

Measurements of the applied load and the displacement of the top half of the shear box allows a shear stress versus displacement curve to be generated. Typical shear stress versus displacement curves have a hyperbolic shape allowing an initial interface shear modulus, and ultimate and residual values of interface shear stress to be obtained. Using the ultimate shear, calculation of a soil/geosynthetic interaction friction angle from direct shear tests can be defined by the following equation:

$$\delta = \frac{\tau_{ult}}{\sigma_n} \quad (2.1)$$

where δ is the soil/geosynthetic friction angle, τ_{ult} is the ultimate interface shear stress and σ_n is the applied vertical normal confinement. Comparisons of the interaction coefficient calculated using direct shear devices to those calculated using pull-out devices have been conducted by many (Ingold, 1982; Fourie and Fabian, 1987). Lower values of soil/geo-

synthetic interaction from pull-out tests were reported for woven and non-woven geotextiles while similar values were reported for geogrids when compared with each of the three types of direct shear tests (Ingold, 1982).

Pull-Out Test Devices

Pull-out tests have also been used to define soil/geosynthetic interaction. In pull-out tests, an embedded geosynthetic specimen is either monotonically or cyclically drawn from confining soil. Measurements from this test include load applied to the front of the geosynthetic sample and displacement of the geosynthetic along various positions of its length. The end not attached to the pulling device is typically free. Studies performed by Chang (1977), Holtz (1977) and Schlosser and Elias (1978), laid the necessary groundwork for subsequent pull-out testing.

Three primary mechanisms of soil/geosynthetic interaction are illustrated in Figure 2.8 and have been best described by Jewell et al. (1984) as:

1. Friction developed by the shearing of soil over the planar surfaces of the geosynthetic parallel to the direction of movement,
2. Soil bearing on geosynthetic material that is normal to the direction of movement creating passive pressures on those regions and
3. Soil-to-soil shearing action within the apertures.

Mechanism one corresponds to any planar geosynthetic material, while mechanisms two and three correspond to geosynthetics where apertures are present, i.e., geogrids.

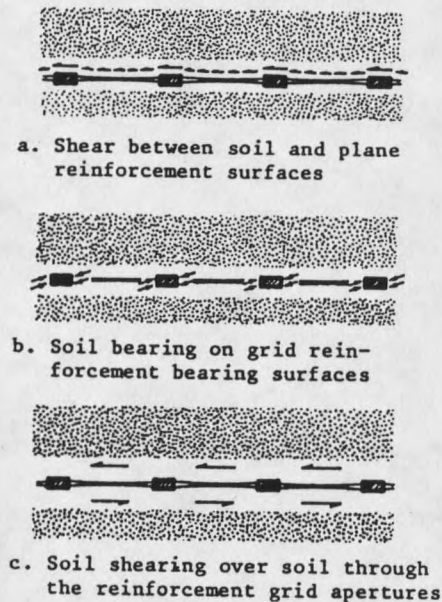


Figure 2.8: Three principle mechanisms of soil/geosynthetic interaction (Jewell et al., 1984).

Pull-Out Apparatus Design

Many different apparatus geometries, testing methods and analysis techniques have been employed. Presently, only a draft ASTM standard exists for these tests. The two primary issues of concern for the design and use of a pull-out box are: 1) the method of loading the geosynthetic and 2) the boundary conditions imposed by the apparatus.

Loading Methods

Methods of loading the geosynthetic affects the determined material properties. The two types of loading apparatuses that will be considered for discussion are displacement-rate controlled and load controlled. Displacement-rate controlled apparatuses pull the geosynthetic sample at a constant rate independent of the applied load. Rates of loading in

previous pull-out tests ranged between 0.1 mm/minute (Brand and Duffy, 1987) to 10 mm/minute (Bauer et al., 1991) with most tests being conducted at 1 mm/minute. A rate of 1 mm/minute is suggested by the ASTM draft standard test method for "Determining Geosynthetic Pull-out Resistance in Soil". Results of the investigative analysis regarding displacement-rate controlled tests performed by Farrag (1991), where tests were conducted at rates ranging from 0.1 to 15 mm/minute, illustrated that changes in displacement rates under 6 mm/minute did not drastically affect pull-out resistance. However, studies conducted by Yasuda et al. (1992) indicated that pull-out rate had minimal to no effect on the ultimate pull-out force for rates ranging between 1 and 40 mm/minute as shown in Figure 2.9. In this figure, SR-2 and SS-2 are two different types of geogrid, σ_n is the applied normal confinement and Dr is the relative density of the soil. The results by Yasuda et al. (1992) are unusual since the load/strain characteristics of most geosynthetic materials are strain-rate dependant.

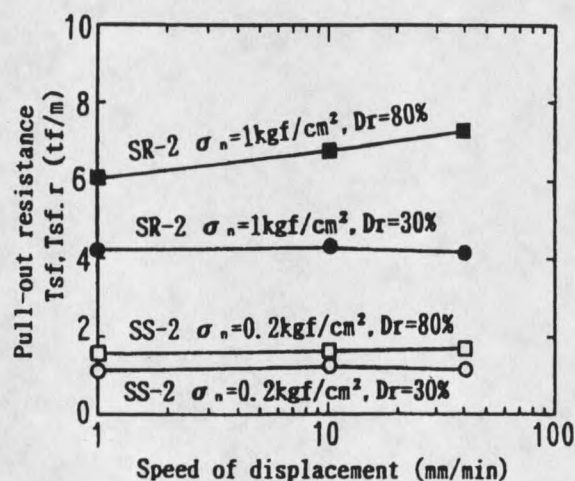


Figure 2.9: Effects of rate of pull on pull-out resistance (Yasuda et al., 1992).

Since repeated traffic loading creates cyclic loads in pavements, determining the geosynthetic's in-soil material properties using cyclic loads is necessary for quantifying reinforcement for this load case. For load-controlled tests, the loading apparatus delivers either a constant load or a cyclic load to the geosynthetic sample. Bathurst and Cai (1994) claimed that to determine the load/strain behavior of a HDPE and PET geogrid under cyclic loads correctly, load amplitude and frequency must represent loading in the field. Cyclic loading of geosynthetics in soil differs from cyclic in-air tests since stresses in the material may be locked in by the surrounding confinement. For in-soil cyclic loading of geosynthetics, part of the peak applied load is locked in, changing the stiffness of the unloading/reloading loop (McGown et al., 1995). Tests performed by Hanna and Touahmia (1991) indicated that after testing a reinforcement sample under cyclic loads, monotonic pull-out load may be effected. Their study concluded that monotonic pull-out resistance was decreased for smooth steel strips, increased for a high adherence ribbed steel strips, and unchanged in a uniaxial geogrid.

Boundary Conditions

Interaction between the soil/geosynthetic mass and the boundaries imposed by the test facility may influence pull-out test results. Defining specific components of the design that can be modified to reduce these effects is necessary. Three primary design components to consider are the gap interface at the front wall, the walls and the top boundary of the pull-out box.

Gap Interface

The most critical design component of the front face of the pull-out box is the slit or opening from which the geosynthetic sample is drawn from the soil. As the geosynthetic sample is being pulled from the soil, lateral and vertical stresses increase causing the soil to dilate and react with the front face and top of the pull-out box. In a poorly constructed pull-out apparatus, this increase of vertical stresses increases the resistance at the soil/geosynthetic interface, thereby increasing the pull-out load. Using a stationary, rigid plate to apply normal confinement, Johnston (1985) made measurements of normal stress at different stages of pull-out. Johnston (1985) found that for densely compacted confinement soil, normal stresses at the front of the box increased as pull-out progressed. Studies using load cells to measure the change in the horizontal pressure at the front of the box due to pull-out have also been conducted and have shown that horizontal stresses increase as pull-out develops (Raju, 1995; Farrag, 1991).

Designing the gap at the front of the pull-out box to reduce this effect insures that results are less affected by internal stress buildup. To reduce these stress increases, some designs included load-transfer sleeves at the front slit. Load-transfer sleeves consist of two thin, tapered wedges extending horizontally from the opening at the front wall into the soil. Rather than transferring load from the soil into the front face of the pull-out box, the sleeves allow this load to be transferred into surrounding soil by effectively moving the front opening into the soil. According to the ASTM draft standard the load-transfer plates should be no more than 13 mm thick near the front wall and no more than 3 mm thick at the point of load

transfer. It also states that these plates must have sufficient space between them to allow the sample to pass between them and must extend into the box a distance equal to half of the depth of soil above or below the geosynthetic. Another issue concerning the gap design is preventing the loss or lodging of soil particles between the load-transfer plates during pull-out. The slit must allow unrestricted movement of the confining soil, yet, prevent the loss of soil particles as the geosynthetic is pulled through it. The lodging of soil material between the load-transfer plates can artificially increase the measured pull-out load. In the discussion by Lo (1990) concerning the gap design, the gap was packed with several layers of lubricated rubber strips allowing the reinforcing material to be pulled through it while preventing the loss of soil.

Researchers have attempted to quantify the effects of the design of the load-transfer sleeve on pull-out test results. Studies were conducted by Farrag (1991) to examine the effect of the load-transfer sleeve's length on pull-out behavior. The results indicated that pull-out load increased as the length of the load-transfer sleeve was decreased as seen in Figure 2.10. The magnitude of load increase was also affected by the rate of pull. When the rate of pull was slower, the sleeve length effected pull-out load less. Raju (1995) suggests that measurement of lateral stress at the front wall is the best method for understanding its influence on test results. Tests by Farrag (1991) also utilized stress cells at the front wall to quantify lateral earth pressures due to geosynthetic pull-out. These stress cells indicated that increasing the length of the load-transfer sleeve significantly decreased the horizontal pressures developed at the front of the wall (Farrag, 1991). Raju (1995) claimed that the load-transfer sleeve/sample interface is difficult to understand since it complicates the stress

distribution along its edge and that the addition of the geosynthetic clamp in the soil further complicates test results. Pull-out tests performed by Chang (1977) did not use a front wall, thus providing no lateral restraint to the soil.

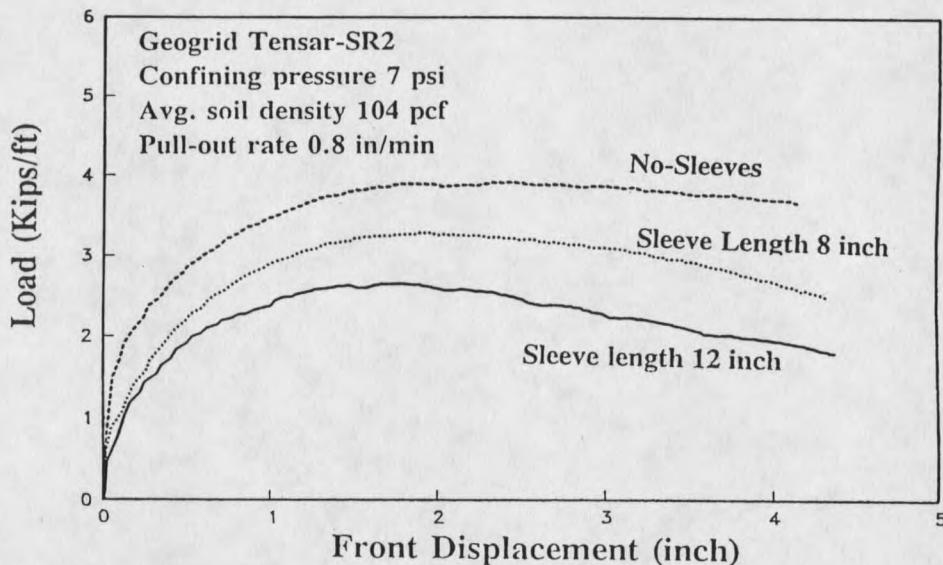


Figure 2.10: Effect of load-transfer sleeve length on pull-out resistance of geogrid (Farrag, 1991).

Walls

Boundary conditions associated with the walls may affect pull-out test results in one of two ways. The first is their effect on soil movement due to dilation. Studies conducted on the frictional characteristics of the front boundary wall have shown that decreasing the front wall friction reduces induced stresses on dilative material, thereby decreasing the geosynthetic pull-out resistance (Palmeira and Milligan, 1989) as shown in Figure 2.11. This figure shows that the use of lubricated layers can be used to reduce shear stresses at the boundaries of the pull-out box. Consequently, side walls should also be lubricated to reduce shear stresses caused by soil movement and pressure. Lowering the friction angle of the side

walls or keeping the geosynthetic edges at a minimum distance of 150 mm from the box sides is recommended by the ASTM draft standard and Farrag (1991). The use of a low adhesion grease and latex sheeting on the side walls allowed soil to move with minimal friction effects (Leshchinsky et al., 1994).

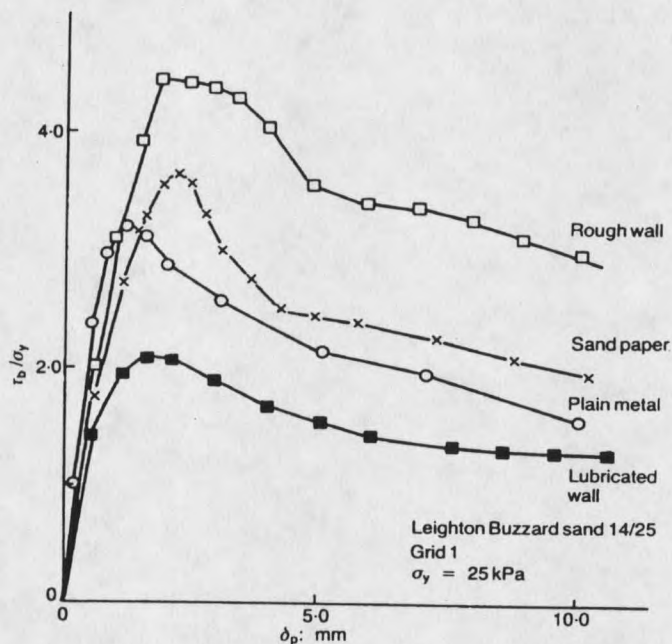


Figure 2.11: Effect of front wall friction on normalized pull-out resistance for a geogrid (Palmeira and Milligan, 1989).

The second manner in which the boundary conditions imposed by the walls can affect pull-out resistance is through a reduction in the applied normal stresses due to friction between the surrounding soil and the walls. Farrag (1991) used pressure cells to compare normal stress levels between locations adjacent to the wall and adjacent to the edge of the geosynthetic sample. It was noticed that vertical pressures were lower near the side walls as shown in Figure 2.12. Figure 2.13 shows the effect of the sample width on the measured pull-out resistance. This difference in vertical pressure indicates that pull-out resistance is

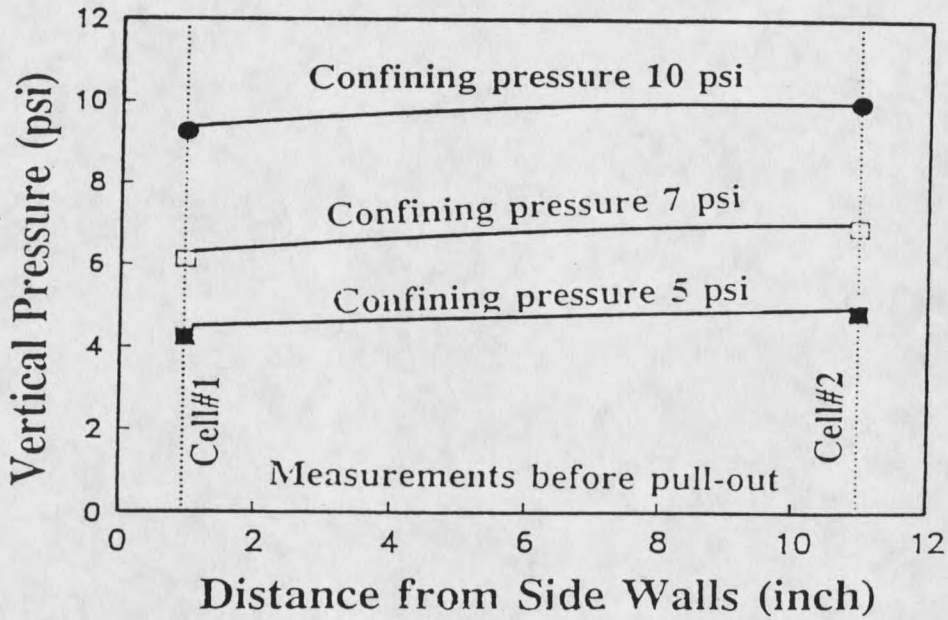


Figure 2.12: Effect of side-wall friction on vertical pressure (Farrag, 1991).

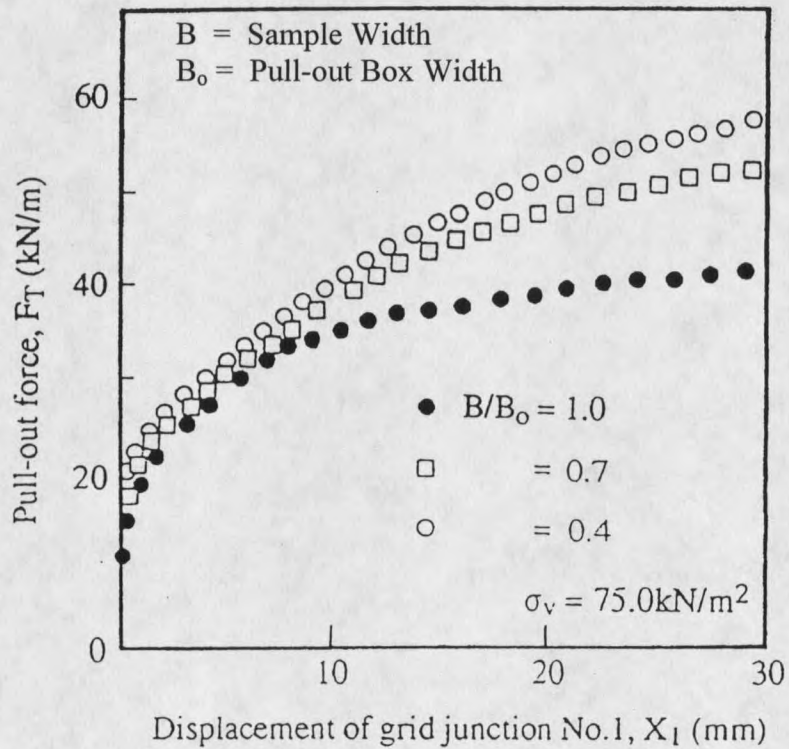


Figure 2.13: Effect of specimen width on pull-out resistance (after Ochiai et al., 1996).

reduced by shear stresses developed along the side walls and that this effect will yield conservative interaction coefficients (Farrag, 1991; Lentz and Pyatt, 1988; Ochiai et al., 1996).

Top

Boundary conditions at the top of the pull-out box are also important since the depth of soil and the method of applying the normal confinement may also influence test results. The method of applying normal confinement may be done using either a rigid plate or a flexible diaphragm. When a rigid plate is used, dilation of the soil at the front of the box is restricted. During pull-out tests the soil at the front of the box dilates, as discussed by Johnston (1985). If the boundary on the top is rigid, the normal confinement will increase in the front and increase pull-out resistance (Palmeira and Milligan, 1989). The use of a flexible diaphragm to alleviate the buildup of excess normal pressures at the front is advisable. Figure 2.14 shows that when a flexible diaphragm is used to apply the normal confinement, the ultimate pull-out load will be lower than if a rigid plate is used (Palmeira and Milligan, 1989). The use of a flexible diaphragm, which maintains a constant, uniform normal stress over the entire pull-out box area is also advised by the ASTM draft standard.

The depth of soil thickness above and below the sample may also affect the pull-out resistance of the geosynthetic material. Experiments performed by Farrag (1991) demonstrated that decreasing the soil thickness increases the pull-out resistance and increasing the soil thickness more than 12 inches does not significantly affect pull-out resistance, as shown in Figure 2.15. The ASTM draft standard suggests that the depth of soil

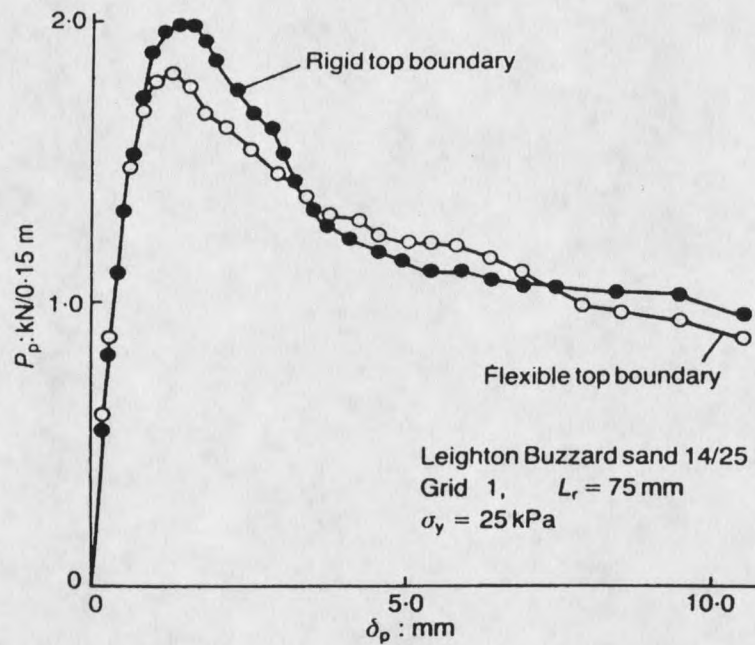


Figure 2.14: Effects of top boundary on pull-out resistance (Palmeira and Milligan, 1989).

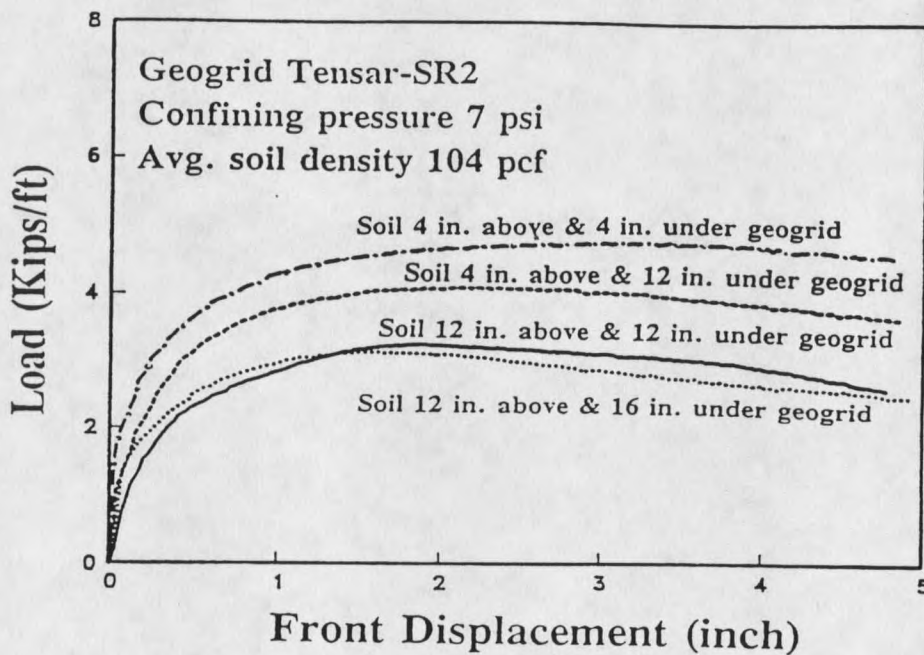


Figure 2.15: Effect on depth of soil, above and below reinforcement, on pull-out resistance (Farrag, 1991).

above and below the soil be at least six times the D_{85} of the soil or three times the maximum particle size of the soil, whichever is greater.

Analysis of Pull-Out Results

It has been well established that geosynthetic materials exhibit a non-uniform shear stress distribution along the length of the specimen during pull-out. The distribution of shear stress along a geosynthetic sample is dependent on soil and geosynthetic material properties, the material type (geomembrane, geogrid, etc.) and the level of applied normal stress. Interpretation of pull-out test results has typically been complicated by the large number of variables involved. Several methods of interpreting pull-out tests to define soil/geosynthetic interaction have been suggested. To define the interaction parameters, the mobilized shear stress at the soil/geosynthetic interface must be quantified. Presently, the four principal methods used to evaluate soil/geosynthetic interface shear resistance are: 1) the 'Mobilizing Process Method' (Ochiai et al., 1996), 2) the 'Average Resistance Method' (Bonczkiewicz et al., 1988; Ochiai et al., 1996), 3) analytical or numerical techniques (Clough and Duncan, 1971, Yuan and Chua, 1991, Alobaidi et al., 1997) and 4) the finite element method (Helwany, 1987; Yuan and Chua, 1990; Wilson-Fahmy and Koerner, 1993; Chan et al., 1993; Yogarajah and Yeo, 1993).

Mobilizing Process Method

The mobilizing process method of determining the relationship between the shear stress and the displacement at the soil/geosynthetic interface attempts to account for increased mobilization of the geosynthetic sample during pull-out as seen in Figure 2.16. As

the pull-out test progresses, mobilized length of the geosynthetic increases making shear along the length of the sample also increase. Using the mobilizing process method, a non-linear shear stress distribution along the sample can be estimated. The relationship between the shear stress and the shear displacement is calculated using load applied to the front of the geosynthetic and the displacement of several points along its length.

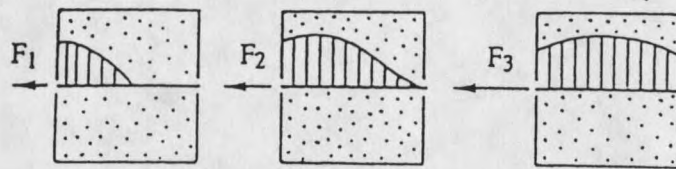


Figure 2.16: Graphical representation of the mobilizing process method (Ochiai et al., 1996).

The four steps in the mobilizing process method can be illustrated by Figure 2.17. These four steps are: 1) discretize the displacement versus distance distribution, 2) determine strain from the discretized elements, 3) determine load in the geosynthetic from the strain and 4) calculate shear using load differencing.

The distribution of displacement versus distance along the length of the embedded geosynthetic is obtained from actual pull-out test data. Since only a few measurements of displacement along the length can be made, this distribution consists of only a few points. Discretizing this distribution can be done in one of two ways. The first is to assume a linear distribution of displacement between measurement points, thus discretizing the distribution into j elements, where j is one less than the number of displacement measurements as shown in Figure 2.17(a). The second method of discretization is to use a curve fit that best represents this distribution. This will allow for smaller discretization and possibly an

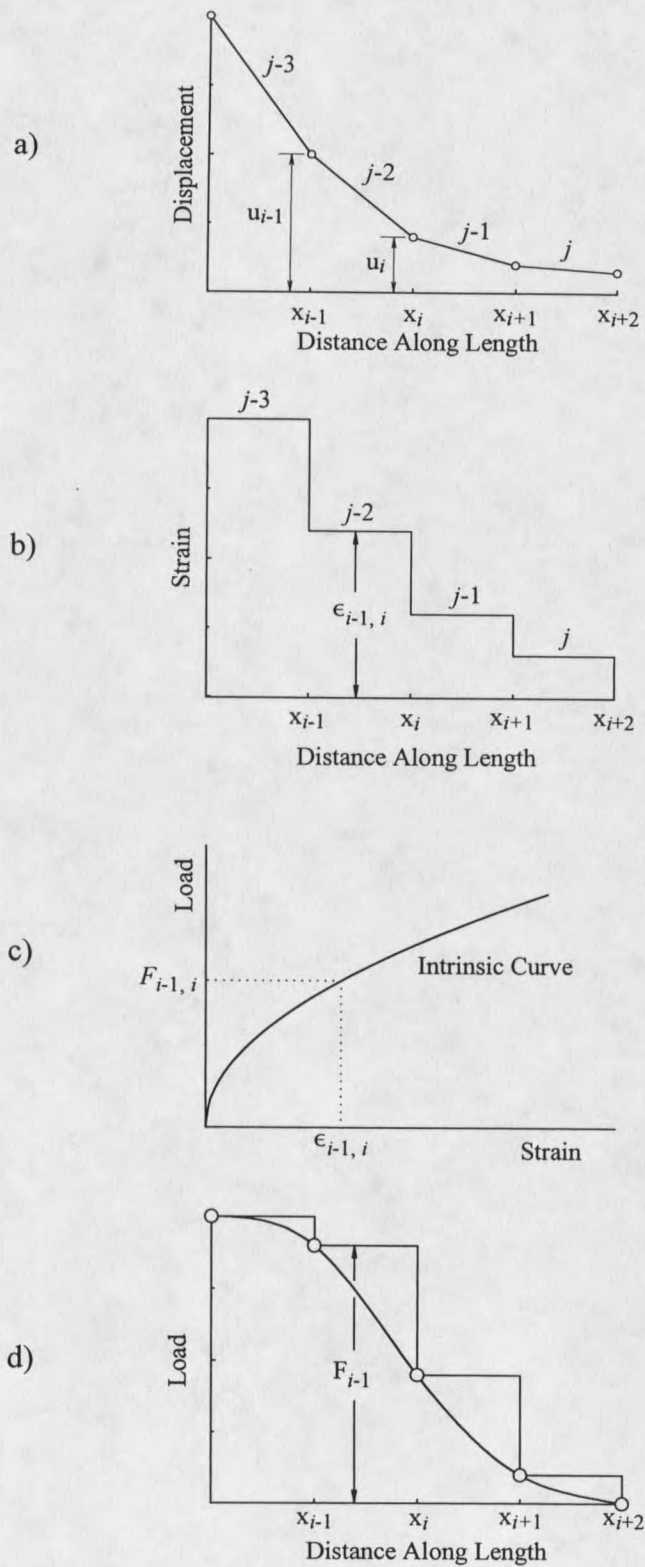


Figure 2.17: Concepts in evaluating pull-out resistance using the mobilizing process method (redrawn after Ochiai et al., 1996).

equation fit of the data.

The second step is to determine the strain across elements formed in step one. A linear distribution of displacement between measurement points results in a constant strain along those elements. Strain between any two discretized points can be calculated using Equation 2.2 as follows:

$$\epsilon_{i, i+1} = \frac{u_i - u_{i+1}}{x_i - x_{i+1}} \quad (2.2)$$

where $\epsilon_{i, i+1}$ is the strain calculated between nodes i and $i+1$, u is the displacement of nodes i and $i+1$ and x is the original position of nodes i and $i+1$. Figure 2.17(b) shows a graphical representation of this step. The definition of strain is simply the change in length, du , divided by the original length of the element, dx . As mentioned earlier, by having an equation to describe the displacement versus distance distribution, strains at any point can be found by taking the derivative, du/dx , of the equation. Determining the correct form of the equation to represent the displacement distribution may be difficult since displacement measurements along the embedded sample are few. Also, strains calculated from the derivative of this equation are sensitive to the form of the equation used to fit the distribution.

Step three in the mobilizing process method is to use the strain to calculate load in the material. This method uses strains to calculate loads in the material since actual load measurements within the material cannot be made. Calculating the load can be accomplished in one of two ways. The first is to use the load/strain curve of the geosynthetic from in-air

tests. The second method of obtaining the load in the material is to develop a load/strain curve from the in-soil test data. The in-soil load/strain curve is defined using the load applied to the front of the sample and the strain calculated in the first element (element 'j-3') from step two. Although the in-soil load/strain curve is developed using results obtained from the front of the sample, it is utilized for all points along the length of the geosynthetic. The distribution of load versus strain in the material, used to calculate the load, is defined as the intrinsic curve as shown in Figure 2.17(c).

Calculated loads are for the element and not nodes, so to simplify the notation used, the load is transferred to the nodes. The force for any particular element will be transferred to the node at the front of the element, where the front node is closest to the applied load. For example, $F_{i-1, p}$ the load for the element spanning nodes $i-1$ and i , becomes F_{i-1} . Figure 2.17(d) shows the graphical representation of the force versus distance along the geosynthetic.

The final step is to calculate the shear stress along each element. The free-body diagram in Figure 2.18 shows how Equation 2.3 is developed. The shear is defined as the difference in the load from one end of the element to the other, dF , divided by the original length of the element, dx . Since the load at all points is known, shear between nodes $i-1$ and i is calculated by Equation 2.3.

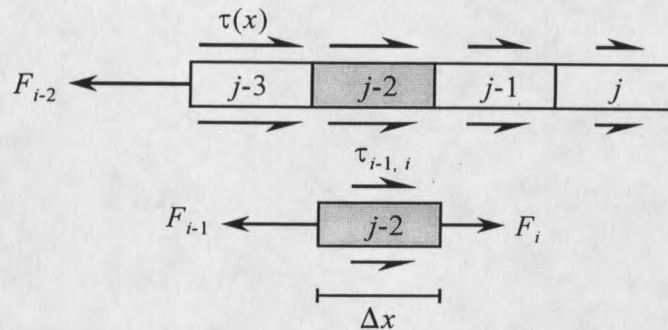


Figure 2.18: Free-body diagram of shear calculation.

$$\tau_{i-1, i} = \frac{F_{i-1} - F_i}{2 \cdot b \cdot \Delta x} \quad (2.3)$$

In this equation F_{i-1} is the transferred force at node $i-1$, F_i is the transferred force at node i , b is the width of the sample and Δx is the length of the element spanning between nodes $i-1$ and i . The shear stress, τ , can now be plotted versus the shear displacement, where shear displacement is defined as the displacement of node i (Farrag, 1991; Ochiai et al., 1996). From the shear stress versus shear displacement diagram, the interaction coefficient, $\tan(\delta)$, may be calculated using the ultimate value of shear stress and the applied normal stress, as described in the direct shear analysis using Equation 2.1.

Average Resistance Method

Within the average resistance method are three different 'sub-methods' of interpreting soil/geosynthetic interaction; the total area method, the effective area method and the maximum slope method (Figure 2.19). These methods will be described individually in the following sections. Figure 2.20 shows graphical representations of each evaluation method.

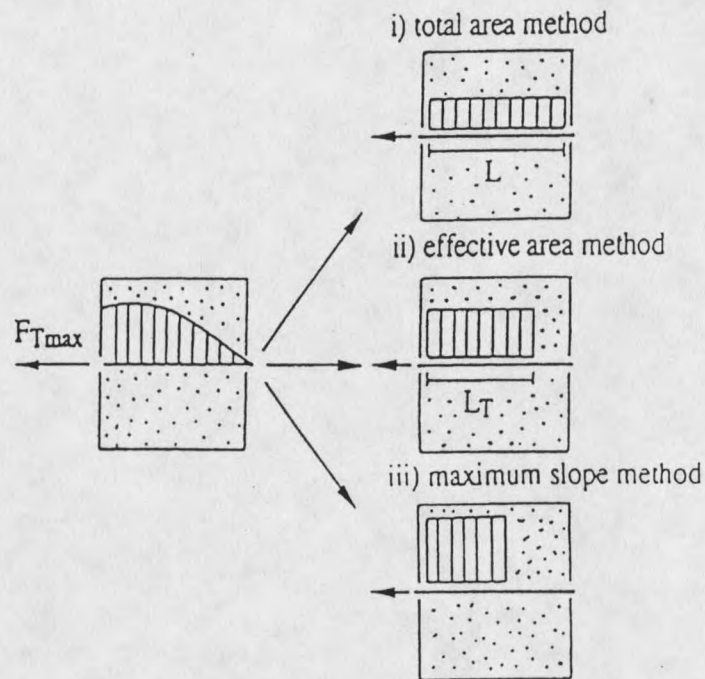


Figure 2.19: Three average resistance methods used to evaluate pull-out resistance (Ochiai et al., 1996).

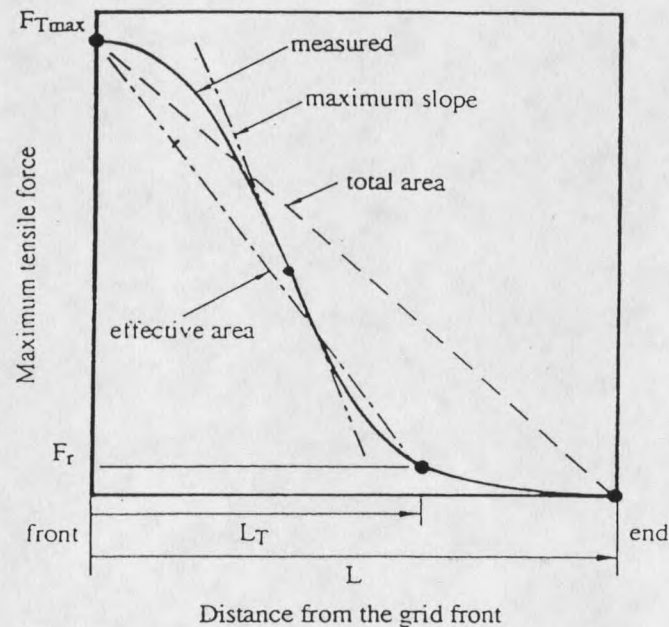


Figure 2.20: Graphical representation of the methods used to evaluate pull-out resistance (Ochiai et al., 1996).

Total Area Method

The total area method is perhaps the most simple method to use but is limited to use for pull-out specimens that can be completely mobilized in pull-out with only slight elongation. Since geosynthetic reinforcement may elongate extensively under high confinement levels, the shear stress is non-uniform along the length. Commonly the shear stress distribution along the soil/geosynthetic interface exhibits a softening effect. Values of ultimate shear stress depend on the level of softening behavior experienced at this interface. If the soil/geosynthetic interface exhibits a softening versus non-softening behavior, the value of ultimate shear may be unconservative to conservative, respectively. The equation used to calculate the average shear for the total area method is:

$$\tau_{average} = \frac{F_{Tmax}}{2 \cdot b \cdot L} \quad (2.4)$$

where F_{Tmax} is the maximum tensile force applied to the front of the sample and b and L are the width and length of the sample, respectively.

Effective Area Method

The effective area method provides a more reasonable value of average shear stress than the total area method since the effective force and area are used. This method allows the sample to be divided into lengths that are experiencing pull-out mobilization. The equation for this method is as follows:

$$\tau_{average} = \frac{F_{Tmax} - F_r}{2 \cdot b \cdot L_{eff}} \quad (2.5)$$

where the quantity $(F_{Tmax} - F_r)$ is the effective force, F_{Tmax} is the maximum tensile force at the front of the sample and F_r is the residual force at L_{eff} , the effective length of the specimen. The effective length of the sample at a particular level of pull-out can be determined by measurements made along its embedded length.

Maximum Slope Method

The maximum slope method uses the maximum tangent slope of the force distribution curve as seen in Figure 2.20. This method of calculating average resistance in the material is unconservative since it often overestimates the interaction. The equation used to calculate the maximum slope is as follows:

$$\tau_{average} = \left(\frac{dF}{dL} \right)_{max} \quad (2.6)$$

To use this method properly, a distribution of the load versus displacement must be developed using the steps outlined in the mobilizing process method. This method uses the maximum tangent slope of this distribution as the average shear stress.

Each of the effective area methods returns a somewhat different result of average shear. Figure 2.21 shows a comparison between the average shear calculated from each of these three methods with respect to normal confinement. The maximum slope method is the

least conservative approach. The effective area method is equivalent to the total area method for samples that pull-out without excessive elongation.

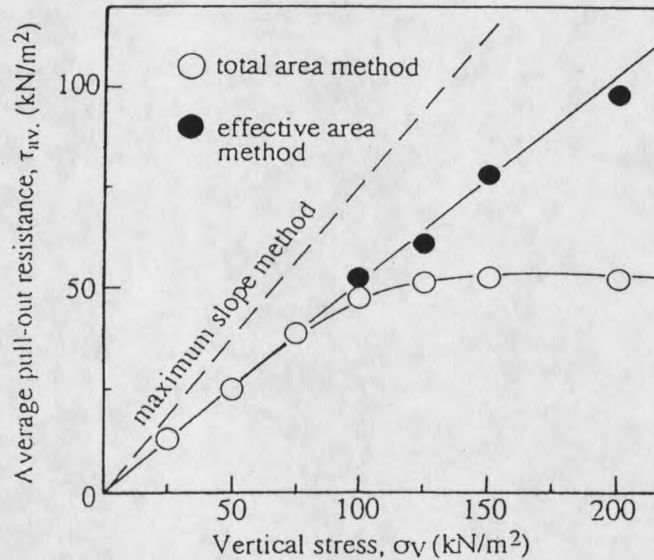


Figure 2.21: Effect of analysis method on average pull-out resistance (Ochiai et al., 1996).

Numerical Method

The numerical method takes into consideration the difficulties in interpreting pull-out tests associated with large displacements and extension of the geosynthetic material. Approaches using differential equations to solve the interaction problem associated with the pull-out test have been attempted. These approaches take into consideration nonlinear distributions of shear stress along the length of the reinforcement. The numerical solution is a function of the pull-out displacement, the axial load-strain characteristics, the interface shear stiffness, the vertical normal confinement and the ultimate and residual interface shear strength.

Formulation of the numerical method begins with the definition of the assumed shape of the function describing the shear stress versus displacement curve. The shear stress versus

displacement distribution may be defined in one of two ways. The first form by Clough and Duncan (1971) is shown in Equation 2.7 as:

$$\tau(u) = \frac{u}{C_1 + C_2 u} \quad (2.7)$$

where τ is the shear stress, u is the interface shear displacement and C_1 and C_2 are experimentally determined coefficients. In this equation, the constant C_1 can be shown to be equal to the inverse of the initial tangent slope of the shear modulus and C_2 can be shown to be equal to the inverse of the asymptotic value of the ultimate shear stress at infinite displacement (Clough and Duncan, 1971; Wilson-Fahmy and Koerner, 1993). The second form of the equation by Juran and Chen (1988) is shown in Equation 2.8 as:

$$\tau(u) = \sigma_y C_3 u \frac{u - C_1}{(u + C_2)^2} \quad (2.8)$$

where $\tau(u)$ is the shear stress in kPa, u is the interface shear displacement in meters and C_1 , C_2 and C_3 are constants which can be determined using Equations 2.9, 2.10 and 2.11 as:

$$C_1 = -4 \frac{\sigma_y d}{G} \frac{[\tan^2 \psi_p \cdot J^2]}{\tan \psi_r} \quad (2.9)$$

$$C_2 = 2 \frac{\sigma_y d}{G} [\tan \psi_p \cdot J] \quad (2.10)$$

$$C_3 = \tan \psi_r \quad (2.11)$$

where σ_v is the vertical normal confinement, d is the thickness of the interface layer, G is the initial tangent shear modulus at the soil/geosynthetic interface, ψ_p is the peak soil/geosynthetic friction angle, ψ_r is the residual soil/geosynthetic friction angle and J is defined using the following equation:

$$J = 1 + \left[1 - \frac{\tan \psi_r}{\tan \psi_p} \right]^2 \quad (2.12)$$

The governing differential equation was formed through the following steps which resemble the mobilizing process method described earlier. Step one is to define strain in terms of displacement using the following relationship:

$$\epsilon = \frac{du}{dx} \quad (2.13)$$

Step two defines the load in the material as a function of the strain, $F(x) = f(\epsilon)$. Step three defines the shear as a function of the load using the following relationship:

$$\tau(x) = \frac{1}{2} \frac{dF}{dx} \quad (2.14)$$

Step four is to combine the load/strain equation with Equation 2.14 to form the governing differential equation shown in Equation 2.15, which relates the interface shear stress at any point x , along the reinforcement length, to the load carried in the reinforcement:

$$\tau(x) = \frac{1}{2} \left[\frac{dF\left(\frac{du}{dx}\right)}{dx} \right] \quad (2.15)$$

Combining Equations 2.15 and 2.8 defines the final form of the governing differential equation describing the relationship between the shear stress and the shear displacement for soil/geosynthetic interaction. Since a closed form solution for this equation does not exist at the present, it must be solved using numerical or analytical techniques.

Finite Element Modeling

Finite element modeling has also been used to describe pull-out behavior. This method allows and accounts for shear displacement in the soil and does not need to assume that shear movement is only along the soil/geosynthetic interface. The finite element method of analysis is a powerful method for solving boundary value problems and for determining the progressive and nonlinear stress and strain in the geosynthetic and the soil/geosynthetic interface. Several models with different geometries and boundary conditions have been developed and compared with actual test data. Typically a plane-strain analysis is used to model pull-out tests (Yuan and Chua, 1990; Wilson-Fahmy and Koerner, 1993; Chan et al., 1993; Yogarajah and Yeo, 1993). The load/strain properties of the geosynthetic are generally defined by unconfined, in-air tests. Material models of the geosynthetic may use either a linear or nonlinear load/strain relationship. Soil parameters may be defined using conventional soil testing, such as the triaxial or direct shear devices. The shear stress relationship at the soil/geosynthetic interface can be calibrated using geosynthetic direct

shear tests. Using a finite element program, the shear stress versus shear displacement relationship can be obtained. Comparisons between the interaction determined from the finite element analysis and direct shear tests can be made, allowing the calibration of the parameters describing this interaction.

Due to the difficulty in obtaining interface parameters from pull-out tests, an attempt to match experimental results numerically was taken by Yuan and Chua (1990). They used the four steps listed below to obtain interface parameters:

- 1) Determine properties of soil and geosynthetic and choose reasonable values for the interface parameters,
- 2) Use these interface parameters to perform a numerical simulation with a numerical code,
- 3) Compare the response predicted by the numerical code to data from actual experiments and
- 4) Modify interface parameters until the predicted response matches the measured response.

Therefore, by modifying the shear stress versus shear displacement parameters, the predicted result can be iterated until it agrees with the experimental result. Similarly, Yogarajah and Yeo (1994) stated that by using the inherent material properties of the geosynthetic, a numerical model can be used to calculate loads and strains in the geosynthetic and therefore determine a shear modulus. Comparing the calculated shear modulus with data from actual experiments provided a means of iteration until results matched one another.

CHAPTER THREE

EXPERIMENTAL METHODS AND TEST RESULTS

Introduction

A large test facility was designed and constructed at Montana State University for the purposes of studying geosynthetic behavior both in-air and in-soil. Versatility of the designed equipment allowed one apparatus to be used for obtaining both in-air material properties and soil/geosynthetic interaction properties. Consequently, when the in-air test apparatus is addressed, it refers to how the pull-out box was used as the frame for in-air tests. The instrumentation associated with this apparatus allowed applied load and axial, lateral and local strains of the geosynthetic specimen to be measured in the in-air tests, and pull-out load and displacements along the embedded length of the test specimen to be measured in the in-soil tests. Results from each of the tests are shown, describing pertinent corrections and interpretation techniques.

Design Considerations

When designing a geosynthetic test apparatus, four major criteria must be considered: 1) type of loading to be applied to the geosynthetic, 2) rigid boundary effects, 3) specimen clamping arrangement and 4) instrumentation.

1. Geosynthetic Loading Conditions: Geosynthetic material properties from monotonic and cyclic loading are needed for constitutive modeling. Therefore, the test facility needed to be capable of performing tests under each of these types of loads. For monotonic loading of geosynthetics, a displacement-rate controlled pulling mechanism was required. For cyclic loading of geosynthetics, a load controlled pulling mechanism was necessary.

2. Rigid Boundary Effects: In-soil test results are affected by rigid boundaries imposed by the walls of the pull-out box. To reduce these effects, three major components of design must be considered: 1) the gap interface at the front wall, 2) the walls and 3) the top boundary.

The most critical boundary is the gap at the front face of the pull-out box through which the sample is pulled. The design of this gap must reduce the build up of excessive resistance from moving soil. The use of load-transfer sleeves helps reduce these effects by transferring load from the moving soil/geosynthetic mass to the surrounding soil, allowing the interaction to take place away from the front face of the box, as described in Chapter 2.

To reduce boundary effects along the walls, they should be lined with a low adhesion grease and a flexible membrane. Reducing wall friction allows the embedded geosynthetic sample to experience a uniform distribution of vertical normal confinement. Designing the box to accommodate large sample sizes also reduces effects from wall boundaries. Vertical boundary effects may be reduced by making the soil thickness above and below the sample sufficiently thick and using a flexible membrane to apply normal confinement. Use of a flexible membrane to apply normal confinement also allows the driven soil mass to dilate

unrestricted.

3. Sample Gripping: The three principal concerns associated with choosing a proper sample gripping method were: 1) to deliver a uniform load across the sample width, 2) decrease stress concentrations at the sample/grip interface and 3) allow constant soil/geosynthetic contact area to be maintained during pull-out. For the pull-out tests, a method of gripping the sample within the soil would need to use a low profile design to reduce the effect on pull-out results due to disturbed soil.

IV. Instrumentation and Controls: Instrumentation and controls used to measure displacements, strains, load and pressure need to provide adequate precision so credible values for the necessary parameters can be obtained. Specifically, a measurement of the applied load at the front of the sample and displacement along the embedded length of the geosynthetic sample needed to be recorded during pull-out. For the in-air samples, measurements of local strain should be made using bonded strain gages. Applied air pressure measurements should be provided to allow normal confinement applied to the soil surface to be calculated.

Equipment Description

As indicated earlier, a single test facility was developed to perform both in-air and in-soil testing of geosynthetics. Consequently, many of the apparatus' components were used for both test types. The following section describes the fundamental components of the pull-out box design and gives a detailed overview of its construction. Figure 3.1 presents an illustration of the individual components of the apparatus.

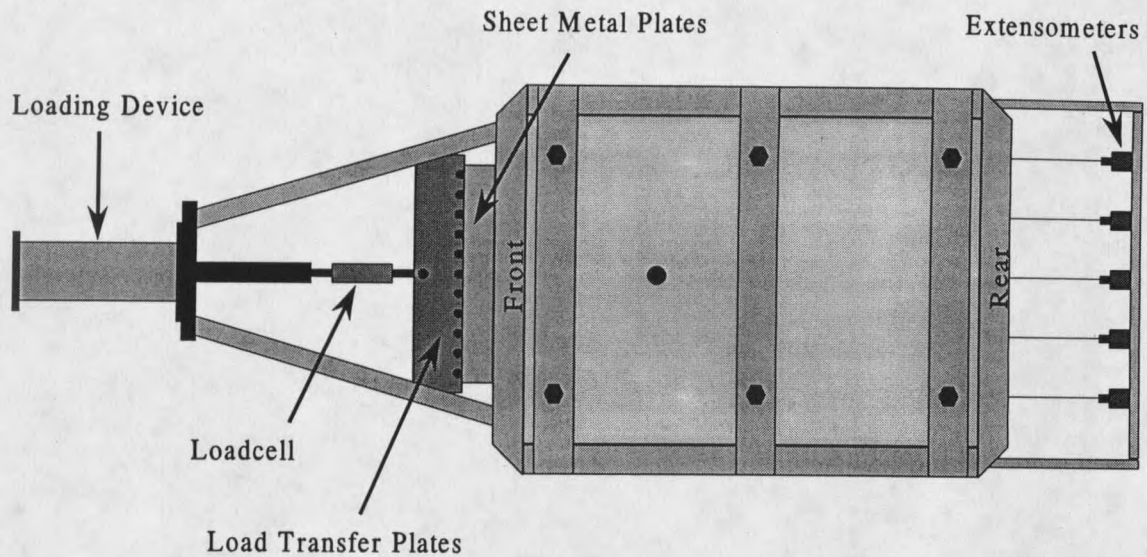


Figure 3.1: Plan view of test apparatus.

Displacement-Rate Controlled Pulling Mechanism

The displacement-rate pulling mechanism used in this study consisted of a mechanical screw driven by a $\frac{1}{8}$ HP, variable speed, DC motor. A complete list of the equipment used in this study along with the manufacturer's name is provided in Appendix D. The screw assembly was capable of applying an ultimate load of 45 kN at a constant rate varying from zero to two millimeters per minute. To achieve these slow rates, a worm gear was placed between the screw and the motor. This pulling mechanism was used when performing monotonic in-air and in-soil tests.

Load Controlled Pulling Mechanism

The load controlled pulling mechanism consisted of two pneumatic cylinders controlled by an automatic binary regulator (ABRTM). Cylinder A had an inside bore diameter of 10.2 cm and a stroke of 25.4 cm. Cylinder B had an inside bore diameter of 15.2

cm and a stroke of 25.4 cm. Because cylinder A had a smaller bore diameter, it could be used at higher pressures giving it finer load control. Because cylinder B had a larger bore diameter, it was used for tests requiring high load levels. Both cylinders were fitted with low friction seals to increase pressure response.

The ABRTM valve, used to control the air pressure to the cylinders, could proportion any value of air pressure into fifteen, equally spaced, increasing pressure levels. The ABRTM valve contained four solenoids controlled by binary signals. Computer driven software was used to arrange the binary signals, thereby dictating the shape of the pulse delivered to the geosynthetic sample.

Clamping Method

An epoxy casting method was used to grip the sample. This method was chosen since it met all of the desired criteria mentioned earlier. Geosynthetic sample ends were cast between two eighteen-gauge pieces of sheet metal using a high strength epoxy. Casting the sample in this way allowed the load to be distributed across the width evenly and reduced high stress concentrations at the sheet metal/geosynthetic interface. This gripping method also had a smooth, low profile design making it very compatible for the in-soil tests. This allowed the sheet metal/geosynthetic interface to be extended into the confining soil without agitating the confinement soil or reducing the soil/geosynthetic contact area during pull-out.

Instrumentation and System Controls

The three primary components of the instrumentation associated with the apparatus consisted of a loadcell, extensometers and foil strain gages. For both in-air and in-soil tests,

a tension loadcell, with an ultimate capacity of 45 kN measured the applied load at the front of the sample. The loadcell was connected between the loading device and the geosynthetic sample using clevis-type couplers. These couplers were offset 90 degrees from each another, to prohibit torquing of the loadcell. Extensometers were used in the in-air tests to measure displacement differences used to calculate strain. Extensometers were used in the in-soil tests to measure displacement along the length of the embedded geosynthetic samples. Five extensometers were available for use; three of them had a range of 25.4 cm, and the remaining two had a range of 12.3 cm. The longer range extensometers were used near the loaded end of the sample since greater displacements occurred there. The extensometers with a shorter range were more accurate and used in areas where smaller displacements occurred. The longer range extensometers were accurate to 0.06 mm and the shorter range extensometers were accurate to 0.04 mm. Each extensometer contained an internally sprung drum onto which the measurement cable was wound. This sprung cable provided constant tension to the extension cables mounted directly to the geosynthetic. For some experiments, strain gages were mounted directly to the geosynthetic material to measure local strains directly.

The two major components of the system controls are the loading system controls and the data acquisition. The computer regulated control system served two purposes: 1) to control the loading system for the cyclic loading case and 2) to collect voltages from each sensor, multiply them by a scale factor and store them in the computer. The loading system control for the monotonic tests was simply an electronic device used to vary the speed of the DC motor.

In-Soil, Pull-Out, Apparatus

The pull-out box was constructed exclusively of steel components. This allowed for future modifications to be made easily. Due to the flexibility of the steel and the high stresses imposed on the structure during testing, deformation of the individual components was the primary criterion governing the design. The components of the pull-out box were designed to be sufficiently rigid so that global movement of the structure did not affect experimental results. The front, back and side walls of the box were made from $\frac{1}{4}$ " steel plating stiffened by $\frac{1}{2}$ " X $3\frac{1}{2}$ " steel flat-bars. The inside dimensions of the finished pull-out box were 1.10 m high by 0.90 m wide by 1.25 m long. Bolted connections were used to fasten individual components of the pull-out box together.

The three major boundaries of concern in the pull-out box design were the gap interface at the front wall, the walls and the top. Comments made concerning their influence on experimental results were presented in Chapter 2. A detailed summary of the designs used for each of these boundaries is discussed in the following three sub-sections.

Gap Interface

As described in Chapter 2, the gap interface at the front of the pull-out box affects experimental results due to induced pressures from pull-out. The design of the gap interface in this study attempted to reduce these effects. The front face of the pull-out box consisted of two components as shown in Figure 3.2. The two components were separated by a distance of 4.0 cm to allow passage of the geosynthetic sample during pull-out. Load-transfer sleeves were attached on the inside face of the front walls (Figure 3.3).

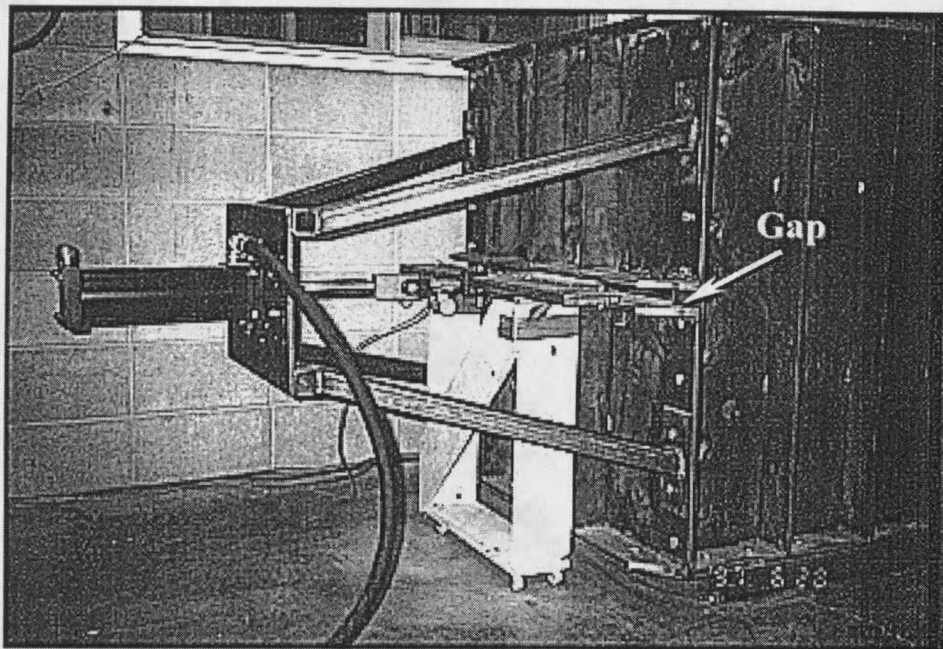


Figure 3.2: Front view of pull-out apparatus showing gap formed by separated upper and lower halves.

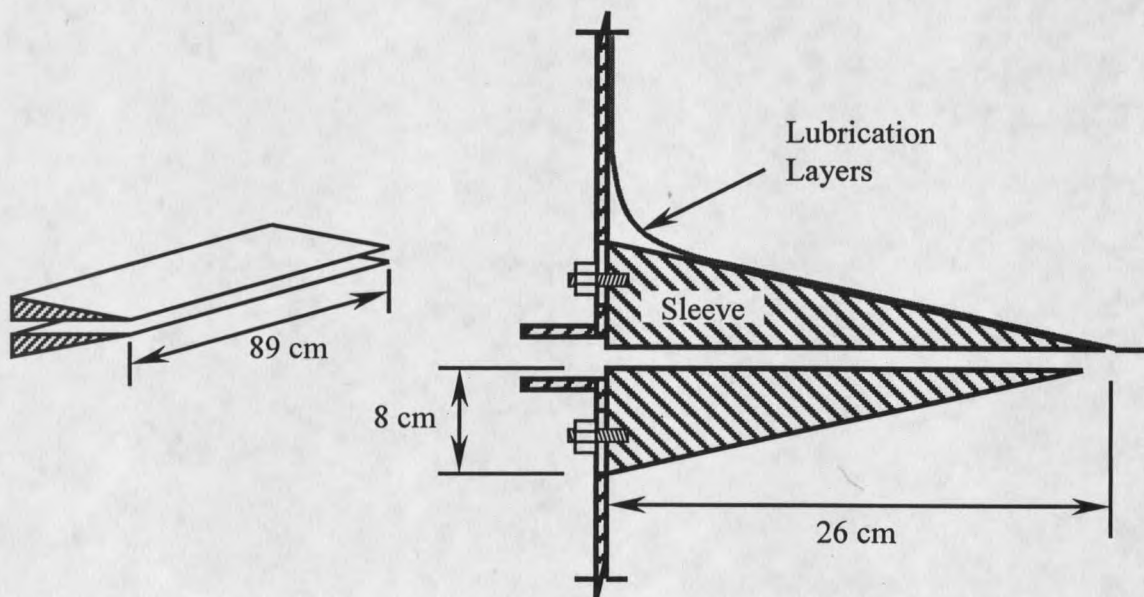


Figure 3.3: Orientation, dimensions and method of attachment of load-transfer sleeves at the front wall.

The load-transfer sleeves were designed to reduce lateral stresses transferred to the front wall during pull-out. Figure 3.3 illustrates their orientation, dimensions and method of attachment to the front wall. The load-transfer sleeves were made entirely of $\frac{1}{4}$ " plate steel. The two sleeves were identical in size and shape being 26 cm in length, 89 cm in width. In their length direction, each sleeve was tapered from a depth of 8 cm at the wall/sleeve interface to an apex at the opposite end. The threaded stud bolts used to connect the sleeves to the front wall were free to move, when loosened, in vertically milled slots in the front wall. This allowed the sleeves to move vertically, making it possible to adjust the gap between them. The tapered construction of the sleeves allowed for natural dilation of the soil during pull-out. As discussed in Chapter 2, allowing upward dilation of the soil above the sample was an especially important component of design.

Walls

Wall boundaries affect normal confinement distribution and pull-out resistance due to friction along their surfaces. All interior surfaces, excluding the lower half of the front and rear walls, were lined to reduce their friction. The lining system consisted of three layers: a smooth geomembrane, low adhesion silicone grease and 1 mm thick latex sheeting.

The semi-rigid geomembrane was rigidly attached to the inside of each of these interior surfaces with rivets. The smooth surface of the geomembrane was thoroughly coated with the silicone grease. The latex rubber membrane was then placed against the greased surface. The two side walls, the top halves of the front and rear faces and the top surface of the upper load-transfer sleeve, were all lubricated using this method.

Top

Application of vertical normal confinement must not restrict dilation of the confining soil during pull-out. A custom-made, pneumatic bladder fitting the plan area of the pull-out box was used for this purpose. Figure 3.4 shows a diagram of the setup used to apply normal confinement to the top of the soil surface. The pneumatic bladder was made of a flexible rubber capable of applying confinement pressures of 200+ kPa. During a pull-out test, the bladder was placed on top of the confining soil with its two pressure inlet hoses facing upward. A reaction lid, made from $\frac{1}{4}$ " plate steel, was placed on top of the bladder. Holes drilled through the reaction lid allowed the inlet pressure lines to pass through without being pinched. Three 2" by 4" by $\frac{1}{4}$ " steel tubes were placed on top of the reaction lid to reduce out of plane bulging along its surface. The three stiffening members were braced by a reaction frame bolted to the top of the pull-out box. The reaction frame, connected to the top of the side walls of the pull-out box, consisted of three 4" by 4" by $\frac{1}{4}$ " steel tubes. Each reaction frame member contained two large bolts to carry the force and allow the thickness of the pneumatic bladder to be varied. A pneumatic pressure regulator delivered constant air pressure to the pneumatic bladder and a high accuracy pressure gauge was used to set and monitor this air pressure.

In-Air Apparatus

In-air tests used the pull-out box frame to house the geosynthetic sample during loading. The fixed end of the in-air sample was held through the gap at the rear of the box and the loaded end was gripped in the same way as the in-soil samples. Loads were applied

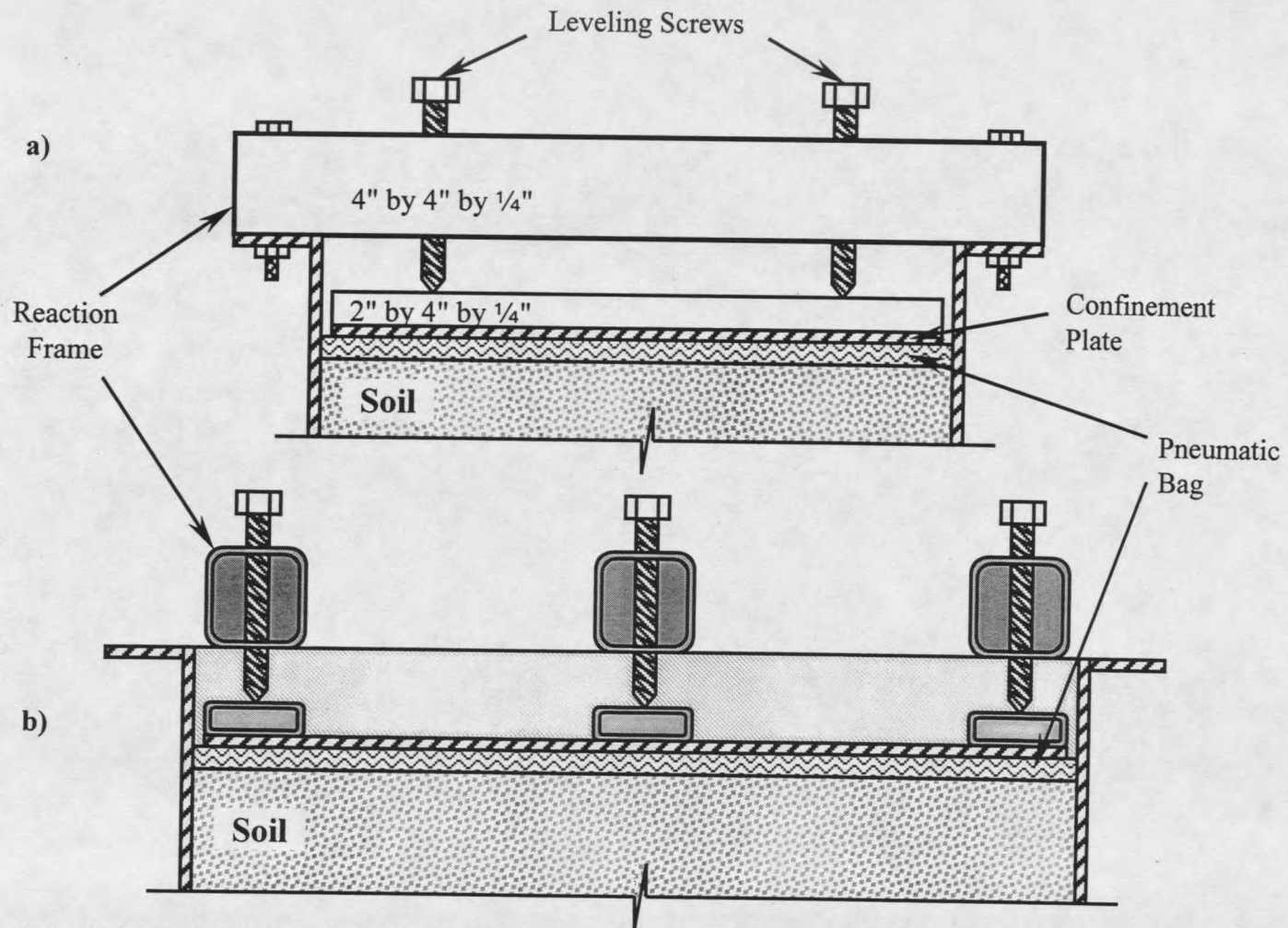


Figure 3.4: Schematic of normal confinement apparatus: a) view from end b) view from side.

using either the displacement-rate controlled or the load controlled pulling mechanisms. The instrumentation and system controls were the same as those used for in-soil tests. The configuration of the extensometers differed since they were used to measure axial and lateral displacement differences to calculate axial and lateral strains. A detailed discussion will follow in subsequent sections of this chapter.

Test Procedures

The test procedures in this study were developed to ensure that geosynthetic test specimens were created and tested using the same techniques. Uniformity between samples and test methods provided assurance that experimental results represented the material's true behavior. The steps outlining the procedures for the in-air tests and the in-soil tests are provided in the following sections. Except minor differences, the same procedure for preparing the sample and connecting the loading device was used for both the in-air and in-soil tests.

In-Air Test Procedure

Fast monotonic, slow monotonic and cyclic in-air tests were performed using essentially the same procedure. Slight differences exist in some steps and will be noted as required. The three steps to performing an in-air test were: 1) preparing the sample, 2) connecting the loading device and instrumentation and 3) running the test.

Sample Preparation

Sample preparation for all of the in-air tests is the same, regardless of the loading method used. The six steps to preparing the sample to be tested are: 1) selecting, trimming and cleaning the geosynthetic, 2) preparing the sheet metal, 3) placing the materials within the jig, 4) mixing and preparing the epoxy, 5) gluing the sample, and 6) drilling the holes for the pulling system.

A biaxial, polypropylene geogrid and a biaxial, woven, polypropylene geotextile were the two commercial geosynthetic materials used in this study. These materials have different properties in the machine directions (MD) and the cross-machine direction (XD). Table 3.1 shows the pertinent characteristics of each of these materials in each of these directions, as reported by the manufacturer.

Product Type	Structure	Polymer Composition	Weight (g/m ²)	Aperture Size MD/XD (mm)	Strength at 5% Strain MD/XD (kN/m)	Ultimate Strength MD/XD (kN/m)
Tensar BX1000 Geogrid	Punched Sheet Drawn Biaxially	Polypropylene	215	25/33	9/13	13/20
Amoco 2006 Geotextile	Woven	Polypropylene	250	N/A	10/22	34/37

Table 3.1: Material properties of the geogrid and geotextile as reported by the manufacturer.

It is well known that geosynthetic materials are slightly nonhomogeneous. Since this is true, two criteria for choosing a representative sample from the roll were developed. The first was to maintain a certain distance from the edges of the roll. Visual inspection showed

that a higher concentration of dissimilarities occurred near the roll edge. Maintaining a distance of 0.5 meters from the edge of the geogrid roll and 0.1 meters from the edge of the geotextile roll was sufficient distance to ensure uniformity between test samples. Related to this, the second criterion was to visually inspect the sample region for other manufacturing abnormalities. It was desired that the geosynthetic samples were as uniform as possible so that results between tests would be more comparable.

Once a sample was taken from the roll, it was cut to size with respect to either the machine direction or the cross-machine direction. The machine direction is the direction in which the material is rolled from the manufacturer as it comes out of the manufacturing machine, the cross-machine direction is orthogonal to the machine direction, making it longitudinal to the roll. For all of the in-air tests, the final size of the extensible portion of the samples was approximately 75 cm in length and 28 cm in width as shown in Figure 9.5. This corresponds to an aspect ratio (width/length) of approximately 0.37. The sheet metal plates bonded to the front and rear of the sample were 15.0 and 18.0 cm in length, respectively, making the total length that the unbonded geosynthetic needed to be cut 108 cm. For the geogrid material, bonding between the geosynthetic and the sheet metal was not a concern since the open apertures allowed epoxy to flow around the material and bond to each of the sheet metal plates. However, the epoxy did not bond well to the tightly woven surface of the geotextile. This was partially remedied by using sulfuric acid to remove residue oils deposited from the manufacturing process and also to etch the geotextile surface slightly.

Once the sample was selected and prepared, the next step was to prepare the sheet metal ends. Eighteen-gauge sheet metal was used because transferring loads through its plane without excessive deformation was possible. Each of the sheet metal pieces was slightly wider than the geosynthetic sample, being 30.5 cm in width. The side of the sheet metal that was to have the epoxy bonded to it was roughened with a belt sander and cleaned with acetone to give the glue a good bonding surface.

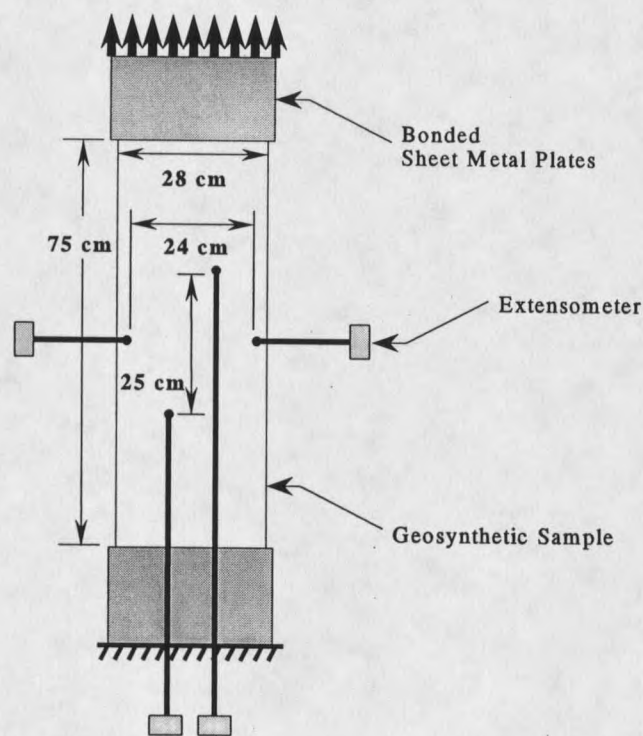


Figure 3.5: Schematic of in-air specimen showing overall dimensions and sensor location.

A sample jig was used to align the sample with respect to its sheet metal ends. Figure 3.6 shows a photograph of the sample jig set up. The lower halves of the sheet metal plates were set in the jig and the epoxy was mixed and poured onto them. Then the sample was stretched between the two sheet metal ends and the top pieces of sheet metal were coated

with glue and placed onto the lower pieces. For the geotextile samples, the glue was massaged into the fibers to attempt to increase bonding through the sample to each of the sheet metal plates. Weights were stacked onto the sheet metal ends to assist in distributing the glue evenly. The epoxy was allowed to dry for at least twelve hours and the excess was trimmed off the edges.

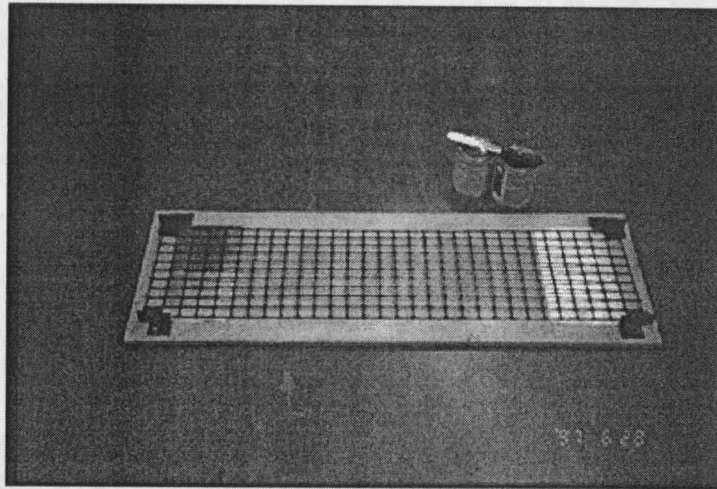


Figure 3.6: Sample jig for preparing in-air test specimens.

The sheet metal ends were then ready to be drilled for placement into the test apparatus. Holes were drilled near the outward edges of the two sheet metal ends. One set of holes allowed two pieces of angled steel to be attached to it that would react against the box frame, thus constituting the fixed end. The other end was connected to the loading apparatus. The front of the sample was bolted between two pieces of eighteen-gauge sheet metal used as extensions due to the large size of the pull-out box. The other ends of the sheet metal extensions were attached to two load distribution members with dimensions 12.7 cm by 89.5 cm in width and length, respectively. Figures 3.7 and 3.8 show each of these setups.

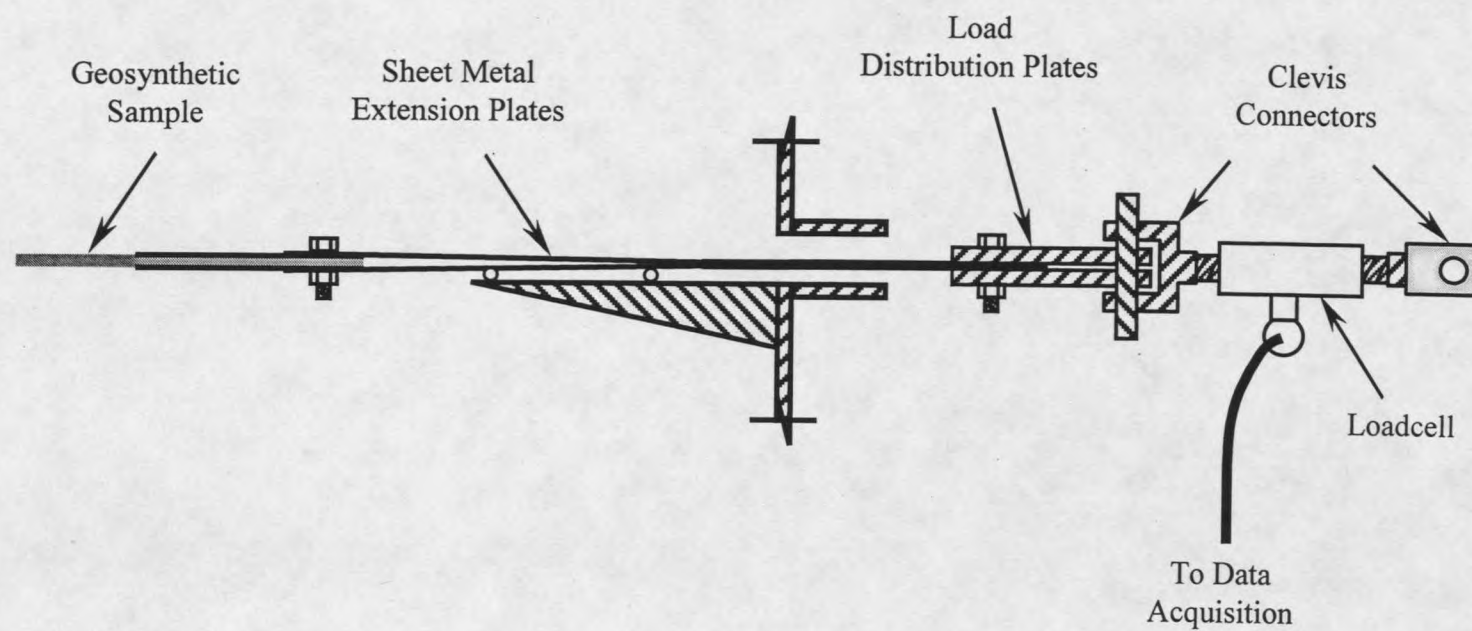


Figure 3.7: Side view of in-air sample clamping method including loadcell connection.

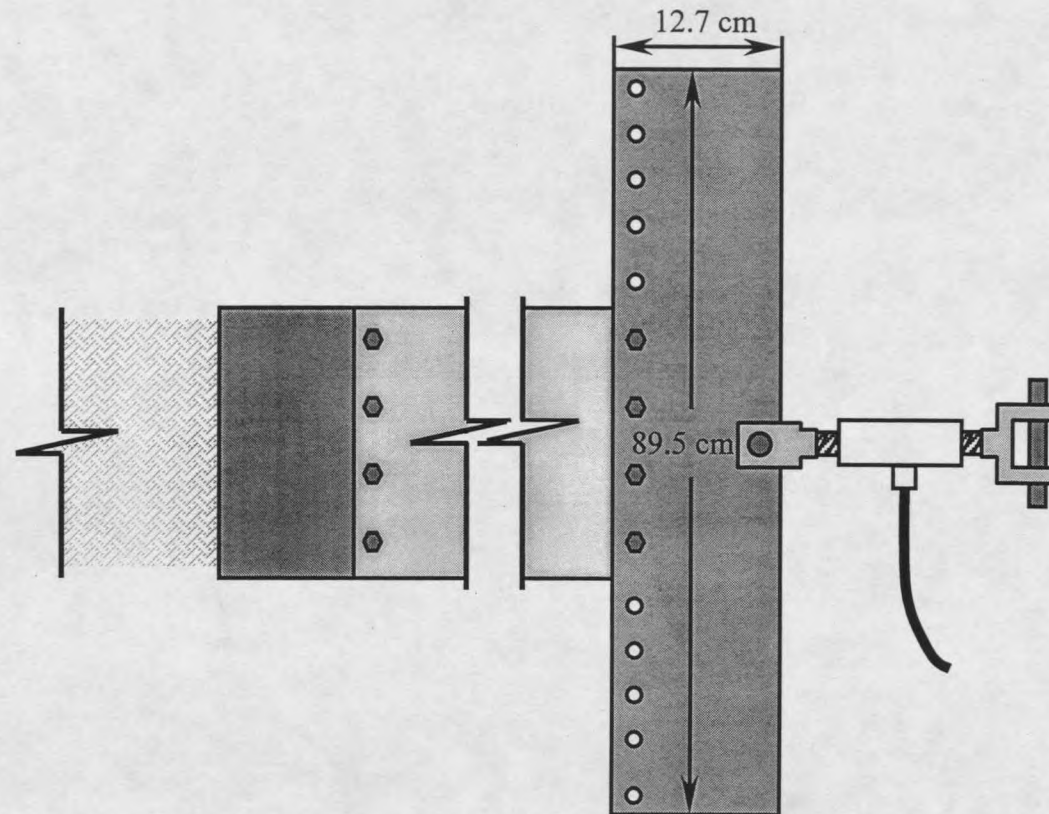


Figure 3.8: Plan view of sample clamping method including loadcell connection.

A centered hole, with respect to the bolted connection, was made through the load distribution members to allow the tension loadcell to be pinned to it. The other end of the loadcell was attached to either the monotonic or cyclic pulling device.

Loading Device and Instrumentation Connection

The instrumentation used in the in-air tests consisted of one loadcell, four extensometers and bonded, foil strain gages. One millimeter diameter extension wires with special metal clips on one end, were used to bridge the connection between the extensometers and the measurement point on the geosynthetic sample. The extension cables were run through grooves cut into the angled pieces that provided the fixity of the rear of the sample. Since the sheet metal ends prohibited lateral strains from occurring near the geosynthetic/sheet metal interface, the extensometers used to measure axial strain were positioned to span the middle third of the sample. The extensometers used to measure lateral displacements were positioned at the centerline of the sample near the edge. The gage length of the axial and lateral strain measurements were approximately 25 and 24 cm, respectively, as previously shown in Figure 3.5. When direct measurements of strain were needed, foil strain gages were bonded directly to the material using a high elongation epoxy. These gages were not used for finite element model calibration but simply to determine how well predictions of global strain in the material can be made through local strain measurements. Appendix A contains specific information regarding methods used for bonding foil gages to the geosynthetic and a comparison of typical results from the global and local strain measurements.

Fast Monotonic Tests

Fast monotonic tests were performed at high rates of displacement. To achieve these high rates, the pneumatic cylinder was programmed to deliver a linearly increasing load to the sample in a very short time. The maximum load was supposed to be able to rupture the sample at its peak pressure in less than one second. However, since the capacity of the load controlled pulling mechanism was lower than the ultimate strength of the geogrid in the cross-machine direction and the geotextile in either direction, rupture was not attainable for these materials in these directions.

Slow Monotonic Tests

For the slow monotonic tests, the samples were pulled at a rate of 2 mm per minute corresponding to a strain rate of approximately 0.26 % per minute. These tests were continued until failure of the sample occurred. Failure was defined as the point when either the geosynthetic material ruptured or peak strength was reached. Overall, the geogrid materials used in this study ruptured but geotextile samples did not due to premature failure at the epoxy/geotextile interface.

Cyclic Tests

Loads in the series I and II, cyclic tests, were applied using either of the two pneumatic cylinders. The data acquisition/load control system was used to control the air pressure input to the pneumatic cylinder in the form of a triangular wave. Each applied load cycle consisted of four stages as shown in Figure 3.9. Stage one consisted of a linearly increasing pressure, stage two, a constant pressure, stage three, a linearly decreasing pressure and stage four, a zero load, time delay between cycles. Figure 3.10 shows the input cycles

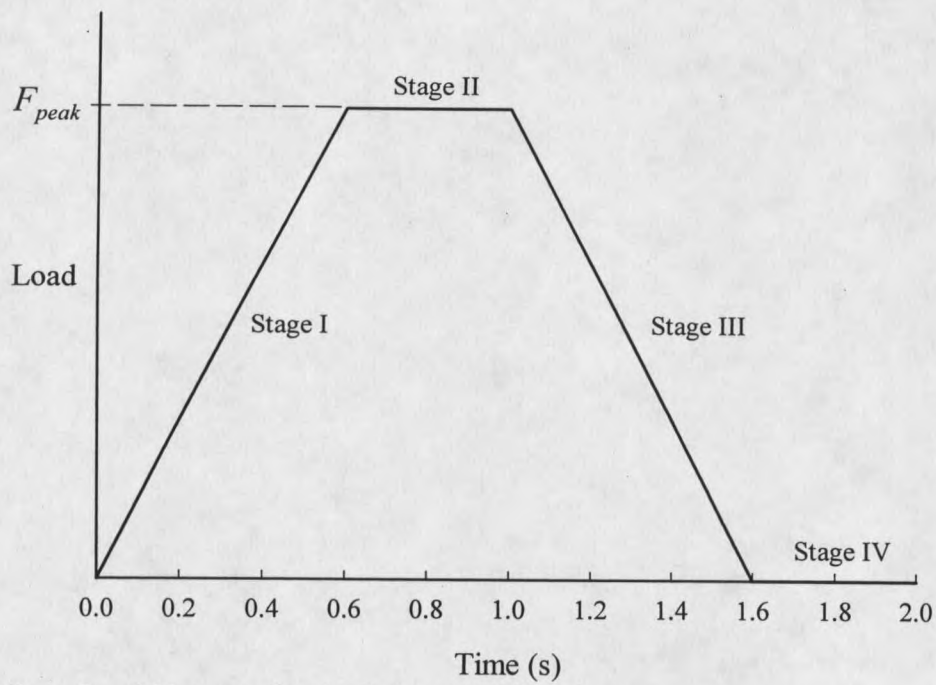


Figure 3.9: Load history of a single input load cycle.

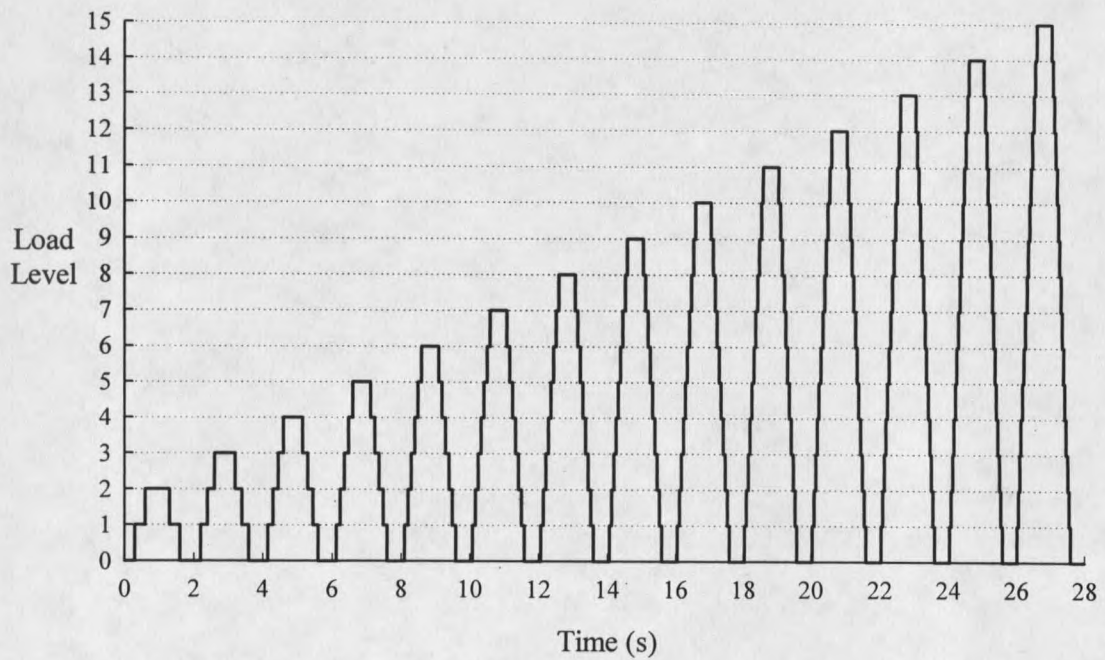


Figure 3.10: Load levels for the cyclic load input pulses.

for the fourteen input pressures used in the two series of cyclic tests. At low pressures, clarity of the linearly increasing and decreasing portions declined due to the low number of pressure increments at those steps. Cyclic, series I tests applied one cycle at each load level for all fourteen load levels and the series II tests applied repeated cycles at each of the fourteen load levels. For the series II tests, cycles were repeated at each load level until the material no longer exhibited creep strains. A list of the number of cycles repeated in the series II test at each load level, is given in Table 3.2. The total number of load cycles to be applied to the in-air series II cyclic tests was 5800. However, for some tests, this total number of cycles was not attained due to the instability of the software controlling the data acquisition/load control system. The system was configured to record the maximum and minimum response values of each sensor for all cycles, and full cycle data for specified cycles.

Load Levels	No. of Cycles
2, 3, 4	100
5, 6, 7, 8	300
9, 10, 11	500
12, 13, 14, 15	700

Table 3.2: Cycle numbers repeated at each load level for the in-air, cyclic, Series II tests.

In-Soil Test Procedures

Monotonic and cyclic in-soil tests were performed using essentially the same procedure. The five steps to performing an in-soil test were: 1) preparing the sample, 2) compacting the soil and placing the sample, 3) connecting the loading device and instrumentation, 4) applying normal confinement and 5) running the test.

Sample Preparation

Sample preparation for the in-soil tests were similar to the in-air tests; the main differences being the size and layout. The same procedure was used to select, trim and clean the geosynthetic and prepare the sheet metal. The sheet metal plates bonded to the geosynthetic samples used in the in-soil tests consisted of two 61 cm square pieces of eighteen-gauge, galvanized steel. For these tests, one end of the geosynthetic sample was connected to the loading apparatus while the other end was free. Figure 3.11 shows a typical in-soil sample placed in the box. To provide a connection to the extensometer measuring the displacement of the front of the sample, a 1 mm cable was glued between the sheet metal pieces, extending from the center of the sheet metal/geosynthetic interface.

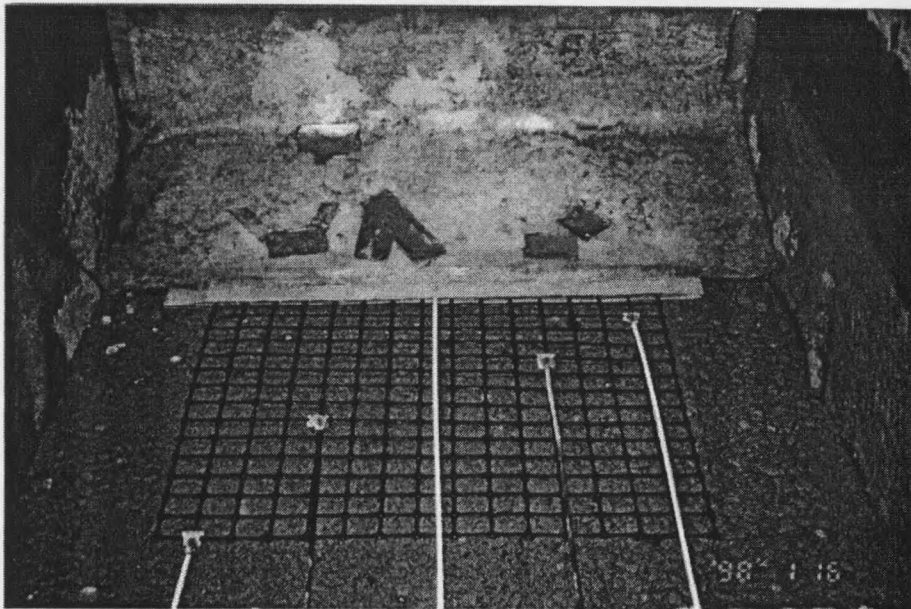


Figure 3.11: In-soil sample placed in pull-out box.

Soil Compaction and Sample Placement

Before the sample could be placed into the pull-out box, soil had to be compacted up to the level of the extended edge of the lower load-transfer sleeve. The soil used in these tests was a well-graded, $\frac{3}{4}$ " minus, base course aggregate having the gradation shown in Figure 3.12. This gravel was compacted to at least 95% of its optimum dry density. Figure 3.13 shows a modified, Proctor, compaction curve for this material. To compact the soil, a vibrating compactor was designed and built. It consisted of a compressed-air driven ball-vibrator attached to a 15 kg mass welded to a 25 cm by 25 cm plate. To ensure that the desired density was achieved, density of the soil was measured using a sand-cone apparatus.

Once the compacted gravel filled the lower half of the pull-out box, the soil was scarified to ensure equal soil/geosynthetic bond on both sides of the material. Then the prepared sample was placed into the apparatus by feeding it through the narrow gap formed by the two load-transfer sleeves at the front of the box. For the monotonic tests, the glued connection between the sheet metal and the geosynthetic was placed into the soil a distance of 15.0 cm from the edges of the load-transfer sleeves. Placing the epoxy/sheet metal interface in the soil ensured that the area of geosynthetic sample in contact with the soil was constant throughout the test. Figure 3.14 shows a schematic of this setup. For the cyclic, in-soil tests, this distance was reduced to 5.0 cm to reduce the friction along the non-geosynthetic surface within the soil. Excavation was taken to a depth of approximately 20 cm below the level of the previous sample to ensure no effects from the previous experiment.

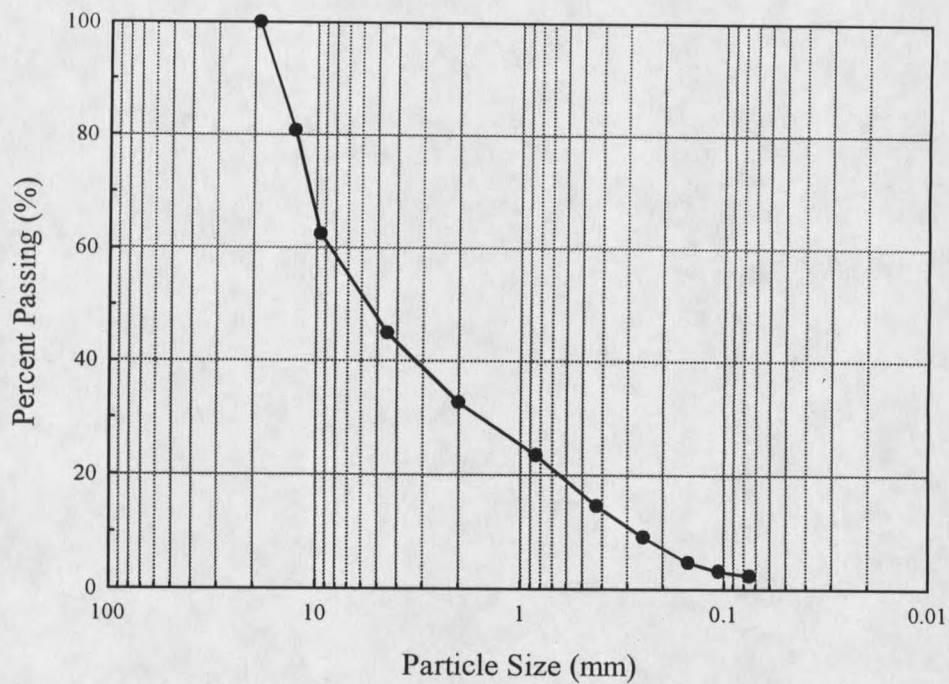


Figure 3.12: Grain-size distribution of the confinement soil used in pull-out experiments.

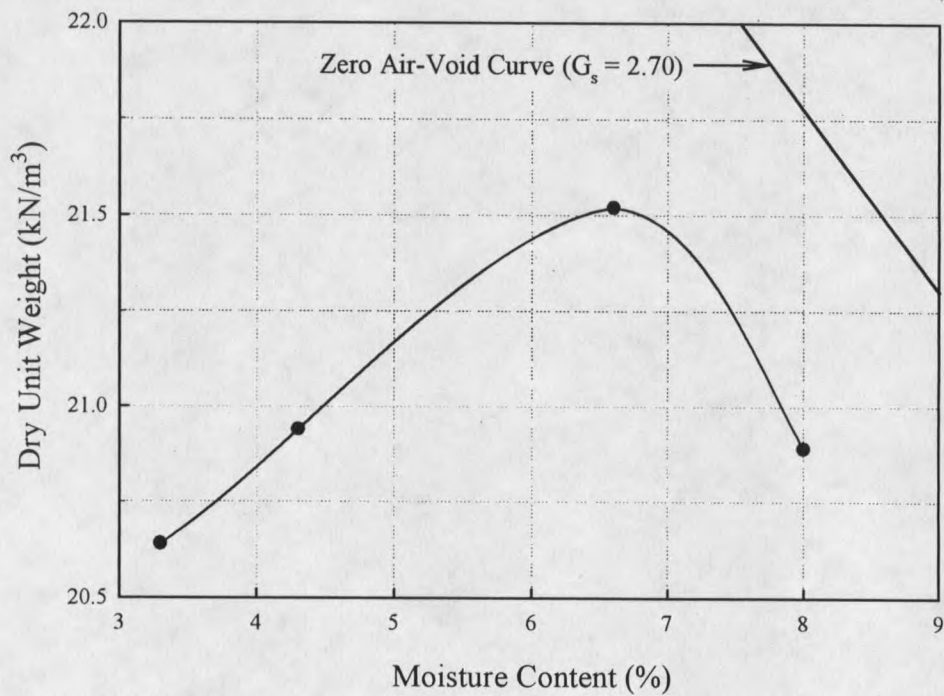


Figure 3.13: Modified Proctor compaction curve for soil used in pull-out experiments.

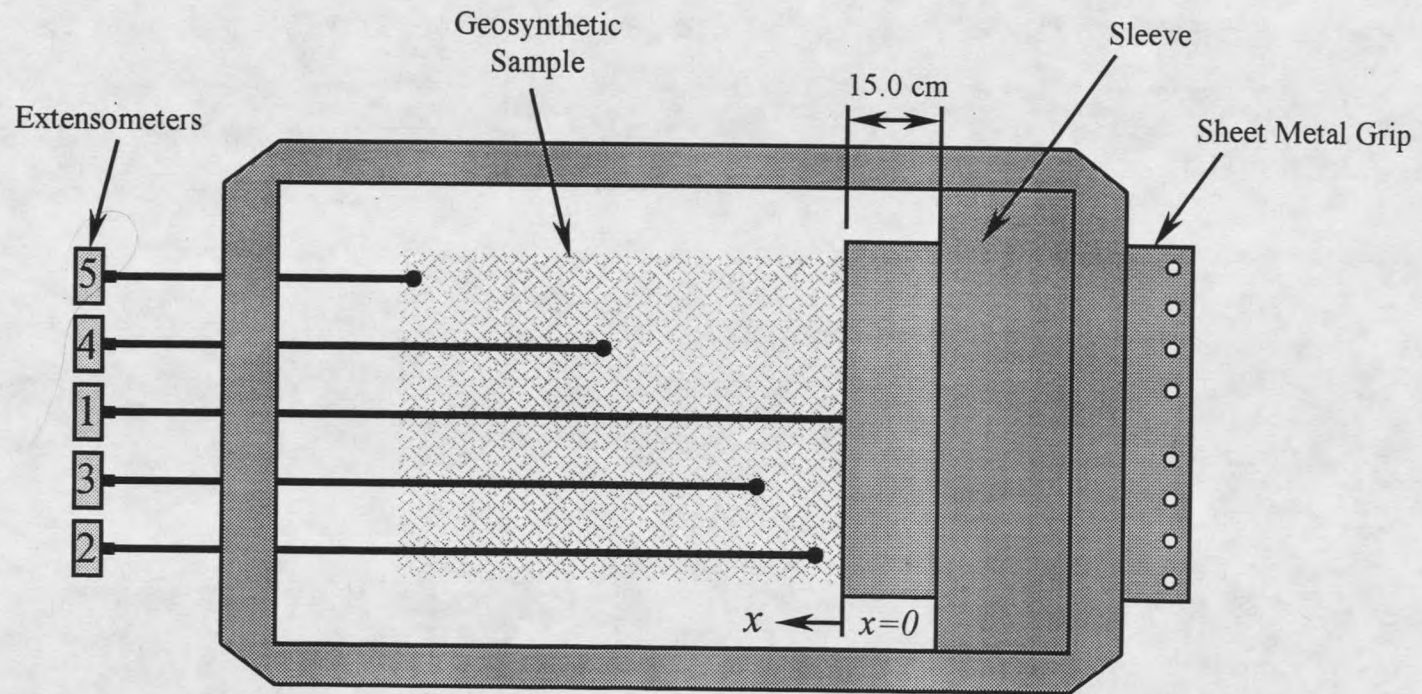


Figure 3.14: Plan view of in-soil sample arrangement.

Loading Device and Instrumentation Connection

Connection of the loading device is like that of the in-air tests, the difference being that the loaded end of the sample was connected directly to the sample without needing sheet metal extensions. For in-soil tests, measurements of displacement were needed from different positions along the embedded length of the geosynthetic including the displacement of the sheet metal plates. The displacement of the sheet metal is measured using the cable glued between the sheet metal as discussed earlier. The other displacement connections were the same as the in-air tests, which were made by a low profile clip connected directly to the geosynthetic. A total of five displacement measurements were made along the embedded length of the sample. Figure 3.14 also shows the approximate location of these measurement connections. For the geogrid tests, connection of the cable clips to the material did not cause excessive resistance at the measurement points. However, this type of clip connection on the geotextile created excessive resistance at the measurement points, making the displacement data invalid. A new method of connecting the measurement cable to the geotextile used a thin cable punched through and glued to the material. A piece of smooth tape was placed over the glued area and a small piece of lubricated latex was placed on the surface of the tape. These connection points had a very low profile making the resulting data more sensible. The cables attached to these clips were housed in reinforced tubing to protect them from the confining soil, allowing them to move freely through the soil. The cables extended through the rear of the pull-out box through a small slit and were attached to the extensometers.

Normal Confinement

Once the loading apparatus and instrumentation were connected to the sample, confining soil was compacted on top of the sample to a depth of approximately 40 cm in lifts of approximately 6 cm in depth. Once a level surface of soil of approximately 40 cm in height above the sample was attained, the confinement system previously described was placed on top of it.

Three confinement levels were used for these tests: 5, 15 and 35 kPa. For the 5 kPa tests, confinement was reached using a depth of approximately 22 cm of soil above the sample. For the 15 and 35 kPa tests, additional confinement was applied using the pneumatic bladder described earlier.

Conducting the Test

Three types of in-soil tests were performed: monotonic, cyclic and sheet metal tests. Monotonic tests were conducted to describe the shear stress distribution along the embedded length of the reinforcement during pull-out. Cyclic tests were performed to help calibrate in-soil cyclic parameters within the finite element model. In-soil tests on sheet metal were performed to provide a load correction to the output data for the monotonic tests. Cyclic tests did not need to be corrected since the length of the sheet metal protruding into the soil was shortened and lubricated layers were used to reduce the friction along its surface.

Monotonic Tests

Monotonic, in-soil tests were performed using the displacement controlled pulling device set at a rate of 1 mm per minute. Monotonic tests were run until failure occurred in the sample. Failure for these tests was defined as either complete mobilization of the embedded geosynthetic or rupture of the sample. Complete mobilization of the sample was attained when the free end of the sample began to move.

Cyclic Tests

Cyclic tests are conducted using the Series II cyclic loading. Like the in-air cyclic tests, the cycles were repeated at fourteen load levels. Table 3.3 shows the number of cycles delivered at each load level. The total number of cycles applied to the geosynthetic for the in-soil cyclic tests was 11,600. Like the in-air cyclic tests the maximum and minimum values for every sensor were recorded for every cycle applied. Full cycle data was also recorded for specified cycles.

Load Levels	No. of Cycles
2, 3, 4	200
5, 6, 7, 8	600
9, 10, 11	1000
12, 13, 14, 15	1400

Table 3.3: Cycle numbers repeated at each load level for the in-soil, cyclic tests.

Test Results

The purpose of this section is to demonstrate the methods used to obtain meaningful results from raw data and to illustrate typical results from each of the experiments performed. Corrections made to the data, to account for various material behaviors and boundary conditions, are explained in detail. A complete set of results from the in-soil tests is contained in Appendix B. A discussion of the results presented in this chapter will be provided in Chapter 4.

In-Air Test Results

To calibrate the constitutive parameters of the geosynthetic materials, in-air tests were conducted. Monotonic tests were used to define the geosynthetic's orthotropic, elastic plastic material properties. Fast monotonic tests were used to define geosynthetic material parameters without creep. Further calibration of the geosynthetic's elastic and plastic material properties and verification of the finite element model's prediction of creep was accomplished using the slow monotonic tests. Cyclic tests were performed to determine the material's kinematic hardening properties.

Test Data Corrections

Two corrections were made to the raw data obtained from in-air tests. The first correction was to account for the reduction in width due to necking and the second was to account for the number of ribs per width spanning the width of the geogrid samples.

Axial loading of geosynthetic samples caused necking to occur. Necking is the reduction in the width of a sample under axial loading. A correction was needed to account for this reduction in width since measured loads had units of force per width. As discussed earlier, lateral displacements were measured in the sample at midlength to calculate lateral strain. Lateral strain measurements were used when calculating the axial load per width by continuously updating the sample width. The updated width was calculated using the following equation:

$$W_{corrected} = W_{initial} (1 - \epsilon_{lat}) \quad (3.1)$$

where $W_{corrected}$ is the corrected width, $W_{initial}$ is the initial width of the geosynthetic sample and ϵ_{lat} is the measured lateral strain at a particular load level.

Geogrid samples containing the same number of ribs may not have the same width due to material nonuniformities created during the manufacturing process. Applied loads are measured in units of force per width, so corrections to the width must be made to account for these material dissimilarities. Using equation 3.2 developed from concepts contained in the ASTM D5262 specification, load values from tests performed on samples of geogrid with different widths can be corrected to a 'true' load per width. This correction can also be used to make valid data comparisons in cases where tests performed on geogrid samples may have had similar aspect ratios but different overall sizes.

$$P_{corrected} = \frac{P_{measured} (n-1)}{n} \quad (3.2)$$

The variable $P_{corrected}$ was the corrected load, $P_{measured}$ was the measured load and n was the number of ribs across the width of the sample.

Fast, Monotonic, In-Air Tests

Measurements made in the fast monotonic in-air tests included axial strain, lateral strain and load. Plots of load versus axial strain were formed using this data for each material as it was pulled in the machine and cross-machine directions, as shown in Figure 3.15. The two curves shown in each of the material's principle strength directions represent repeated tests. Differences between repeated tests are slight and are most likely attributed to geosynthetic nonhomogeneities rather than testing methods. To calculate pertinent material parameters, plots of axial strain versus lateral strain were needed. These plots can be seen in Figure 3.16 for both directions of the geogrid and geotextile, respectively.

Slow, Monotonic, In-Air Tests

Slow monotonic tests were performed to verify the creep parameters used in the finite element model and to further define the material's elastic/plastic parameters. Like the fast monotonic tests, measurements of applied load and axial and lateral strains were made. Pertinent corrections were also implemented to adjust raw data. Figure 3.17 shows plots of load versus axial strain for the geogrid and geotextile in each of their principle strength directions. Plots of axial strain versus lateral strain for these materials are shown in Figure 3.18. To calibrate the material's shear parameters, slow monotonic tests were performed on both materials oriented in the 45 degree direction. Results from the geogrid are shown in Figures B.9 and B.10 in Appendix B. Results from the geotextile indicated that the material had zero strength in the 45 degree direction and therefore will not be shown.

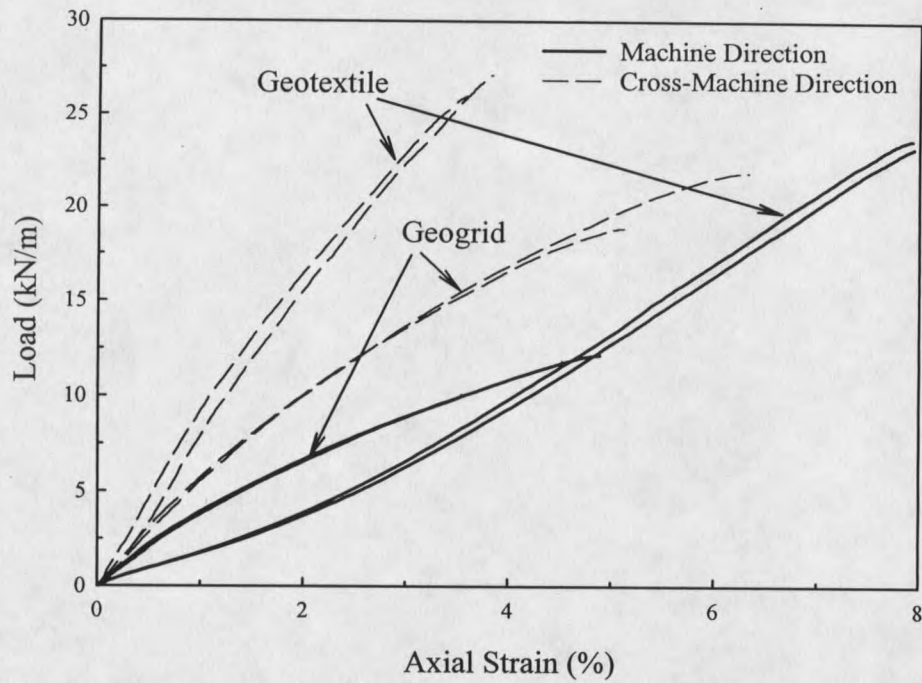


Figure 3.15: Fast, monotonic, load/strain curves for the geogrid and the geotextile in both principle strength directions.

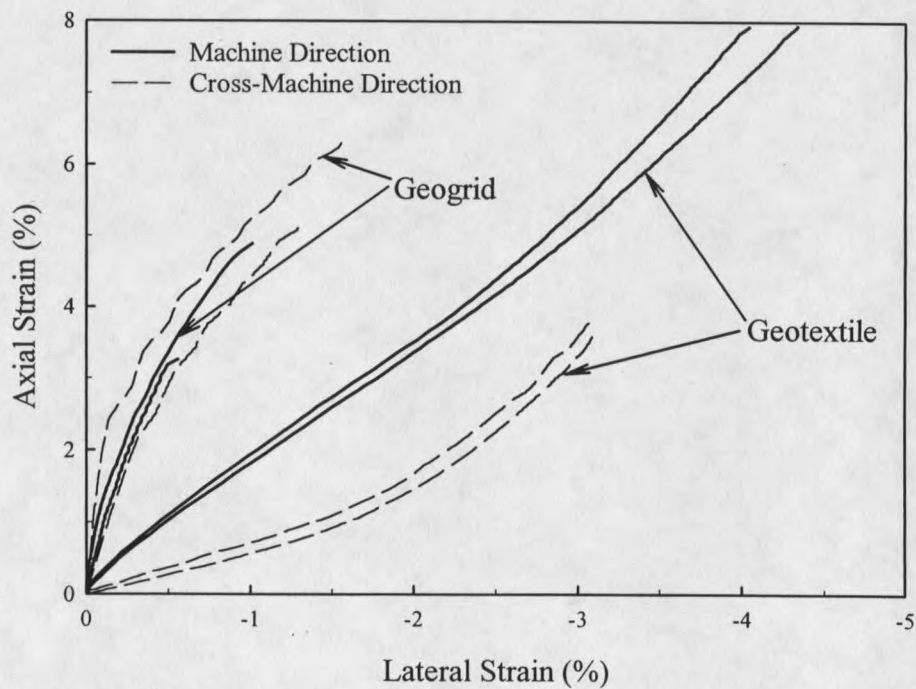


Figure 3.16: Fast, monotonic axial strain versus lateral strain for the geogrid and geotextile in both principle strength directions.

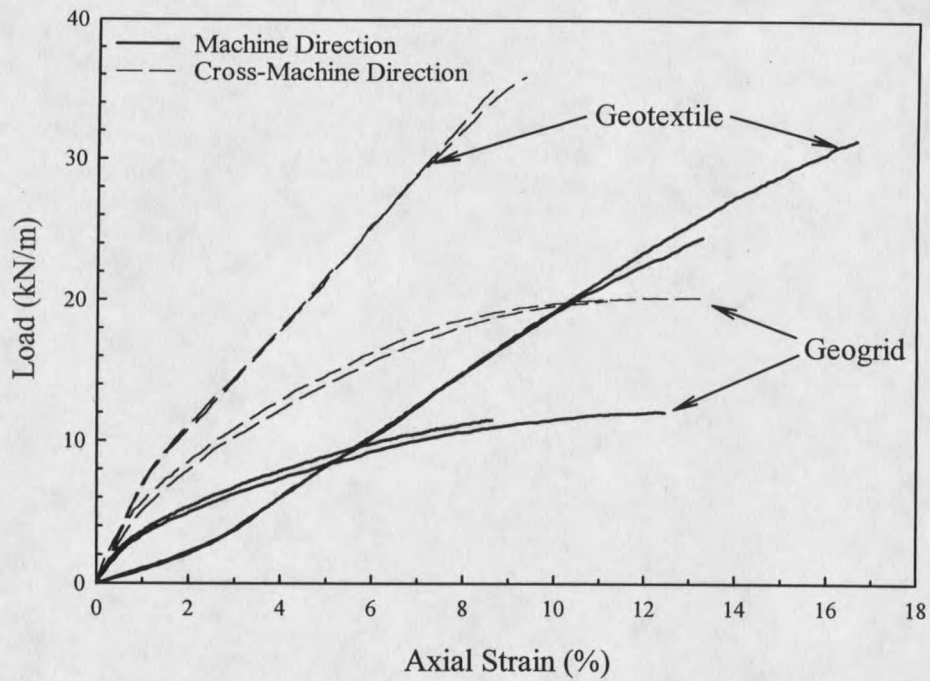


Figure 3.17: Slow, monotonic, load/strain curves for the geogrid and the geotextile in both principle strength directions.

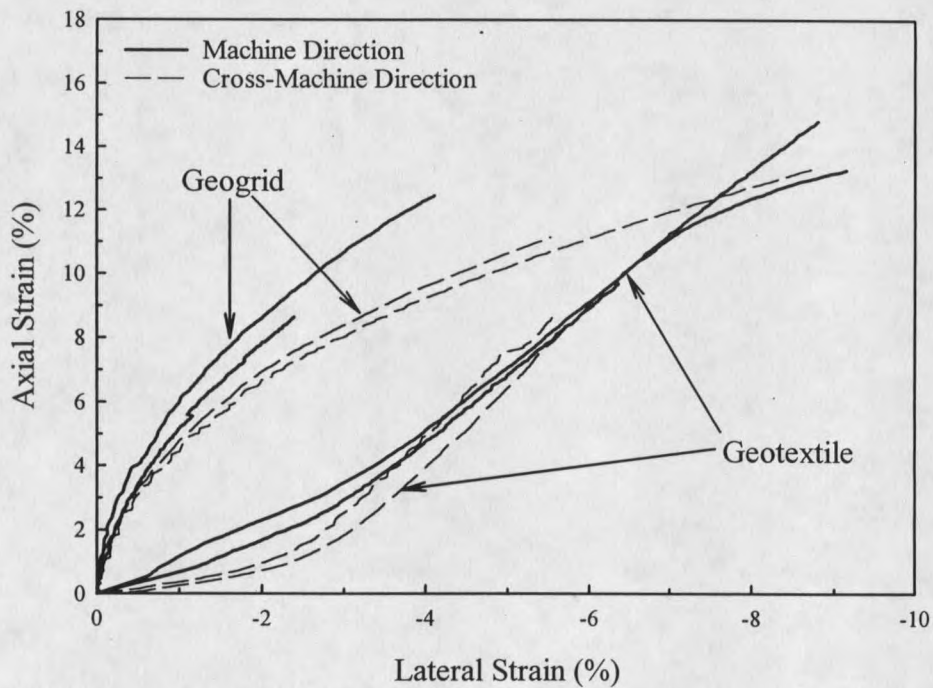


Figure 3.18: Slow, monotonic axial strain versus lateral strain curves for the geogrid and geotextile in both principle strength directions.

Cyclic, In-Air Tests

Cyclic, in-air tests were conducted under two types of loading (Series I and Series II) to define the geosynthetic's kinematic hardening properties. Measurements of applied load and axial and lateral strains allowed plots of load versus strain for each of the material types to be shown. Figure 3.19 shows a plot of load versus axial strain for the geogrid in the machine direction when loaded under the Series I loading, and Figure 3.20 shows a plot of load versus axial strain for the geogrid in the machine direction under the Series II loading. The plot for the Series I loading contains all of the applied cycles, whereas the Series II plot only contains specific cycles. A complete set of plots showing the results from the cyclic, in-air tests is given in Appendix B.

In-Soil Test Results

In-soil tests were conducted to define soil/geosynthetic interaction parameters. Parameters derived from plots of soil/geosynthetic, interface shear stress versus shear displacement for various confinement levels were used to define soil/geosynthetic interaction and were obtained from monotonic tests. Monotonic, pull-out, tests were performed at three levels of confinement: 5, 15 and 35 kPa. Cyclic tests were also performed on each material at a confinement level of 15 kPa. These were used to calibrate the finite element model's prediction of the soil/geosynthetic interaction under this type of loading. Details regarding the interpretation of raw data into meaningful results are given in the following sections.

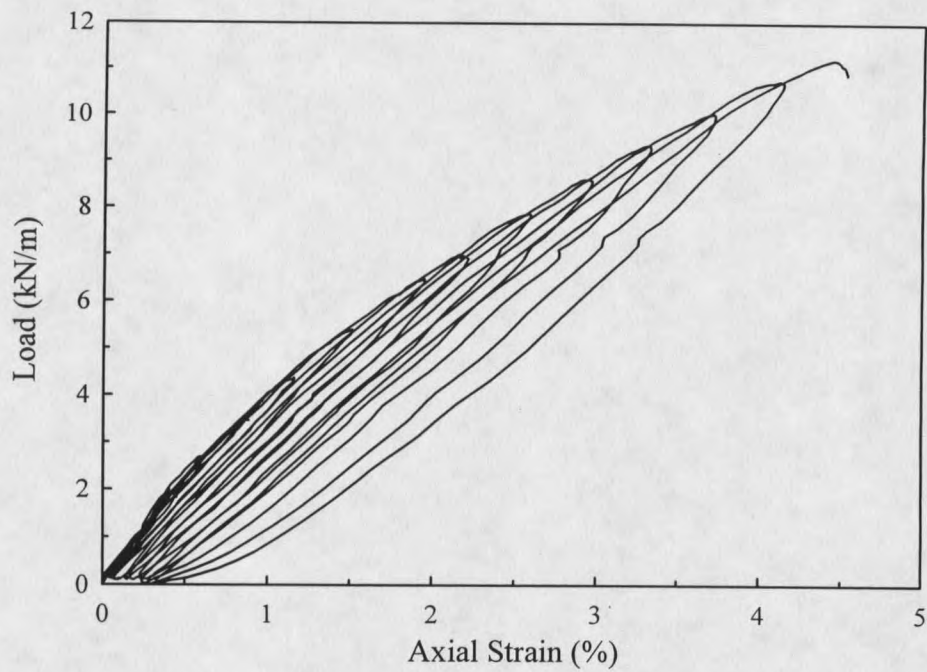


Figure 3.19: Cyclic, Series I, load/strain, loading/unloading curves for the geogrid in the machine direction: ggmd-3.

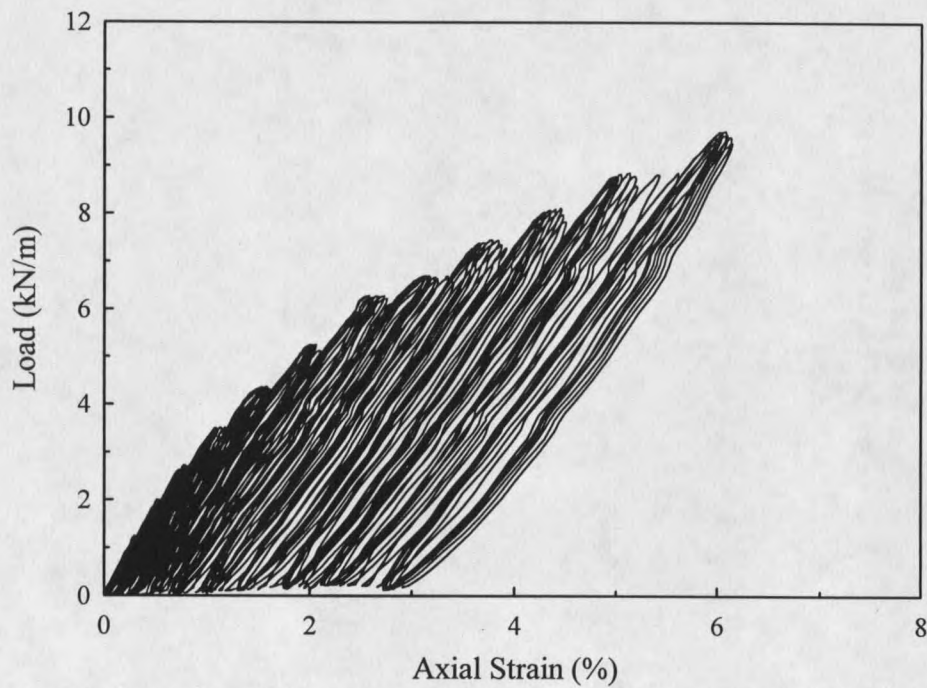


Figure 3.20: Cyclic, Series II, load/strain, loading/unloading curves for the geogrid in the machine direction: ggmd-6.

Test Data Corrections

Two corrections were made to the raw test data. The first correction used Equation 3.2 to account for the number of ribs along the width of the sample, as in the in-air tests. The second correction adjusted the applied pull-out load by accounting for friction carried along the surface of the sheet metal grip within the soil.

To determine the amount of frictional load due to the sheet metal grip in the soil to be subtracted from pull-out tests, monotonic, in-soil tests were performed on samples of sheet metal. Recall from earlier discussion how the sheet metal grips for the geosynthetic extended into the confining soil 15.0 cm from the leading edge of the load-transfer plates, as shown in Figure 3.14. The pull-out tests performed on sheet metal were arranged identically to the geosynthetic samples excluding the geosynthetic materials. In these tests, measurements of applied load and displacement of the embedded edge were made. Tests were performed and repeated on the sheet metal under the same three levels of normal confinement and at the same pull-out rate of 1 mm per minute. This allowed load/displacement curves to be formed for each level of confinement. A computer program was written to average data from repeated sheet metal tests and subtract it from the raw data obtained from monotonic tests conducted on the geosynthetic under the same level of normal confinement. Averaged results from the repeated sheet metal tests at each level of confinement are shown in Figure 3.21.

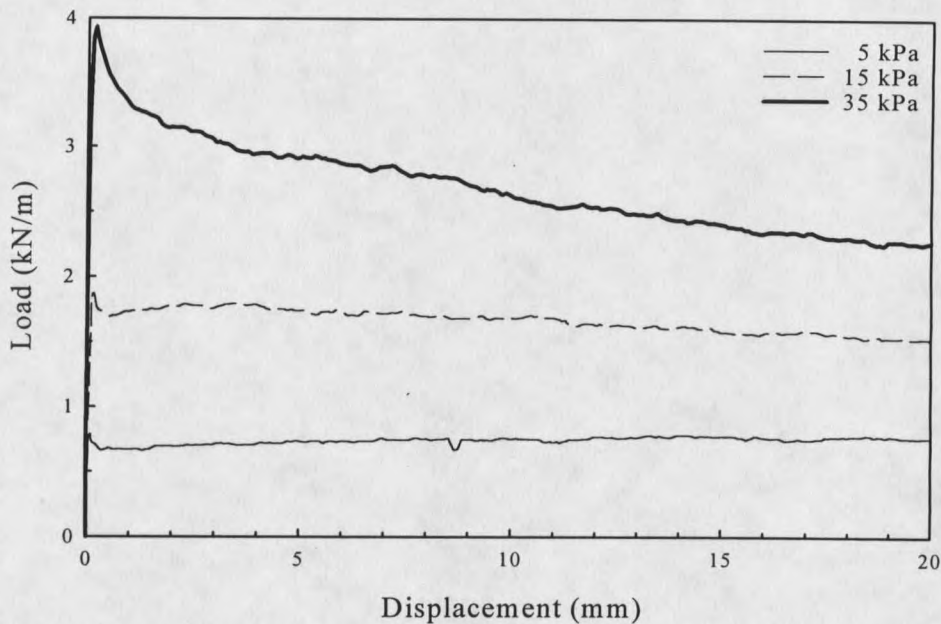


Figure 3.21: Averaged load versus displacement curves for the in-soil, sheet metal tests.

Monotonic, In-Soil Tests

Measurements of applied pull-out load and displacements along the embedded length of the sample allowed load versus displacement curves to be prepared. An example of the load versus displacement for a geogrid sample at three levels of normal confinement is given in Figure 3.22. To analyze the data using either the mobilizing process method or a numerical technique, having a plot of displacement along the embedded length of the sample is necessary. Since the displacement at various points along the length of the embedded sample changes with increasing load, several curves can be shown in one figure. Figure 3.23 shows the displacement along the length for six load levels for a geogrid test pulled in the machine direction at 5 kPa normal confinement. These load levels were calculated based on percentages of the ultimate load carried in the material at failure. The percentages used are 25, 50, 75, 90, and 100 percent of the ultimate load. Figures B.33 through B.48 in Appendix

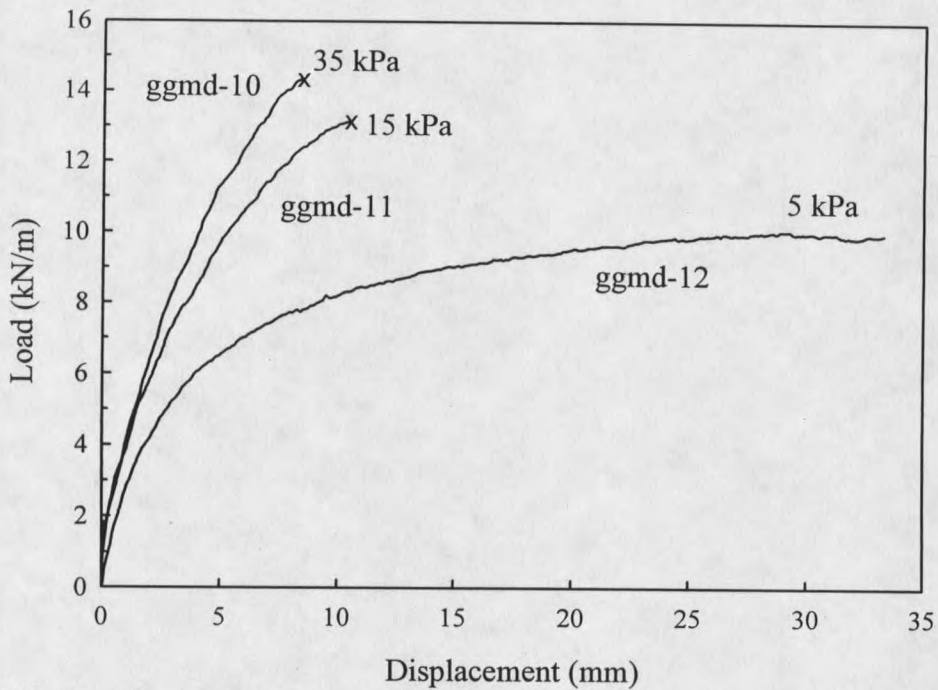


Figure 3.22: In-soil, load versus displacement curves for the geogrid pulled in the machine direction at three levels of normal confinement.

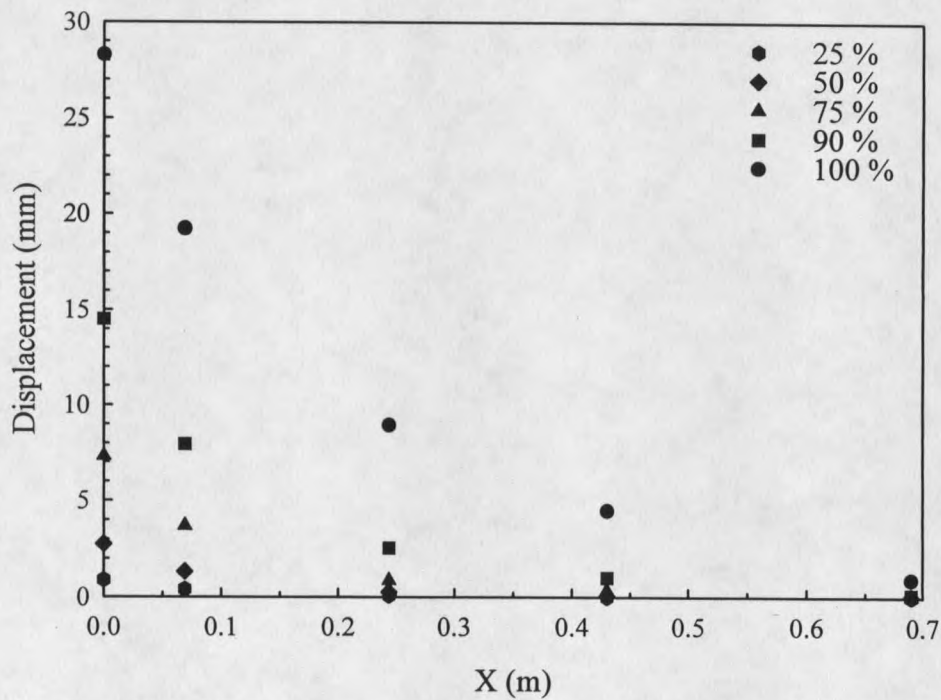


Figure 3.23: In-soil displacement versus distance along reinforcement length for geogrid pulled in the machine direction at 5 kPa normal confinement: ggmd-12.

B contain a complete set of results from the monotonic, in-soil tests.

Cyclic, In-Soil Tests

Cyclic, in-soil tests were setup and run much like the monotonic, in-soil tests with the exception of the loading condition. Cyclic, in-soil test data will be used for future analysis work using the finite element modeling. Cyclic tests on these materials were conducted at 15 kPa normal confinement. Results from these tests include maximum and minimum applied load versus cycle number and maximum and minimum displacements versus cycle number, as shown in Figures 3.24 and 3.25, respectively. The fourteen levels of loading can be clearly defined using Figure 3.24. Figure 3.25 shows the displacement of the front of the material per cycle number. The minimum displacements at the front show how ratchetting occurs in the cyclic tests. Ratchetting allows the material to move in the direction of the load without letting it return fully, as clearly seen in Figure 3.25. Figure 3.26 also illustrates ratchetting by showing the accumulated displacements for the unloading/reloading loops. A complete set of results for the cyclic, in-soil tests is given in Appendix B.

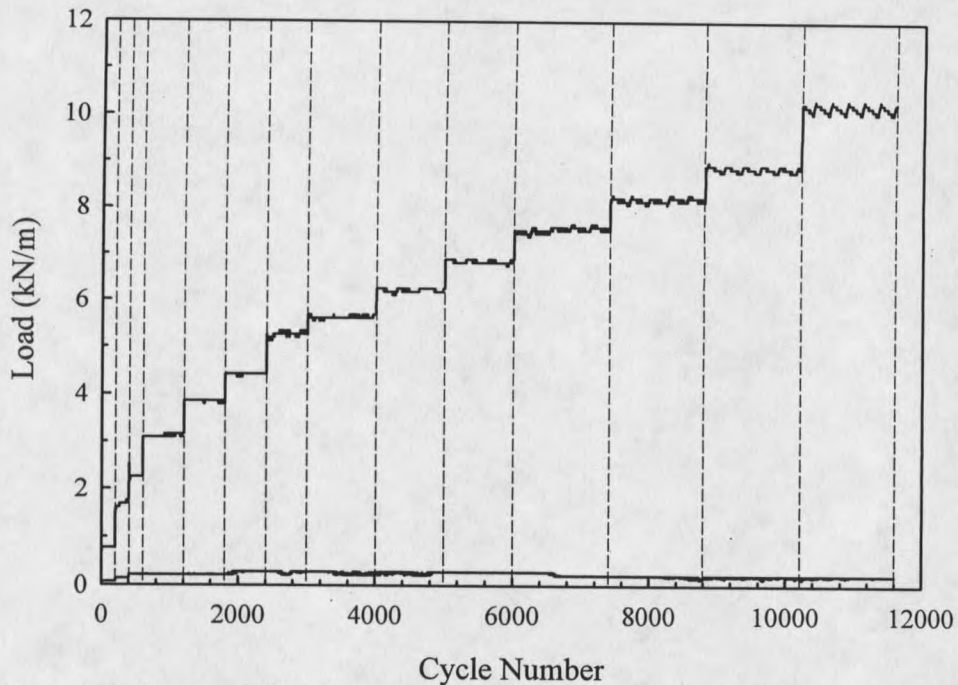


Figure 3.24: Maximum and minimum loads applied to the geogrid in the machine direction at 15 kPa normal confinement: ggmd-13.

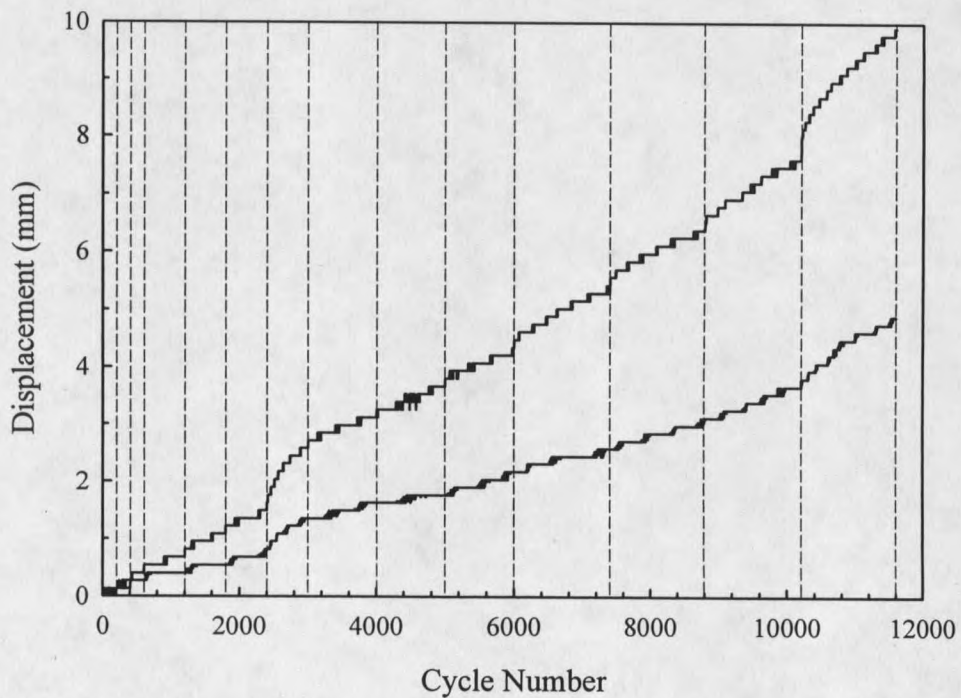


Figure 3.25: Maximum and minimum displacement at the front of a sample of geogrid pulled in the machine direction at 15 kPa normal confinement: ggmd-13.

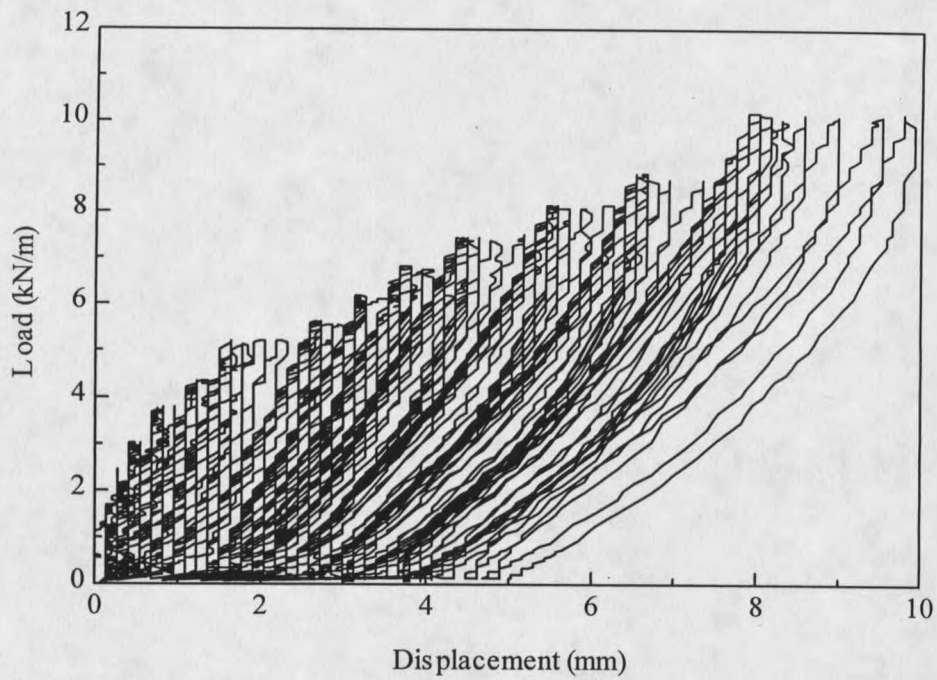


Figure 3.26: Cyclic, in-soil, unloading/reloading load/displacement curves for a sample of geogrid pulled in the machine direction at 15 kPa normal confinement: ggmd-13.

CHAPTER FOUR

DISCUSSION AND ANALYSIS OF TEST RESULTS

Introduction

Comments on data trends and brief comparisons of experimental results from in-air and in-soil tests are made in this chapter. Test data is interpreted and analyzed to determine constitutive material parameters and soil/geosynthetic interaction parameters. Inherent geosynthetic material properties were determined mostly from fast monotonic and cyclic, Series II test results. The method for interpreting in-air test data to determine its material properties is described in detail concluding with a table of appropriate material parameters.

Soil/geosynthetic interaction parameters were determined from monotonic, in-soil, pull-out tests. These interaction parameters described the relationship between shear stress and shear displacement with respect to normal stress confinement. Current methods of calculating this relationship were briefly outlined and described in Chapter 2. Based on available methods, the mobilizing process method was first used. However, problems associated with this method motivated the author to develop a numerical method based on the solution of a governing differential equation to solve the interaction problem. A description of how the mobilizing process method was used, examples of results from this analytical technique and a discussion of those results are given. Lastly, a detailed derivation

of the differential equation and its numerical solution are described. Examples of typical results from the numerical analysis are shown and the resulting interaction parameters are tabulated.

Discussion of In-Air Test Results

Results from in-air tests performed on the geosynthetic materials showed that the materials had different strengths in their principle directions. Figures 3.15 and 3.17 showed that the strength in the cross-machine direction was higher than the strength in the machine direction for both materials. Fast and slow monotonic tests showed that comparable load/strain characteristics were obtained from repeated tests.

Strain levels at failure varied between tests, though load levels at these strains were similar. This was most likely due to material nonhomogeneities since an identical testing method was used for all in-air tests.

Slow monotonic tests performed on the geotextile showed that the slope of the load/strain curve in the machine direction changed drastically at approximately 3 % axial strain. In the cross-machine direction, this slope change occurred at approximately 1 % axial strain. This phenomenon may be explained by the fact that the geotextile was a woven material. Reorientation of the woven fibers may have caused the initial portion of the load/strain curve to exhibit these slope differences. Occurrence of this slope change is still noticed in the fast monotonic tests but not as distinctly. Duplicate tests on the geotextile indicated that this phenomenon was repeatable.

Measured geosynthetic material properties are strain-rate dependent. This can be seen in the different stress/strain characteristics obtained from fast and slow monotonic tests. Standard creep tests were not performed in this study, however, strain differences between fast and slow monotonic tests indicated the tendency of the materials to creep when loaded slowly. Figures 4.1 and 4.2 show this for the geogrid and the geotextile, respectively. It was also noticed that ultimate strains for the fast monotonic tests were nearly half the ultimate strains for the slow monotonic loading. This was caused by viscous behavior of the material. When loaded rapidly, time dependent strains were unable to develop which essentially stiffened the material and caused failure to occur at lower values of strain. Differences in lateral strains between test types can be explained using similar reasoning.

Evidence of viscous and creep dependent behavior can be shown using comparisons between monotonic and cyclic tests. Individual cycles from the cyclic, Series I tests were applied to the sample using the same strain rate as the fast monotonic tests, therefore, creep strains were negligible. Figure 4.3 shows a plot of a fast monotonic test and a cyclic, Series I test performed on the geogrid in the machine direction. Peak loads for the cyclic test correspond well with the load/strain curve from the fast monotonic test. In the same way the slow monotonic tests correlated with results from the cyclic, Series II tests. Repeated cycles in these tests allowed creep strains to develop over time. Figure 4.4 shows a plot of a slow monotonic test and a cyclic, Series II test performed on the geogrid in the machine direction. Again, peak loads for the cyclic test match the load/strain curve from the slow monotonic test.

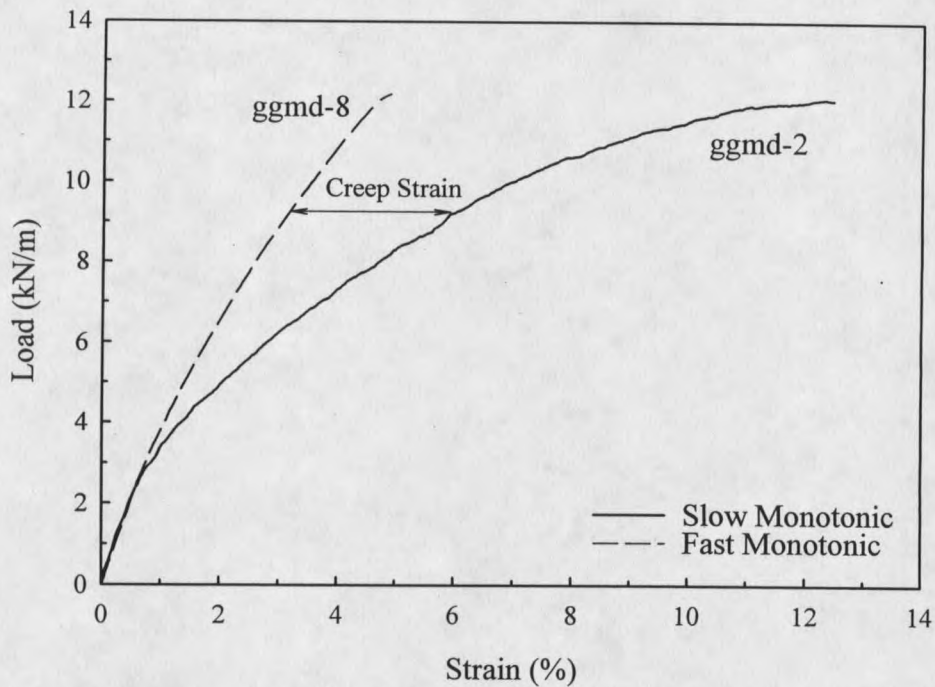


Figure 4.1: Comparison between fast and slow monotonic tests performed on the geogrid in the machine direction.

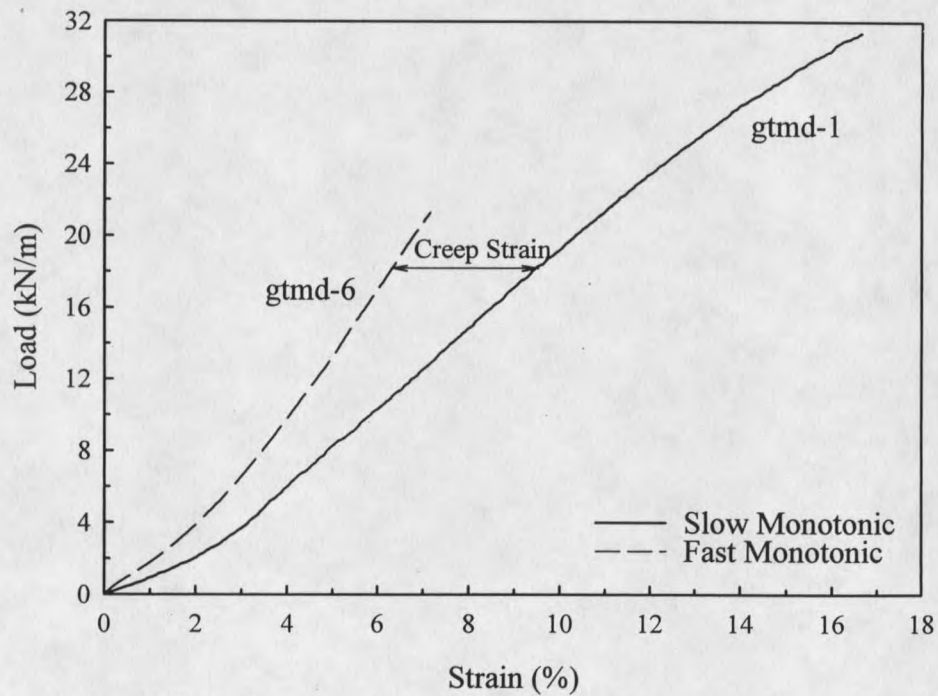


Figure 4.2: Comparison between fast and slow monotonic tests performed on the geotextile in the machine direction.

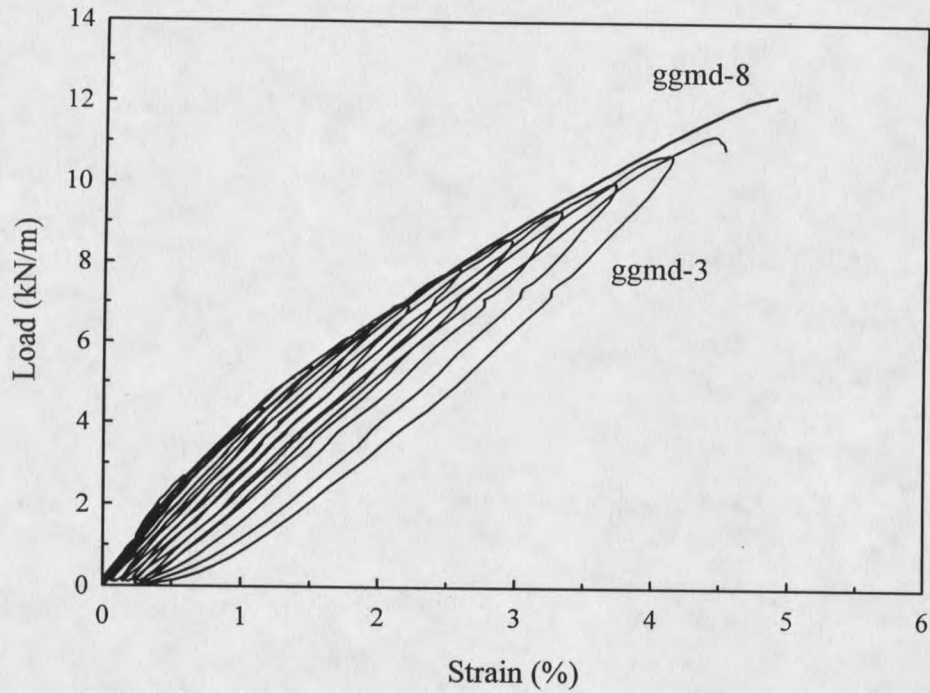


Figure 4.3: Comparison between fast monotonic and cyclic, Series I tests performed on the geogrid in the machine direction.

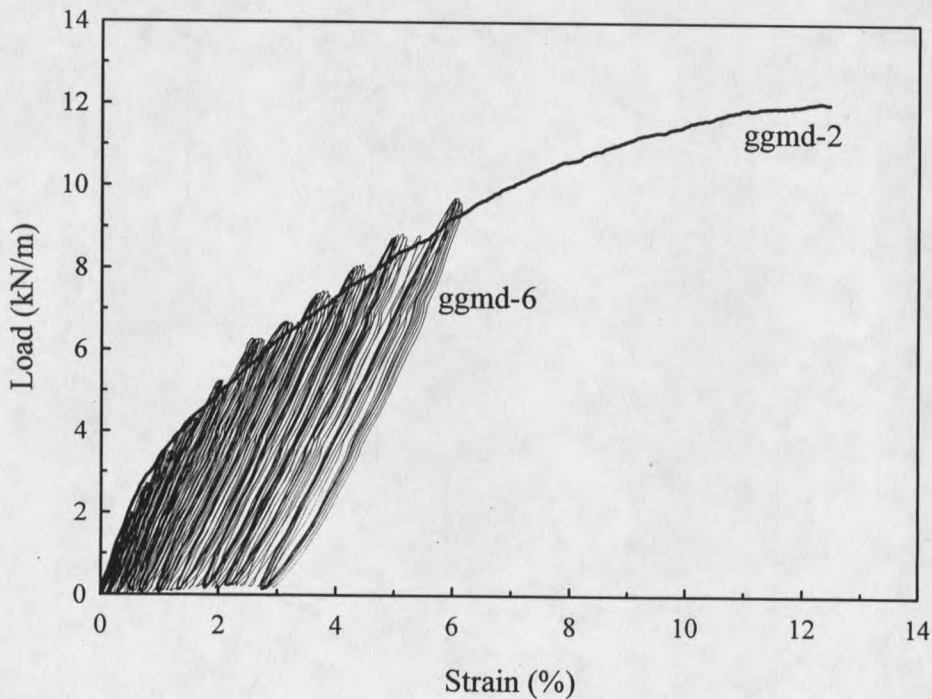


Figure 4.4: Comparison between slow monotonic and cyclic, Series II tests performed on the geogrid in the machine direction.

Analysis of In-Air Test Results

The geosynthetic materials in this study exhibited orthotropic, elastic/plastic material properties. To describe these material parameters within the context of a finite element model properly, a constitutive model must first be formed. Calibrating the constitutive parameters can be done using appropriate tests conducted on the materials.

A pressure independent, anisotropic, yield plasticity constitutive model (Hill mechanism) along with a 3-D, orthotropic elastic response was used to characterize geosynthetic material behavior. The 3-D, elastic, orthotropic constitutive matrix in Equation 4.1 shows the "engineering constants" that defined the material's orthotropic elastic properties.

$$\begin{Bmatrix} \epsilon_x \\ \epsilon_m \\ \epsilon_z \\ \gamma_{mx} \\ \gamma_{mz} \\ \gamma_{xz} \end{Bmatrix} = \begin{bmatrix} 1/E_x & -\nu_{mx}/E_m & -\nu_{zx}/E_z & 0 & 0 & 0 \\ -\nu_{xm}/E_x & 1/E_m & -\nu_{zm}/E_z & 0 & 0 & 0 \\ -\nu_{mz}/E_m & -\nu_{xz}/E_x & 1/E_z & 0 & 0 & 0 \\ 0 & 0 & 0 & 1/G_{mx} & 0 & 0 \\ 0 & 0 & 0 & 0 & 1/G_{mz} & 0 \\ 0 & 0 & 0 & 0 & 0 & 1/G_{xz} \end{bmatrix} \begin{Bmatrix} \sigma_x \\ \sigma_m \\ \sigma_z \\ \sigma_{xm} \\ \sigma_{mz} \\ \sigma_{xz} \end{Bmatrix} \quad (4.1)$$

In this matrix the subscript m denotes the machine direction, x , the cross-machine direction and z , the out-of-plane direction. The terms E_x , E_m and E_z are elastic moduli for the cross-machine, machine and the out-of-plane direction, respectively. Parameters G_{mx} , G_{mz} and G_{xz} are shear moduli pertaining to the machine/cross-machine, machine/out-of-plane and the cross-machine/out-of-plane planes, respectively. The terms ν_{xm} and ν_{mx} are Poisson's ratios pertaining to the machine/cross-machine plane. More specifically, ν_{xm} is the Poisson's

ratio that characterizes the lateral strain in the machine direction when uniaxial load is applied in the cross-machine direction and ν_{mx} is the Poisson's ratio that characterizes the lateral strain in the cross-machine direction when the material is loaded in the machine direction. Since lateral strains were not measured in the z -direction nor did the material experience appreciable strain in the z -direction, ν_{mz} , ν_{zm} , ν_{xz} and ν_{zx} were arbitrarily set to zero. The terms σ_x , σ_m , and σ_z are the applied stresses pertaining to the cross-machine, machine and out-of-plane directions, respectively, and σ_{xm} , σ_{mz} and σ_{xz} are the applied shear stresses in the cross-machine/machine, machine/out-of-plane and cross-machine/out-of-plane directions, respectively.

To define these elastic/plastic orthotropic material parameters, data obtained from fast monotonic tests was used. Results from these tests did not include creep since the samples were tested under a high rate of deformation. Tests performed on the geosynthetic samples oriented in both of their principle axes allowed load/strain relationships to be determined. Using the slope of the initial portion of the load versus axial strain curves and the axial strain versus lateral strain curves, elastic material properties, like elastic moduli and Poisson's ratios, were calculated. Tests performed on geosynthetic samples oriented 45 degrees from each of their principle strength directions allowed the shear modulus to be obtained using the following equation:

$$G_{mx} = \frac{\bar{\sigma}}{2(\bar{\epsilon} - \underline{\epsilon})} \quad (4.2)$$

where $\bar{\sigma}$ is the axial load carried in the geosynthetic, and $\bar{\epsilon}$ and $\underline{\epsilon}$ are the strains measured in the axial and lateral directions, respectively, for this test. Elastic material properties from

tests oriented in the 45 degree direction also allowed G_{mx} to be calibrated.

The geosynthetic materials also exhibited anisotropic yield characteristics. The pressure independent, anisotropic, plasticity model used a potential function which required that six yield ratios be defined. These ratios were defined as the ultimate yield strength, expressed as a normal or shear stress in each of the three material directions, divided by an ultimate reference normal stress. The ultimate reference normal stress, σ_0 , was defined as the yield strength in one of the principle strength directions. In this case, the ultimate reference normal stress was defined as $\bar{\sigma}_x$, the ultimate value of axial stress measured in the cross-machine direction. These yield stress ratios were denoted as R_x , R_m , R_z , R_{mx} , R_{mz} and R_{xz} . The yield stress ratios were evaluated from the load versus axial strain results in the machine and cross-machine directions presented in Chapter 3. The following equations define these stress ratios as:

$$R_x = \frac{\bar{\sigma}_x}{\sigma_0}, \quad R_m = \frac{\bar{\sigma}_m}{\sigma_0}, \quad R_z = \frac{\bar{\sigma}_z}{\sigma_0}, \quad R_{xm} = \frac{\bar{\sigma}_{xm}}{\tau_0}, \quad R_{mz} = \frac{\bar{\sigma}_{mz}}{\tau_0}, \quad R_{xz} = \frac{\bar{\sigma}_{xz}}{\tau_0} \quad (4.3)$$

where, $\bar{\sigma}_x$, $\bar{\sigma}_m$ and $\bar{\sigma}_z$ are the ultimate yield strength measured in uniaxial tension in the cross-machine, machine, and out-of-plane directions, respectively. The terms $\bar{\sigma}_{xm}$, $\bar{\sigma}_{mz}$, $\bar{\sigma}_{xz}$ are ultimate values of shear stress measured in the cross-machine/machine, machine/out-of-plane and cross-machine/out-of-plane planes, respectively. The ultimate reference shear stress, τ_0 , was defined as:

$$\tau_0 = \frac{\sigma_0}{\sqrt{3}} \quad (4.4)$$

Values for each of these parameters were determined using the data from the fast monotonic in-air tests on the geogrid and geotextile. Tables 4.1 and 4.2 show the tabulated values for each of these parameters.

Moduli (kPa)		Poisson's Ratios		Yield Stress Ratios	
E_x	645,000	ν_{mx}	0.03	R_x	1
E_m	600,000	ν_{xm}	0.032	R_m	0.584
E_z	1,000,000	ν_{mz}	0	R_z	0.8
G_{mx}	30,000	ν_{zm}	0	R_{xm}	0.091
G_{mz}	30,000	ν_{xz}	0	R_{mz}	0.091
G_{xz}	30,000	ν_{zx}	0	R_{xz}	0.091

Table 4.1: In-air constitutive material parameters for the geogrid.

Moduli (kPa)		Poisson's Ratios		Yield Stress Ratios	
E_x	960,000	ν_{xm}	2	R_x	1
E_m	239,000	ν_{mx}	0.498	R_m	0.74
E_z	1,000,000	ν_{mz}	0	R_z	1
G_{mx}	≈ 0	ν_{zm}	0	R_{xm}	≈ 0
G_{mz}	≈ 0	ν_{xz}	0	R_{mz}	≈ 0
G_{xz}	≈ 0	ν_{zx}	0	R_{xz}	≈ 0

Table 4.2: In-air constitutive material parameters for the geotextile.

Discussion of In-Soil (Pull-Out) Test Results

Results from in-soil consisted of load/displacement curves and displacement versus distance along the embedded length of the geosynthetic. It was desired that the entire length of the geosynthetic sample be mobilized in pull-out at the peak strength of the material. This would only happen at a particular sample length. To estimate the proper length of the sample, rough estimations of shear stress along the geosynthetic surface were made and compared to the ultimate capacity of the material. Results from these approximations were poor and many times resulted in failure of the sample before the sample was completely mobilized, as seen in Figure 3.22. This is important to note since values of load in the material are dependent upon the embedded length of the sample. If a short and long sample of geosynthetic are pulled from soil at the same level of confinement, the shorter one will obviously require less load. Based on this reasoning, load/displacement curves cannot be directly compared to one another since samples would have to be fully mobilized in pull-out at their ultimate strength.

During pull-out, mobilization of the embedded length of the sample was progressive making displacements along the embedded length also progressive. Figure 3.23 shows how displacements get progressively larger for higher levels of load. At 100 % of the ultimate load, the free end of the geosynthetic just begins to move. Displacement along the geosynthetic is nonlinear making strains in the material also nonlinear. This nonlinearity becomes more pronounced at higher confinement levels since shear stresses along the surface of the geosynthetic increases with confinement.

Comparisons between monotonic and cyclic, in-soil tests show that pull-out mobilization is effected by rate and method of applying the pull-out load. Figure 4.5 shows load/displacement results from a monotonic and a cyclic tests performed on a sample of geogrid at 15 kPa normal confinement. For the cyclic test results shown, the displacement is measured at the front of the sample. The free end of the sample in the monotonic case began to move when the front of the sample had displaced approximately 28 millimeters. For the cyclic case, the free end of the sample began to move when the front had moved approximately five millimeters. Not surprisingly, this seems to indicate that applying cyclic loads to the material lowers the load at which complete mobilization of the sample occurs. This is of particular concern to those interested in designing structures reinforced with geosynthetic within seismically active areas.

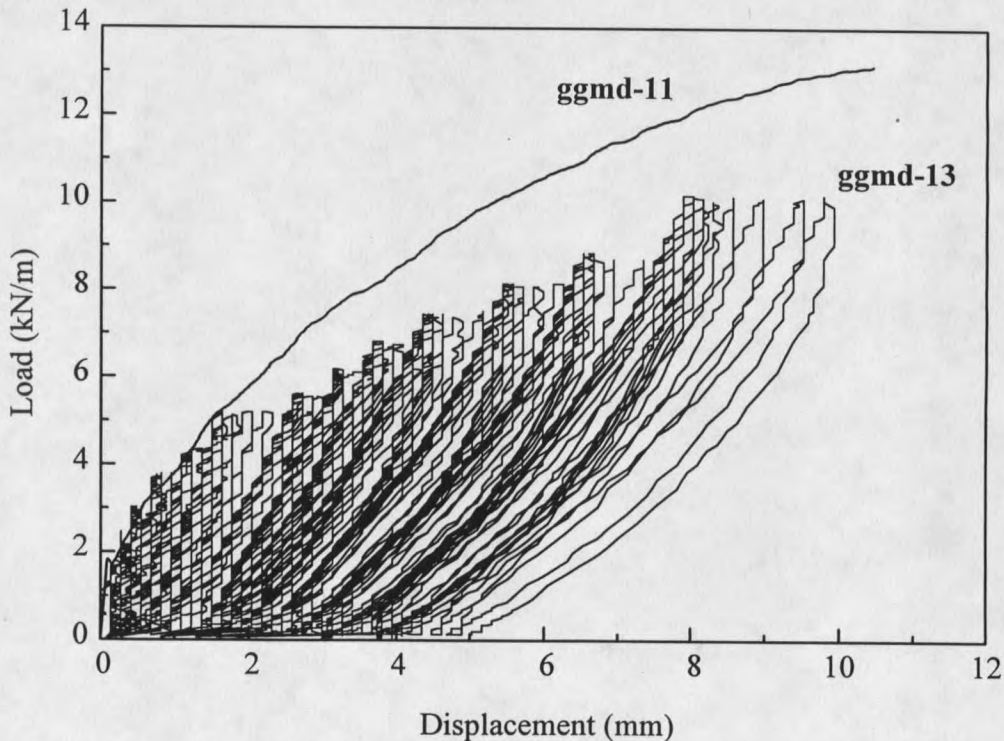


Figure 4.5: Comparison of monotonic and cyclic pull-out tests performed on the geogrid in the machine direction at 15 kPa normal confinement.

Analysis of In-Soil (Pull-Out) Test Results

Pulling geosynthetic materials from confining soil creates a non-uniform distribution of displacement along the material's length. Consequently, the development of shear stress along the length is also non-uniform. Developing a relationship that describes the relationship between the displacement of the material and the resulting shear stress, requires that certain parameters be defined. These interaction parameters will be used within the finite element model to describe shear stresses at the soil/geosynthetic interface as a function of the displacement of the geosynthetic reinforcement. Estimating parameters necessary to describe the relationship between shear stress and shear displacement can be accomplished using the methods outlined in Chapter 2.

Of the four methods currently used to determine the relationship between shear stress and displacement along the embedded reinforcement outlined in Chapter 2 (the Mobilizing Process Method, Average Resistance Method, Numerical Techniques and the Finite Element Method), the mobilizing process method was initially used. When it was determined that this method did not produce reliable results, a numerical approach was taken to solve the problem. A discussion outlining the mobilizing process method and the numerical method is given in the following sections.

Mobilizing Process Method

The mobilizing process method calculated the shear stress along the length of the embedded geosynthetic based on strains within the material. To accomplish this, the mobilizing process method required that the load applied to the front of the sample and various displacements along the embedded length be measured. Pull-out experiments conducted in this study measured the load applied to the front and displacement at five locations along the embedded length of the geosynthetic. Strain in the geosynthetic was determined from the slope of an assumed displacement distribution. The strain was then used to calculate the load in the material. From the load, shear was calculated and plotted with respect to the displacement. Lastly, the shear stress versus shear displacement distribution was formed, allowing the interaction parameters describing its shape to be obtained for use in the finite element model.

The four steps in the mobilizing process method are: 1) to discretize the displacement distribution along the length of the embedded material, 2) determine strain from the discretized elements, 3) determine load in the material from the strain and 4) calculate the shear stress using load differencing. These four steps were previously illustrated in Chapter 2 using Figure 2.17.

Approximating the Displacement Distribution

Measurements of displacement during pull-out testing allowed a distribution of displacement along the reinforcement length to be made. Since only a limited number of measurement points were made, displacement was known only at discrete points. To

describe the displacement between these points, certain assumptions had to be made. Previous researchers assumed a linear distribution between measurement points, as discussed in Chapter 2. Using a linear distribution between measurement points results in a conservative estimate of strain and only allows discretization of the distribution into j elements, where j is one less than the number of displacement measurements. Another alternative was to use a non-linear equation through the measurement points. This would allow displacement to be calculated at all points along the length.

In this study, it was desired to use an equation to approximate the displacement distribution for two reasons: 1) having an equation would allow the displacement distribution to be discretized into an infinite number of elements, and 2) strains could be calculated by taking the derivative of the equation. However, it was found that determining a suitable equation that accurately represented the displacement distribution was difficult.

Ideally, a unique equation form would be able to accurately represent the displacement distribution at all load levels. This was difficult since strains at the front of the sample increased at greater rates than strains along the embedded length. The reason for having the same equation for all curves was to ensure that strain distributions also had similar shapes. Figure 4.6 shows an example of a linear, a cubic spline and a sum-of-exponentials approximation used to define displacement between the experimental data points. There is a noticeable difference between them, yet, all can be considered reasonable approximations. Figure 4.7 shows the difference in the calculated strain from each of these approximations. These plots show that strain calculations are highly dependent on the method of approximating the displacement distribution. More specifically, strains at the

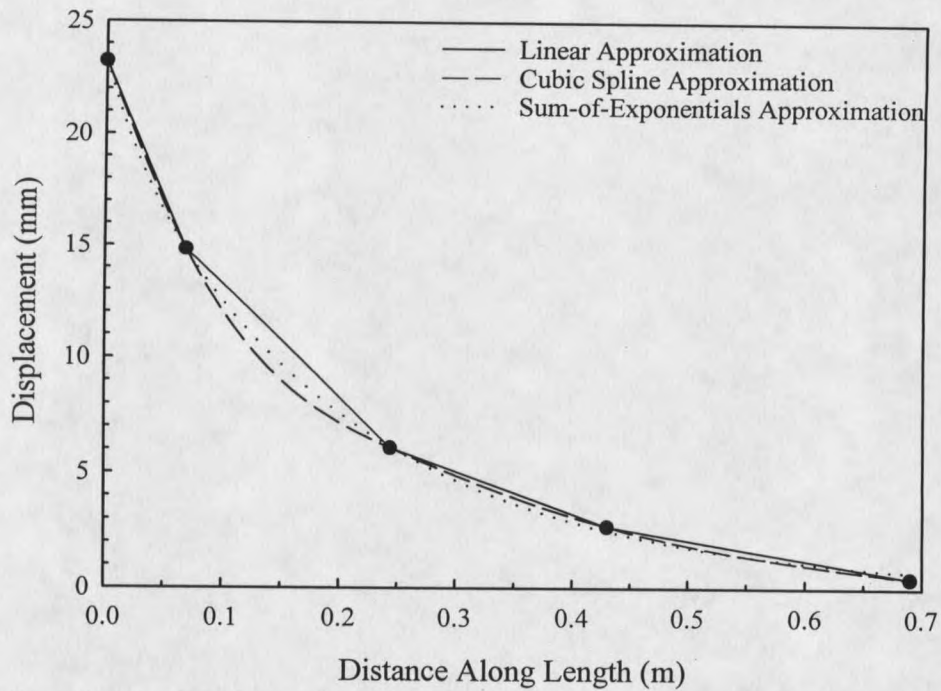


Figure 4.6: Approximations of a displacement distribution using linear, cubic spline and sum-of-exponentials approaches.

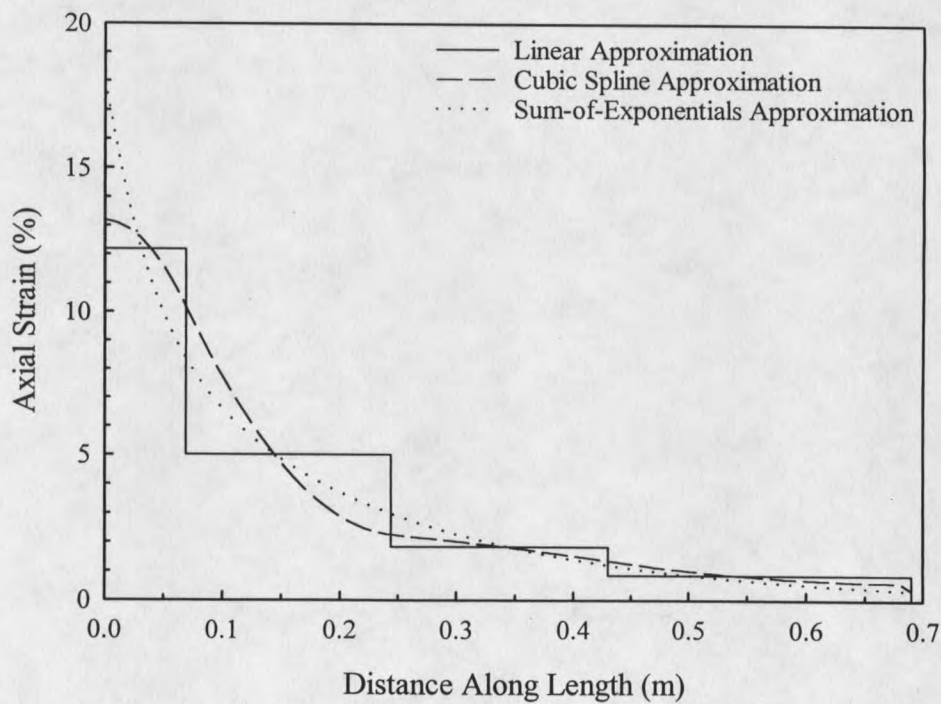


Figure 4.7: Strain distributions calculated using a linear, cubic spline and sum-of-exponentials approximation of a displacement distribution.

front were very sensitive to small changes in the curve fit equation coefficients. This was undesirable since strains at the front of the sample were used to define the load/strain properties of the material.

Determining Strains in the Material

Strains at the front of the sample were calculated using the method described in Chapter 2. Briefly, this method calculated strains based on displacement differences in the material using:

$$\epsilon_{i,i+1} = \frac{u_i - u_{i+1}}{x_i - x_{i+1}} = \frac{d}{dx} u(x) \quad (4.5)$$

where $\epsilon_{i,i+1}$ is the strain calculated between nodes i and $i+1$, u is the displacement of nodes i and $i+1$, as a function of x , and x is the original position of nodes i and $i+1$. Having an equation that describes the displacement would allow its derivative to be used to calculate the strain.

Determining Load in the Material

Strains at the front of the sample were plotted against the measured pull-out load to define an "in-soil" load/strain curve. Even though this load/strain curve was determined from the strains at the front of the sample, it was used as the intrinsic curve for the entire length of the material. Alternatively, the in-air load/strain properties could also be used to define the intrinsic load/strain curve. Having the load/strain information was necessary since shear along the material was determined from the load carried in the material. By using the

strain distribution and the load/strain curve, load can be determined for any point along the embedded geosynthetic. Differences between the in-air and in-soil load/strain curves was dependent upon the method of determining strain at the front of the sample for the in-soil case. Figure 4.8 shows an example of the difference between an in-air and an in-soil load/strain curves for the geogrid oriented in the machine direction. For the presented case, the displacement distribution was approximated using a sum-of-exponentials equation. The majority of the differences between these curves was attributed to the method of calculating the strains, with the remainder due to effects of confinement. Quantifying the error between these two curves was difficult since direct strain measurements were not made, making an assessment of the accuracy of the strain calculations impossible.

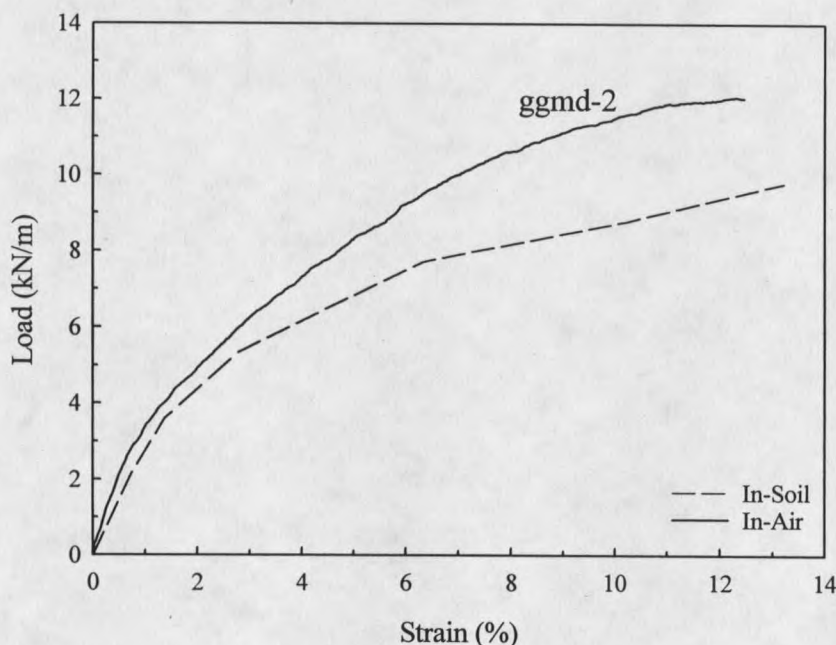


Figure 4.8: Comparison of in-air and in-soil load/strain curves for the geogrid pulled in the machine direction.

Alternatively, load/strain behavior can be predicted using in-air test results. Geosynthetic material properties were well defined by in-air tests and exhibit the materials inherent behavior. In-air tests cannot reflect changes due to confinement, but it can be argued that the difference between each of these load/strain curves was mainly due to soil/geosynthetic interaction, assuming the strain has been calculated properly. In this study, the load was calculated using results from in-air tests performed on the geosynthetic.

Knowing the strain distribution along the embedded length allowed the distribution of the load in the material to be calculated. From the load distribution, the shear stress could be calculated using the following equation:

$$\tau_{i-1,i} = \frac{F_{i-1} - F_i}{2 \cdot b \cdot \Delta x} \quad (4.6)$$

where $\tau_{i-1,i}$ is the shear stress between nodes $i-1$ and i , F_{i-1} and F_i are the loads at nodes $i-1$ and i , respectively, b is the width of the material and Δx is the length of the element between nodes $i-1$ and i . Equation 4.6 is based on the free-body diagram shown in Figure 4.9 which states that the force in a particular element is transferred into shear along its surface. A multiplier of two in the denominator proportions the shear over both sides of the material. To create a shear stress versus shear displacement diagram, values of shear displacement corresponding to these shear stresses were needed.

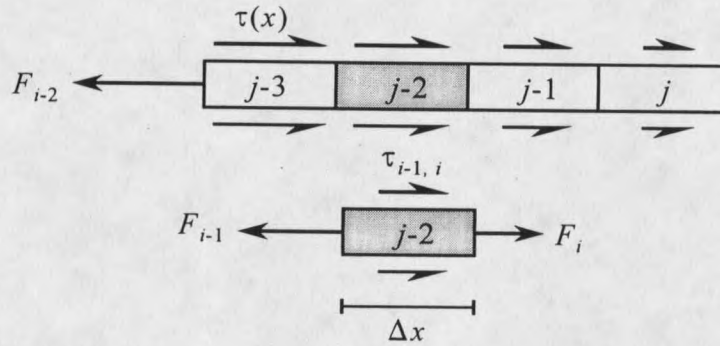


Figure 4.9: Free-body diagram of shear interaction.

There are two methods of calculating the shear displacement of the material. The method proposed by Farrag (1991) and Ochiai (1996) discussed in Chapter 2, defined the shear displacement as the displacement of the rear node of an element of material. Shear displacements in this study were defined as the average displacement of an element. Using an average displacement accounted for displacement due to differential straining of the element. Straining of the element also created shear stress which needed to be accounted for in the analysis. As the elements became smaller, the values of the shear displacement from each of these two methods approached one another.

Results

Results from the mobilizing process method were inconsistent due to difficulties regarding the approximation of the displacement distribution. In several cases the strains calculated from the best fit equations were greater than ultimate strains measured in the material during the in-air tests. This clearly indicated that strain calculations were incorrect. A shear stress versus shear displacement diagram for the geogrid pulled in the machine

direction at 5 kPa confinement, that used a linear approximation of the displacement between the measurement points, is given in Figure 4.10. For the curve fit approximation of the displacement, results were most consistent when a sum-of-exponentials equation was used. Using a sum-of-exponentials curve fit equation, a plot of shear stress versus shear displacement for the same material and conditions was determined, as shown in Figure 4.11. The displacement distribution was discretized into elements one centimeter in length. Each data point on the shear stress versus shear displacement curve corresponded to a particular level of applied load. Creating more points along the shear stress versus shear displacement distribution was very laborious since equations describing the displacement distribution had to be calibrated for each new load level.

In the linear analysis only three elements could be used for the analysis since there were only five measurement points of displacement. The sum-of-exponentials analysis used an equation to fit the displacement data and therefore could be discretized into an infinite number of elements. For the sake of comparison, only three elements were plotted from the sum-of-exponentials analysis. The analysis actually contained 69, one centimeter elements. There is a significant difference between the linear approximation results and the sum-of-exponentials results. The ultimate value of shear stress for the front element using the linear approximation was approximately 18 kPa. For the sum-of-exponentials approximation the ultimate value of shear stress for the front element was approximately 30 kPa. Shear stress versus shear displacement curves from each element should be the same. The values for the initial slopes for the linear approximation curves are 25,900 kPa, 8200 kPa and 23,800 kPa, respectively. For the sum-of-exponentials approximation these values were 35,600 kPa,

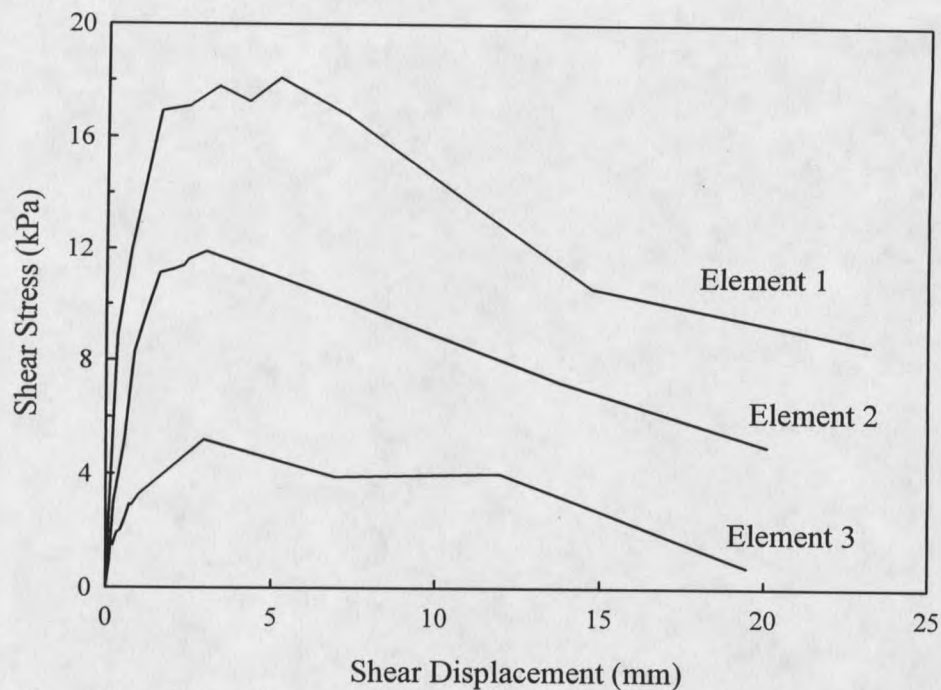


Figure 4.10: Shear stress versus shear displacement for the geogrid pulled in the machine direction at a normal confinement of 5 kPa; computed using a linear approximation of the displacement distribution.

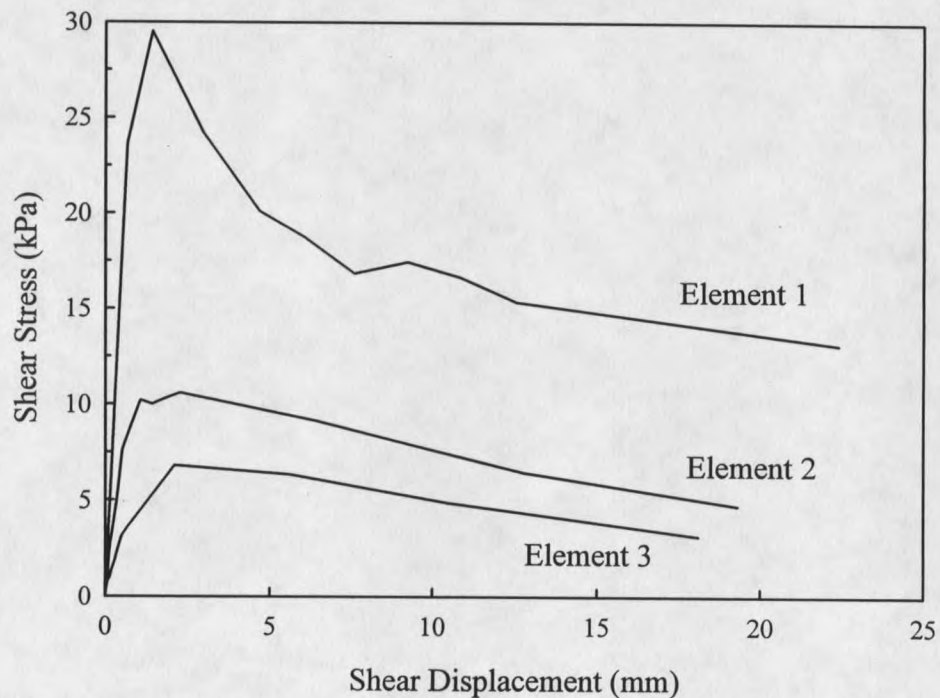


Figure 4.11: Shear stress versus shear displacement for the geogrid pulled in the machine direction at a normal confinement of 5 kPa; computed using a sum-of-exponentials approximation of the displacement distribution.

15,000 kPa and 10,400 kPa, respectively. From these values it is obvious that the two methods of approximating the displacement distribution yielded shear/displacement curves that were not consistent with one another.

Another major flaw with the mobilizing process method was that shear stress decreases excessively at higher levels of shear displacement. It is possible for the shear stress to soften, but not this drastically. This figure indicated that at high strains, caused by large displacements, very little load is required to pull the geosynthetic from the soil, which is clearly not the case as indicated in the load/displacement diagram. This phenomenon can be explained by the fact that the mobilizing process method is based on the strain differences carried across an element of material. As strain levels increase, the load/strain curve, used to calculate loads from strains in the geogrid, flattened. Therefore, at higher levels of strain, load differences decreased, which made the calculated shear stresses decrease.

Numerical Method

Difficulties associated with determining accurate or consistent parameters describing the soil/geosynthetic interaction using the mobilizing process method motivated the author to develop a numerical method of determining the distribution of shear stress versus shear displacement based on the solution of a governing differential equation. In this section the derivation of the differential equation, definitions of variables within the equation, and the method used to find a numerical solution are discussed. The numerical solution is a function of the pull-out displacement, the load/strain characteristics of the material, the interface shear stiffness, the vertical normal confinement and the ultimate and residual interface shear strength.

Derivation of Differential Equation

As the geosynthetic reinforcement was pulled from the soil, shear stresses developed along the soil/geosynthetic interface. The interface shear stress along the length of the reinforcement can be illustrated using the free-body diagram previously shown in Figure 4.9. Using this diagram it can be shown that the interface shear stress at a point along the length of the reinforcement is related to the load at that point by the differential equation, shown in general form, as:

$$\tau(x) = \frac{1}{2} \left[\frac{dF(x)}{dx} \right] \quad (4.7)$$

where x is a distance along the length of the reinforcement, $\tau(x)$ is the interface shear stress at a point x and $F(x)$ represents the load in the material at x which is, in turn, a function of the strain at point x . Strains in this equation were based on the derivative of the distribution of displacement along the embedded length of the geosynthetic, as detailed earlier in this chapter and in Chapter 2. The solution to Equation 4.7 required that the functions be in the form of displacements, which would allow a back-calculation of the displacement distribution along the length.

Definition of Functions and Variables in Differential Equation

To back-calculate the distribution of displacement along the length of the reinforcement, the shear stress and the load within the material needed to be expressed in terms of displacement. The development of shear stress along the area of the geosynthetic

was a function of the amount of displacement at that point as previously noted in the mobilizing process method. The relationship used to describe the shear stress as a function of displacement consisted of the modified hyperbolic equation developed by Juran and Chen (1988) introduced in Chapter 2 as:

$$\tau(u(x)) = \sigma_y C_3 u(x) \frac{[u(x) - C_1]}{[u(x) + C_2]^2} \quad (4.8)$$

where $\tau(u(x))$ is the shear stress, $u(x)$ is the interface shear displacement as a function of x , σ_y is the vertical normal confinement and C_1 , C_2 and C_3 are constants which can be determined using equations 4.9, 4.10 and 4.11.

$$C_1 = -4 \frac{\sigma_y d}{G_i} \frac{[\tan^2 \psi_p \cdot J^2]}{\tan \psi_r} \quad (4.9)$$

$$C_2 = 2 \frac{\sigma_y d}{G_i} [\tan \psi_p \cdot J] \quad (4.10)$$

$$C_3 = \tan \psi_r \quad (4.11)$$

where σ_y is the vertical normal confinement, d is the thickness of the shear interface layer, G_i is the initial tangent shear modulus at the soil/geosynthetic interface, ψ_p is the peak friction angle at the soil/geosynthetic interface, ψ_r is the residual soil/geosynthetic friction angle and J is defined using the following equation:

$$J = 1 + \left[1 - \frac{\tan \psi_r}{\tan \psi_p} \right]^2 \quad (4.12)$$

The load in the material was based on the strain. Strains were calculated in terms of displacement as shown in Equation 4.5 presented earlier. Substituting Equations 4.5 and 4.8 into 4.7 results in the following equation:

$$\sigma_y C_3 u(x) \frac{[u(x) - C_1]}{[u(x) + C_2]^2} = \frac{1}{2} \left[\frac{dF}{dx} \left(\frac{d}{dx} u(x) \right) \right] \quad (4.13)$$

Equation 4.13 is the generic form of the governing differential equation in terms of displacement, where displacement is a function of x .

Load in the material was determined using data from in-air tests. It was necessary to define equations that describe the load/strain relationship in each of the material's orthogonal directions using best fit approximations. Figures 4.12 and 4.13 show the load-strain curves for the material's orthogonal directions and their respective curve fit approximations. The curve fit equations were not necessarily the most accurate approximations of the data but were chosen to limit the governing differential equation to a second order equation. Approximations of the load/strain curves for both directions of the geogrid and the cross-machine direction of the geotextile were made using a hyperbolic equation of the form:

$$F(u(x)) = \frac{\left(\frac{d}{dx} u(x) \right)}{a + b \left(\frac{d}{dx} u(x) \right)} \quad (4.14)$$

where a and b are constants. The approximation of the geotextile load/strain curve in the

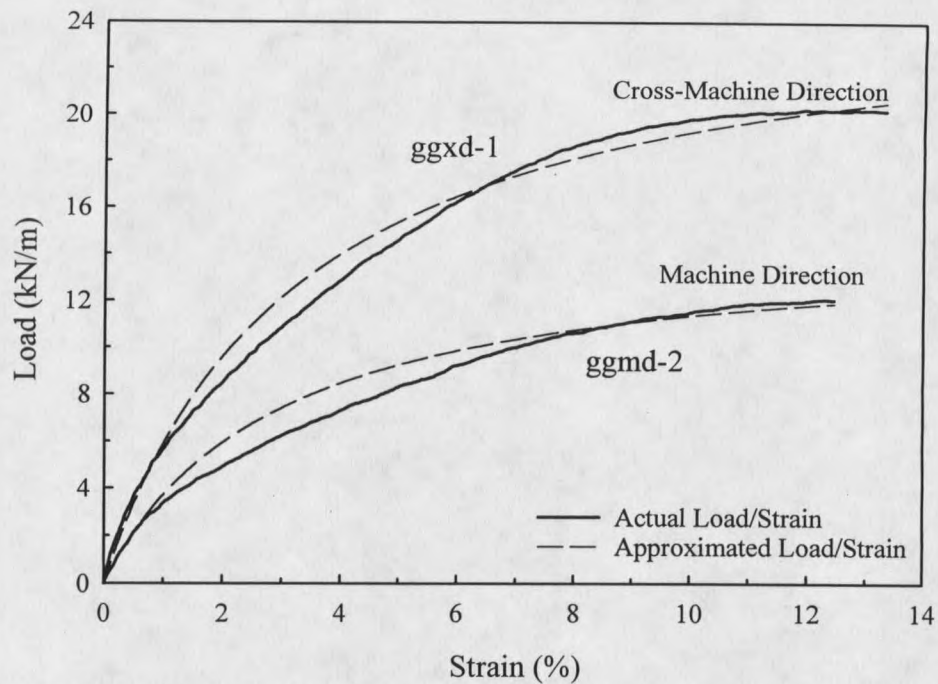


Figure 4.12: Curve fit approximations of the in-air, load/strain curves for the geogrid.

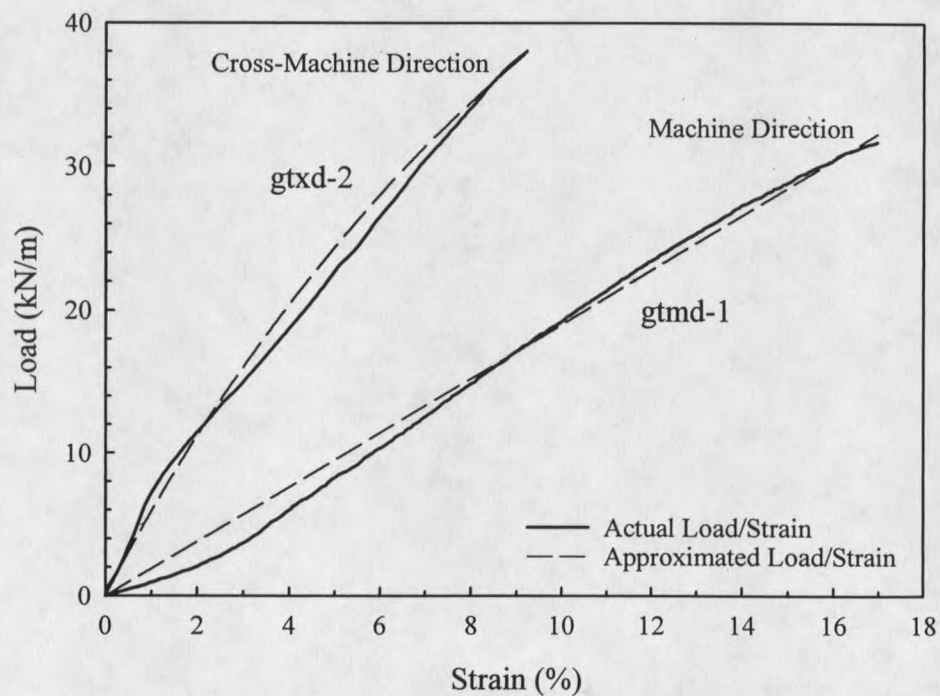


Figure 4.13: Curve fit approximations of the in-air, load/strain curves for the geotextile.

machine direction was made using a linear equation of the form:

$$F(u(x)) = a \left(\frac{d}{dx} u(x) \right) \quad (4.15)$$

For illustration purposes, Equation 4.14 will be used in the following derivation. To substitute the force equation (Equation 4.14) into Equation 4.13, its derivative must first be taken. The derivative of Equation 4.14 with respect to x is:

$$\frac{dF(u(x))}{dx} = \frac{a \left(\frac{d^2}{dx^2} u(x) \right)}{\left(a + b \left(\frac{d}{dx} u(x) \right) \right)^2} \quad (4.16)$$

Substituting Equation 4.16 into 4.13 results in Equation 4.17 to be solved in terms of displacement.

$$\sigma_y C_3 u(x) \frac{[u(x) - C_1]}{[u(x) + C_2]^2} = \frac{a \left(\frac{d^2}{dx^2} u(x) \right)}{2 \left(a + b \left(\frac{d}{dx} u(x) \right) \right)^2} \quad (4.17)$$

Solution of Equation

An explicit solution to the governing differential equation (Equation 4.17) in terms of displacement was not available so a numerical method was formulated. By solving the equation in terms of displacement a relationship between the shear stress and the shear

displacement can be formed based on the parameters G_i , ψ_p and ψ_r , the initial soil/geosynthetic interface shear modulus, the peak soil/geosynthetic friction angle and the residual soil/geosynthetic friction angle, respectively. Figure 4.14 illustrates the general shape of the shear stress versus shear displacement relationship, showing the pertinent details.

The first step in solving this equation was to define the pertinent boundary conditions. Next, the derivatives of displacement with respect to x needed to be approximated using difference equations. Lastly, the parameters listed above must be calibrated using interpolative techniques and comparisons between calculated and measured displacements and loads.

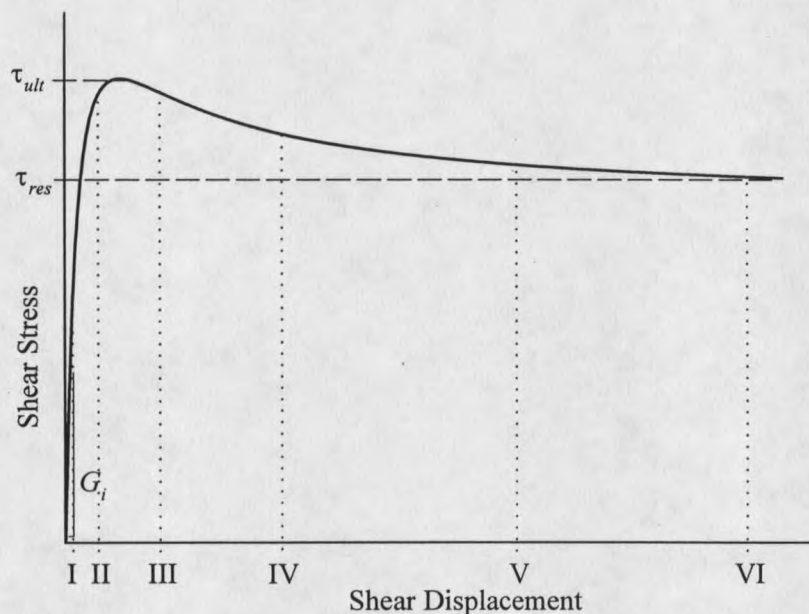


Figure 4.14: Generic shear stress versus shear displacement curve.

Boundary Conditions

The two main boundary conditions used in the analysis consisted of the displacement of the front of the sample ($x=0$) and the strain in the sample at $x=L$, where L is the length of

the embedded geosynthetic. At the front of the sample, the displacement was known since it was measured during the test. At the free end of the geosynthetic sample, $x=L$, the strain was always zero since stresses at $x=L$ were zero. Recall that the strain is calculated from the slope of the displacement versus x -distance curve. In other words, when the strain was equal to zero at $x=L$, it indicated that the slope of the displacement distribution was zero.

Numerical Analysis

To solve Equation 4.17 for displacement, approximations of the derivatives of $u(x)$ were needed. Values for the first and second derivatives of $u(x)$ with respect to x were estimated using center differencing approximations. The center differencing equation used to estimate the first derivative was:

$$\frac{d(u(x_i))}{dx} = \frac{u(x_{i+1}) - u(x_{i-1}))}{2 \cdot \Delta x} \quad (4.18)$$

where Δx is the length of a discretized element of geosynthetic. The equation used to determine the second derivative was:

$$\frac{d^2(u(x_i))}{dx^2} = \frac{u(x_{i+1}) - 2u(x_i) + u(x_{i-1}))}{\Delta x^2} \quad (4.19)$$

The three points, designated $u(x_{i-1})$, $u(x_i)$ and $u(x_{i+1})$, were defined as the displacement of nodes $i-1$, i and $i+1$, respectively. By substituting Equations 4.18 and 4.19 into Equation 4.17 and simplifying, the equation became:

$$\sigma_y C_3 u(x_i) \frac{[u(x_i) - C_1]}{[u(x_i) + C_2]^2} = \frac{1}{2} \frac{a [u(x_{i+1}) - 2u(x_i) + u(x_{i-1}))]}{\Delta x^2 \left(a + \frac{1}{2} \frac{b [u(x_{i+1}) - u(x_{i-1}))]}{\Delta x} \right)^2} \quad (4.20)$$

General Solution

The general solution of Equation 4.20 began by defining $u(x_{i+1})$ in terms of the remaining variables. The applied normal stress, σ_y , the in-air load/strain approximation equation constants, a and b , the length of the discretized elements, Δx and the displacement at the front of the sample, $u(x_{i-1})$, were known. This left the analytical constants C_1 , C_2 , C_3 and the displacement at node two, $u(x_i)$, to be determined. The constants C_1 , C_2 and C_3 contained the interaction parameters G_p , ψ_p and ψ_r that described the shape of the shear stress versus displacement relationship. The solution was determined at $i=2$, making $u(x_{i-1})$, $u(x_i)$ and $u(x_{i+1})$ equal to $u(x_1)$, $u(x_2)$ and $u(x_3)$, respectively, where x_1 was the location of the node at the front of the sample, $x=0$. The displacement at the front of the sample was chosen based on a particular level of applied load.

The length of the embedded geosynthetic specimen was discretized into 1000 equal length elements. The solution to the equation was found when the boundary condition, $\epsilon=0$, at $x=L$ was satisfied. To satisfy this boundary condition values of the displacements at node i were iterated until the slope of the displacement distribution at $x=L$ was zero within a tolerance of 1×10^{-5} . The slope of the displacement distribution at $x=L$ was calculated using equation 4.18. This iterative method was used for all the chosen levels of displacement that

will describe the shear stress/shear displacement relationship. Initially, arbitrary values for G_p , ψ_p and ψ_r were used until the boundary condition at $x=L$ for each level of displacement was satisfied. More appropriate values for these parameters were calibrated using the following procedure.

Calibration of Solution

The hyperbolic equation (Equation 4.8) used for determining the shear stress in terms of displacement included shear softening, as shown in Figure 4.14. The interaction parameters G_p , ψ_p and ψ_r dictate the shape of this curve. By adjusting these parameters and the displacement at node i , numerically calculated loads and displacements could be compared to experimental loads and displacements. The loads were calculated using the approximate load/strain equation for the material and the strain from the first element. The strain in the front was calculated using Equation 4.5 between nodes $i-1$ and i .

Due to obvious difficulties in determining values for G_p , ψ_p and ψ_r simultaneously, it was necessary to develop a more efficient method for their determination. This was done by using the six approximate levels of displacement shown in Figure 4.14. Better initial estimations of these parameters could be made by realizing that certain parameters have greater influence at different levels of displacement. The parameter G_p , the interface shear modulus, described the shear stress at lower displacement levels. Therefore, to calibrate G_p , low values of displacement at $u(x_1)$ were used, like those in Level I. Displacement levels near Levels II and III allowed ψ_p to be calibrated. The parameter ψ_p described the peak shear stress along the geosynthetic. From larger values of displacement, like those in Levels IV,

V and VI, the parameter ψ_r , which described the residual shear stress, could be calibrated. Even though each variable tended to influence a different level of displacement, they were not independent of each other. The interface layer thickness, d , was simply a multiplier to the stiffness modulus, therefore varying d effected the solution in the same manner as changing G_i .

Comparisons between the actual test data and the experimental results were done graphically and consisted of a check between the displacement along the sample length and between the load versus displacement curve from the monotonic pull-out tests. When a satisfactory match was determined for both, the shear stress versus shear displacement relationship was determined. From this relationship, values of G_i , τ_{ult} and τ_{res} were determined and used to describe the constitutive interaction parameters.

This process was repeated for samples tested under all confinement levels. Values of τ_{ult} , τ_{res} and G_i were taken from each of the shear stress versus shear displacement curves and plotted with respect to their corresponding applied normal confinement, σ_v , to describe their relationship to normal confinement.

Results from Numerical Analysis

Typical results from a monotonic test performed on a geotextile sample pulled in the cross-machine direction at a normal confinement of 35 kPa are shown in Figures 4.15 through 4.22. Comparisons between actual and calculated displacement distributions along the sample length are made in Figures 4.15 through 4.20. The comparison between the actual load and the predicted load is shown in Figure 4.21. Lastly, the shear stress versus shear displacement curve is plotted using the calculated interaction parameters (Figure 4.22).

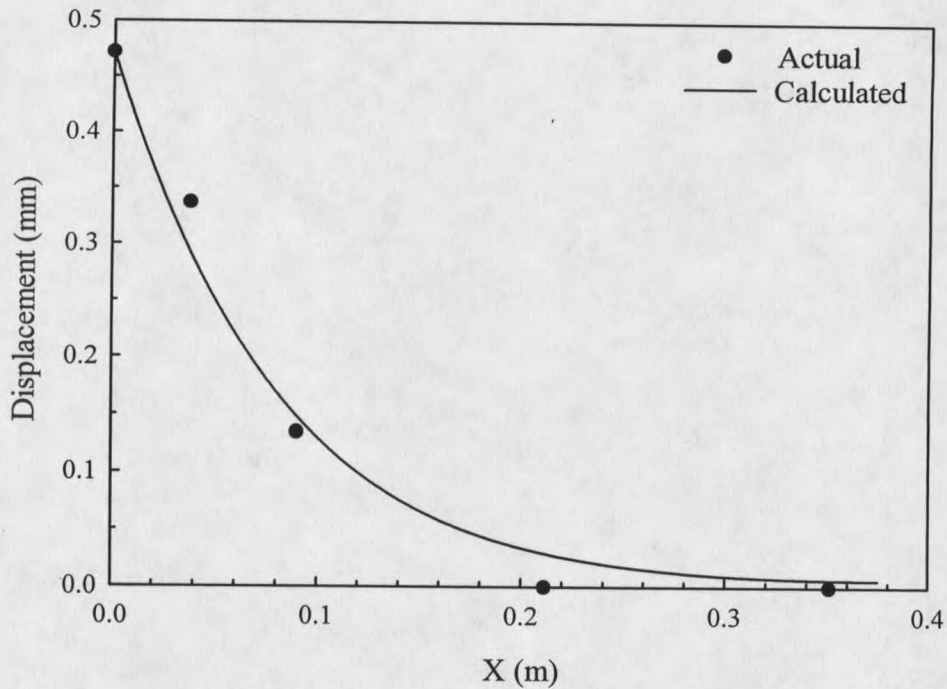


Figure 4.15: Comparison of Level I displacements for the geotextile pulled in the cross-machine direction at 35 kPa normal confinement: gtxd-15.

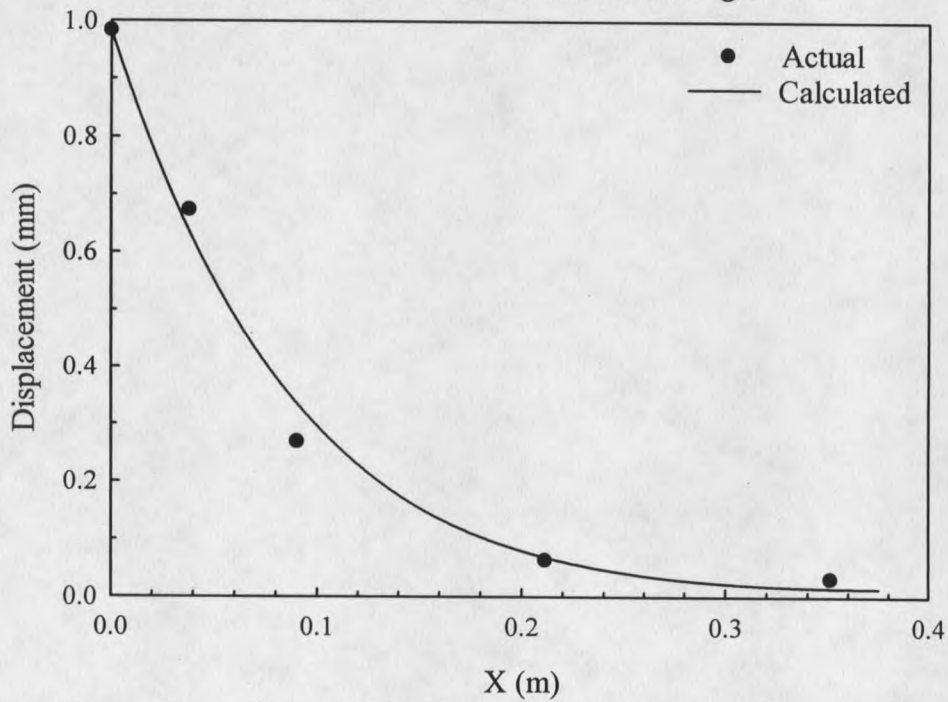


Figure 4.16: Comparison of Level II displacements for the geotextile pulled in the cross-machine direction at 35 kPa normal confinement: gtxd-15.

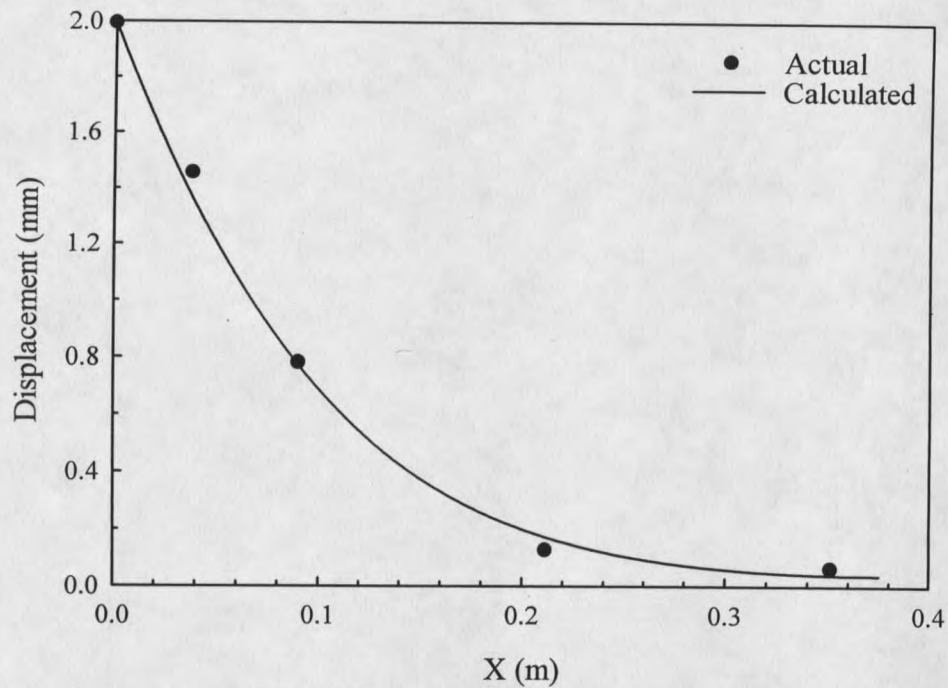


Figure 4.17: Comparison of Level III displacements for the geotextile pulled in the cross-machine direction at 35 kPa normal confinement: gtxd-15.

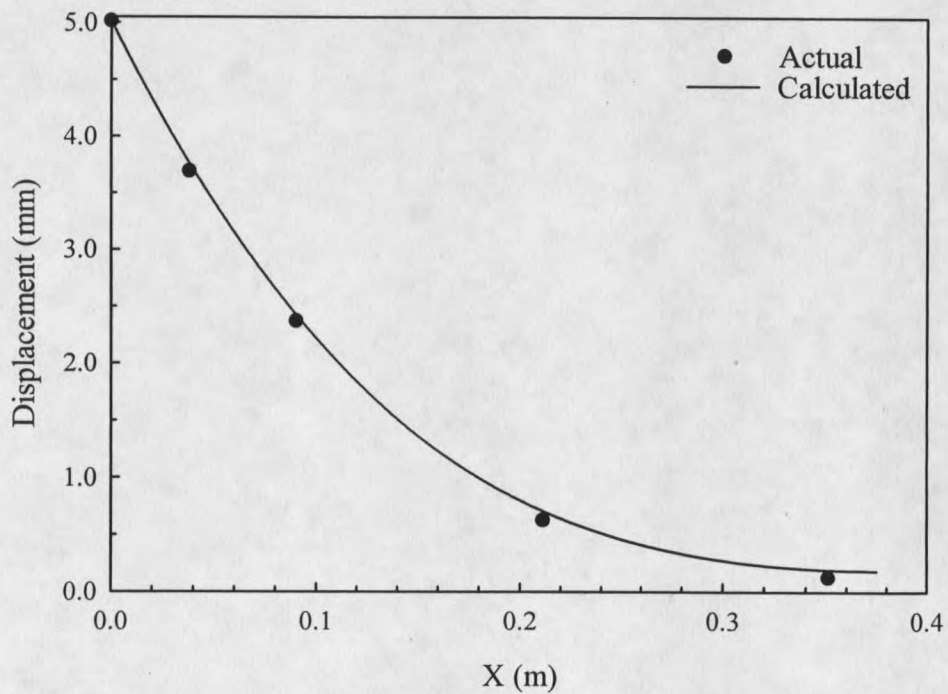


Figure 4.18: Comparison of Level IV displacements for the geotextile pulled in the cross-machine direction at 35 kPa normal confinement: gtxd-15.

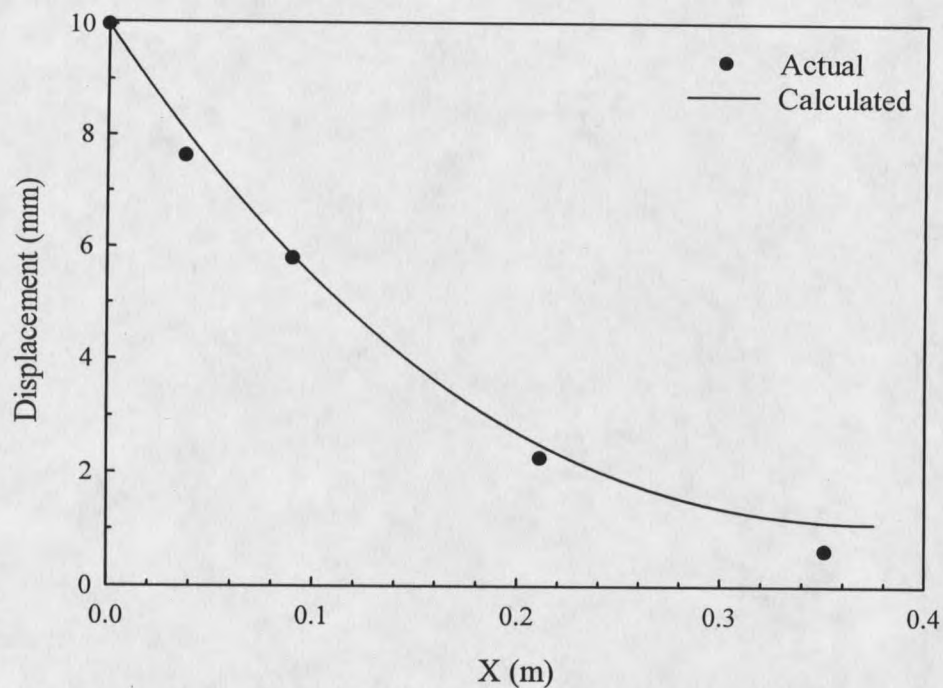


Figure 4.19: Comparison of Level V displacements for the geotextile pulled in the cross-machine direction at 35 kPa normal confinement: gtxd-15.

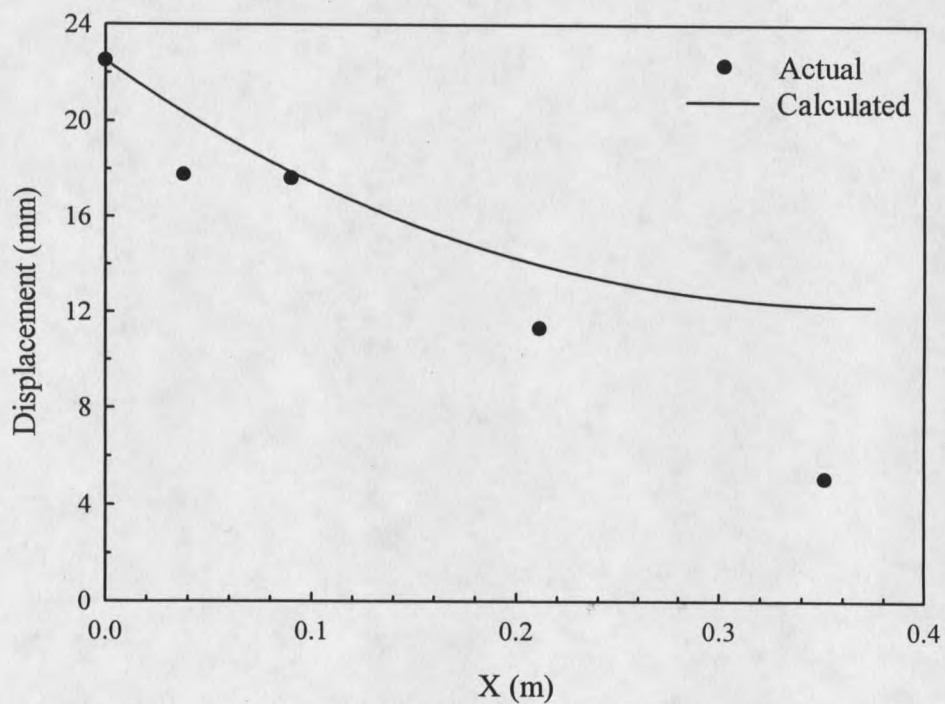


Figure 4.20: Comparison of Level VI displacements for the geotextile pulled in the cross-machine direction at 35 kPa normal confinement: gtxd-15.

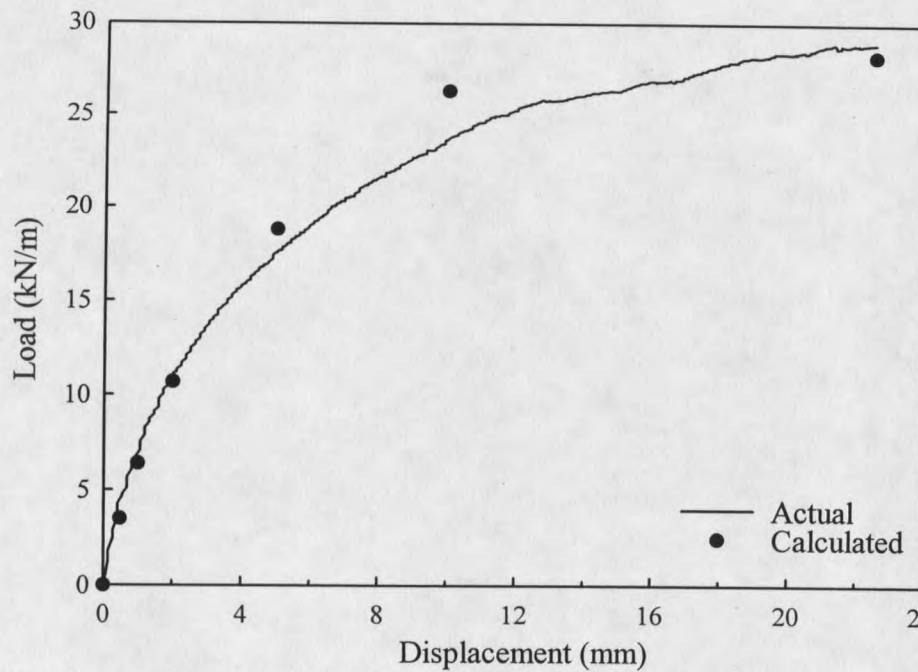


Figure 4.21: Comparison between actual and predicted load/displacement curves for the geotextile pulled in the cross-machine direction at 35 kPa normal confinement: gtxd-15.

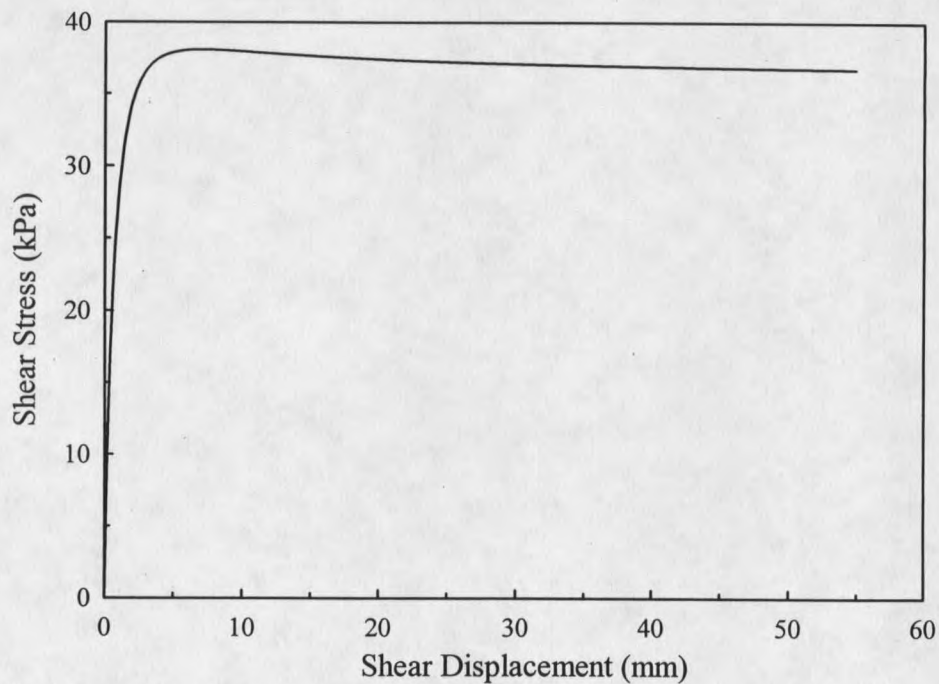


Figure 4.22: Predicted shear stress versus shear displacement curve for the geotextile pulled in the cross-machine direction at 35 kPa normal confinement: gtxd-15.

A reasonable fit for each of the six levels of displacement is made with the exception of Level VI. The predicted load versus displacement curve also provides a satisfactory match. These figures show that the numerical solution to the governing differential equation was effective in determining reasonable values for the soil/geosynthetic interaction parameters. A complete set of figures showing the predicted shear stress versus shear displacement curves with their corresponding predictions of the displacement distributions and load/displacement distributions are given in Appendix C. Table 4.3 shows the interaction parameters associated with these predictions.

Material/Direction	σ_n (kPa)	τ_{ult} (kPa)	τ_{res} (kPa)	ψ_p (deg)	ψ_r (deg)	G_i (kPa)
Geogrid/ Machine	5	8.3	8.3	59	59	230
	15	20.7	19.2	59	52	600
	35	30.2	26.4	47	37	900
Geogrid/Cross-Machine	5	9.4	9.4	62	62	200
	15	20.9	20.6	57	54	400
	35	41.9	41.7	52	50	800
Geotextile/Machine	5	4.3	4.0	45	39	75
	15	16.6	15.5	53	46	250
	35	51.4	32.6	62	43	600
Geotextile/Cross-machine	5	3.9	2.9	45	30	100
	15	15.7	13	53	41	275
	35	38.1	36.2	52	46	600

Table 4.3: Interaction parameters for the geogrid and geotextile.

The relationship between the ultimate and residual values of shear with respect to normal confinement for the geogrid is shown in Figure 4.23. This plot indicates that ultimate and residual values of soil/geosynthetic interface shear stress do not increase linearly for increasing levels of normal confinement in either principle strength direction. Instead, secant friction angles along this curve tend to decrease as normal confinement increases. This phenomenon is similar to the nonlinearity associated with friction angles calculated for dense granular soils. Under low confinement conditions shearing is accompanied by soil particles that dilate while for high confinement conditions, soil particles are restricted to more of a sliding motion. Typically, shearing failure caused by soil dilation requires more energy than sliding particles. Therefore, the suppression of soil dilatency at higher confinements reduces the amount of work required to reorient soil particles at the shear interface. As a result, soil friction angles tend to decrease with increasing confinement levels. This should also be true for the case where a geosynthetic is shearing through soil since shearing mechanisms at the soil/geosynthetic interface are similar. However, the relationship between ultimate interface shear stresses versus normal confinement is more linear for the geogrid in the cross-machine direction. The geogrid in this direction is more stiff making it less resistant to strains induced by shear stresses. Therefore, more energy is imparted by the shearing process into the soil rather than into the geosynthetic. This is evident since there is less reduction of the secant friction angle at higher normal confinements when the material is pulled in its stiffer direction.

For the geotextile pulled in the machine direction, a different phenomenon appeared to be occurring, as shown in Figure 4.24. Monotonic tests on the material in the machine

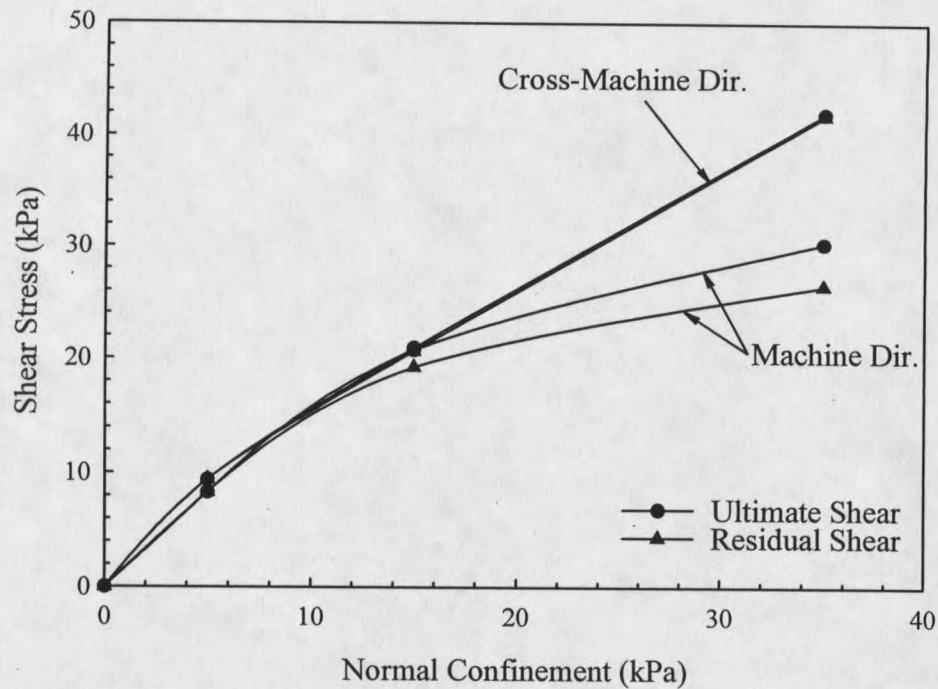


Figure 4.23: Ultimate and residual shear stress versus normal confinement curves for the geogrid.

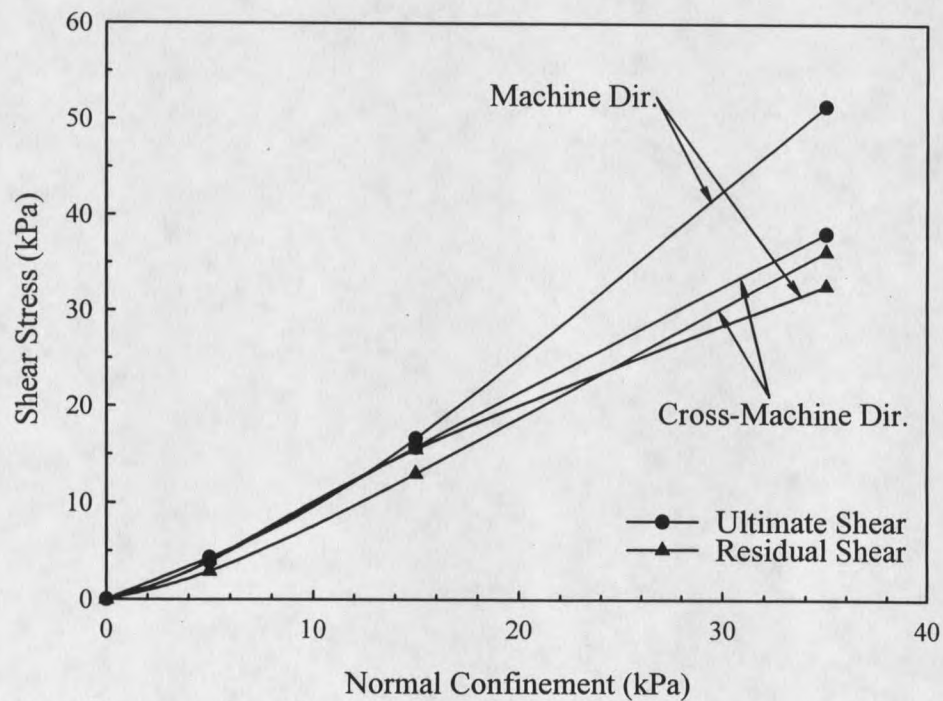


Figure 4.24: Ultimate and residual shear stress versus normal confinement curves for the geotextile.

direction indicated that the secant friction angle increased with increasing normal confinement. This was thought to be due to the ability of soil particles to protrude through and bear passively on the material at higher confinements. The geotextile was thin and non-rigid in the z-direction allowing larger soil particles to protrude through and into the material easily. Passive resistance from protruding soil particles increased soil/geotextile interaction resulting in a higher soil/geosynthetic friction angle at higher confinements. However, this increased interaction did not seem to affect the ultimate shear stress in the cross-machine direction as dramatically as in the machine direction.

Values of the initial interface shear modulus and normal confinement can also be affected by these types of phenomena. Curves showing the relationships between the initial interface shear modulus and the applied normal confinement for the geogrid and the geotextile are shown in Figure 4.25. Values for G_i tended to follow similar patterns to the ultimate and residual interface shear stress relationships commented on earlier. Differences in the shear mechanism at the soil/geosynthetic interface affected relationships between the shear modulus and normal confinement in the same manner as confinement affected the ultimate and residual shear values.

The shear modulus values for the geogrid in the machine direction were higher at higher confinements than in the cross-machine direction. This was thought to be due to the physical differences between the material as it was pulled in these directions. Samples of geogrid had more ribs per length in the machine direction than in the cross-machine direction. This would increase the initial interface modulus since the moving material would have to overcome a greater amount of resistance by soil particles bearing against ribs.

Differences in the interface shear modulus in the geotextile were small since the surface of the material was physically similar in each of the material's principle directions.

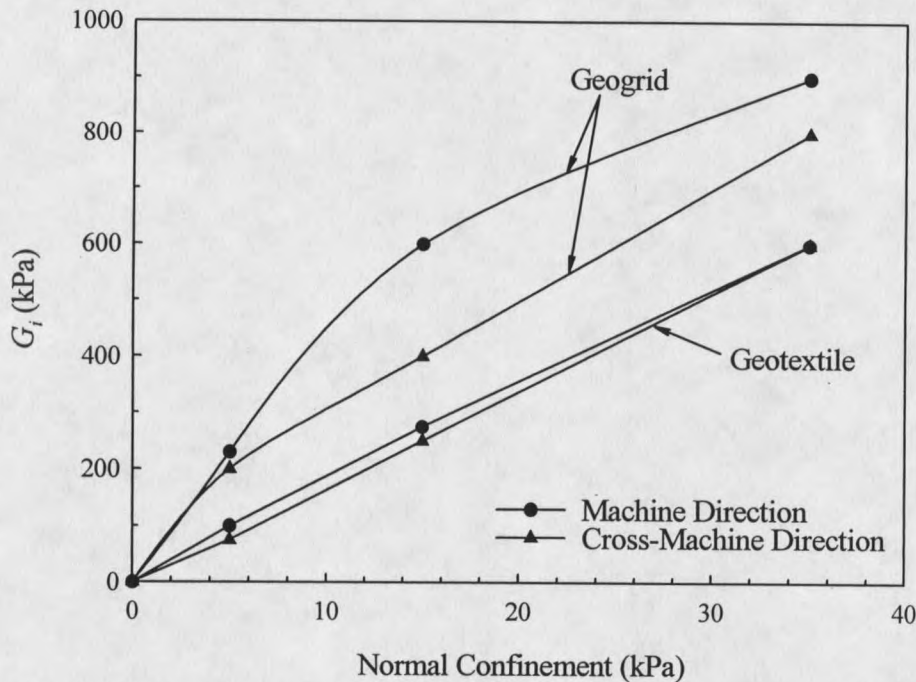


Figure 4.25: Initial interface shear moduli versus normal confinement for the geogrid and the geotextile.

Two material types were used in this research to study differences in pull-out for different materials. The geogrid material provided greater strength at low displacement levels through an increase in the initial interface shear modulus. Ultimate values of resistance were comparable between each of the materials when pulled through the gravel.

Differences between the thickness and flexibility of these materials in the z-direction resulted in different behaviors during pull-out. The geogrid material had substantial thickness in the out-of-plane direction making it resistant to local out-of-plane bending while the geotextile did not have appreciable thickness in the z-direction, making it less resistant to out-of-plane distortions. This effected the ultimate values of shear stress. The ultimate

values of shear stress were effected by the materials ability to resist out-of-plane distortions caused by the confinement soil. Geogrid was able to resist these distortions better since it had greater thickness and rigidity in the z-direction than the geotextile.

For the gravel confinement soil used, it can be concluded that the geogrid provided better pull-out resistance than the geotextile with the exception of the ultimate resistance of the geotextile when pulled in the machine direction at 35 kPa normal confinement. For transportation applications, where there is small relative displacement between the reinforcement fabric and the confinement soil, values of the initial shear modulus will most likely govern pavement reinforcement design. Since the geogrid provides greater resistance at low displacement levels, it would therefore provide the greatest reinforcement.

CHAPTER FIVE

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER RESEARCH

Summary

A large scale test facility was designed, built and commissioned to perform both in-air and in-soil tests on geosynthetic materials. Its design provided two loading methods: monotonic and cyclic. One major criterion of its design was to reduce the effects of the boundary conditions imposed on the test specimen by the apparatus. Ideas collected from previous researchers helped define and reduce boundary condition effects by incorporating certain aspects of design.

The necessary parameters to calibrate the constitutive model were successfully determined using the methods of experimentation and interpretation proposed in this thesis. Geosynthetic constitutive material parameters were determined from in-air, element tests performed on the geosynthetics oriented in the machine, cross-machine and 45 degree directions. Unconfined, uniaxial element tests provided the inherent material parameters of these materials. Performing fast monotonic tests on the geosynthetics was used to determine the material's orthotropic, elastic/plastic properties without creep. To help further define the material's elastic/plastic response with creep, slow monotonic tests were performed. Cyclic

in-air tests facilitated the calibration of the material's nonlinear, combined isotropic/kinematic hardening properties.

In-soil, pull-out tests were performed at three normal confinements and were used to determine soil/geosynthetic interaction parameters. The constitutive model required parameters that described the relationship between shear stress and shear displacement. Determination of these parameters was attempted using the mobilizing process method. Complications with this method required the author to develop a numerical method based on the solution of a governing differential equation that describes the interaction problem. Satisfactory definition of the interaction parameters was achieved using this method and allowed relationships of shear stress versus shear displacement to be formed.

Conclusion

The increased use of finite element models in design has made it necessary to determine more specific material parameters. These new parameters require that new test configurations be used. The research program described in this thesis contributed to this end by helping refine existing methods of testing to define these constitutive material parameters better.

Constitutive parameters describing both the geosynthetic material and soil/geosynthetic interaction can be successfully obtained using in-air element tests and in-soil, pull-out tests, respectively. To define the material's intrinsic material properties within the constitutive matrix, monotonic, in-air tests must be performed on the geosynthetic. To define the material's kinematic hardening properties, cyclic tests must be performed. Creep

tests and slow monotonic tests must be performed to define creep parameters.

Soil/geosynthetic interaction parameters can be successfully obtained using pull-out tests. Measurements of the load applied to the front of the sample and displacements along the embedded length are sufficient for calibrating the newly developed numerical analysis that defines the interaction parameters based on progressive mobilization of the embedded geosynthetic. Upon establishing the reliability of the numerical method used to determine the soil/geosynthetic interaction parameters, its use will provide a valuable tool for the analysis and design of geosynthetic reinforced soils.

During this study, much was learned about the construction, use and interpretation of in-air and in-soil testing of geosynthetics. From the knowledge gained, the following list of miscellaneous conclusions was made:

- ◇ Pull-out tests performed on the geotextile indicated that the method of connecting the displacement cables to the material can cause extra resistance and therefore may temporarily bind material at the measurement point. It was concluded that displacement measurement connections must be made to have a very low profile.
- ◇ The use of bonded foil strain gages mounted directly to the material was an effective method of measuring strains on the sample.
- ◇ Using greased membrane layers along the walls allowed the compactor to densify the soil near the wall more effectively. This provided a more uniform distribution of normal confinement to the underlying sample.

- ◇ The geosynthetic materials used in this study were proven to be strain-rate sensitive materials, showing both creep and viscous behaviors.
- ◇ Pull-out tests produced nonlinear displacements along the embedded length of the geosynthetic.
- ◇ As confinement increases, the ultimate value of shear at the soil/geosynthetic interface also increases, making interaction normal stress dependent.
- ◇ The initial interface shear modulus increases as confinement increased and followed similar patterns to the ultimate and residual values of shear stress for each material.
- ◇ Plots of the initial interface shear modulus versus applied normal confinement for the geogrid indicated a positive decreasing relationship. This relationship for the geotextile was nearly linear.
- ◇ Comparisons between monotonic and cyclic pull-out tests indicated that ultimate pull-out capacity is lower for cyclic loads.

Recommendations for Further Research

The work contained in this thesis falls into three broad categories: 1) design and operation of the test apparatus, 2) execution of in-air and in-soil experiments and 3) interpretation of data obtained from these experiments. The following sections contain suggestions for further research in each of these three areas.

Design and Operation of Test Facility

The design of the test facility in this work was comprised mostly of ideas compiled from other researchers. One main criterion of the design was reducing the effects of the rigid boundaries on test results. Past studies have been devoted solely to defining design techniques to reduce these boundary effects. The purpose of this research was not to define these issues or to develop a standard test procedure. However, comparative studies between this work and others would be of value, since only a draft standard method for pull-out testing of geosynthetics currently exists. To aid in the development of this standard, comparative experiments that vary the pertinent design components discussed in Chapter 3 should be performed. This information would be valuable to those interested in developing future pull-out apparatuses that would reduce the effects of rigid boundaries on soil/geosynthetic interaction.

In this study, pneumatic cylinders were used to apply the cyclic loads to the geosynthetics in the load controlled tests. Due to the compressibility of air, precise control of the pressures delivered to the pneumatic cylinders was difficult. Although the pneumatic cylinders were fitted with low friction seals, at low pressures their response was rough and sluggish. To attain a uniform and timely response of cyclic loads, it is recommended that hydraulically controlled equipment replace the existing pneumatic system. Hydraulic systems give more predictable and timely response, making the resulting data clearer.

The use of the load-transfer sleeves provided a means of reducing the load against the front wall. Another suggested method of reducing front wall boundary effects may be

to use a flexible pneumatic membrane at the front wall to create a non-rigid surface for the dilative soil against which to react. Doing this would require comparisons between tests performed with and without the non-rigid boundary and comparisons between the non-rigid boundary and the load-transfer sleeves.

Sample Preparation and Testing

The monotonic tests performed allowed accurate material parameters describing the material's orthotropic elastic/plastic response to be obtained. Describing the shear parameters of the material was accomplished using samples oriented 45 degrees from the material's principle direction. Buckling of the geogrid material caused the load in the material to level off prematurely. To remedy this, attempts were made to confine the material with greased membranes to reduce the buckling, however, the added friction from the confinement made data interpretation impossible. To describe material behavior in materials oriented in the 45-degree direction accurately, testing methods that reduce or account for buckling need to be devised.

The epoxy/sheet metal bonding technique used to clamp the geosynthetic sample had many advantages, however, bonding the sheet metal plates to the geotextile was difficult. Since the surface of the geotextile used in this study did not have open apertures to allow epoxy to bond through the material like the geogrid, bonding to its surface was difficult. Therefore, the geotextile/sheet metal bond often controlled failure of the test. Developing a new method or modifying the current gluing technique would allow failure of the geotextile occur. One idea might be to create artificial holes in the portion of the sample that is to be

bonded to allow the epoxy to flow through the sample and possibly increase the bond strength. Another alternative is to use a jaw-type clamp to grip the sample for the in-air tests. However, this will not work for the in-soil tests since smooth, low profile clamps are needed.

For the in-air tests, extensometers were used to measure displacements that were later used to calculate strains. The internal springs within the extensometers provided tension to the extension cables attached to the geosynthetic. This was advantageous since external methods of applying tension to these cables was not needed. However, the construction of these extensometers did not allow the cable to be extended and suddenly released. Allowing the cables to snap back freely caused the internal spring to break. Overextension of these gages also caused them to fail. This was an issue in this work since geosynthetic samples were often tested to failure. Upon failure, energy stored in the material often caused violent and unpredictable rupture. Although preventive work was developed to protect these gages, a more robust method of measuring the displacements would alleviate the problem of damaging the instrumentation.

In the cyclic in-soil tests performed on the geotextile a region of high pore water pressure developed at the soil/geosynthetic interface. This was most likely caused by the rapid movement of the geosynthetic as it was being pulled out. This movement induced positive pore water pressures in the surrounding soil creating a region of lower effective stresses that lowered pull-out resistance. This phenomenon was not apparent in the geogrid experiments. This was most likely because the closed structure of the geotextile did not allow pore water to drain freely as the test progressed whereas the geogrid did allow water to pass through it, thereby lessening this effect. Incorporating measurements of the pore

water pressure near the soil/geosynthetic interface may help quantify this behavior. Consequently, pore water pressure buildup needs to be considered in future cyclic, in-soil tests.

Data Interpretation

Within the mobilizing process method, the dilemma of finding an equation to match the displacement measurements was never remedied. To use this method accurately, a better technique for approximating the displacement distribution needs to be formulated. If this cannot be done adequately, results will most likely be based on straight line portions between the measurement points. This will result in a conservative estimate of soil/geosynthetic interaction. Ideally, more measurements along the embedded length would help define the shear/displacement relationship to yield more realistic strain results.

Within the numerical analysis, comparisons between the actual and calculated displacement distributions along the length of the geosynthetic and between the actual and calculated load/displacement curves were the method of calibrating the interaction parameters. To fine tune this numerical process further, a more sophisticated method of comparison should be used. Graphic comparisons are subject to assumptions made by the analyst, where numerical comparisons are more independent of these assumptions.

Lastly, it is recommended that the test equipment and analytical techniques developed in this study be used with materials other than the two used in this study. Two such possibilities might be a uniaxial geogrid and a non-woven geotextile. This would promote further refinement of the testing and analytical techniques used in this study.

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APPENDICES

APPENDIX A

INSTALLATION OF GEOSYNTHETIC BONDED RESISTANCE STRAIN GAGES

Introduction

Direct measurements of local strain were made on both of the geosynthetics using foil strain gages. The strain gages were bonded directly to the material using a high elongation epoxy for the geogrid and a silicone adhesive/sealant for the geotextile. A detailed description of the gages, the methods used for applying these gages to the materials and examples of data received from them are given in this Appendix.

Gage Description

The strain gages consisted of EP-08-500GB-120 gages for the geogrid and EP-08-40CBY-120 gages for the geotextile provided by Measurements Group, Inc. (MGI). MGI also supplied the means of applying them to the material. The EP series gages were chosen since they could withstand the high elongation of the geosynthetics to which they were attached. Literature from MGI said that these gage types showed zero shift under high-cyclic strains. This was beneficial since these gages would be used in the cyclic, in-air tests. However, these gages only had a fatigue life of 10,000 cycles at 1000 microstrain ($\mu\epsilon$). Many gages failed when loaded cyclically since strain levels exceeded the 1000 $\mu\epsilon$ level. The next number in the gage title described the gage alloy type. The metal foil in the 08 series consisted of an annealed constantan for maximum ductility of the strain gage. The next designation group described the gage length and tab geometry of the strain gage. The 500GB gages, were 0.5 inches in length and the 40CBY gages were 4.0 inches in length.

Figure A.1 shows a diagram of each of these gages. The value of 120 on the end indicated the resistance of the gage in ohms.

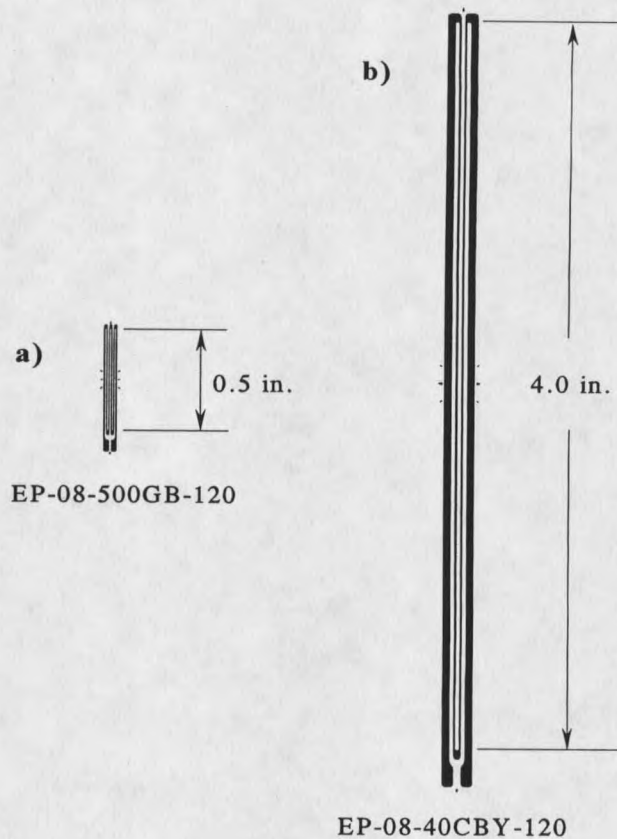


Figure A.1: The two types of foil strain gages used: a) geogrid strain gage and b) geotextile strain gage.

Gage Placement

Gages were attached to the geosynthetics using a multi-step procedure. The procedure used was dependent on the material type. The general procedure for the geosynthetics used in this study consisted of six steps: 1) preparation of the geosynthetic and strain gage surfaces, 2) preparation and placement of the adhesive, 3) placement and alignment of the gage, 4) curing of the glue, 5) connection of the electronics and 6) covering the gage and surrounding area with environmental protection.

Geogrid Gage Placement Procedure

The first step in the procedure was to prepare the surface of the geogrid. To insure proper gage to geogrid bonding, the surface of the geogrid needed to be flat and clean. To account for possible bending of the rib, strain gages were placed on both sides. Therefore, the following processes were performed for both sides of the material simultaneously.

Due to the manufacturing process used for making a biaxial geogrid, its ribs have a non-uniform thickness in both of the principle strength directions. Therefore, the ribs were first filed to a uniform thickness laterally to the gage. This process removed very little material from the rib. Next the rib was thoroughly cleaned using mild solvents provided by MGI. After cleaning, a razor blade was used to score the surface of the rib very lightly to provide a rough surface to adhere to. This scoring process was done very carefully so as not to make an uneven surface. Cutting the surface with the razor must be done carefully since creating too rough of a surface could negatively effect results. This method of roughing the surface seemed to be more controlled than using sandpaper since sandpaper tended to raise the polypropylene fibers from the surface creating an uneven surface. The surface was again thoroughly cleaned using the solvents. A neutralizer was used after the solvent application to neutralize the bond surface before gluing.

High elongation epoxy supplied by MGI was mixed to the correct ratio and applied sparingly to the gage placement areas. The gages were removed from the package and carefully placed on the rib. If the gages were accidentally dirtied through contact with skin or other objects before being placed, they were cleaned using special cleaning solvents

provided by MGI for this purpose. The gages were placed so that the center of the gage corresponded to the middle of the rib. Indicators stamped on the gages (see Figure A.1) allowed precise placement with respect to the rib.

Once the gage was properly placed, a thin piece of Teflon was placed over the gage and taped to the surrounding geogrid. This protected the gage during the curing process and kept the gage stationary. To insure proper adhesion the gages were clamped at a pressure ranging between 35 and 140 kPa, as specified by MGI. Rubber pads were placed on both sides of the ribs containing strain gages to provide a non-rigid surface onto which the weights were placed. The weights were placed to provide the proper confinement pressure.

The epoxy could be properly cured using either of two methods: 1) allow it to air dry for two weeks at +24°C (+75°F) or 2) elevate the glue line temperature to +74°C (+165°F) for two hours. The latter method was used to expedite testing. To elevate the temperature, a hot air dryer was placed in a hole in the top of a box. The box contained adjustable apertures on the sides so that the ambient temperature could be held constant at +74°C. A thermometer placed in the box enabled the temperature to be monitored during the curing process. After the glue had been in this environment for at least an hour, the weights were removed to allow the warm air to reach the gages directly.

Two bondable terminals were also glued to the geogrid near the gages. These terminals provided the connection point for the gage leads and cables connected to the data acquisition unit. The leads were soldered to the bondable terminals.

A Wheatstone $\frac{1}{2}$ bridge completed the circuit and was located near the data acquisition. Having the bridge completed away from the strain gages allowed the precision

resistors to be reused. The 120 ohm, bridge completion resistors provided by MGI had tolerance of $\pm 0.01\%$. Figure A.2 shows a schematic of the electronic setup for these gages.

In this figure, the strain gages are labeled ϵ and the precision resistors are labeled R.

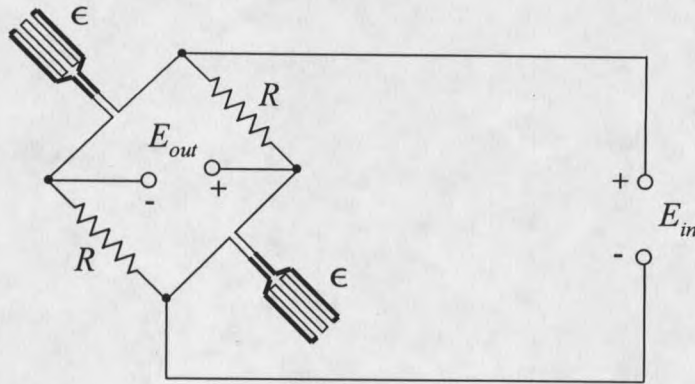


Figure A.2: Wheatstone bridge circuit.

To calculate strain from the output voltages the following expression was used:

$$\epsilon(\%) = \frac{-2e^5 \left(\frac{E_{out}}{G} \right)}{\left(\frac{E_{out}}{G} \right) \cdot F - 1000 \cdot E_{in} \cdot F} \quad (A.1)$$

where ϵ is percent strain of the gage, E_{in} and E_{out} are the input and output voltages in volts, respectively, F is the electronic gain and G is a gage factor given by the manufacturer. The gage factors used were 2.070 for the 500GB gages and 2.080 for the 40CBY gages.

Lastly, environmental protection was used to simulate the protection that was to be used in the cyclic-plate load tests. The environmental protection was needed in these tests to keep moisture and other foreign materials from coming into contact with the strain gages. During the in-air tests, environmental protection was used to negate possible differences in

the strain measurements for samples with and without environmental protection, though it was not necessary. The environmental protection consisted of 3.2 millimeter thick coatings of butyl rubber sealant, a solvent-thinned butyl rubber and clear contact paper. The thicker more rigid butyl rubber sheets were placed on both sides of the geogrid covering the gages on each side. Pressure was applied to them to ensure a thorough bond was formed to and through the geogrid material. The solvent-thinned butyl rubber was then liberally distributed around the edges to negate the possibility of water reaching the enclosed strain gages. The thin contact paper was added to the outside for added protection against larger foreign particles.

After the placement of the environmental protection, the gage was ready to be used. Input voltages were controlled using DC amplifiers and the output voltages were monitored using the data acquisition device.

Geotextile Gage Placement Procedure

Placement procedure of the strain gages on the geotextiles differed from those used on the geogrid due to the drastically different surface characteristics of geotextiles. Longer strain gages were used for the geotextile to allow a more average strain to be obtained.

The bonding surface for the strain gage needed to be as smooth as possible. Since the surface of the woven geotextile was very rough, a smooth surface needed to be formed. The material used to provide a smooth surface needed to be more flexible than the geotextile so reinforcement of the area did not occur. However, the basal substance also needed to be sufficiently rigid to transmit strains from the geotextile to the strain gage. A non-corrosive,

RTV, silicone rubber adhesive/sealant (Dow Corning 3145 RTV) satisfactorily met these criteria and was used as the base and surrounding material for applying these gages to the geotextile.

Thorough cleaning of the geotextile surface was accomplished using acetone. This was used primarily to rid the surface of the oils used in the manufacturing process. After cleaning, a bead of the silicone adhesive/sealant was deposited on the material. This was massaged into the fibers of the material until it could be seen coming through the other side. Since the adhesive/sealant did not form a chemical bond with the geotextile, working it into the material's tight weave allowed a good mechanical bond to be formed. A razor blade was used to smooth out the surface of the glue forming a flat plane of adhesive at a level equal to the top of the geotextile fibers. The gage was placed on top of the glue and aligned with the weave of the material using a placement guide. A piece of Teflon was placed over the top of the gage and the adhesive to protect it from sticking to the workbench as the other side was prepared in the same manner. The adhesive/sealant was not massaged into the fibers since glue from the other side was already protruding through and rubbing would disturb the already placed gage. Once both sides were at the same stage, the Teflon was carefully removed from side one without disturbing the gage to allow the adhesive to dry. The time for the glue to cure fully was 24 hours for a 0.5 mm thickness at 50% relative humidity.

Once the adhesive was sufficiently dry, another layer of adhesive/sealant was placed over the entire gage with the exception of the wire leads extending from the gage. The adhesive/sealant provided the environmental protection, keeping water and other foreign material from touching the gages. This new layer was also allowed to dry. After it was dry,

another very thin layer of the adhesive was placed allowing a piece of latex rubber to be bonded to the top. This provided protection from the larger soil particles that would be experienced in the cyclic-plate load tests.

After these protective layers were placed and completely cured, the electronics were connected as described in the geogrid section. Bondable terminals were not used for the geotextile. Instead, the data acquisition wires were connected directly to the wires extending from the strain gage. After soldering the wire connections, a liberal amount of the adhesive/sealant was applied to the wire connections; thoroughly covering all exposed wires. This was done to protect the strain gage from moisture. After this had cured, a liberal amount of the solvent-thinned butyl rubber was spread over the entire area to ensure a water-tight seal. Finally, a piece of plastic wrap was placed over the butyl rubber as a final protective measure. After the completion of these steps, gage placement was complete and was ready for use.

Results

Results from the foil strain gages were not directly used in the analysis of the material parameters. The foil gage results were used to compare strains obtained from the strain gages to strains calculated using the extensometers. The following plots of load versus strain show the differences between the axially measured strains and the strains obtained by the foil strain gages.

Foil strain gages were used on the slow monotonic and the cyclic Series I and II tests, but were not used on the fast monotonic or the in-soil tests. In general, correlations between

axial strains and the foil strains for the monotonic tests was good (see Figures A.3 and A.4). The geotextile sample showed a larger difference than the geogrid, as seen in Figure A.5. However, the initial foil strains on the geotextile were satisfactory. Foil strains measured in the geogrid for the cyclic Series I tests tended to slightly overestimate strain, as shown in Figures A.6 and A.7. Inconsistencies in the strain data from the geotextile was most likely due to the transfer of strains through the adhesive/sealant. In the machine direction the foil gage stopped operating most likely due to debonding between the adhesive and the geotextile (see Figure A.8). Conversely, results from the geotextile pulled in the cross-machine direction indicated that a better bond existed in that direction, as seen in Figure A.9. Since in the Series II cyclic tests, cycles were repeated thousands of times at relatively high strains, it was likely that these foil gages would not last the duration of the test. As expected, for most of the strain gages used in the cyclic, Series II tests, failure occurred in the gages before loading was complete (see Figures A.10 and A.11). Surprisingly, strains in the cross-machine direction on the geotextile continued to be output for the entire duration (see Figure A.12). Results from the gages used in the Series II tests indicated that higher plastic strains occurred in the gage than in the material. Further investigation using strain gages on these materials would allow a better hypothesis to be drawn regarding the different strain behaviors observed from these gages.

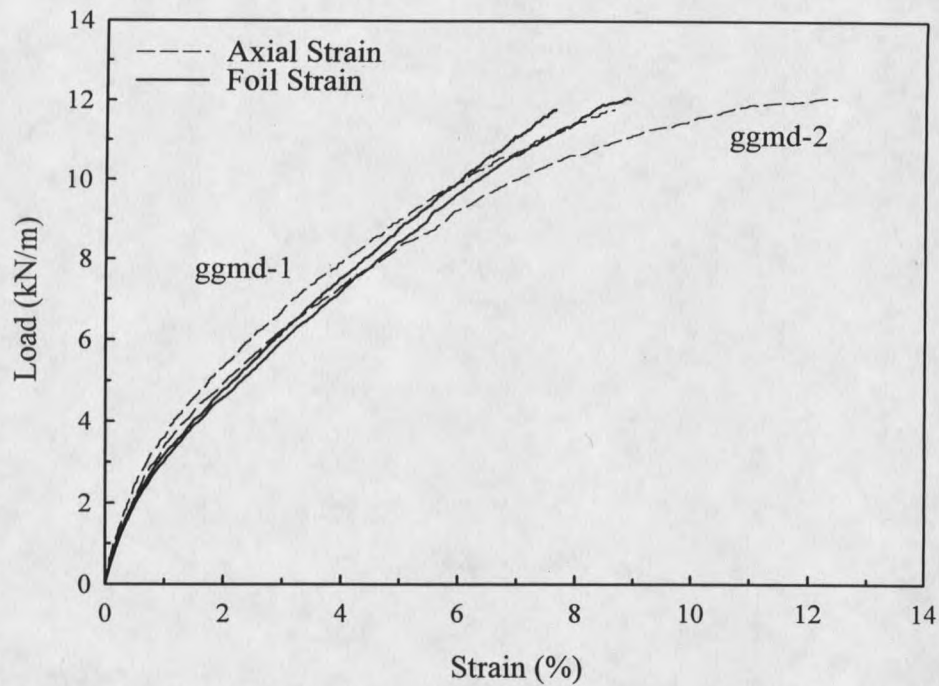


Figure A.3: Comparison of the foil and axial strain results from slow monotonic tests performed on the geogrid in the machine direction.

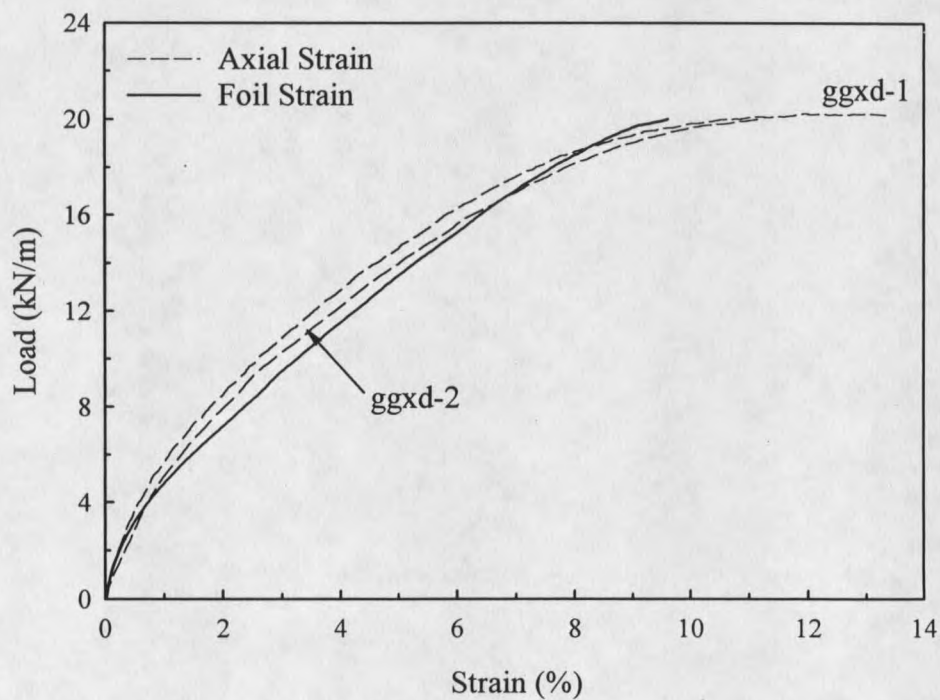


Figure A.4: Comparison of the foil and axial strain results from slow monotonic tests performed on the geogrid in the cross-machine direction.

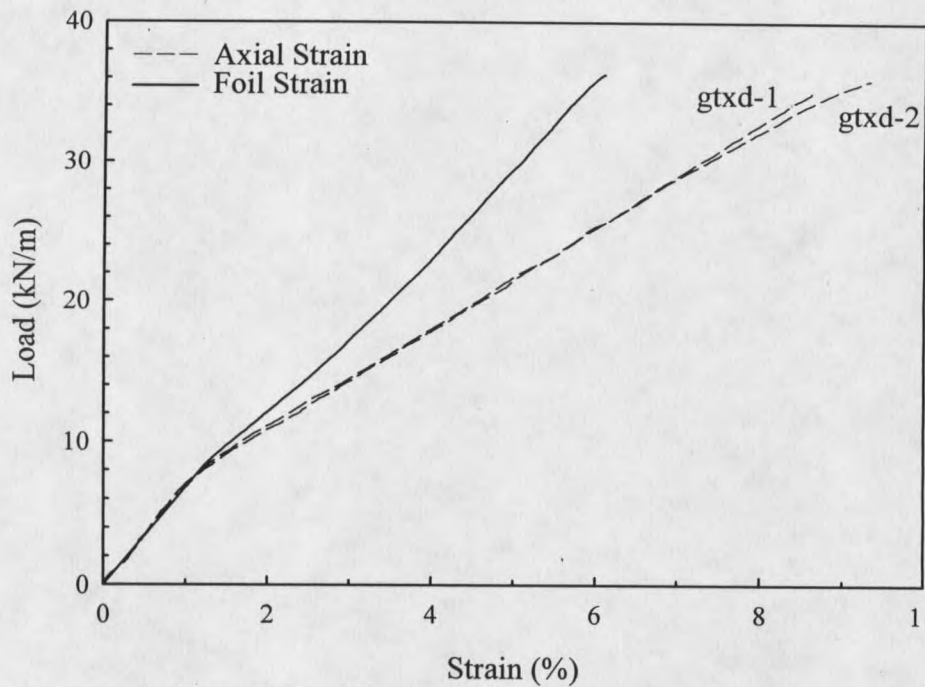


Figure A.5: Comparison of the foil and axial strain results from slow monotonic tests performed on the geotextile in the cross-machine direction.

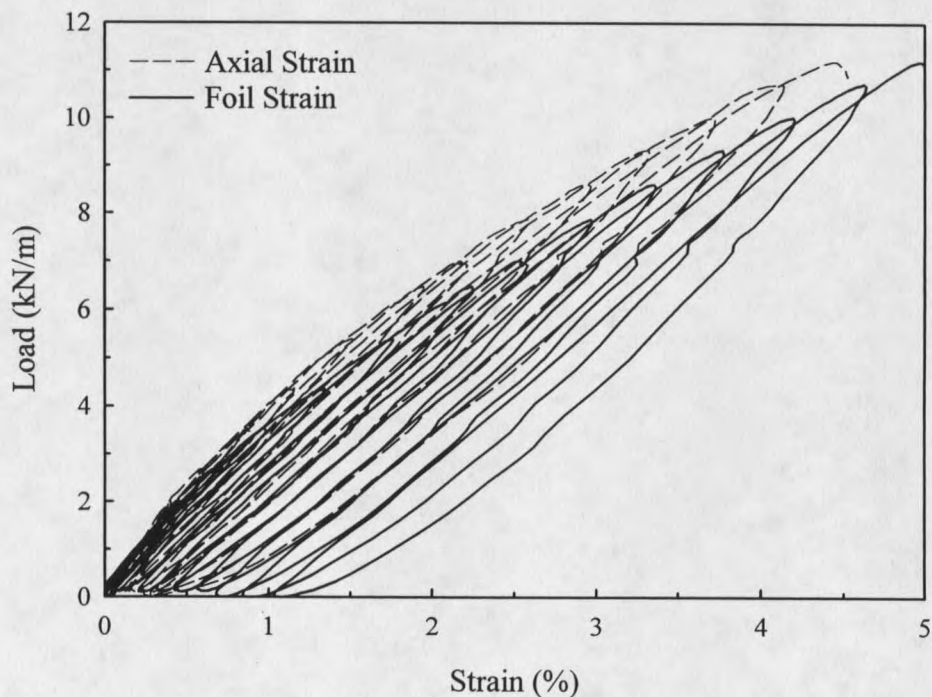


Figure A.6: Comparison of the foil and axial strain results from a cyclic, Series I test performed on the geogrid in the machine direction (ggmd-3).

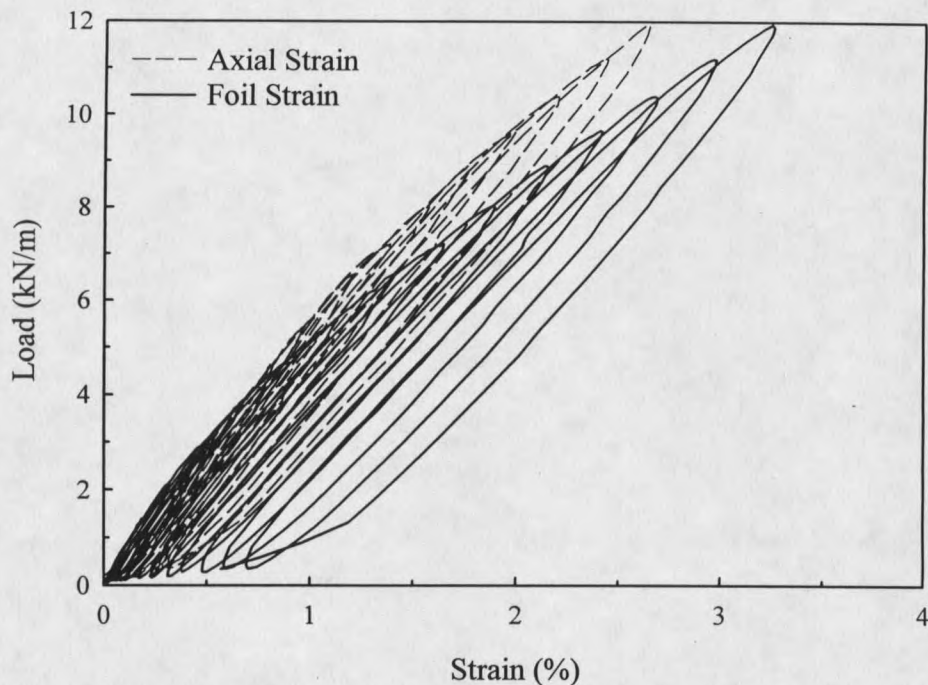


Figure A.7: Comparison of the foil and axial strain results from a cyclic, Series I test performed on the geogrid in the cross-machine direction (ggxd-4).

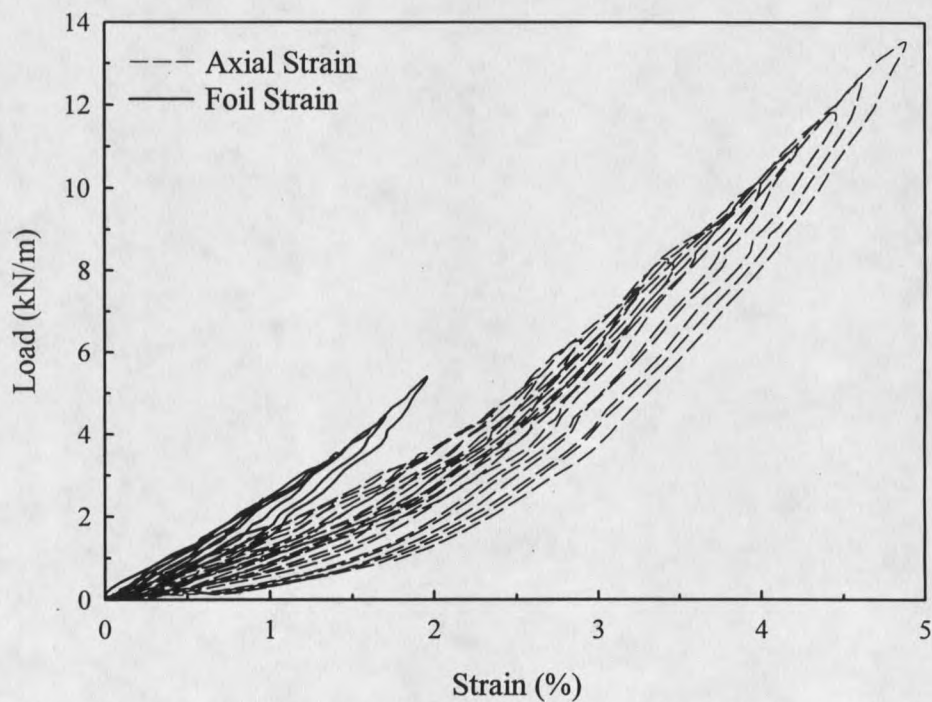


Figure A.8: Comparison of the foil and axial strain results from a cyclic, Series I test performed on the geotextile in the machine direction (gtmd-4).

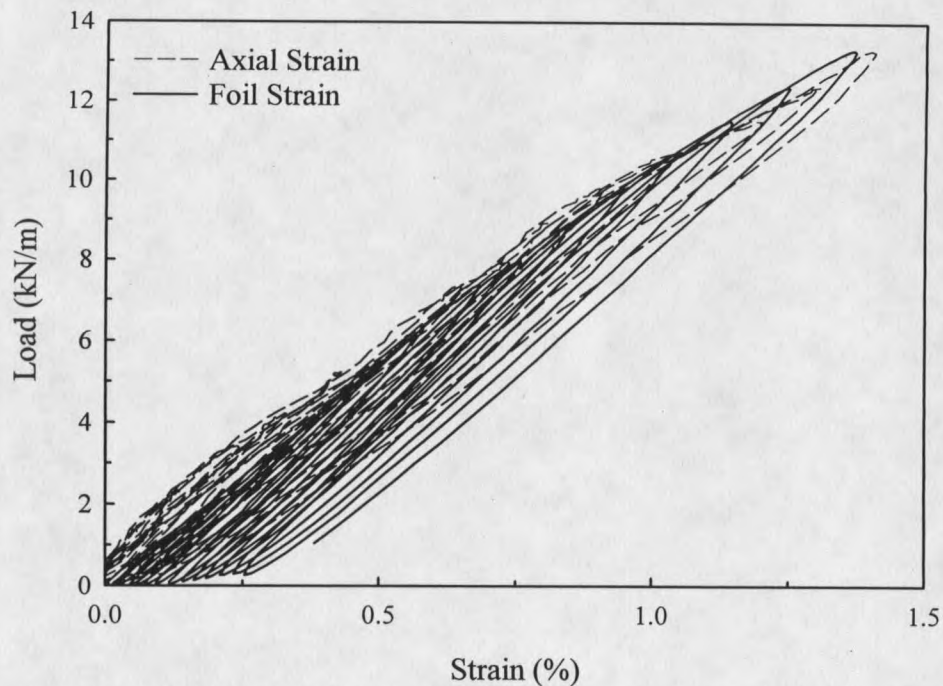


Figure A.9: Comparison of the foil and axial strain results from a cyclic, Series I test performed on the geotextile in the cross-machine direction (gtxd-3).

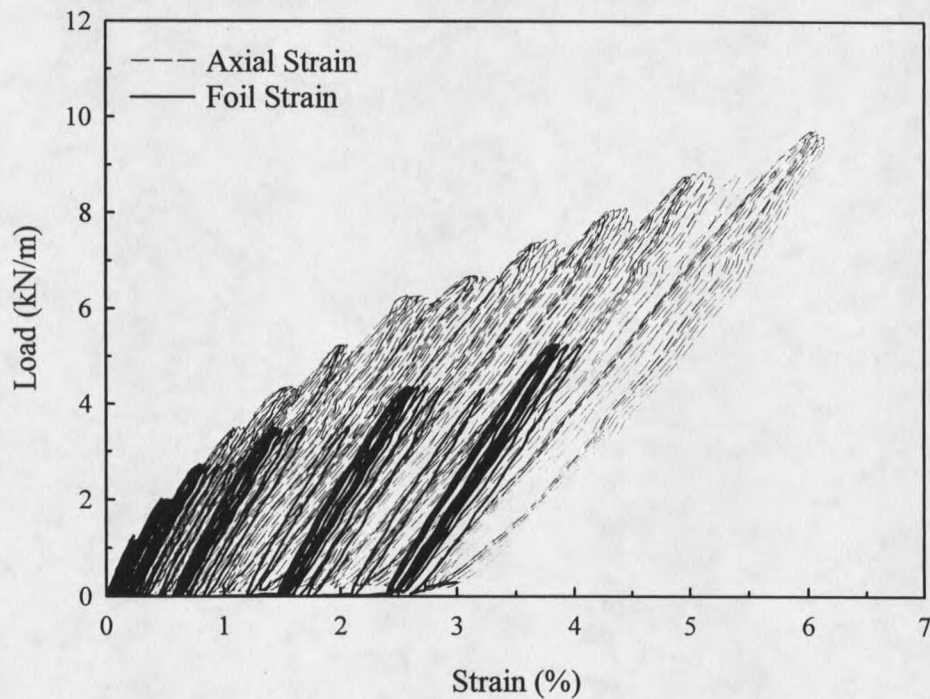


Figure A.10: Comparison of the foil and axial strain results from a cyclic, Series II test performed on the geogrid in the machine direction (ggmd-6).

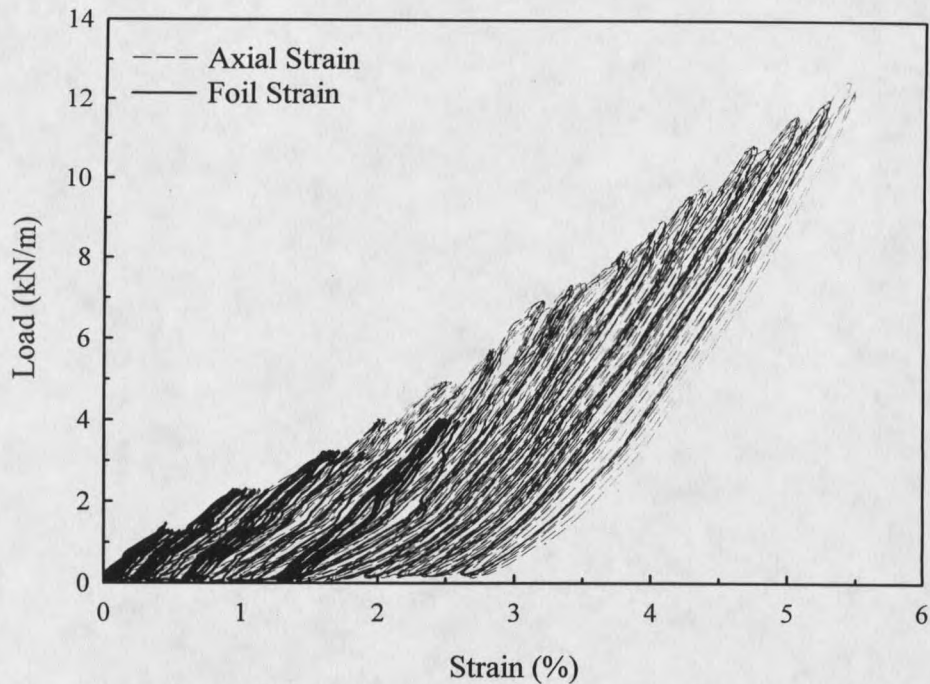


Figure A.11: Comparison of the foil and axial strain results from a cyclic, Series II test performed on the geotextile in the machine direction (gtmd-5).

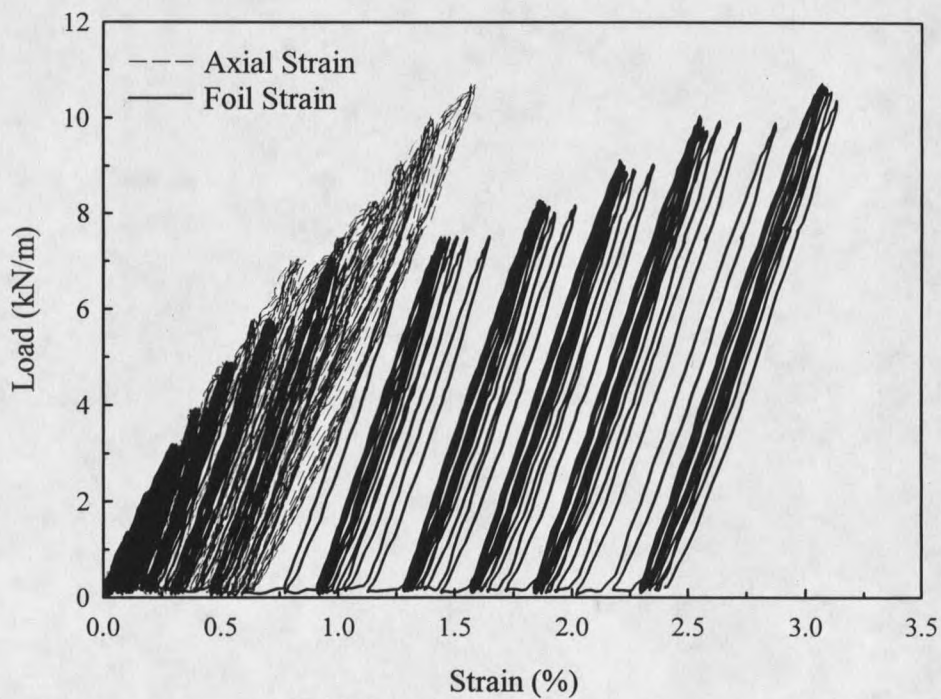


Figure A.12: Comparison of the foil and axial strain results from a cyclic, Series II test performed on the geotextile in the cross-machine direction (gtxd-5).

APPENDIX B

COMPLETE RESULTS FROM THE IN-AIR AND IN-SOIL TESTS

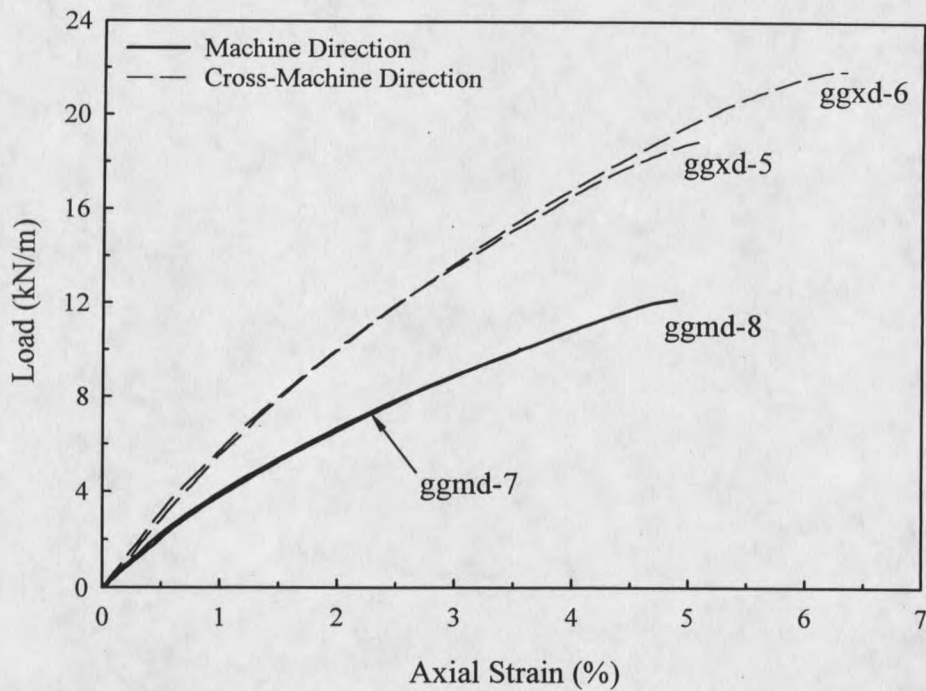


Figure B.1: Fast, monotonic, load/strain curves for the geogrid pulled in the machine and cross-machine directions.

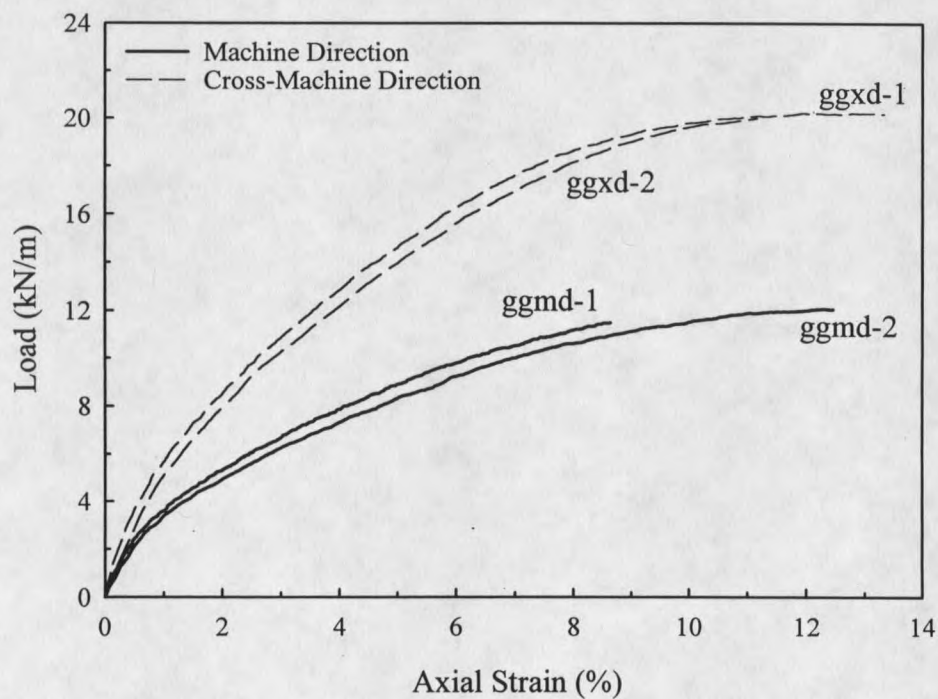


Figure B.2: Slow, monotonic, load/strain curves for the geogrid pulled in the machine and cross-machine directions.

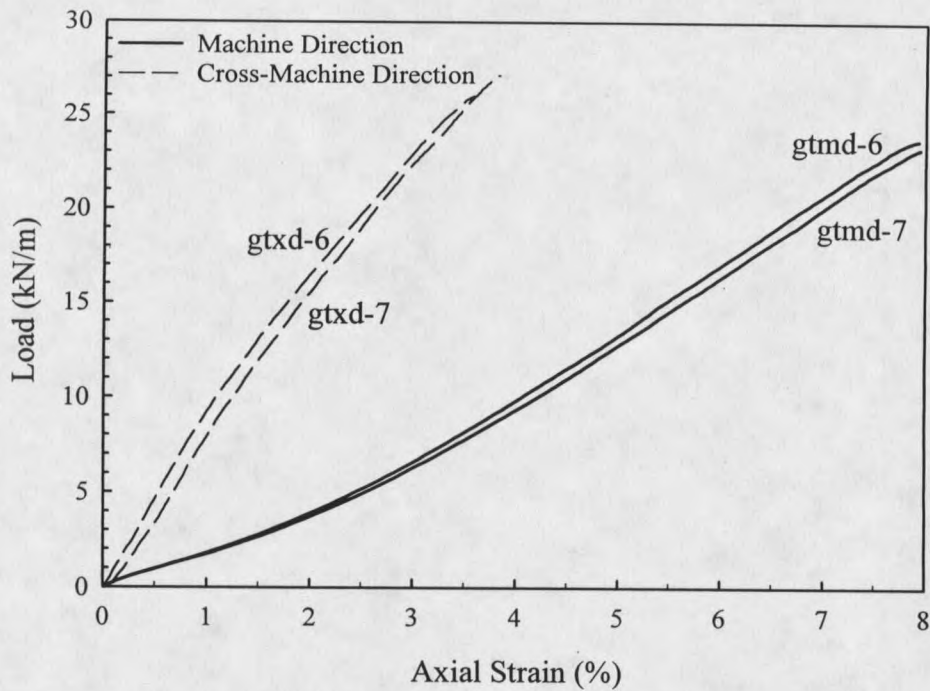


Figure B.3: Fast, monotonic, load/strain curves for the geotextile pulled in the machine and cross-machine directions.

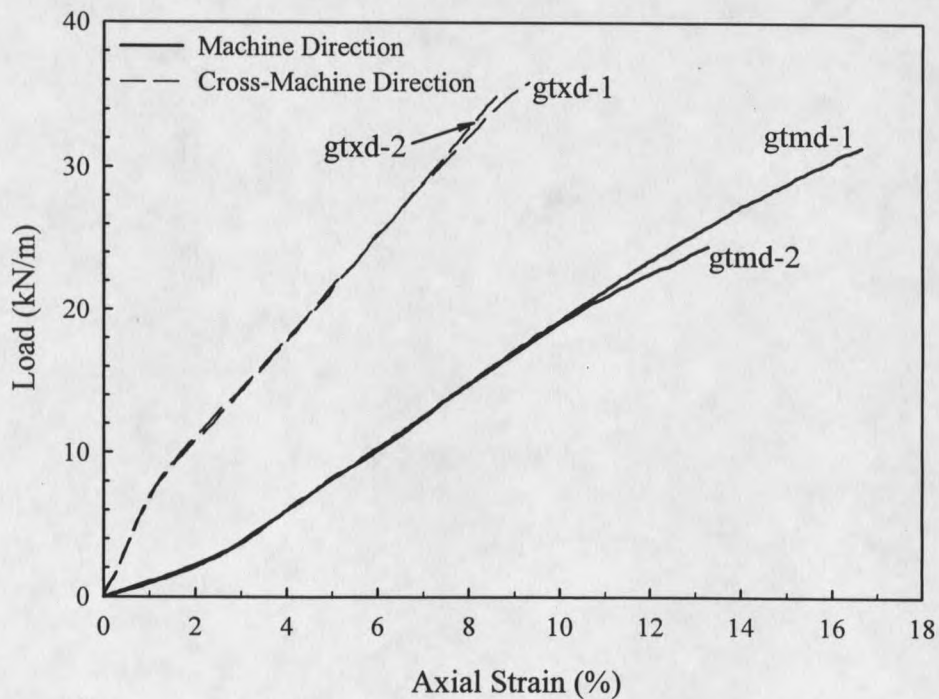


Figure B.4: Slow, monotonic, load/strain curves for the geotextile pulled in the machine and cross-machine directions.

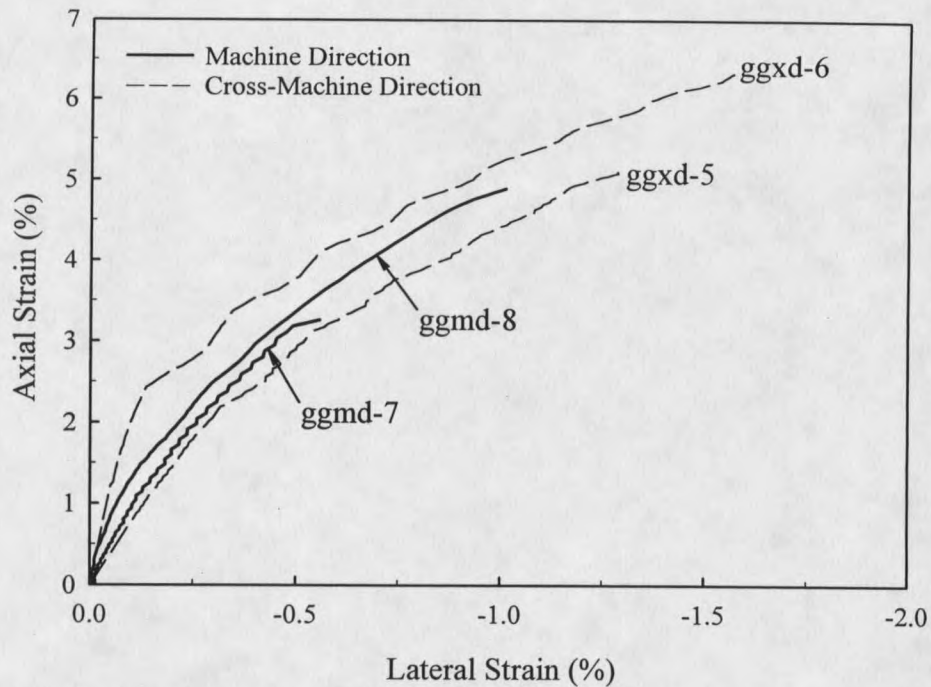


Figure B.5: Fast, monotonic axial strain versus lateral strain curves for the geogrid pulled in the machine and cross-machine directions.

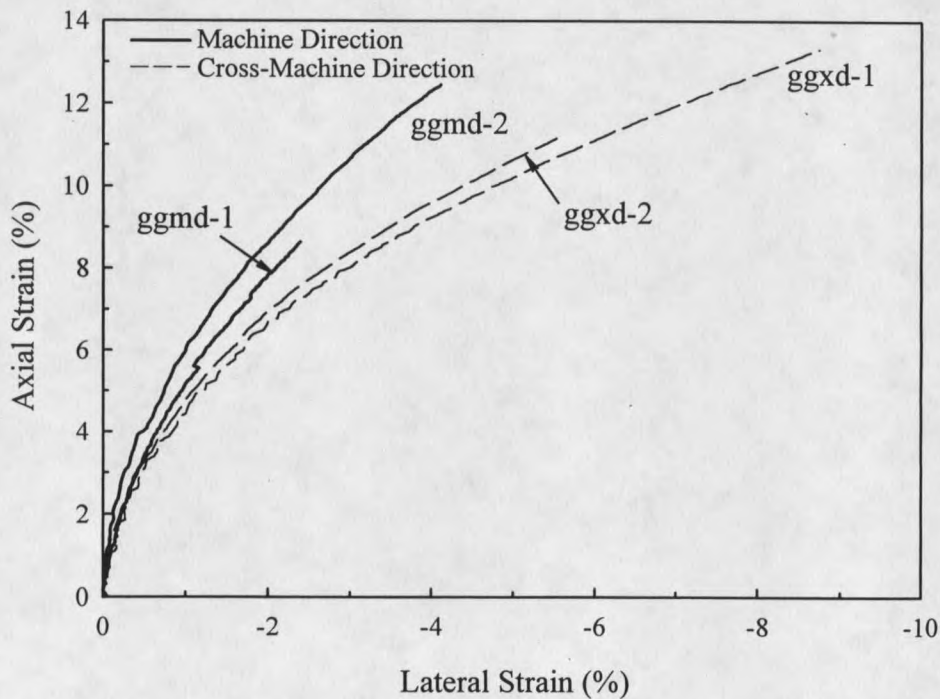


Figure B.6: Slow, monotonic axial strain versus lateral strain curves for the geogrid pulled in the machine and cross-machine directions.

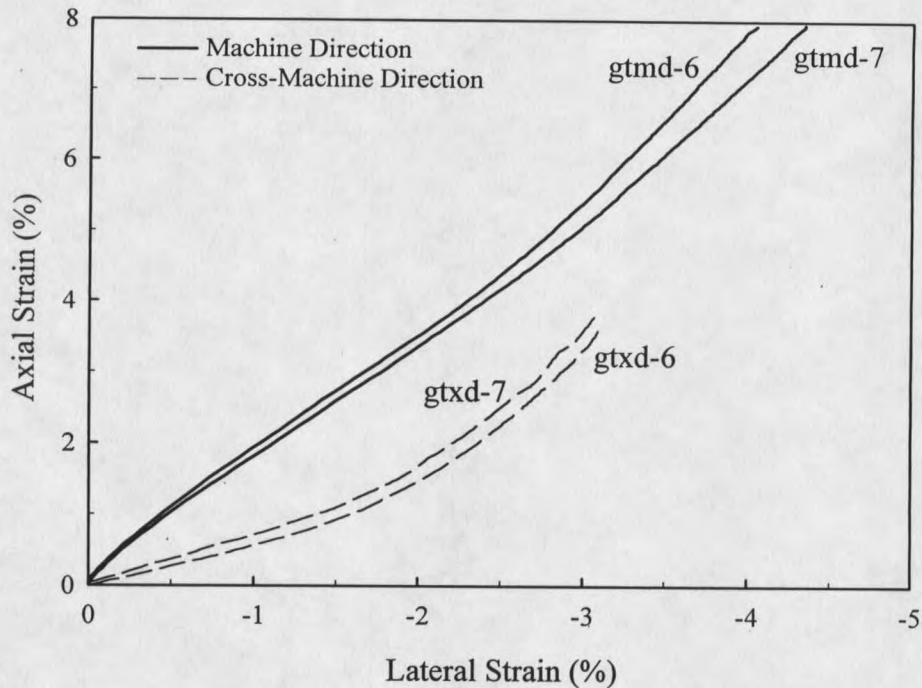


Figure B.7: Fast, monotonic axial strain versus lateral strain curves for the geotextile pulled in the machine and cross-machine directions.

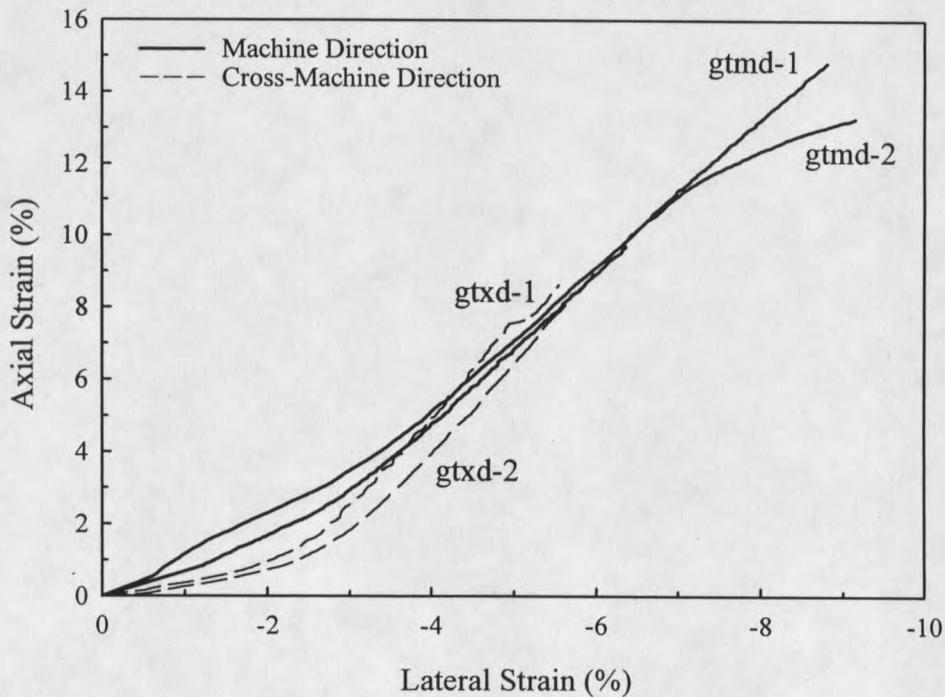


Figure B.8: Slow, monotonic axial strain versus lateral strain curves for the geotextile pulled in the machine and cross-machine directions.

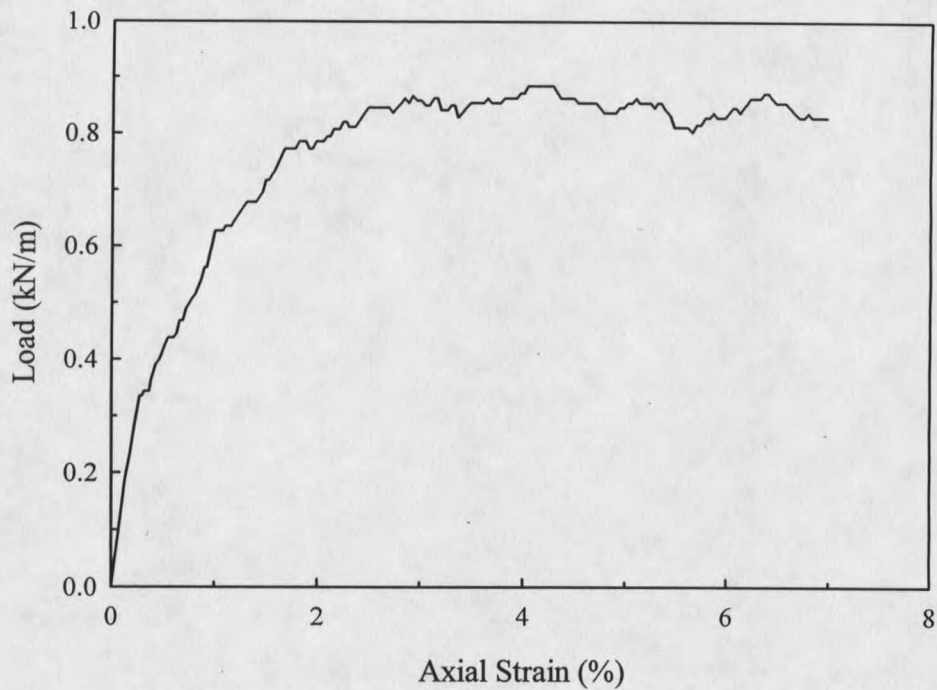


Figure B.9: Load versus axial strain in the geogrid oriented at 45 degrees from its principle strength directions: gg45-1.

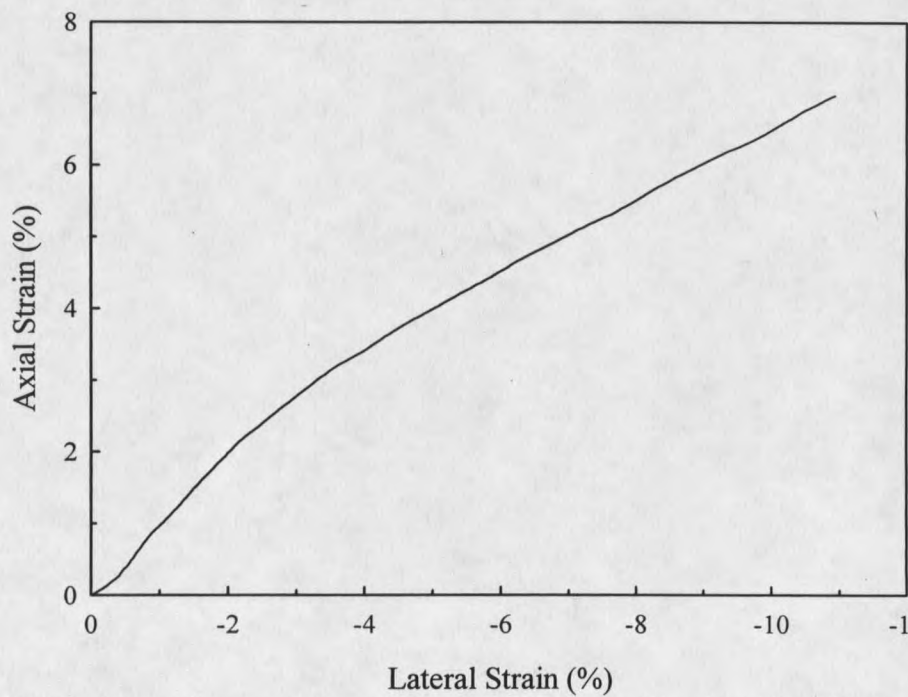


Figure B.10: Axial strain versus lateral strain in the geogrid oriented at 45 degrees from its principle strength directions: gg45-1.

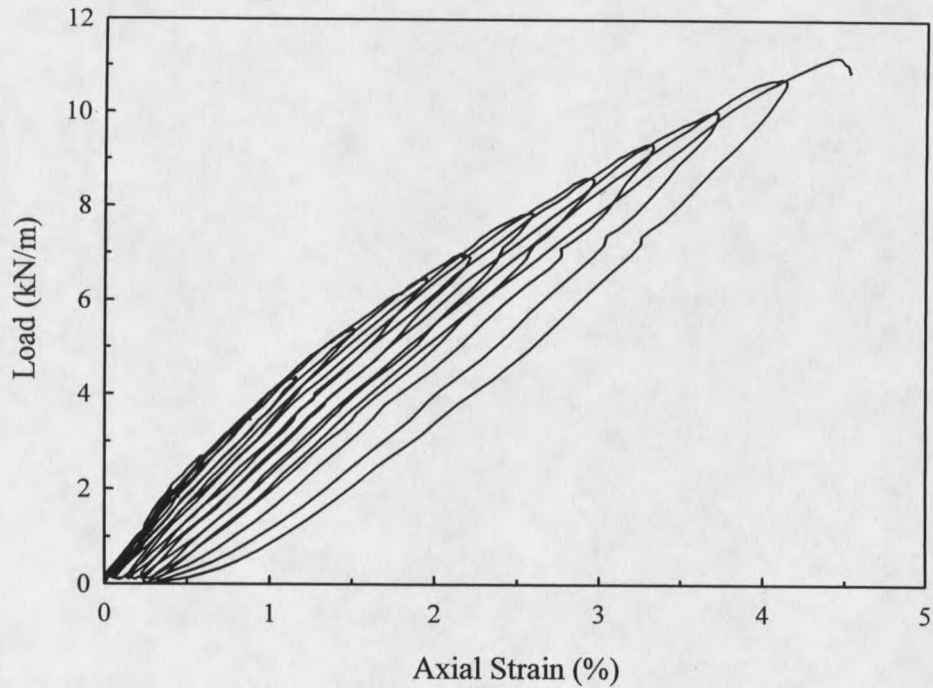


Figure B.11: Cyclic, Series I, unloading/reloading load/strain curves for the geogrid pulled in the machine direction: ggmd-3.

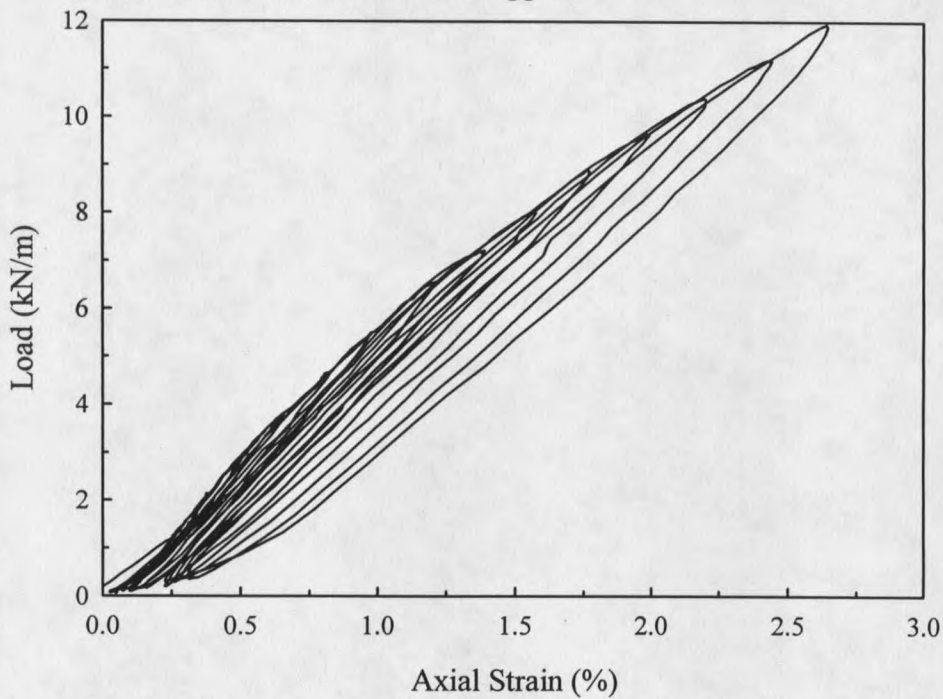


Figure B.12: Cyclic, Series I, unloading/reloading load/strain curves for the geogrid pulled in the cross-machine direction: ggxd-4.

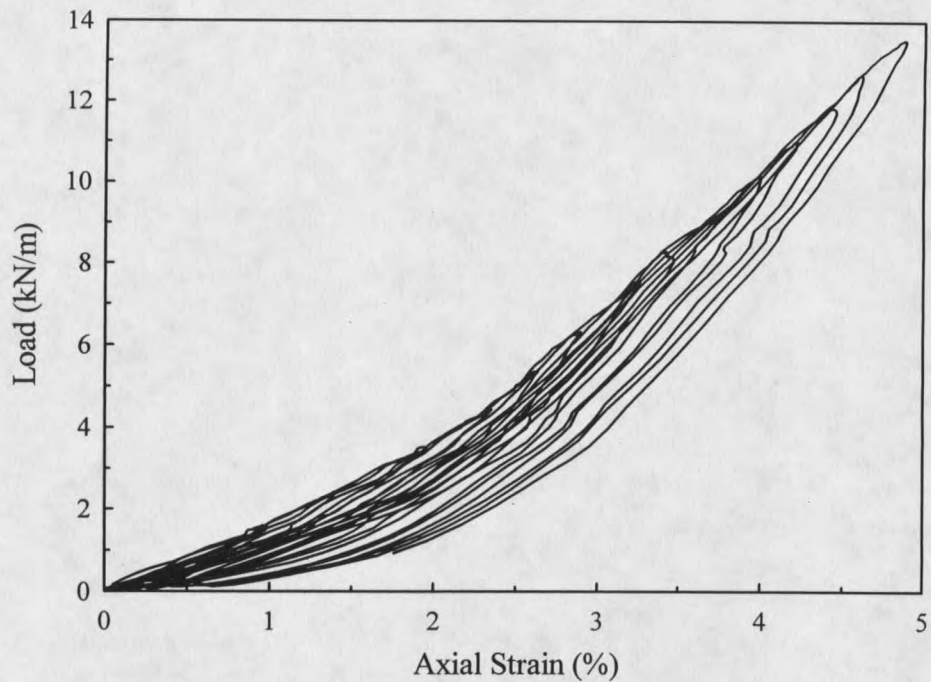


Figure B.13: Cyclic, Series I, unloading/reloading load/strain curves for the geotextile pulled in the machine direction: gtmd-4.

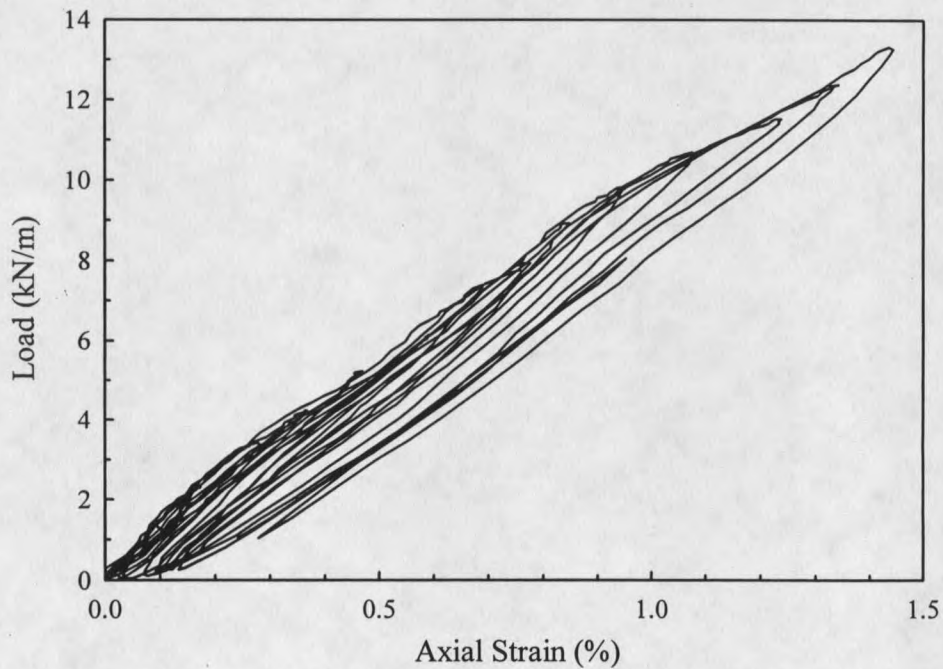


Figure B.14: Cyclic, Series I, unloading/reloading load/strain curves for the geotextile pulled in the cross-machine direction: gtxd-3.

Load Level	Cycle Number											Total Cycles Applied
	1	2	4	8	16	32	64	100				
2												100
3	101	102	103	104	108	116	132	164	200			100
4	201	202	204	208	216	232	264	300				100
5	301	302	304	308	316	332	364	480	600			300
6	601	602	604	608	616	632	664	780	900			300
7	901	902	904	908	916	932	964	1080	1200			300
8	1201	1202	1204	1208	1216	1232	1264	1380	1500			300
9	1501	1502	1504	1508	1516	1532	1564	1620	1750	2000		500
10	2001	2002	2004	2008	2016	2032	2064	2120	2250	2500		500
11	2501	2502	2504	2508	2516	2532	2564	2620	2750	3000		500
12	3001	3002	3004	3008	3016	3032	3064	3120	3250	3500	3700	700
13	3701	3702	3704	3708	3716	3732	3764	3820	3950	4200	4400	700
14	4401	4402	4404	4408	4416	4432	4464	4520	4650	4900	5100	700
15	5101	5102	5104	5108	5116	5132	5164	5220	5350	5600	5800	700

Table B.1: Cycle numbers collected and plotted for the in-air, cyclic, Series II tests.

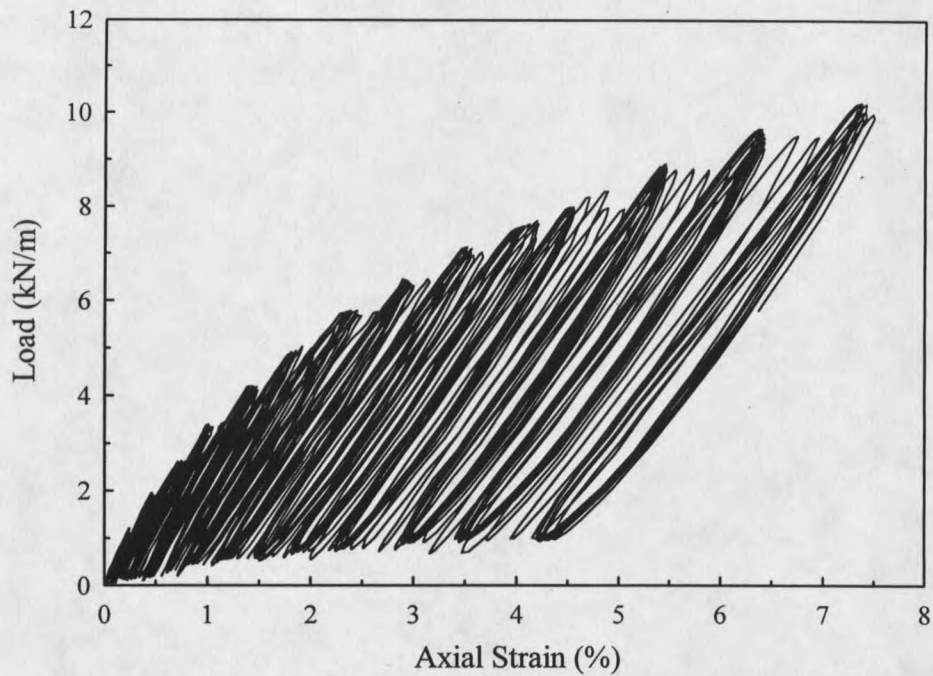


Figure B.15: Cyclic, Series II, unloading/reloading, load/strain curves for the geogrid pulled in the machine direction: ggmd-5.

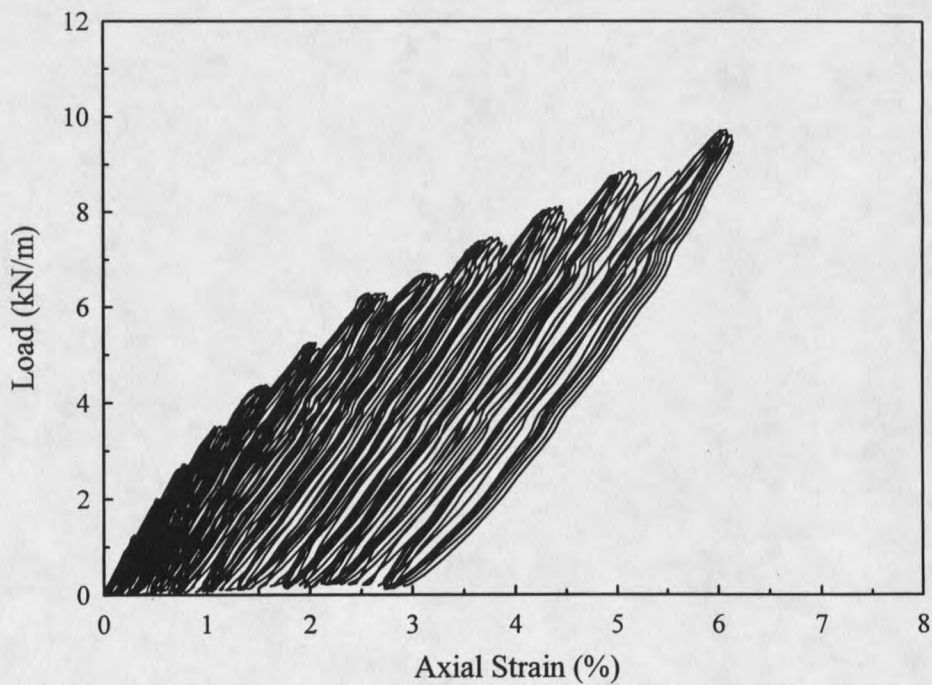


Figure B.16: Cyclic, Series II, unloading/reloading, load/strain curves for the geogrid pulled in the machine direction: ggmd-6.

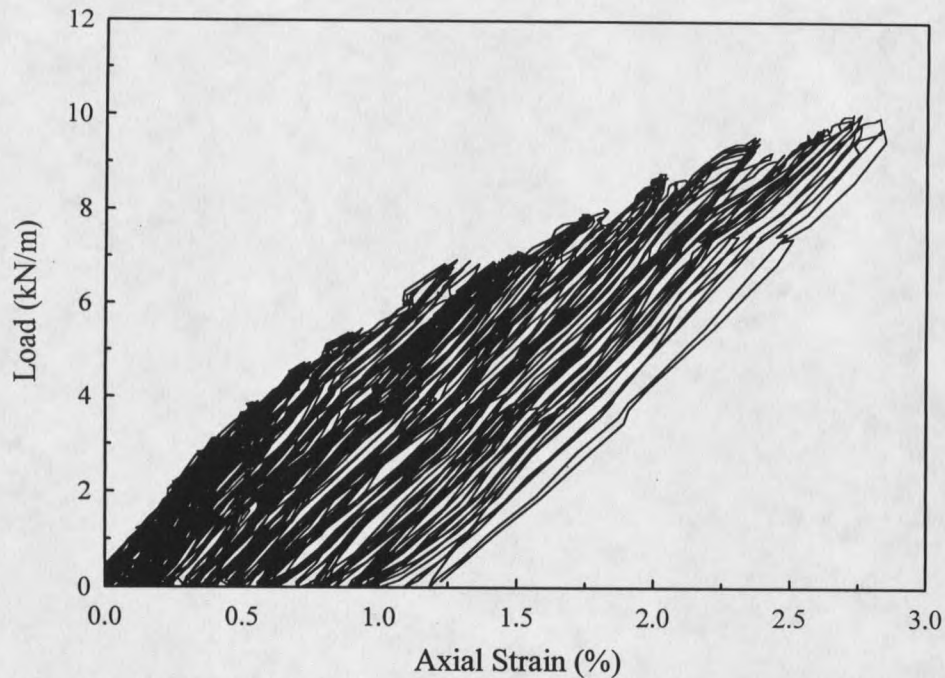


Figure B.17: Cyclic, Series II, unloading/reloading, load/strain curves for the geogrid pulled in the cross-machine direction: ggxd-3.

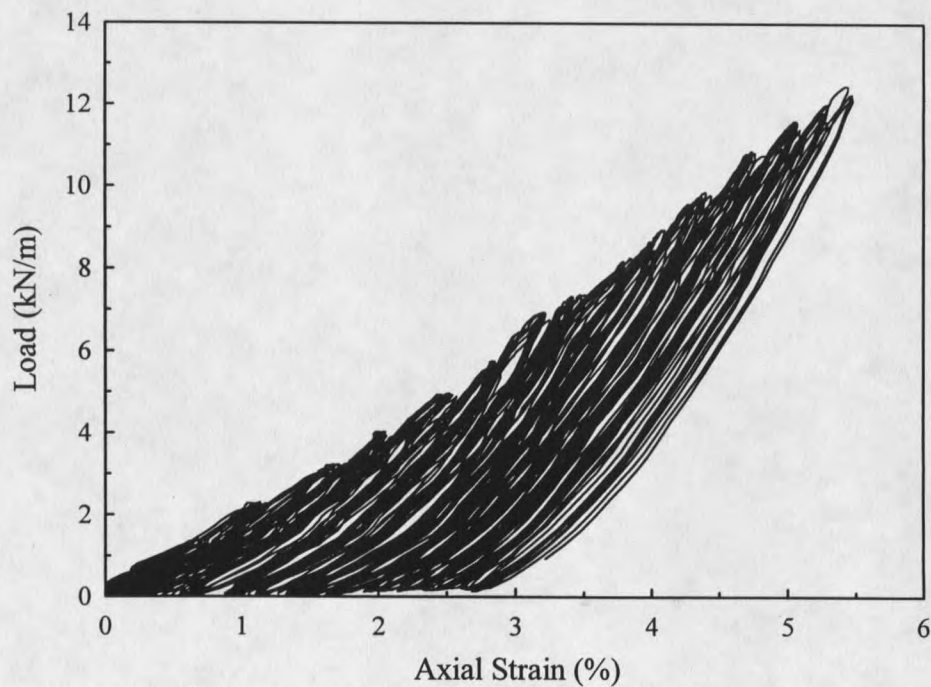


Figure B.18: Cyclic, Series II, unloading/reloading, load/strain curves for the geotextile pulled in the machine direction: gtmd-5.

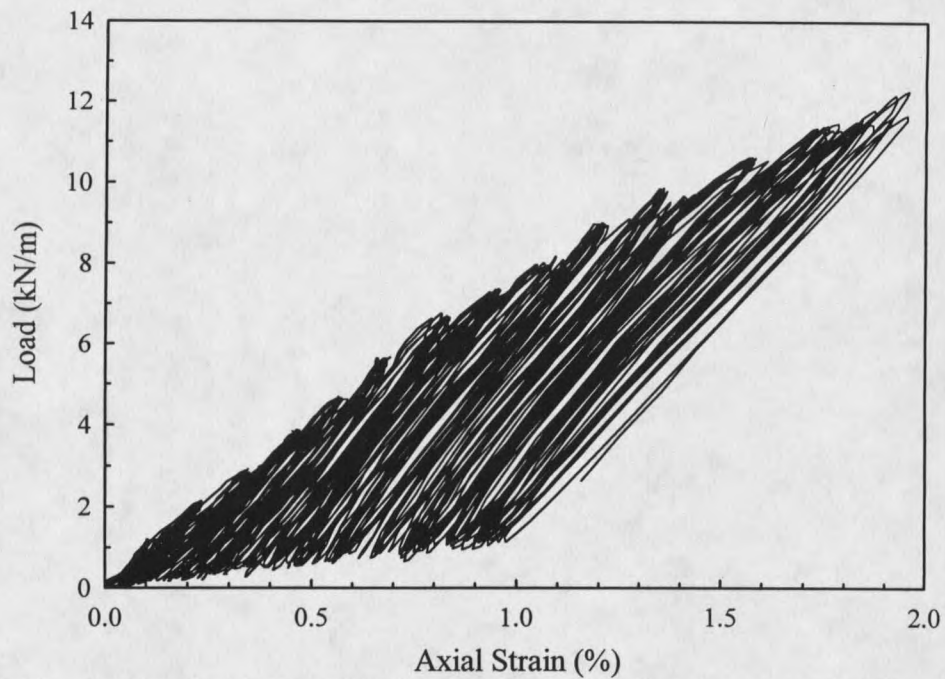


Figure B.19: Cyclic, Series II, unloading/reloading, load/strain curves for the geotextile pulled in the cross-machine direction: gtxd-4.

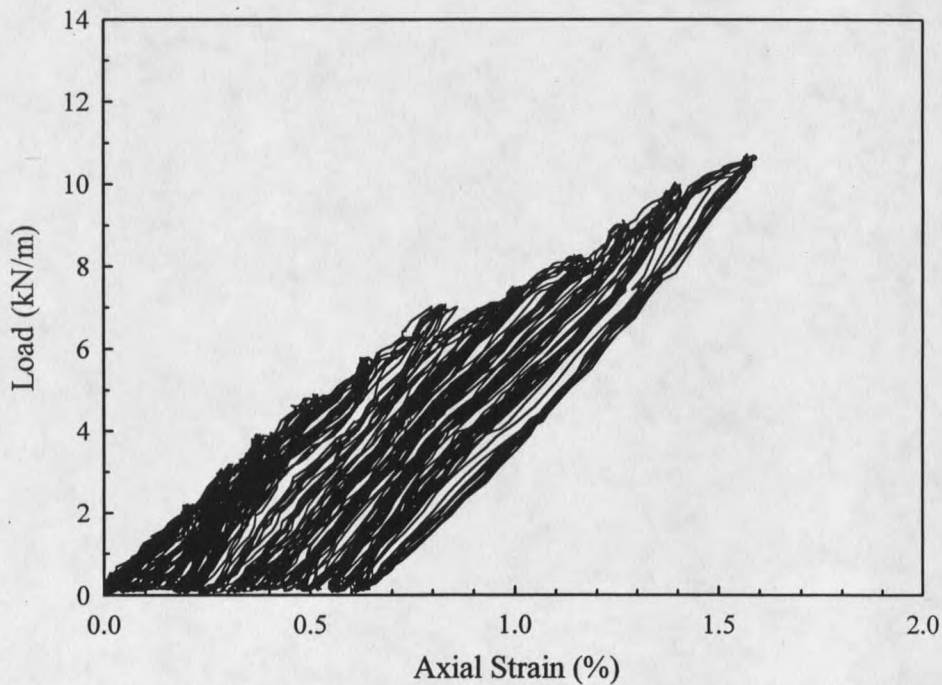


Figure B.20: Cyclic, Series II, unloading/reloading, load/strain curves for the geotextile pulled in the cross-machine direction: gtxd-5.

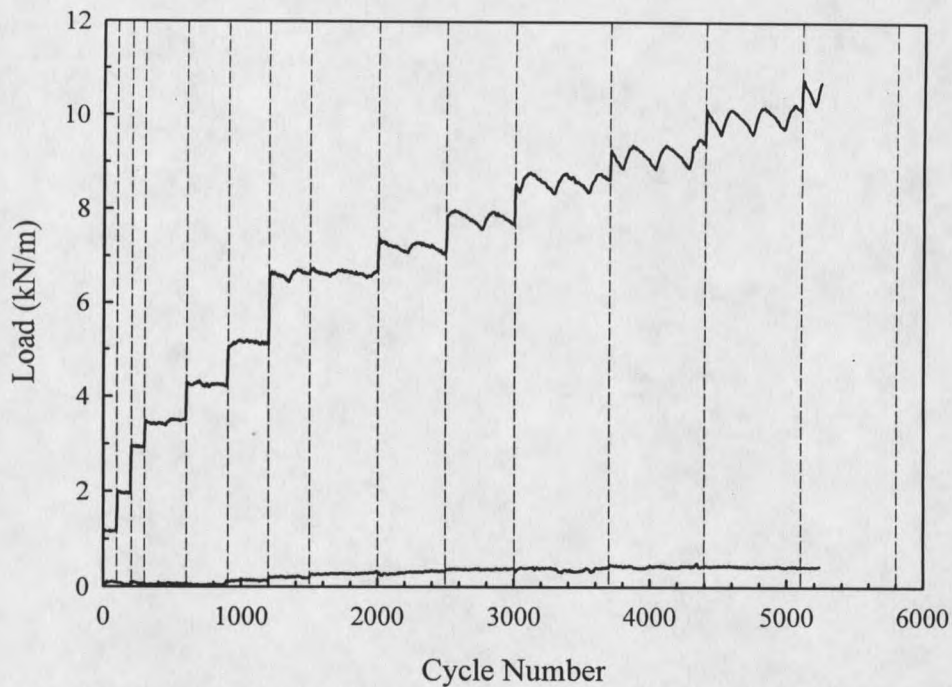


Figure B.21: Maximum and minimum loads applied to the geogrid in the machine direction for an in-air, cyclic, Series II test: ggmd-5.

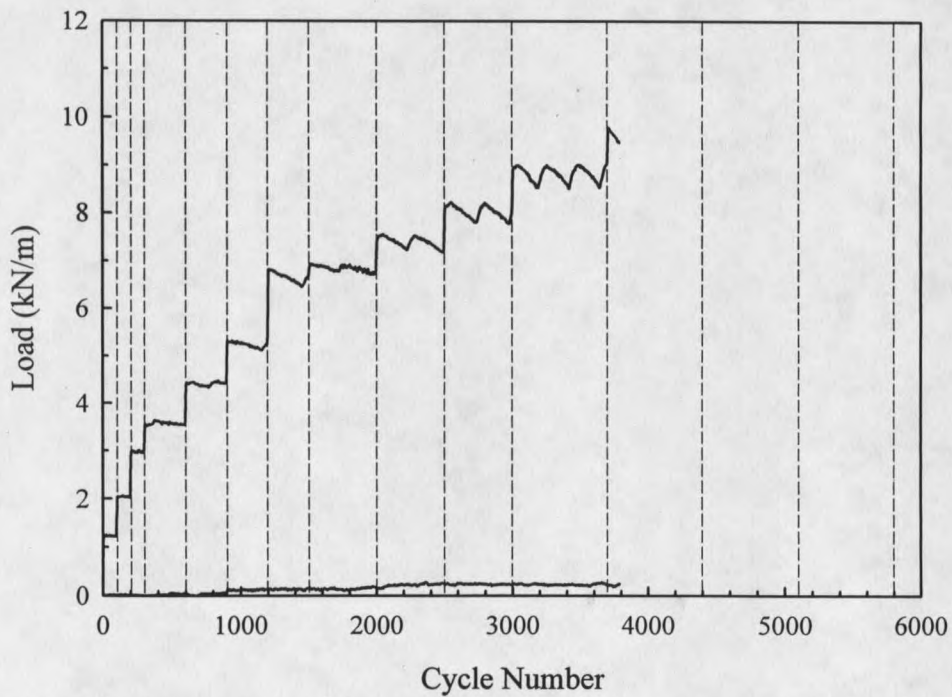


Figure B.22: Maximum and minimum loads applied to the geogrid in the machine direction for an in-air, cyclic, Series II test: ggmd-6.

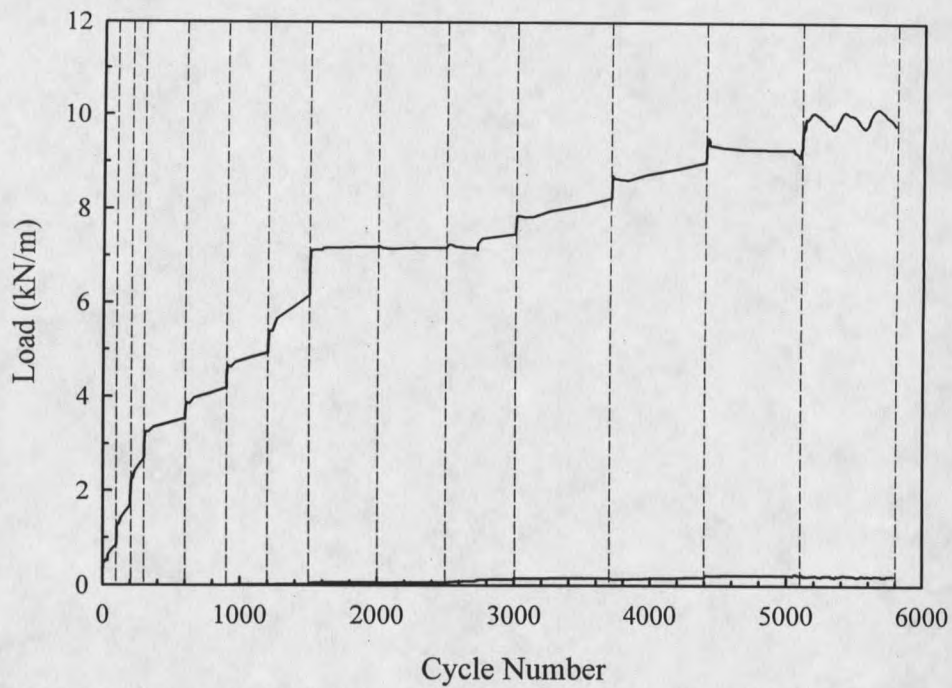


Figure B.23: Maximum and minimum loads applied to the geogrid in the cross-machine direction for an in-air, cyclic, Series II test: ggxd-3.

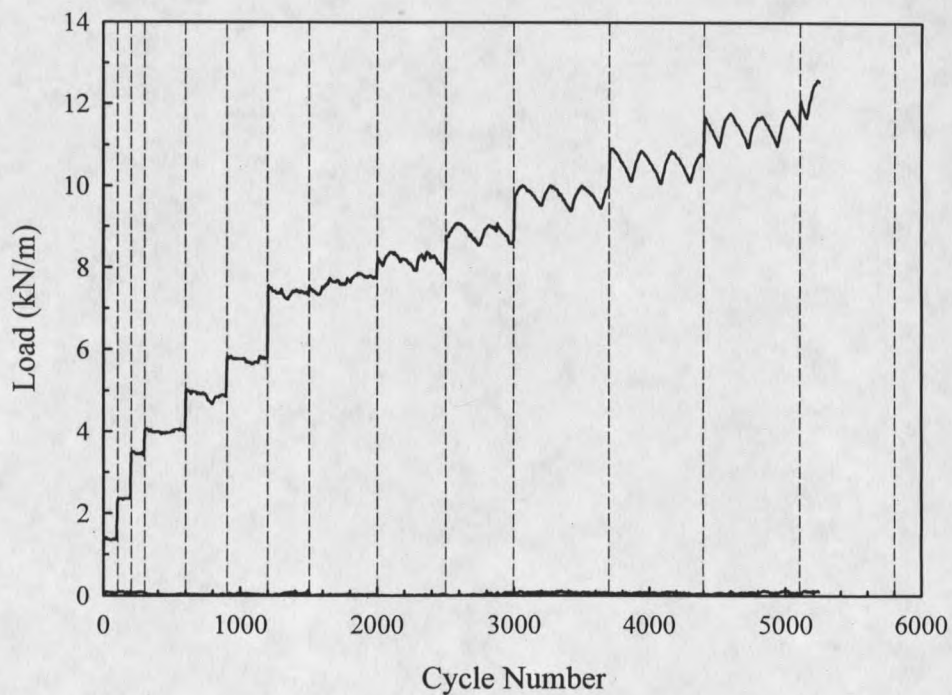


Figure B.24: Maximum and minimum loads applied to the geotextile in the machine direction for an in-air, cyclic, Series II test: ggmd-5.

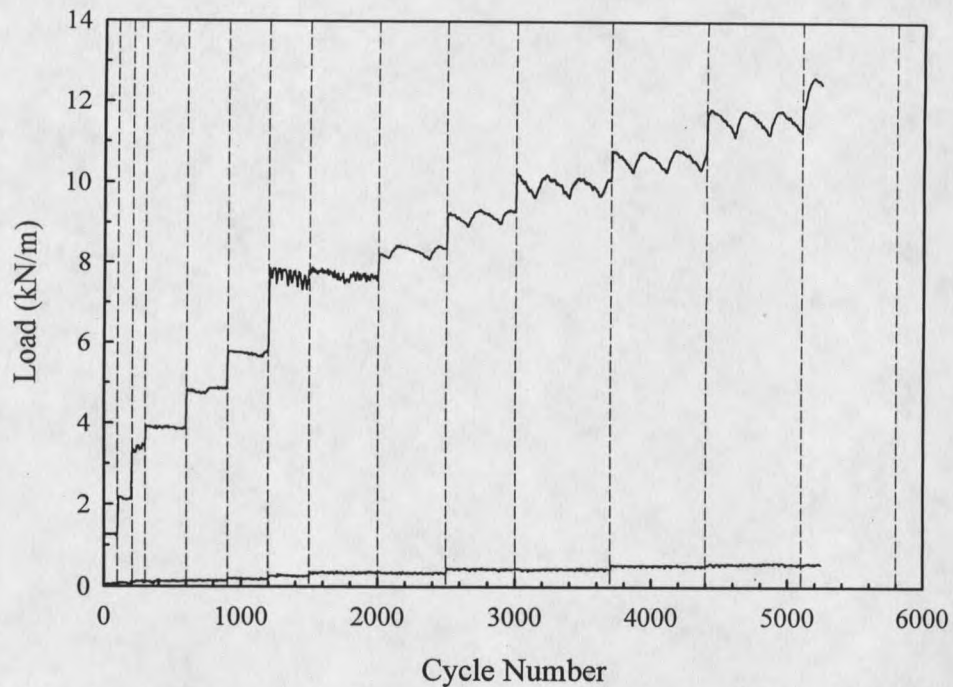


Figure B.25: Maximum and minimum loads applied to the geotextile in the cross-machine direction for an in-air, cyclic, Series II test: gtxd-4.

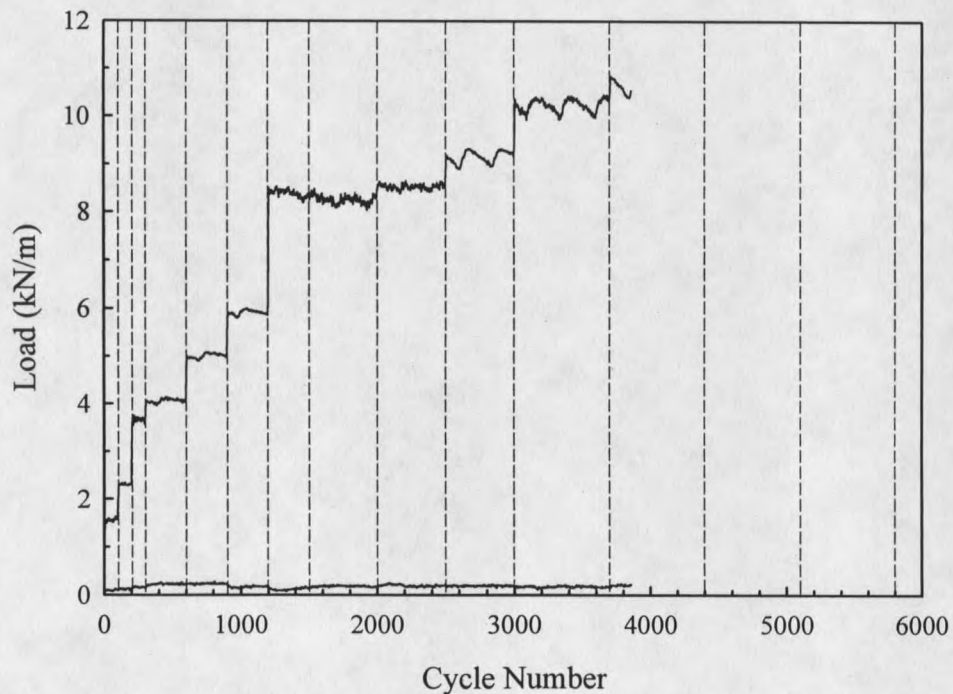


Figure B.26: Maximum and minimum loads applied to the geotextile in the cross-machine direction for an in-air, cyclic, Series II test: gtxd-5.

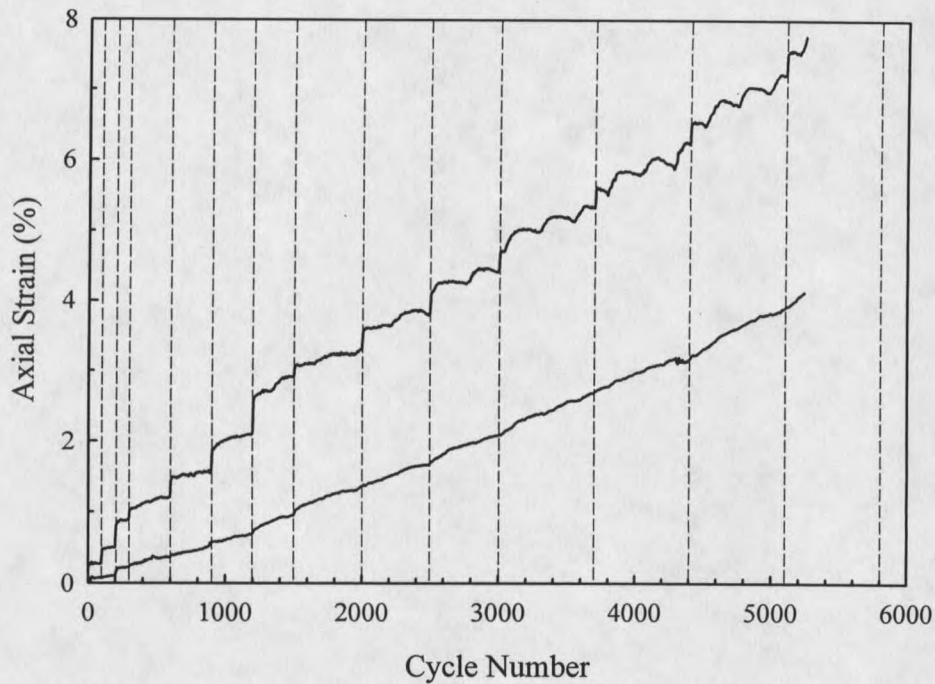


Figure B.27: Maximum and minimum axial strains of the geogrid pulled in the machine direction for an in-air, cyclic, Series II test: ggmd-5.

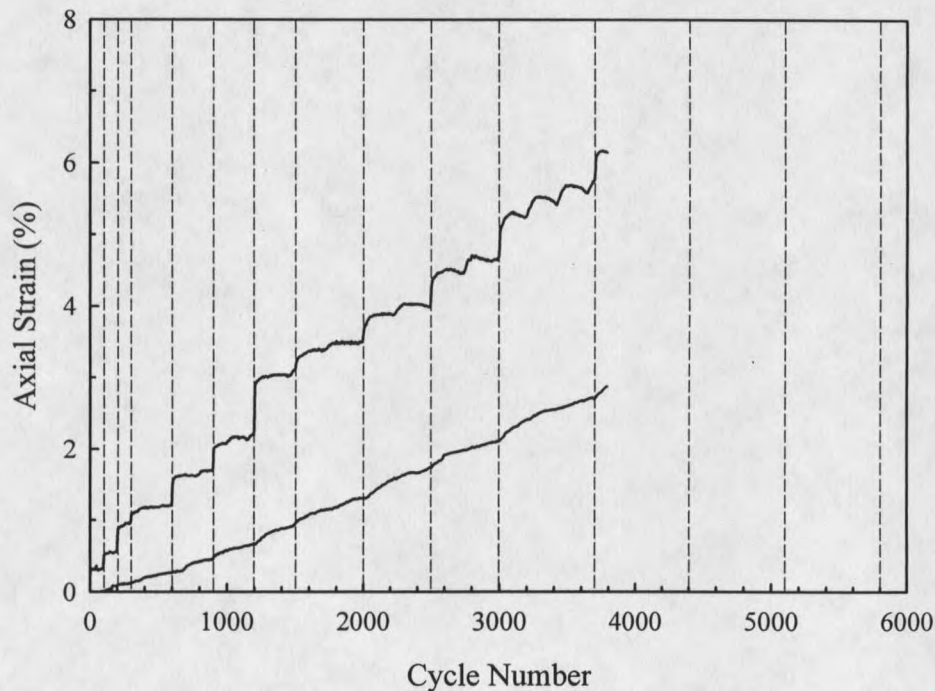


Figure B.28: Maximum and minimum axial strains of the geogrid pulled in the machine direction for an in-air, cyclic, Series II test: ggmd-6.

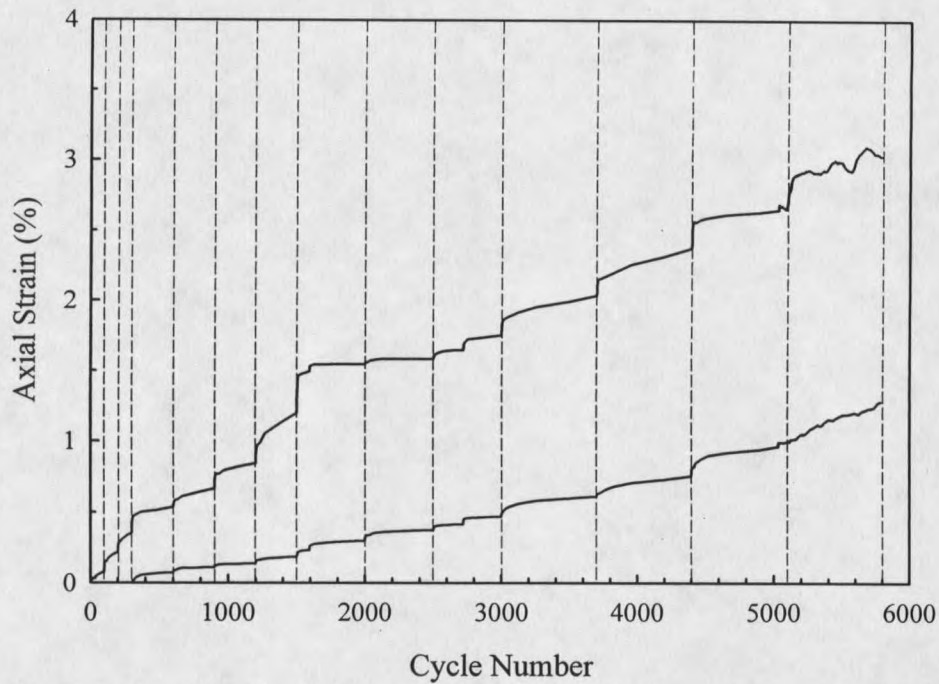


Figure B.29: Maximum and minimum axial strains of the geogrid pulled in the cross-machine direction for an in-air, cyclic, Series II test: ggxd-3.

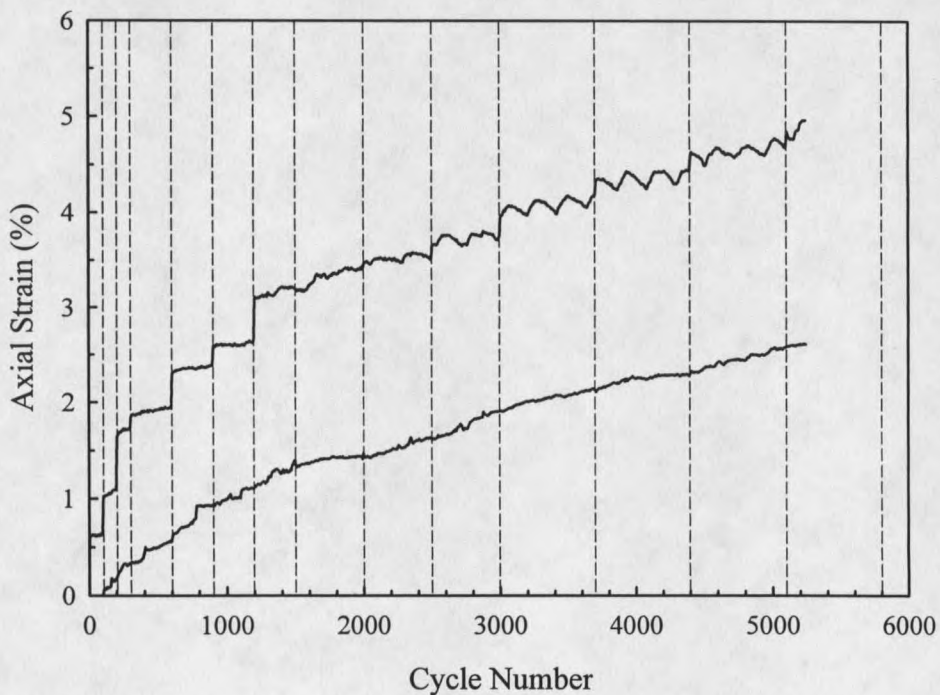


Figure B.30: Maximum and minimum axial strains of the geotextile pulled in the machine direction for an in-air, cyclic, Series II test: gtmd-5.

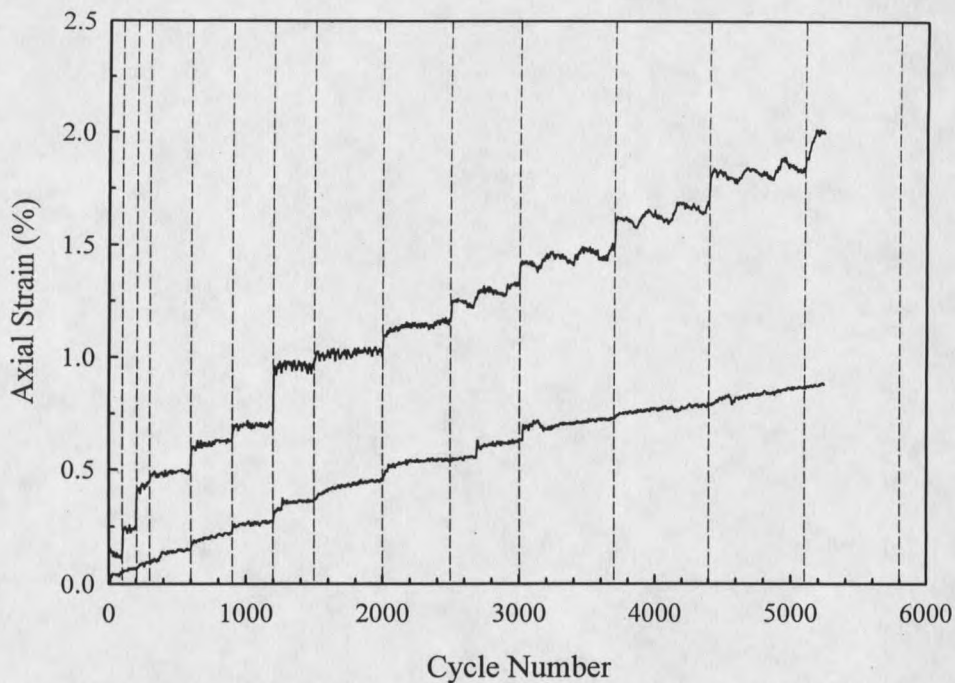


Figure B.31: Maximum and minimum axial strains of the geotextile pulled in the cross-machine direction for an in-air, cyclic, Series II test: gtxd-4.

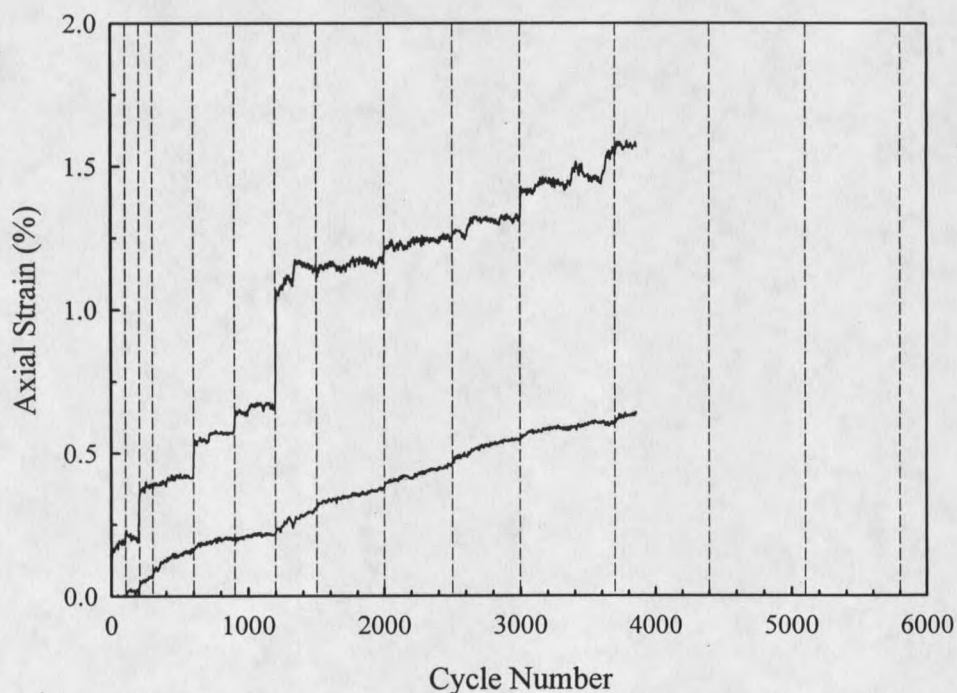


Figure B.32: Maximum and minimum axial strains of the geotextile pulled in the cross-machine direction for an in-air, cyclic, Series II test: gtxd-5.

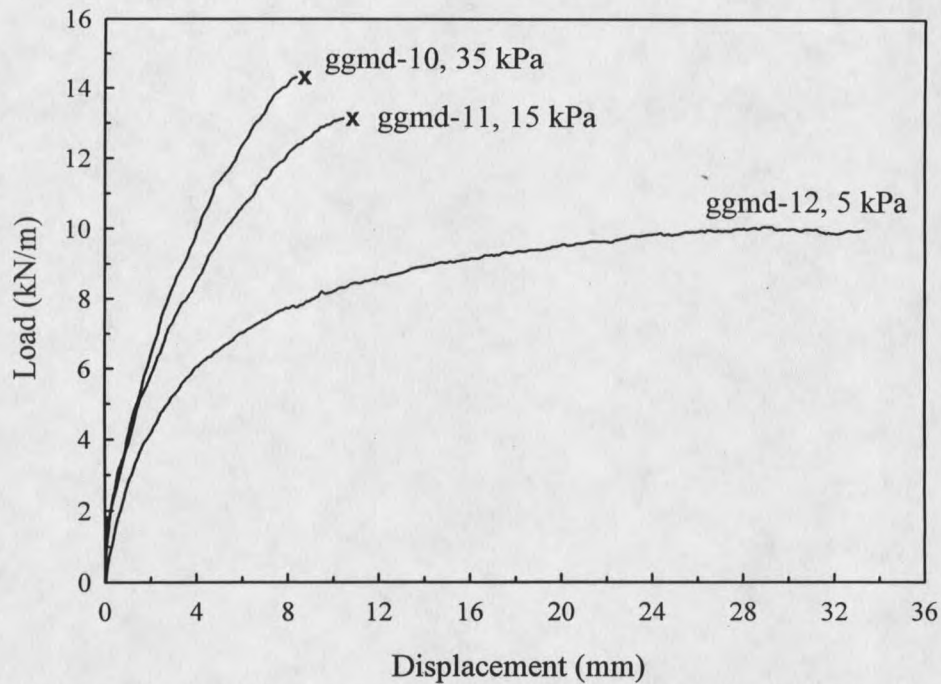


Figure B.33: In-soil, load/displacement curves for the geogrid pulled in the machine direction for three levels of normal confinement.

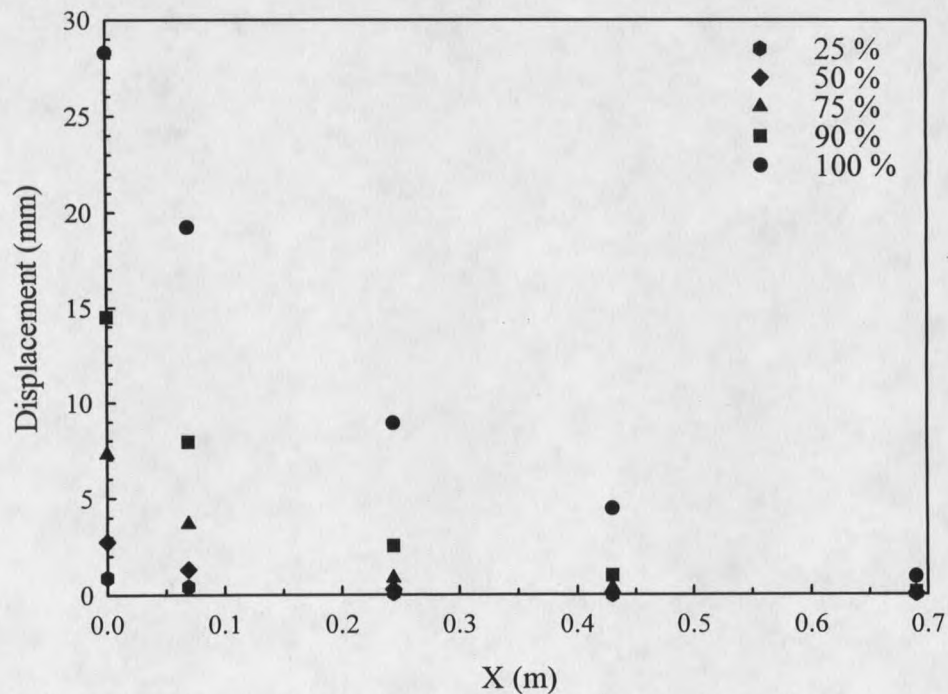


Figure B.34: Displacement along the length of the geogrid pulled in the machine direction at 5 kPa normal confinement: ggmd-12.

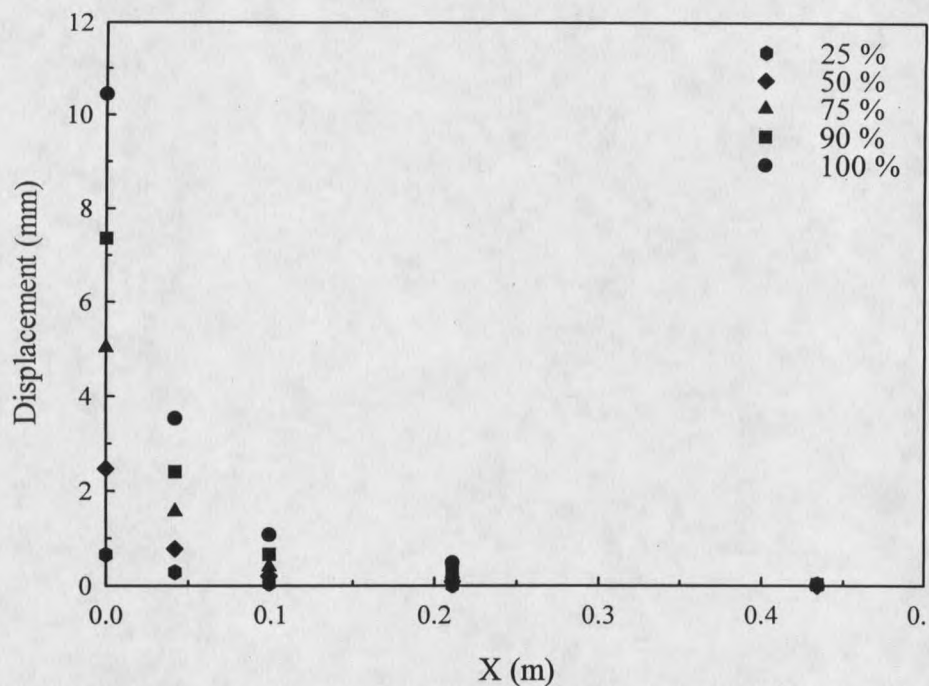


Figure B.35: Displacement along the length of the geogrid pulled in the machine direction for 15 kPa normal confinement: ggmd-11.

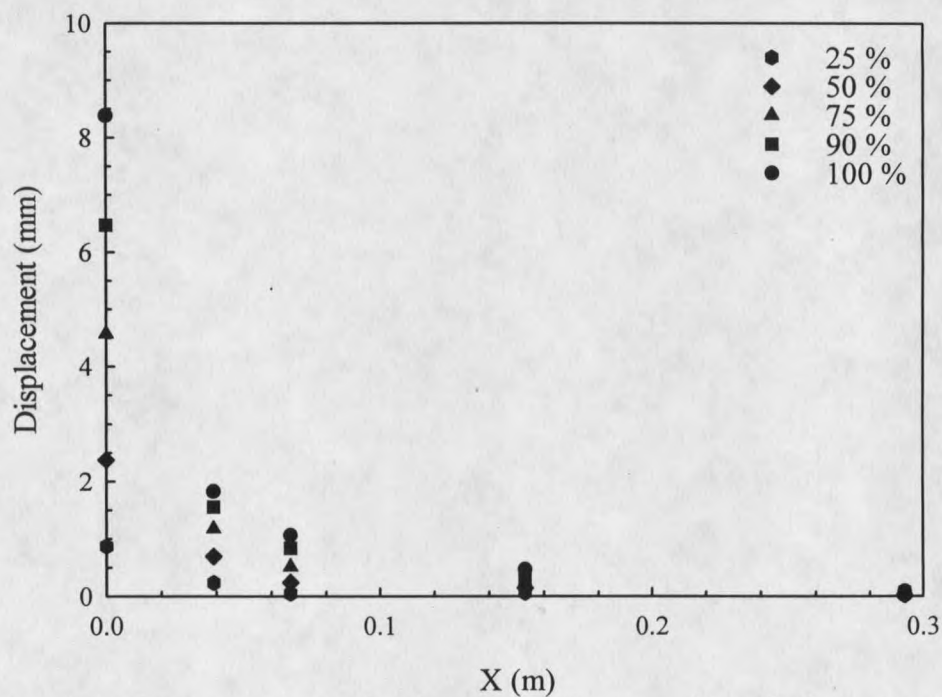


Figure B.36: Displacement along the length of the geogrid pulled in the machine direction for 35 kPa normal confinement: ggmd-10.

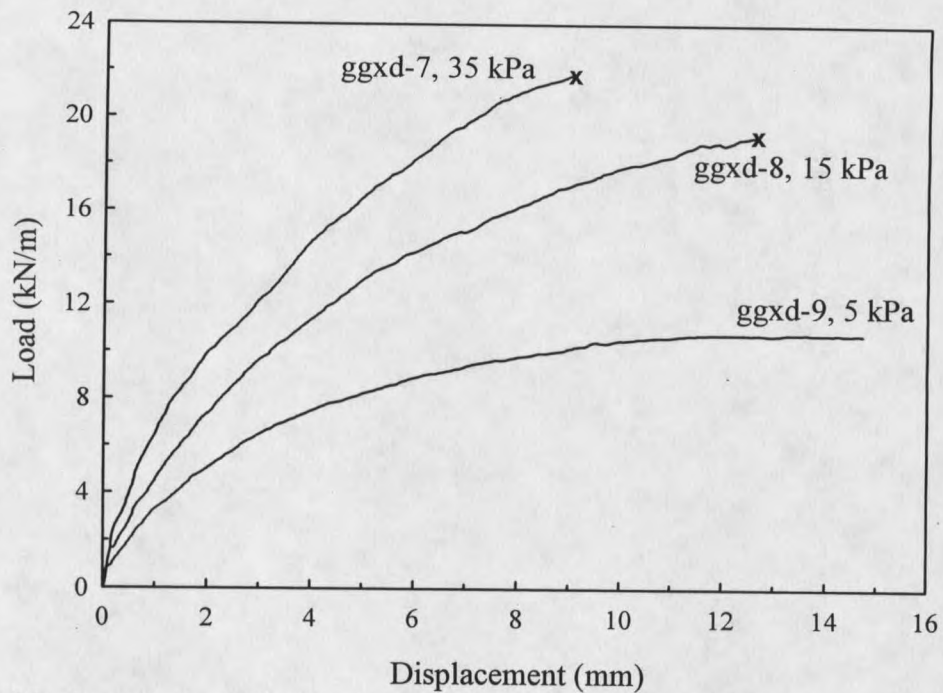


Figure B.37: In-soil, load/displacement curves for the geogrid pulled in the cross-machine direction for three levels of normal confinement.

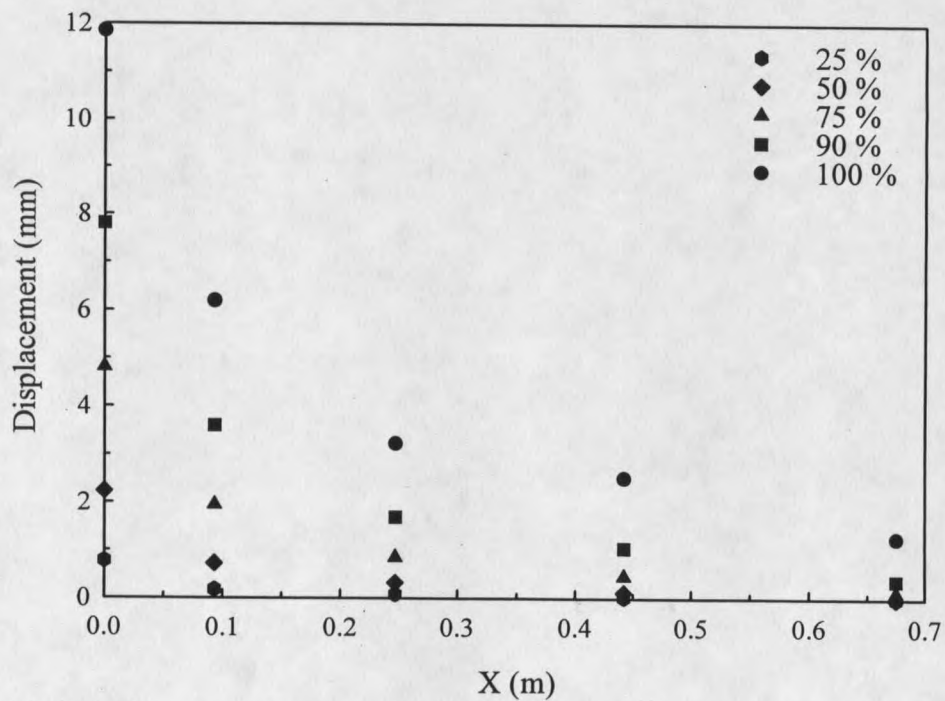


Figure B.38: Displacement along the length of the geogrid pulled in the cross-machine direction for 5 kPa normal confinement: ggxd-9.

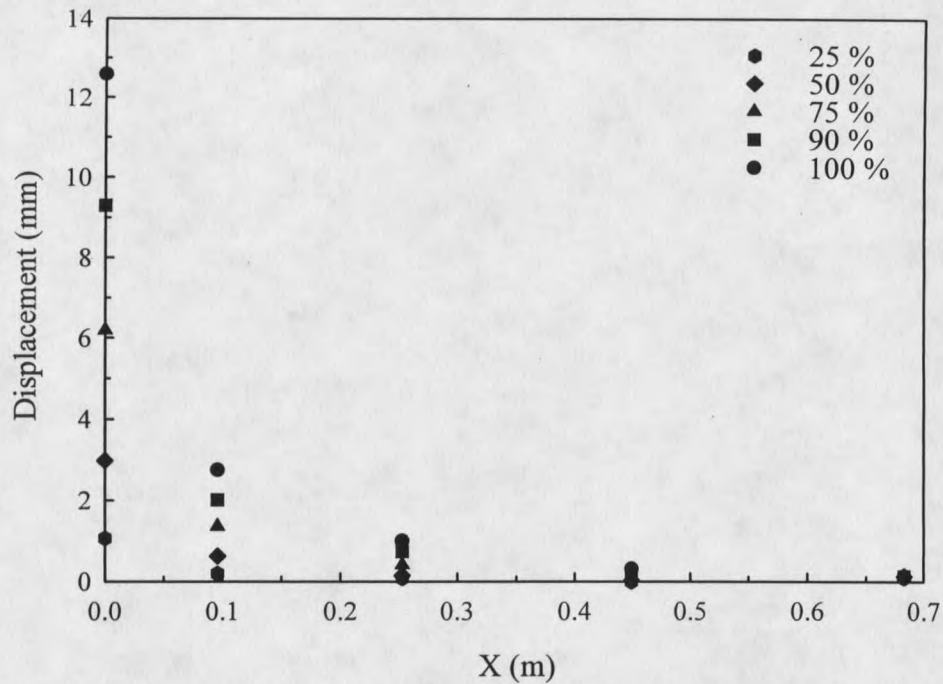


Figure B.39: Displacement along the length of the geogrid pulled in the cross-machine direction for 15 kPa normal confinement: ggxd-8.

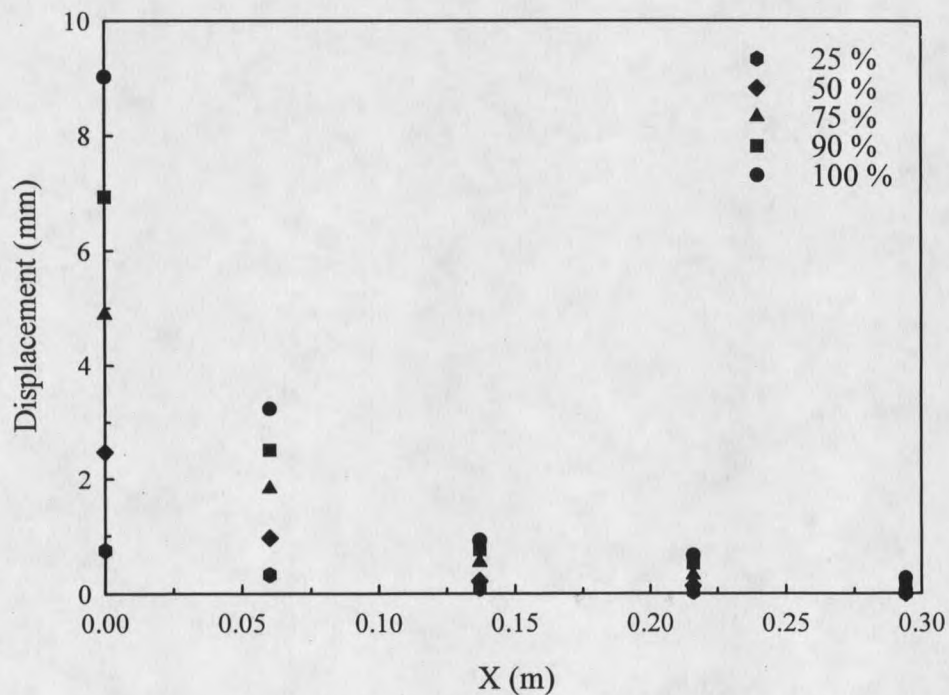


Figure B.40: Displacement along the length of the geogrid pulled in the cross-machine direction for 35 kPa normal confinement: ggxd-7.

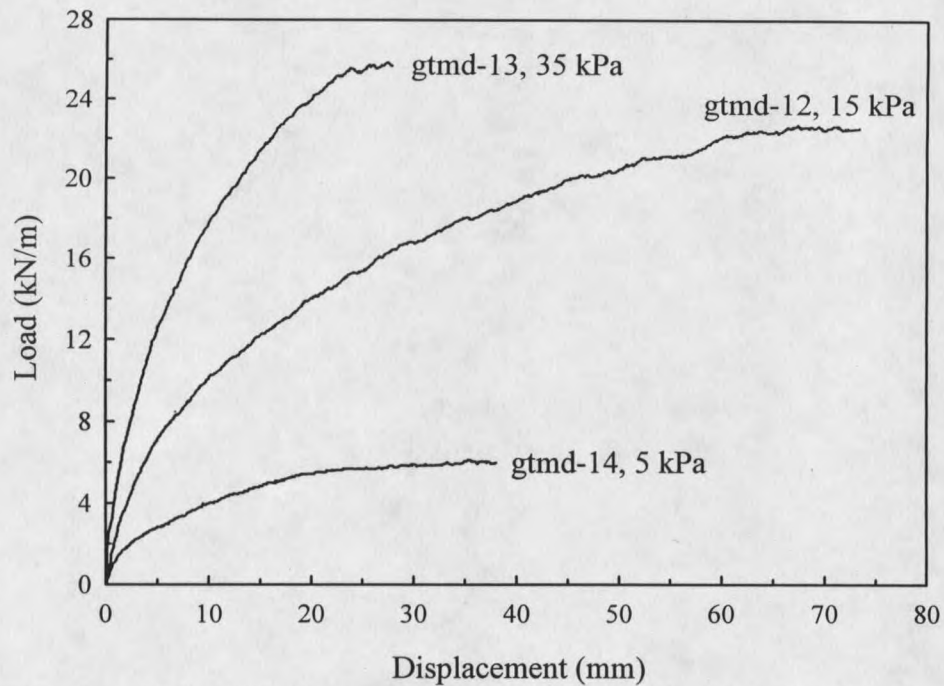


Figure B.41: In-soil, load/displacement curves for the geotextile pulled in the machine direction for three levels of normal confinement.

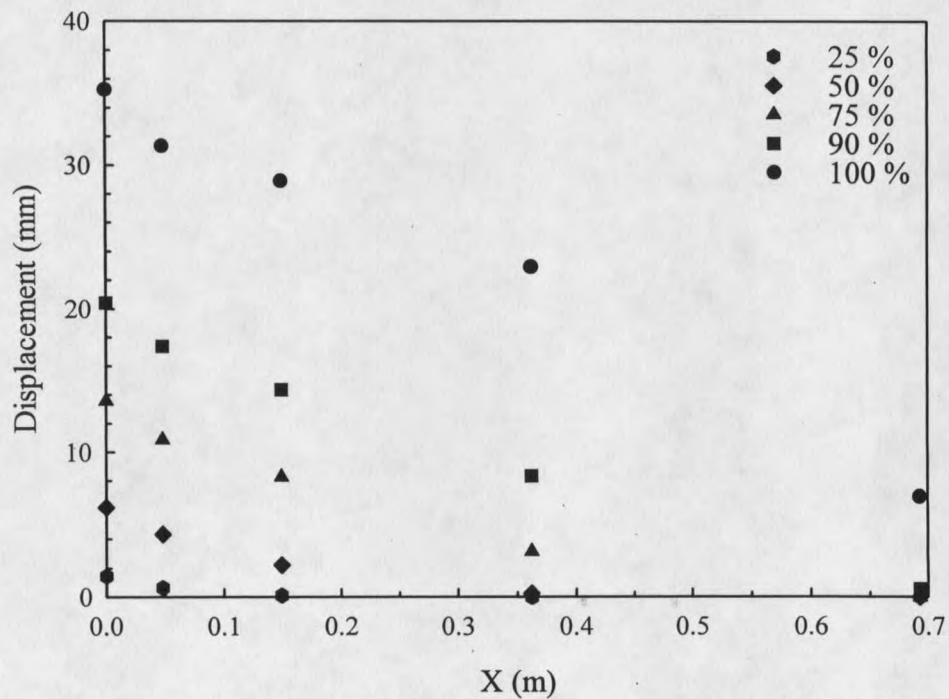


Figure B.42: Displacement along the length of the geotextile pulled in the machine direction for 5 kPa normal confinement: gtmd-14.

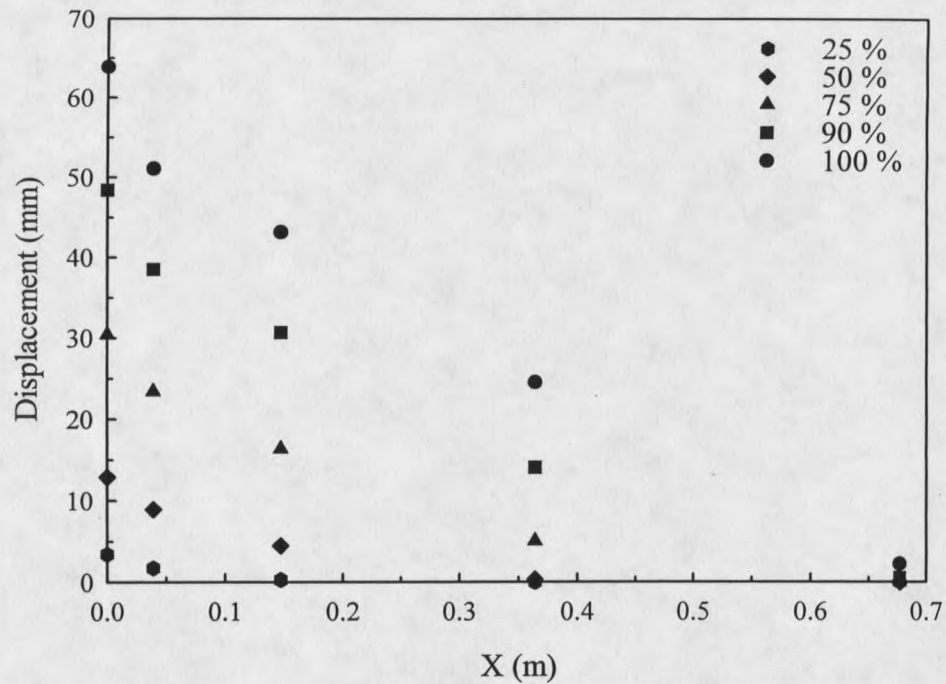


Figure B.43: Displacement along the length of the geotextile pulled in the machine direction for 15 kPa normal confinement: gtmd-12.

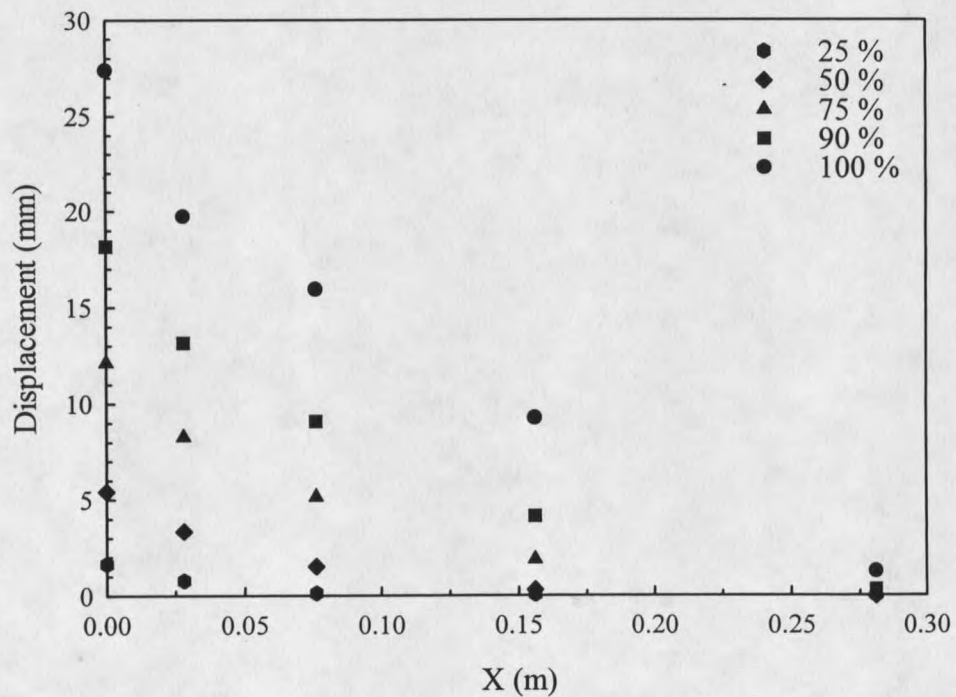


Figure B.44: Displacement along the length of the geotextile pulled in the machine direction for 35 kPa normal confinement: gtmd-13.

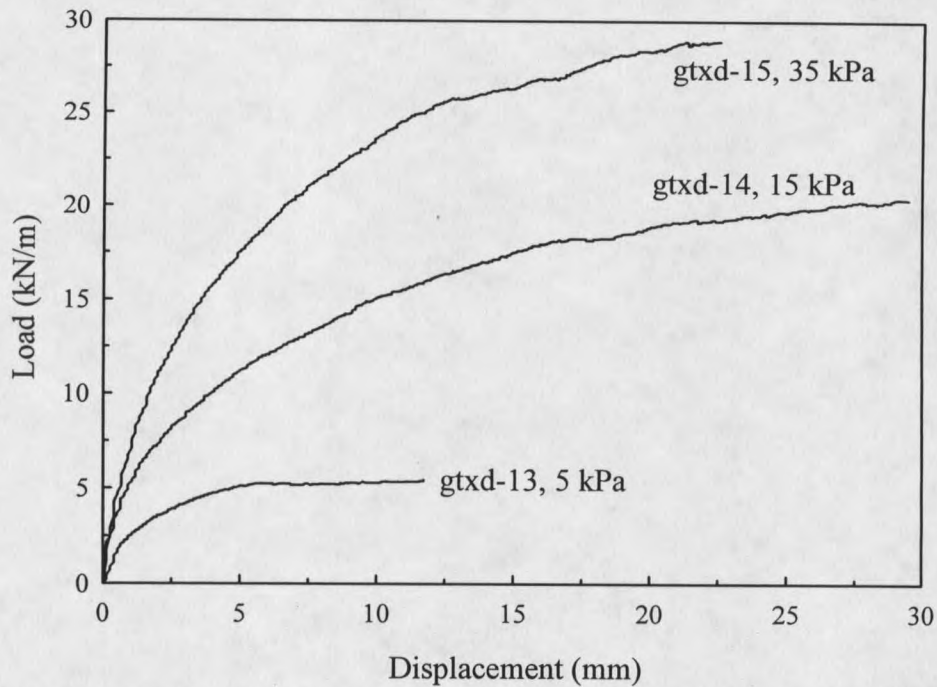


Figure B.45: In-soil load/displacement curves for the geotextile pulled in the cross-machine direction for three levels of normal confinement.

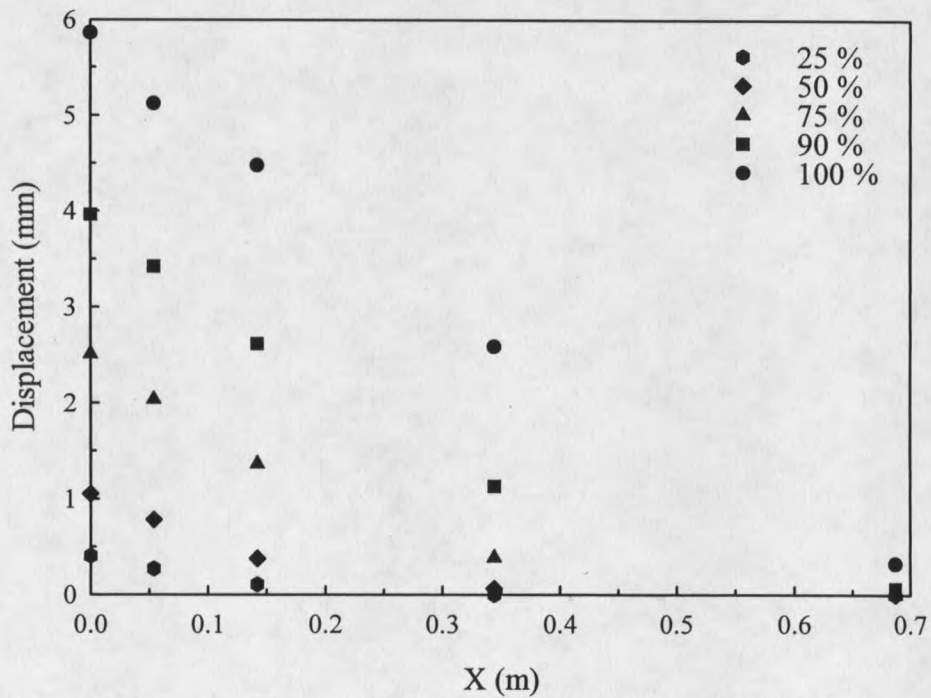


Figure B.46: Displacement along the length of the geotextile pulled in the cross-machine direction for 5 kPa normal confinement: gtxd-13.

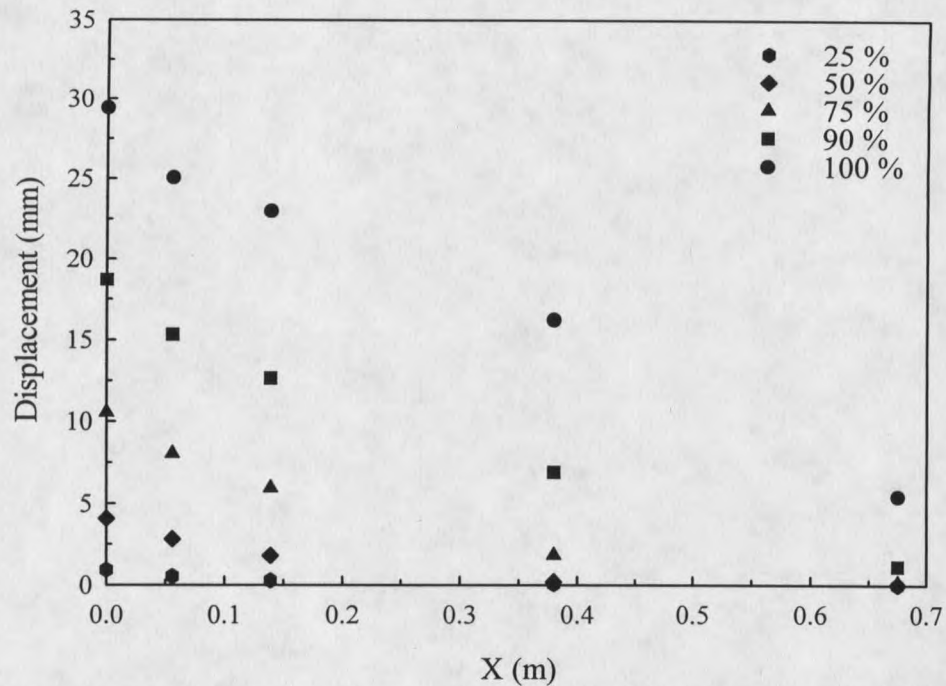


Figure B.47: Displacement along the length of the geotextile pulled in the cross-machine direction for 15 kPa normal confinement: gtxd-14.

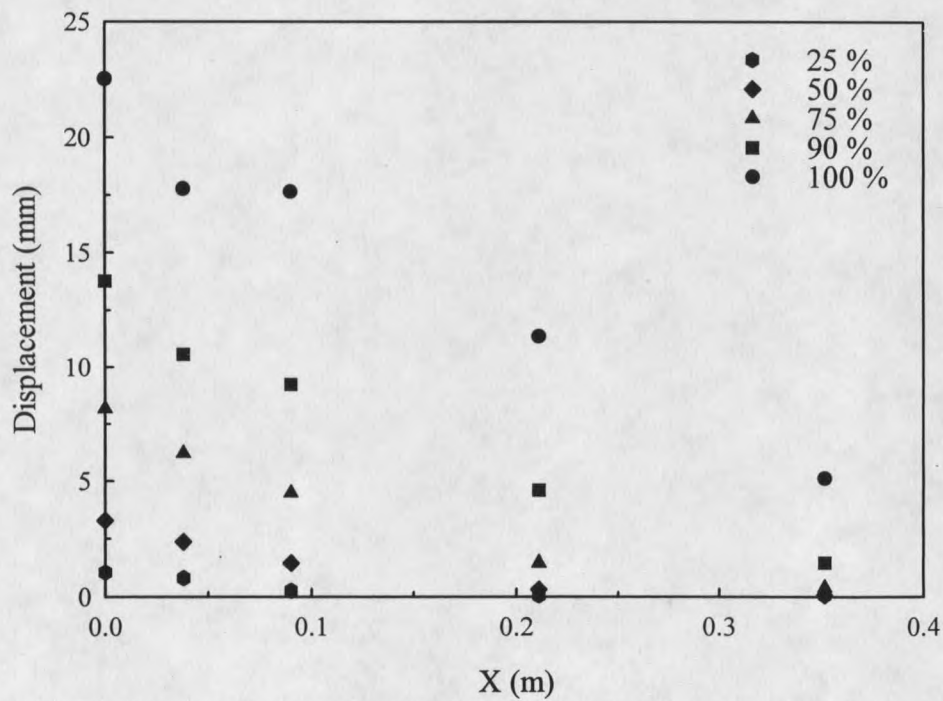


Figure B.48: Displacement along the length of the geotextile pulled in the cross-machine direction for 35 kPa normal confinement: gtxd-15.

Load Level	Cycle Number												Total Cycles Applied
	1	2	4	8	16	32	64	100	150	200			
2													200
3	201	202	204	208	216	232	264	300	350	400			200
4	401	402	404	408	416	432	464	500	550	600			200
5	601	602	604	608	616	632	664	728	856	1200			600
6	1201	1202	1204	1208	1216	1232	1264	1328	1456	1800			600
7	1801	1802	1804	1808	1816	1832	1864	1928	2056	2400			600
8	2401	2402	2404	2408	2416	2432	2464	2528	2656	3000			600
9	3001	3002	3004	3008	3016	3032	3064	3128	3256	3512	4000		1000
10	4001	4002	4004	4008	4016	4032	4064	4128	4256	4512	5000		1000
11	5001	5002	5004	5008	5016	5032	5064	5128	5256	5512	6000		1000
12	6001	6002	6004	6008	6016	6032	6064	6128	6256	6512	7000	7400	1400
13	7401	7402	7404	7408	7416	7432	7464	7528	7656	7912	8400	8800	1400
14	8801	8802	8804	8808	8816	8832	8864	8928	9056	9312	9800	10,200	1400
15	10,201	10,202	10,204	10,208	10,216	10,232	10,264	10,328	10,456	10,712	11,200	11,600	1400

Table B.2: Cycle numbers collected and plotted for the in-soil, cyclic, Series II tests.

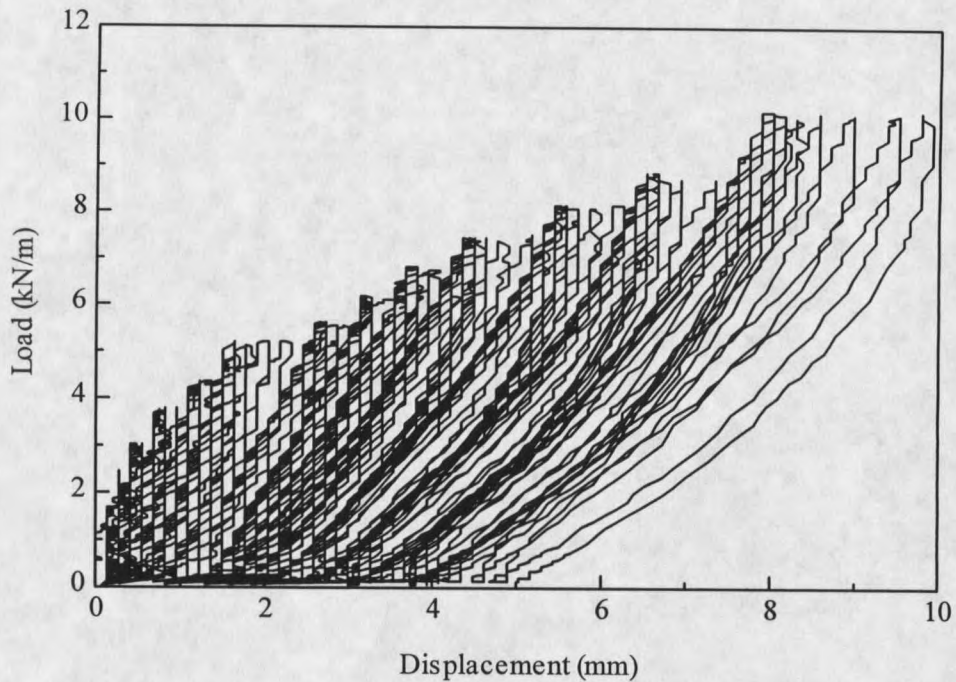


Figure B.49: Cyclic, in-soil, unloading/reloading load/displacement curves for the geogrid pulled in the machine direction at 15 kPa normal confinement: ggmd-13.

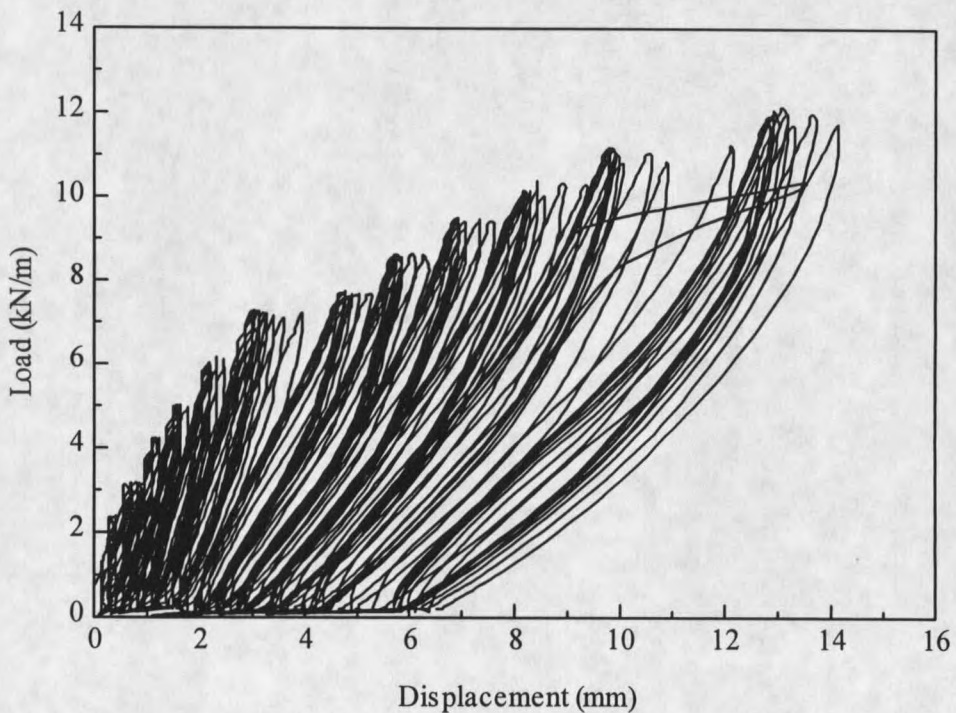


Figure B.50: Cyclic, in-soil, unloading/reloading load/displacement curves for the geogrid pulled in the cross-machine direction at 15 kPa normal confinement: ggxd-10.

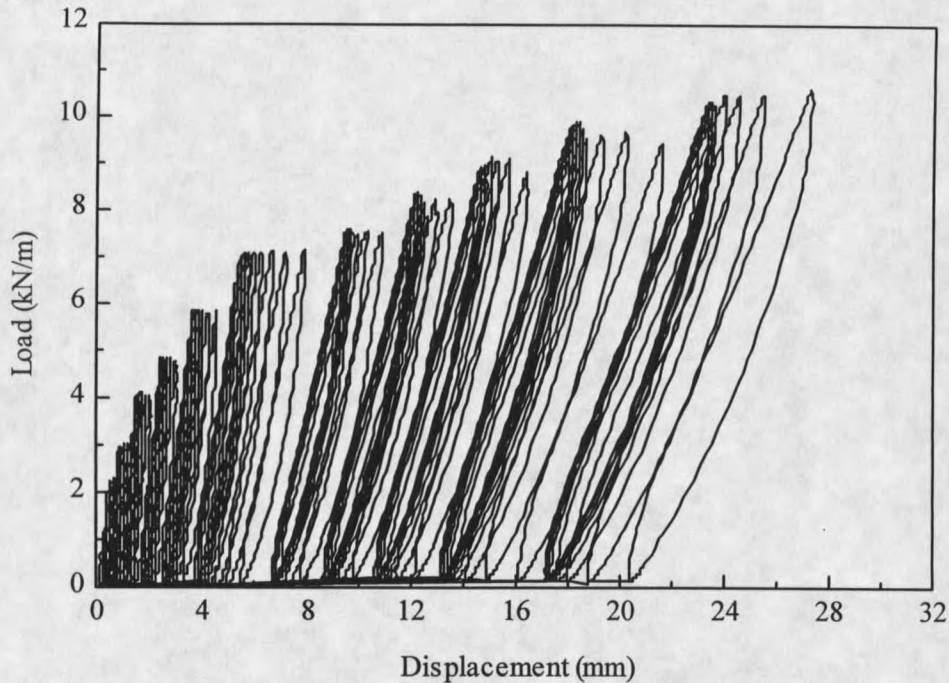


Figure B.51: Cyclic, in-soil, unloading/reloading load/displacement curves for the geotextile pulled in the machine direction at 15 kPa normal confinement: gtmd-15.

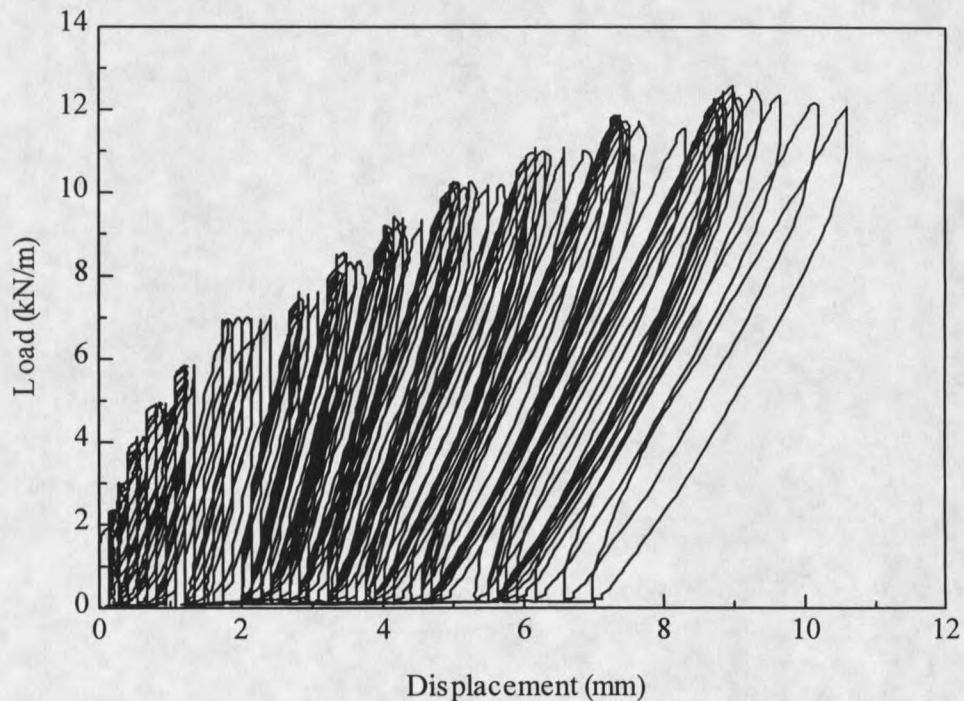


Figure B.52: Cyclic, in-soil, unloading/reloading load/displacement curves for the geotextile pulled in the cross-machine direction at 15 kPa normal confinement: gtxd-16.

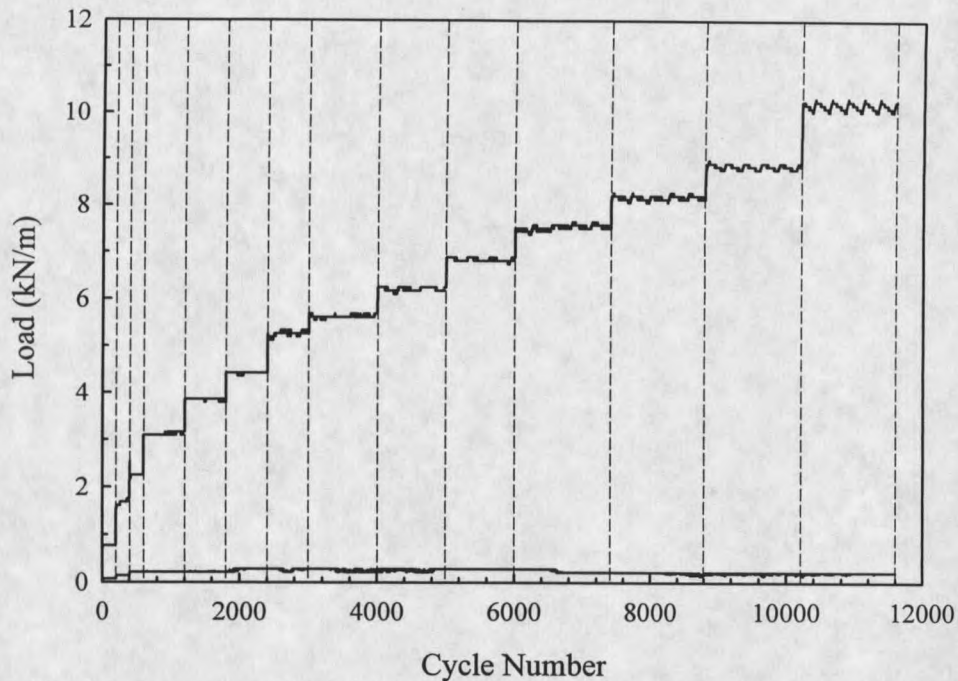


Figure B.53: Maximum and minimum loads applied to the geogrid in the machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: ggmd-13.

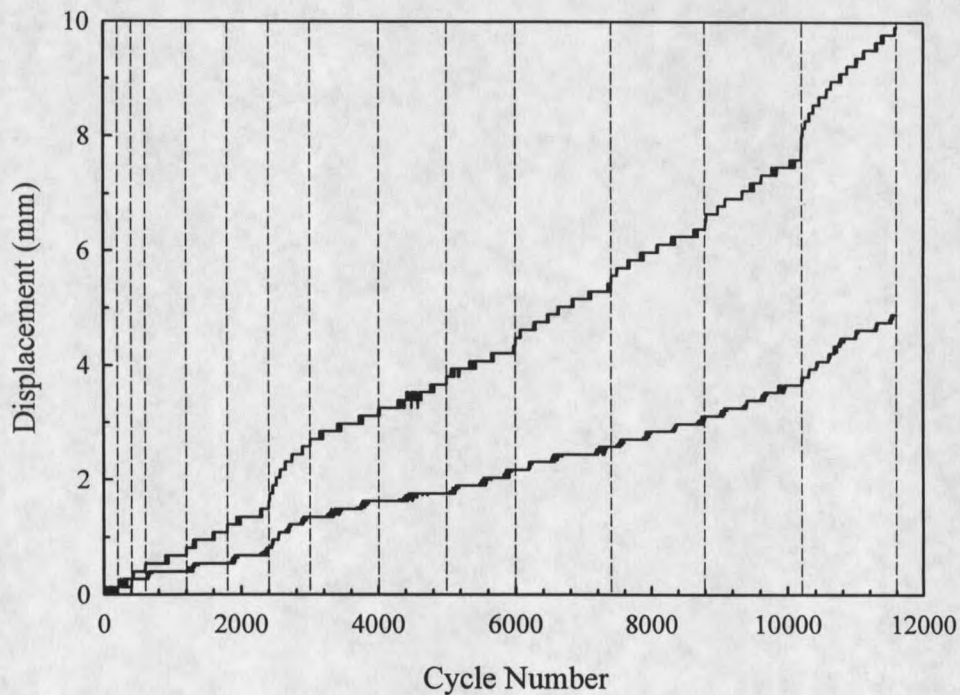


Figure B.54: Maximum and minimum displacements at the front of a sample of geogrid pulled in the machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: ggmd-13.

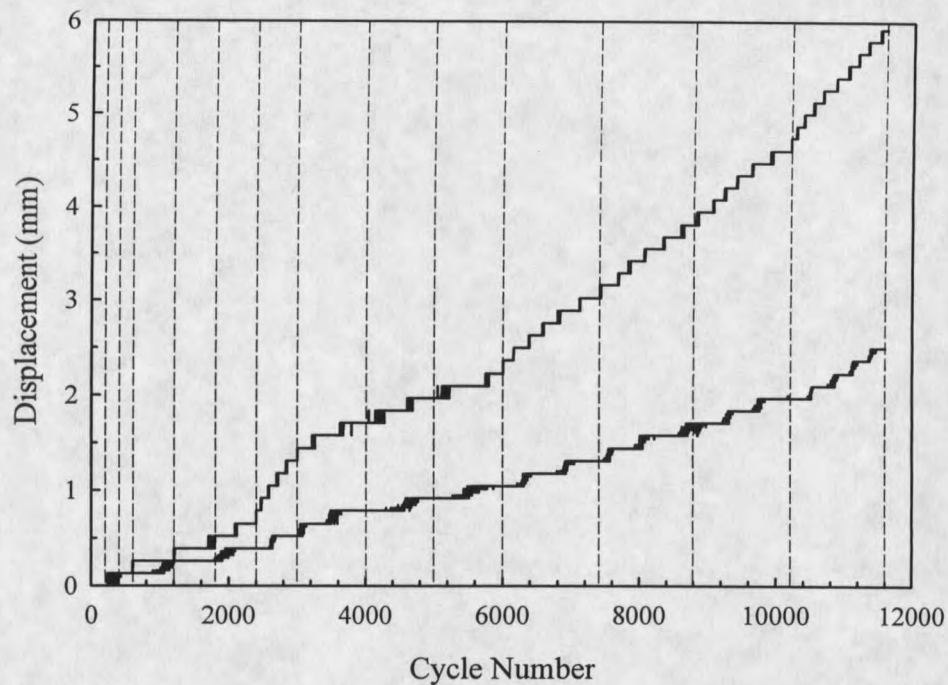


Figure B.55: Maximum and minimum displacements at position two along the length of a sample of geogrid pulled in the machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: ggmd-13.

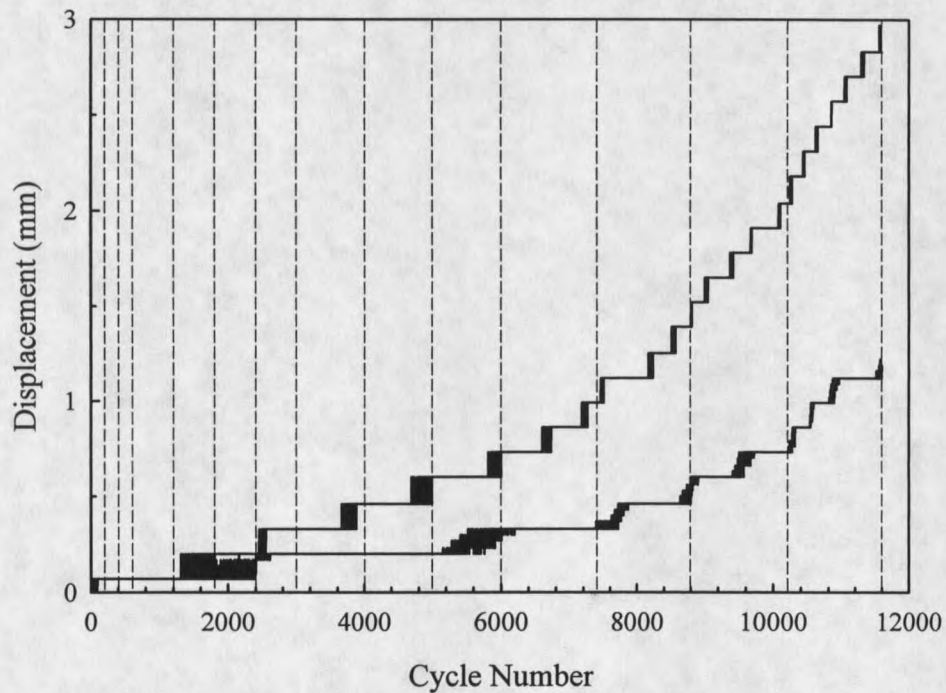


Figure B.56: Maximum and minimum displacements at position three along the length of a sample of geogrid pulled in the machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: ggmd-13.

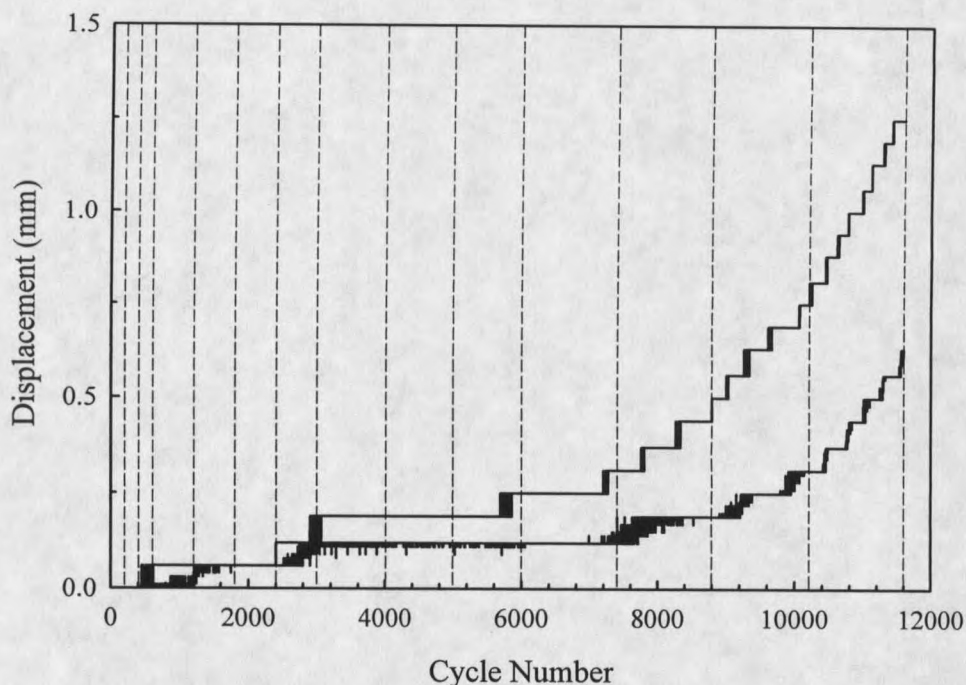


Figure B.57: Maximum and minimum displacements at position four along the length of a sample of geogrid pulled in the machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: ggmd-13.

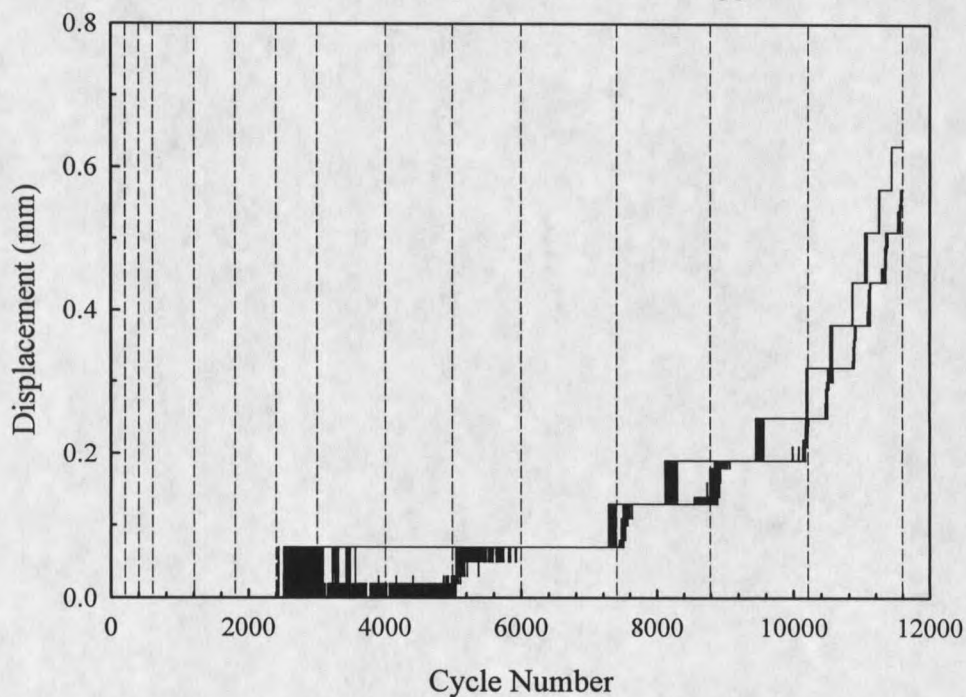


Figure B.58: Maximum and minimum displacements at position five along the length of a sample of geogrid pulled in the machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: ggmd-13.

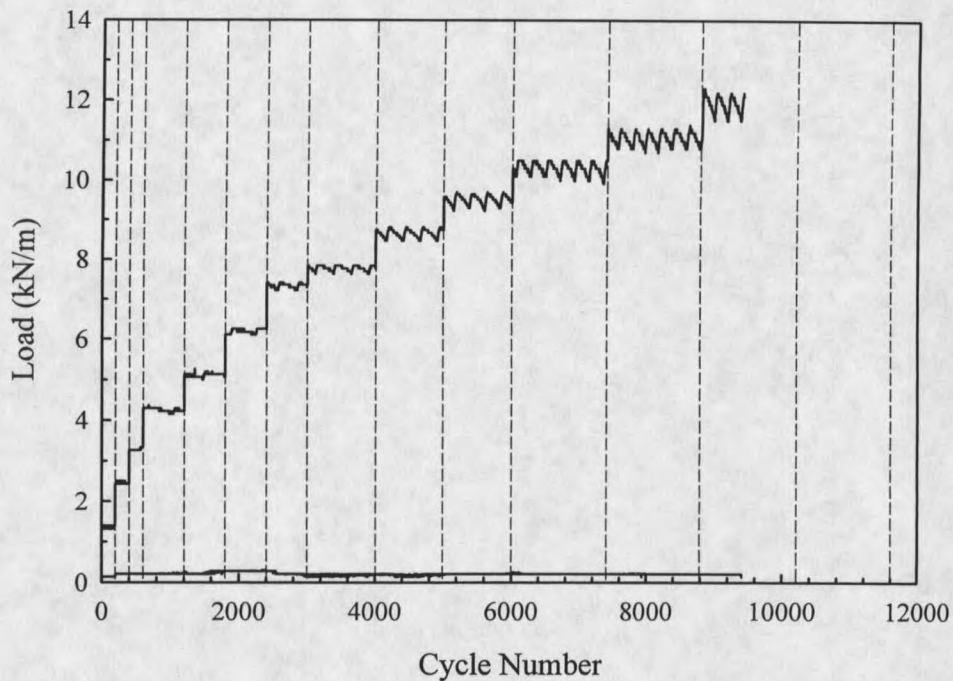


Figure B.59: Maximum and minimum loads applied to the geogrid in the cross-machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: ggxd-10.

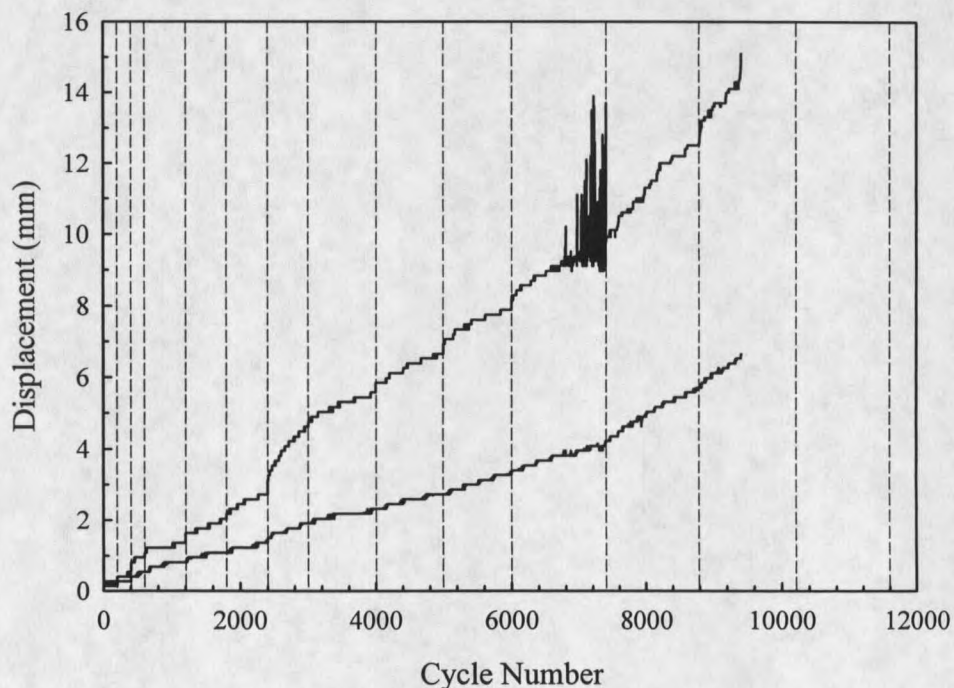


Figure B.60: Maximum and minimum displacements at the front of a sample of geogrid pulled in the cross-machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: ggxd-10.

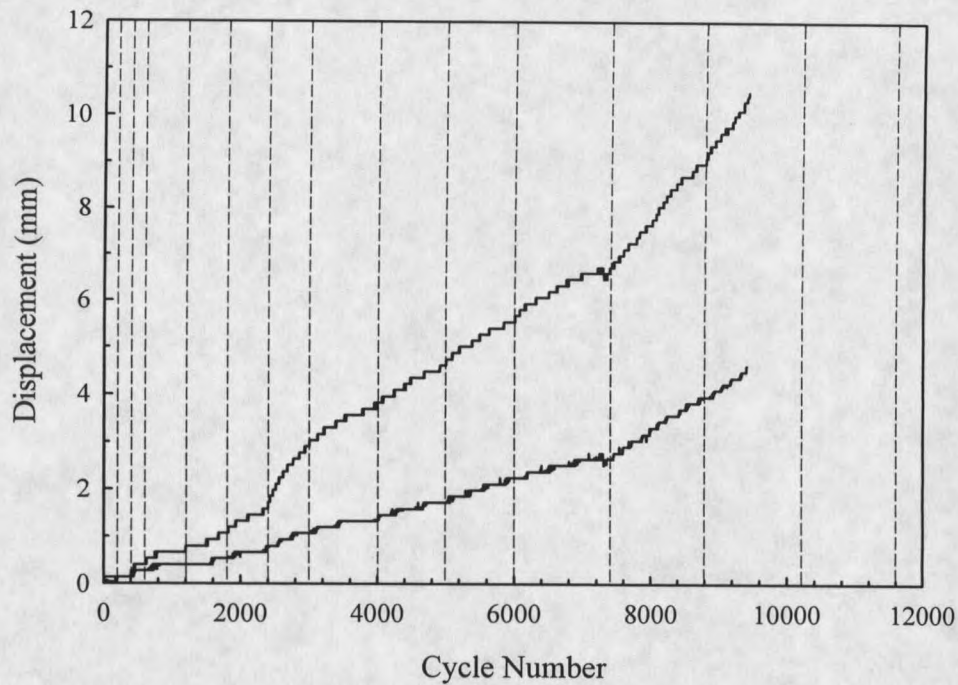


Figure B.61: Maximum and minimum displacements at position two along the length of a sample of geogrid pulled in the cross-machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: ggxd-10.

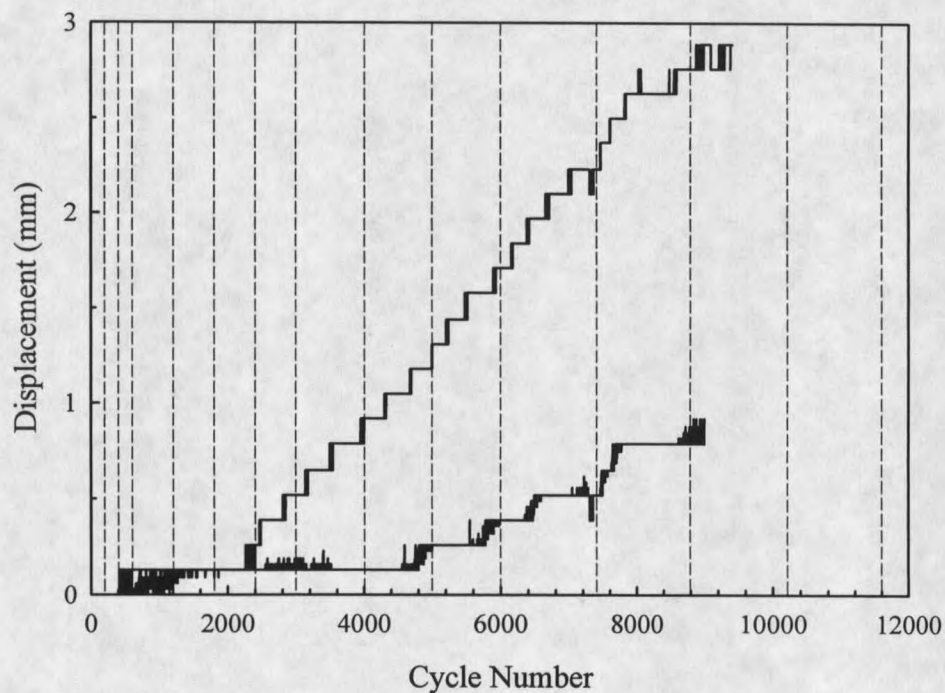


Figure B.62: Maximum and minimum displacements at position three along the length of a sample of geogrid pulled in the cross-machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: ggxd-10.

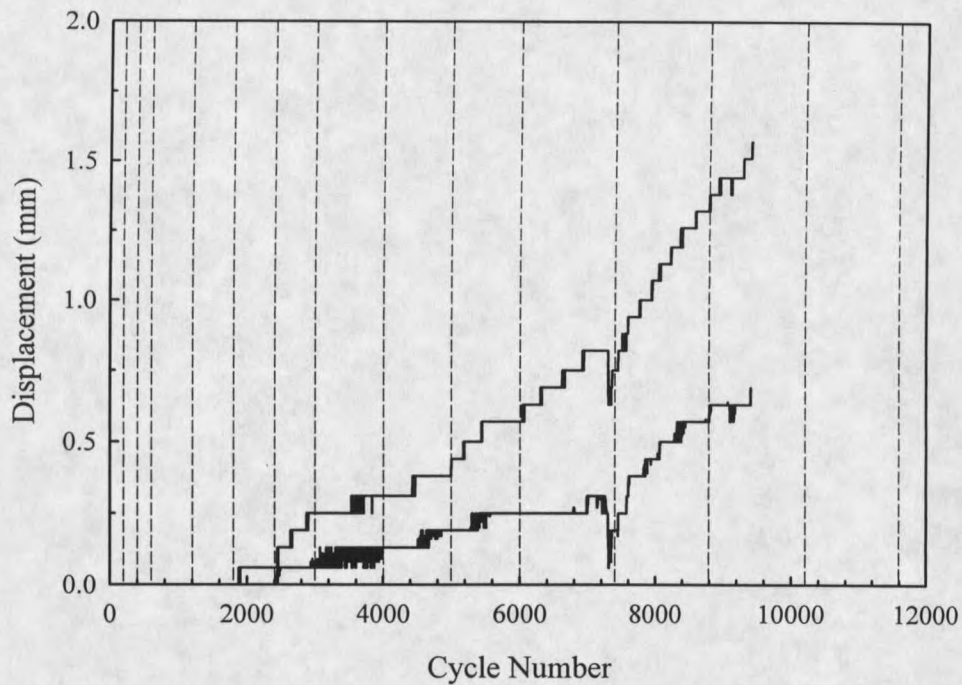


Figure B.63: Maximum and minimum displacements at position four along the length of a sample of geogrid pulled in the cross-machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: ggxd-10.

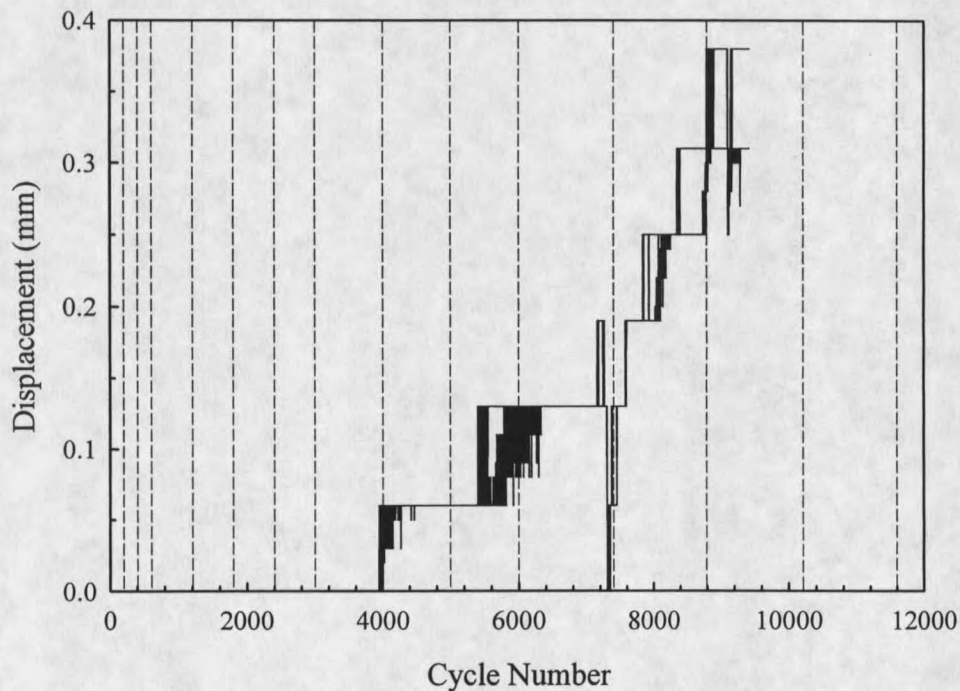


Figure B.64: Maximum and minimum displacement at position five along the length of a sample of geogrid pulled in the cross-machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: ggxd-10.

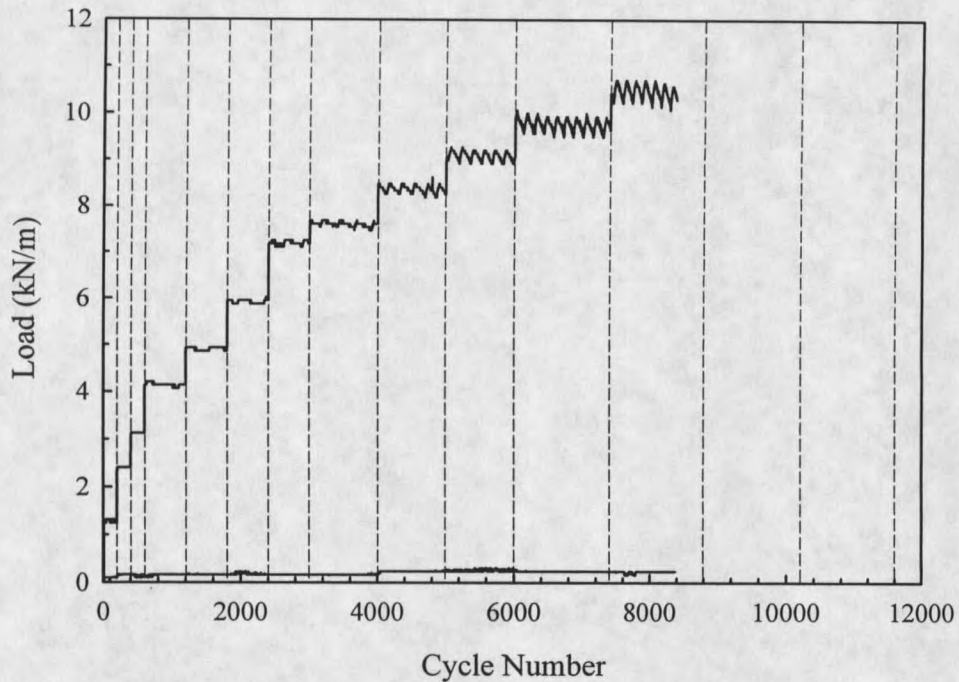


Figure B.65: Maximum and minimum loads applied to the geotextile in the machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: gtmd-15.

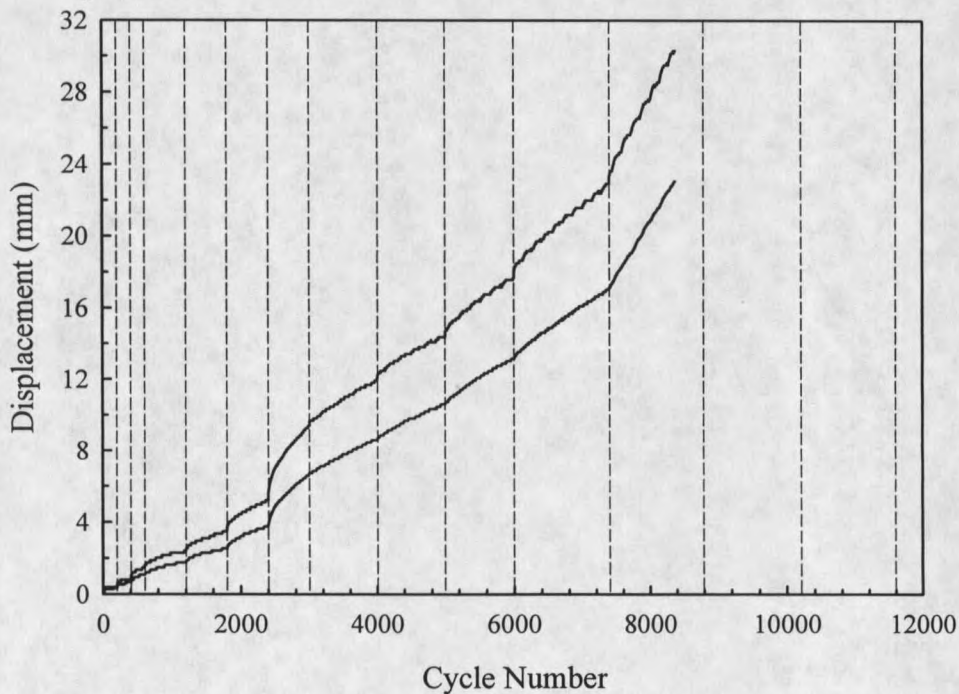


Figure B.66: Maximum and minimum displacements at the front of a sample of geotextile pulled in the machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: gtmd-15.

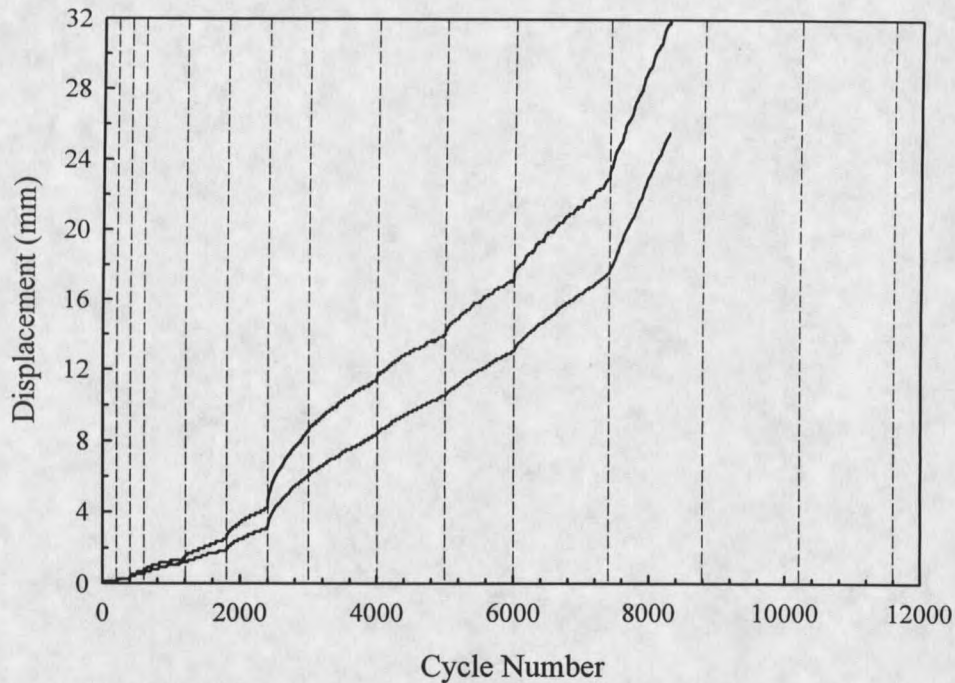


Figure B.67: Maximum and minimum displacements at position two along the length of a sample of geotextile pulled in the machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: gtmd-15.

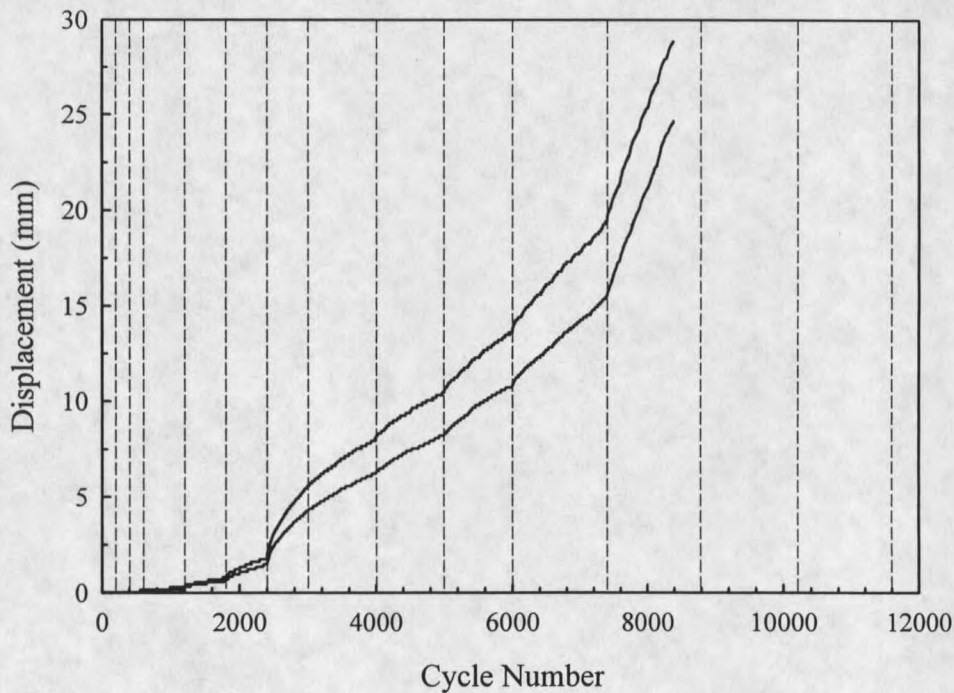


Figure B.68: Maximum and minimum displacements at position three along the length of a sample of geotextile pulled in the machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: gtmd-15.

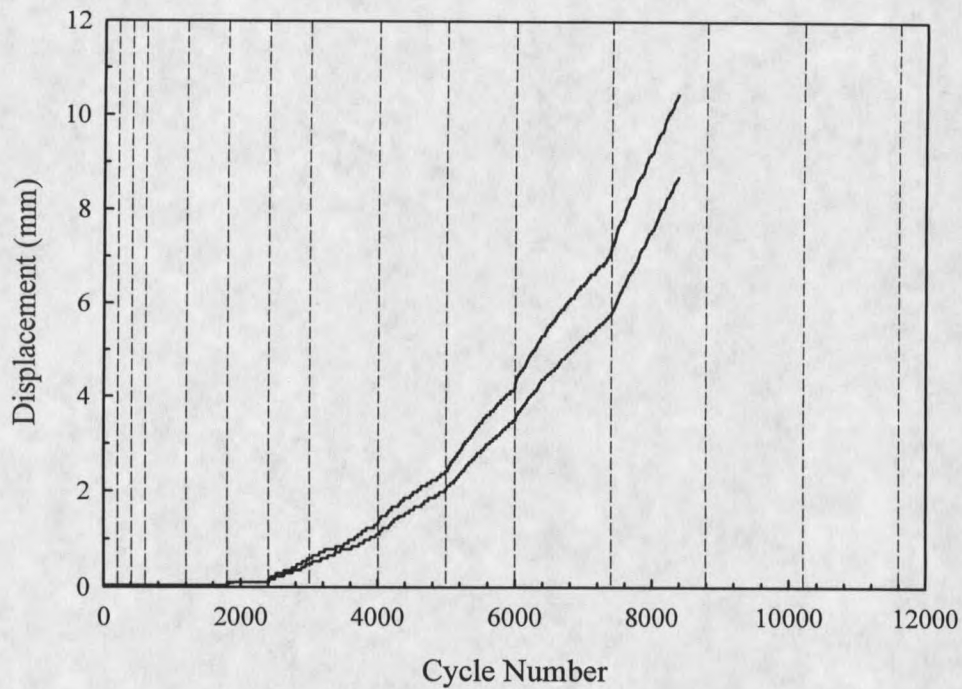


Figure B.69: Maximum and minimum displacements at position four along the length of a sample of geotextile pulled in the machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: gtmd-15.

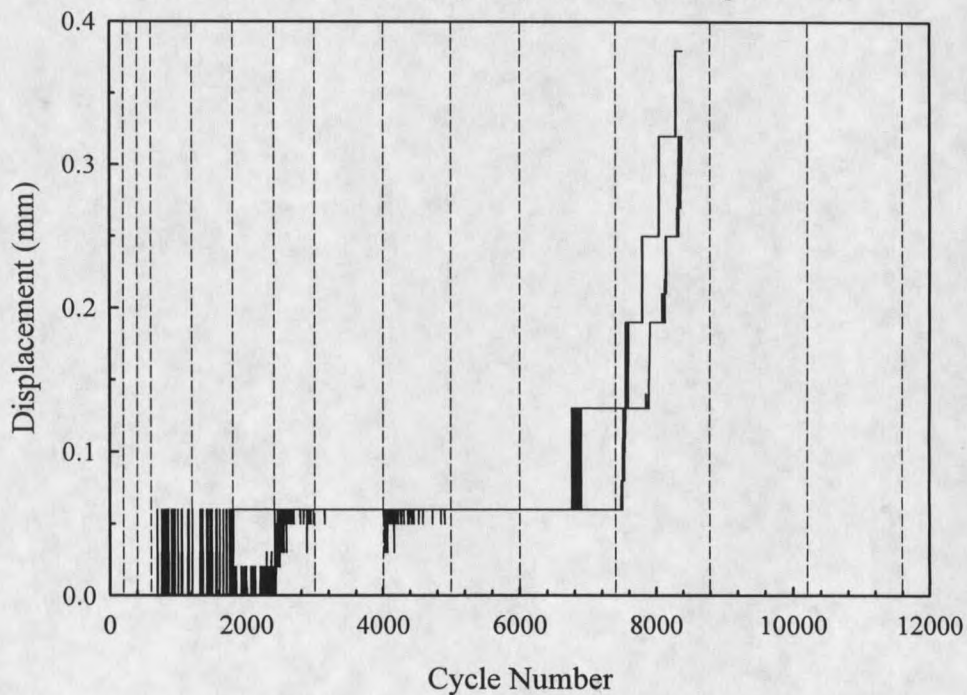


Figure B.70: Maximum and minimum displacement at position five along the length of a sample of geotextile pulled in the machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: gtmd-15.

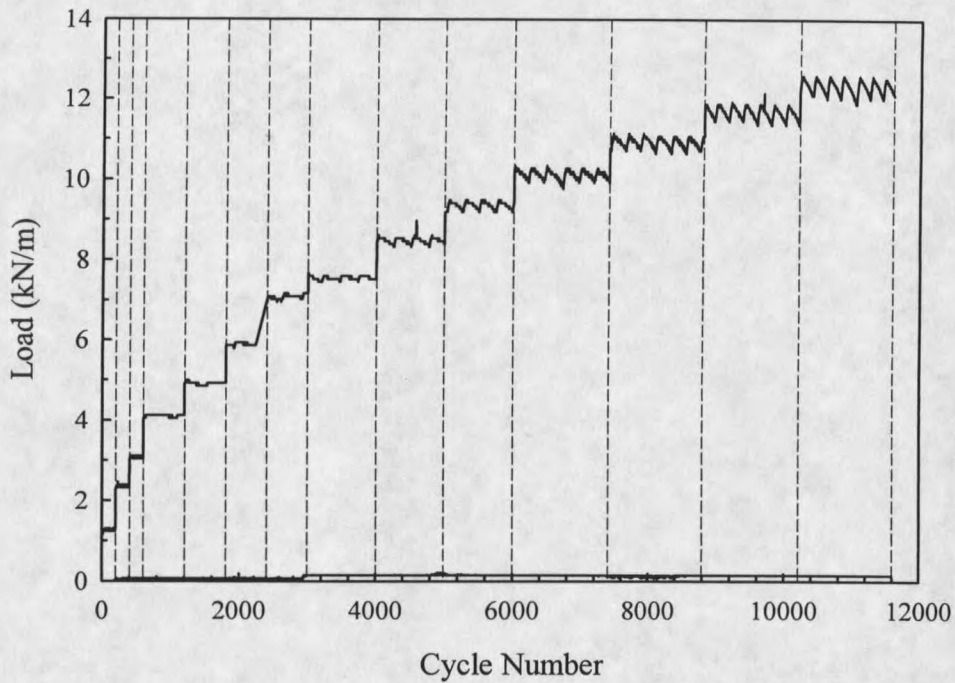


Figure B.71: Maximum and minimum loads applied to the geotextile in the cross-machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: gtxd-16.

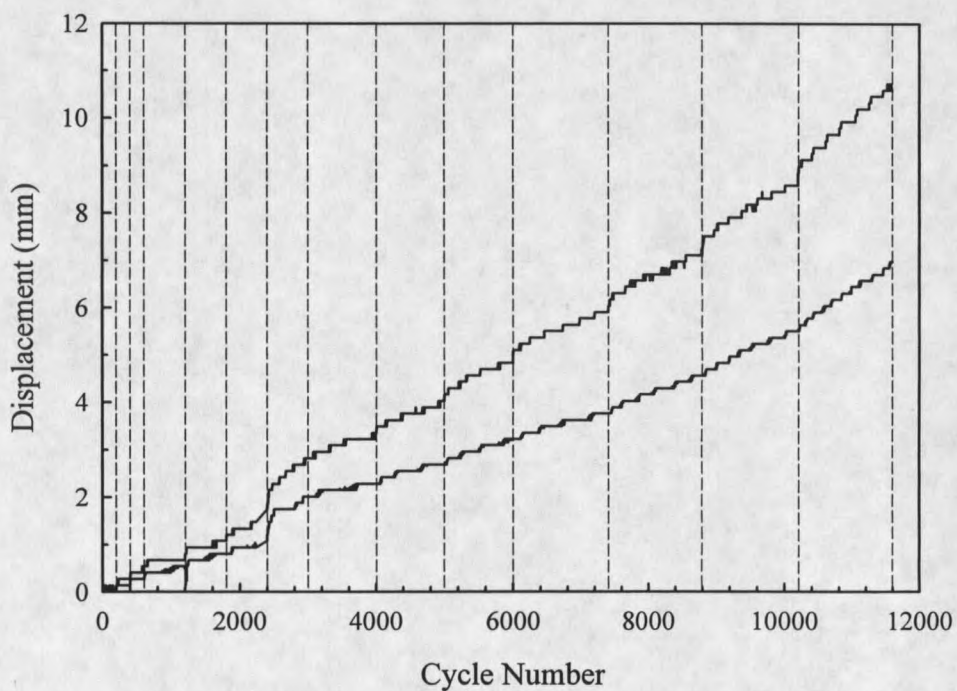


Figure B.72: Maximum and minimum displacements at the front of a sample of geotextile pulled in the cross-machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: gtxd-16.

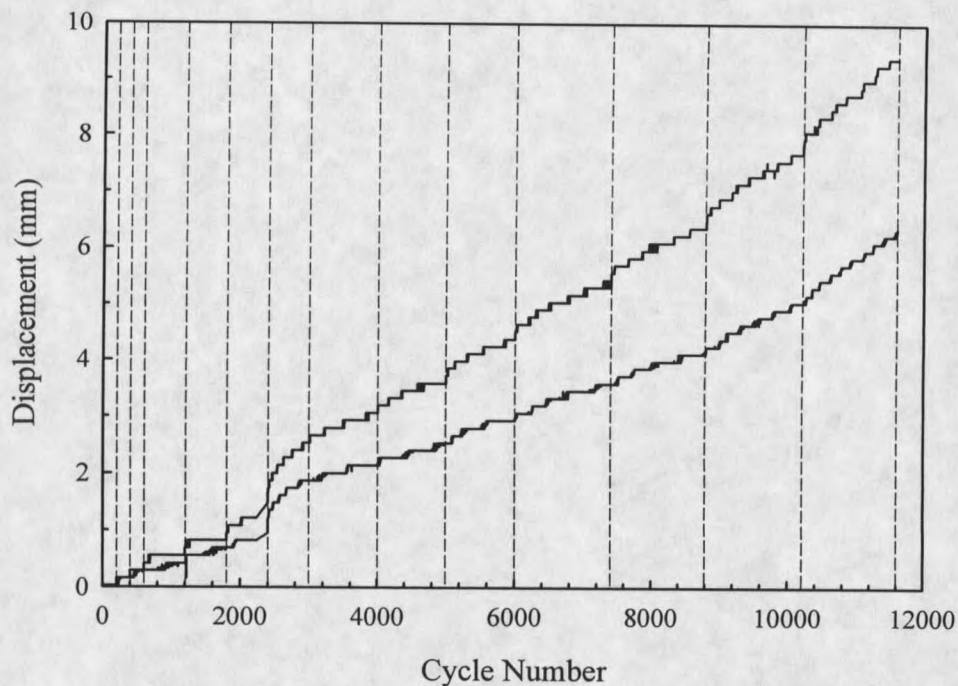


Figure B.73: Maximum and minimum displacements at position two along the length of a sample of geotextile pulled in the cross-machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: gtxd-16.

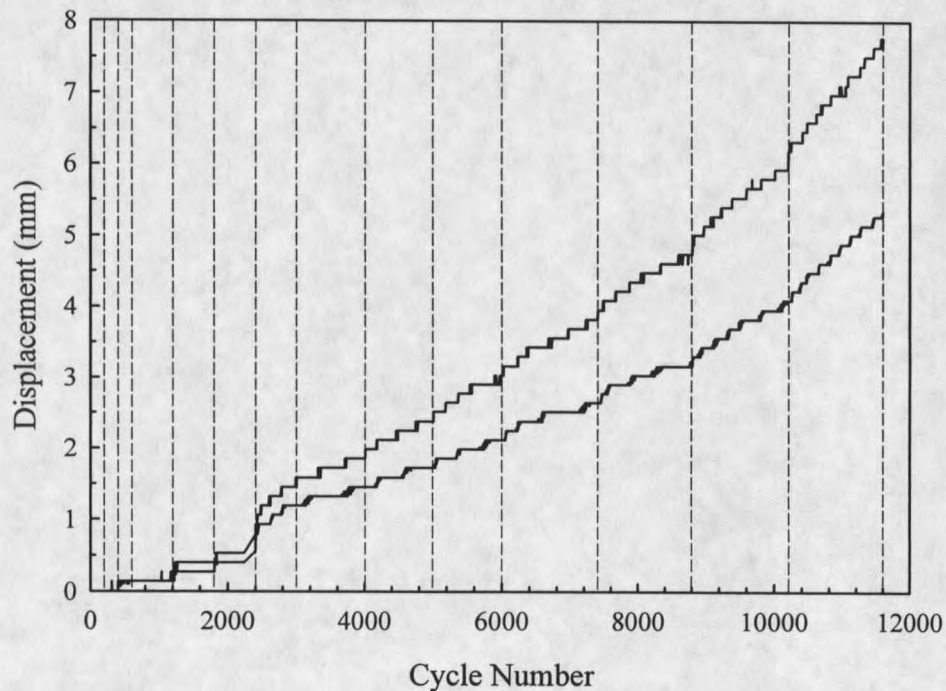


Figure B.74: Maximum and minimum displacements at position three along the length of a sample of geotextile pulled in the cross-machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: gtxd-16.

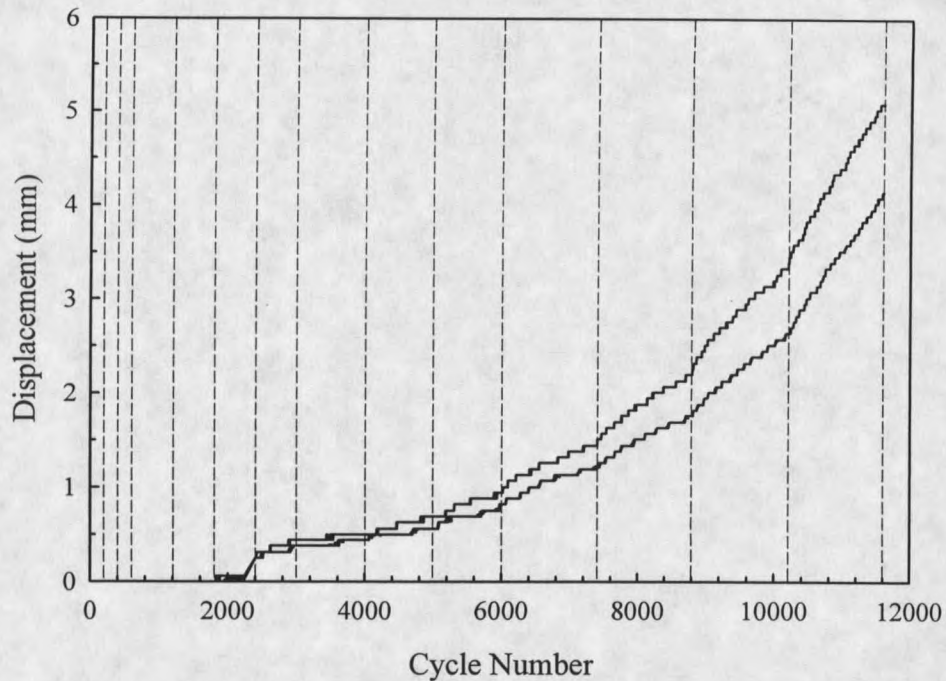


Figure B.75: Maximum and minimum displacements at position four along the length of a sample of geotextile pulled in the cross-machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: gtxd-16.

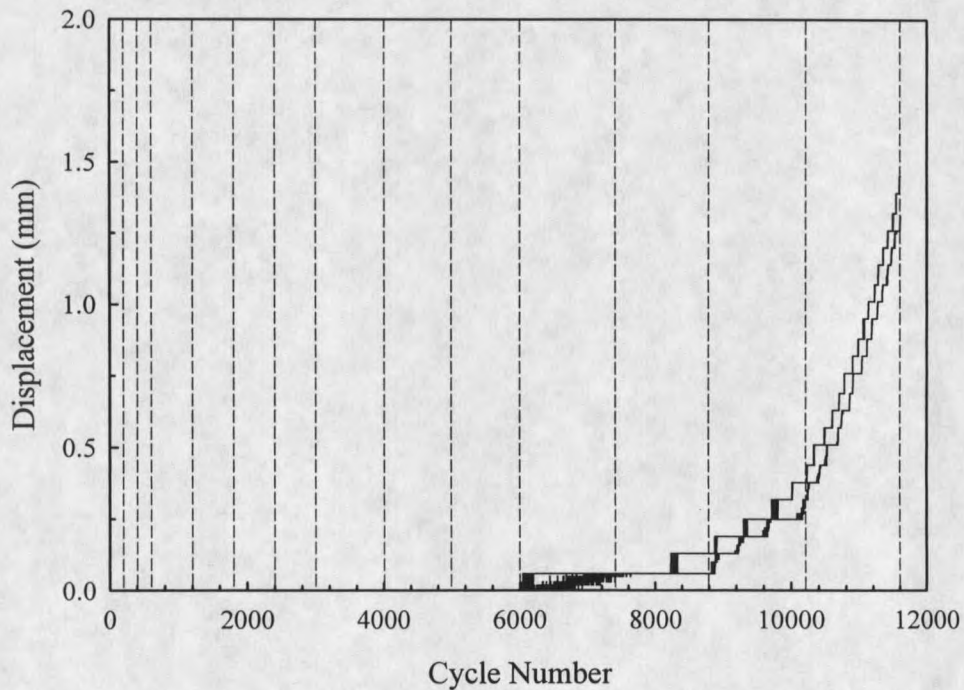


Figure B.76: Maximum and minimum displacement at position five along the length of a sample of geotextile pulled in the cross-machine direction at 15 kPa normal confinement for an in-soil, cyclic, Series II test: gtxd-16.

APPENDIX C

COMPLETE RESULTS FROM THE IN-SOIL ANALYSIS

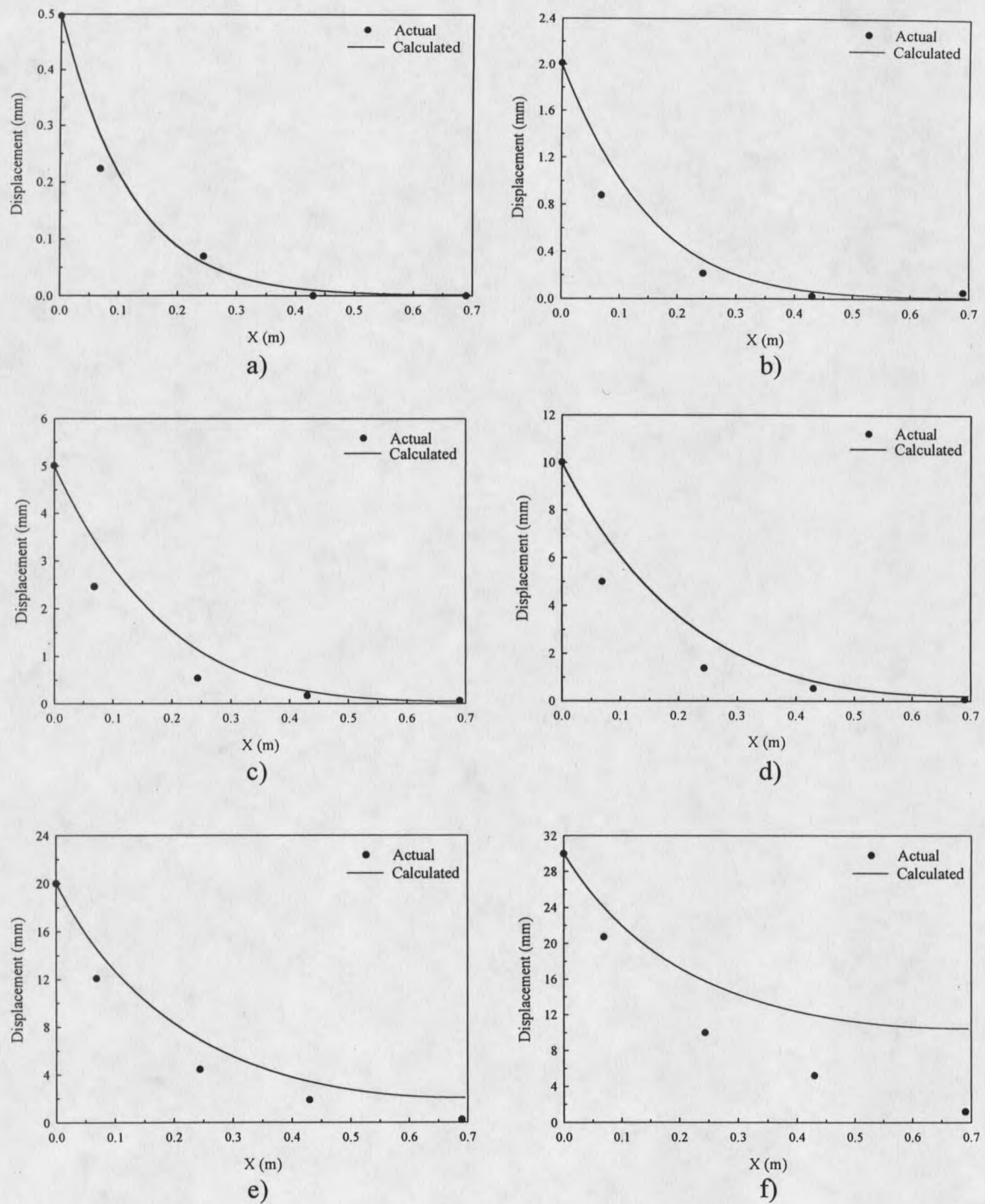


Figure C.1: Comparisons between actual and predicted displacement curves for the geogrid pulled in the machine direction at $\sigma_n = 5$ kPa: a) Level I, b) Level II, c) Level III, d) Level IV, e) Level V, f) Level VI: ggmd-12.

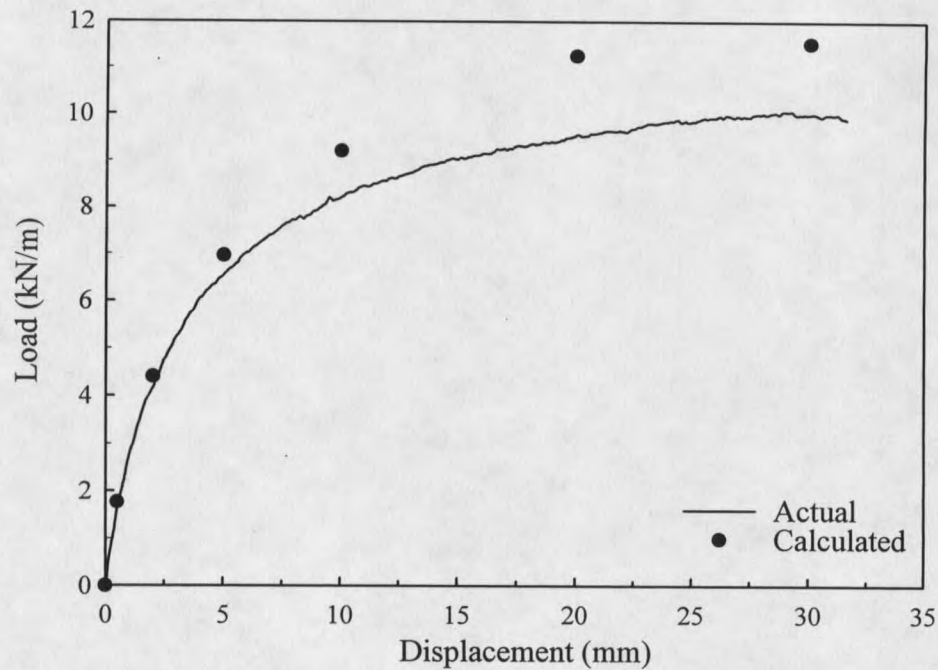


Figure C.2: Comparison between actual and predicted load/displacement curves for the geogrid pulled in the machine direction at $\sigma_n = 5$ kPa: ggmd-12.

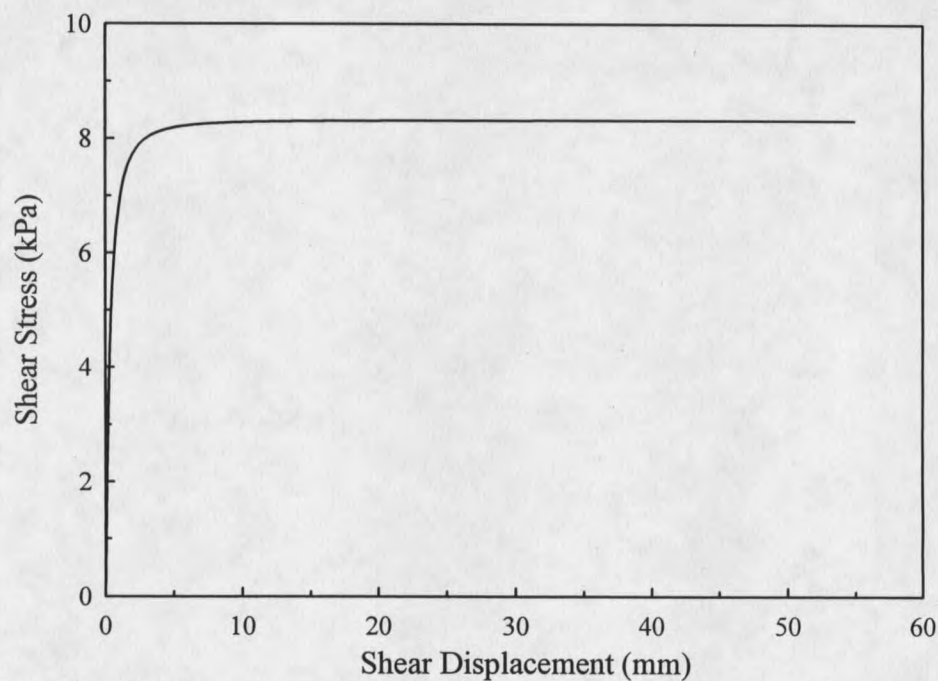


Figure C.3: Predicted shear stress versus shear displacement curve for the geogrid pulled in the machine direction at $\sigma_n = 5$ kPa: ggmd-12.

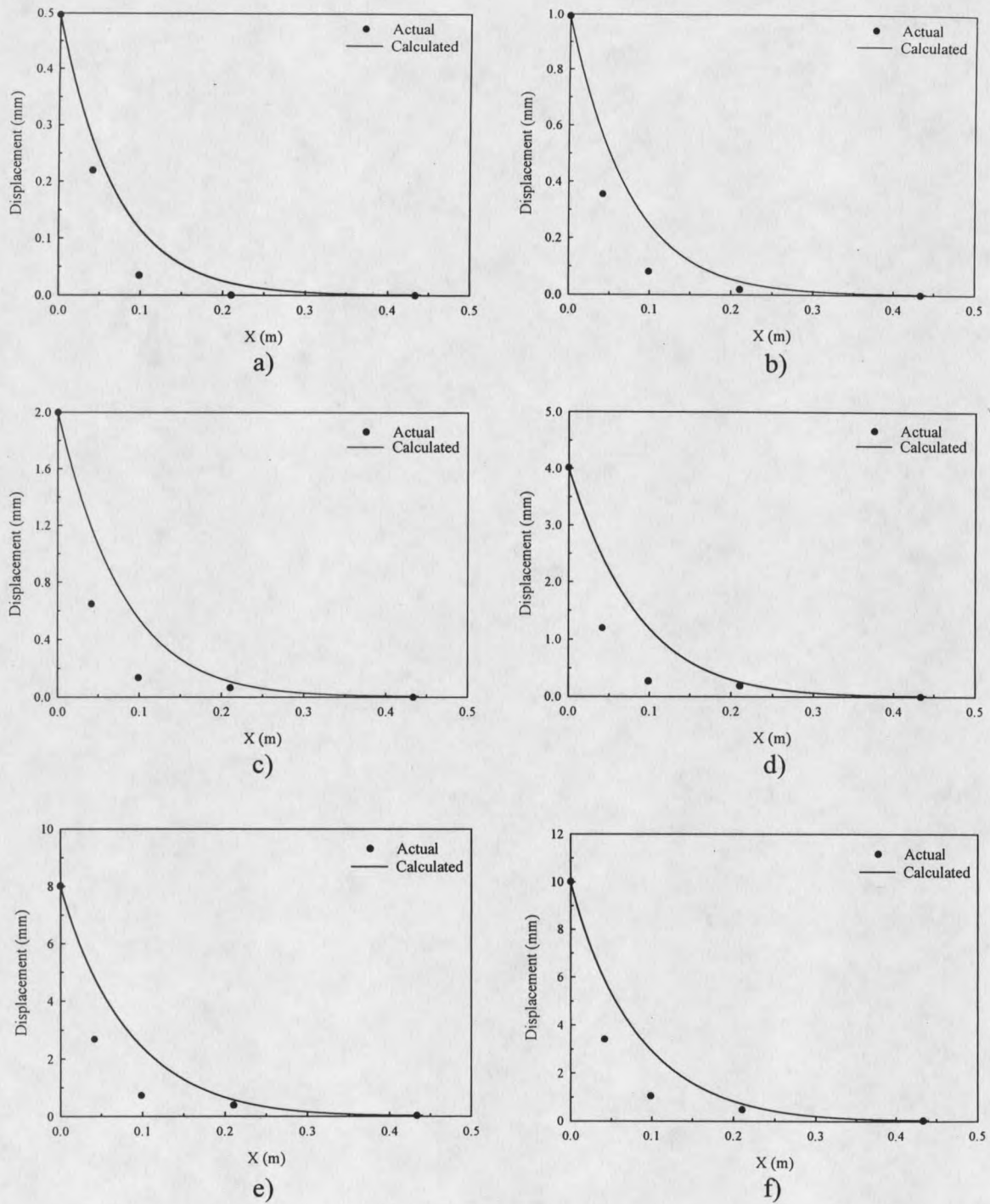


Figure C.4: Comparisons between actual and predicted displacement curves for the geogrid pulled in the machine direction at $\sigma_n = 15$ kPa: a) Level I, b) Level II, c) Level III, d) Level IV, e) Level V, f) Level VI: ggmd-11.

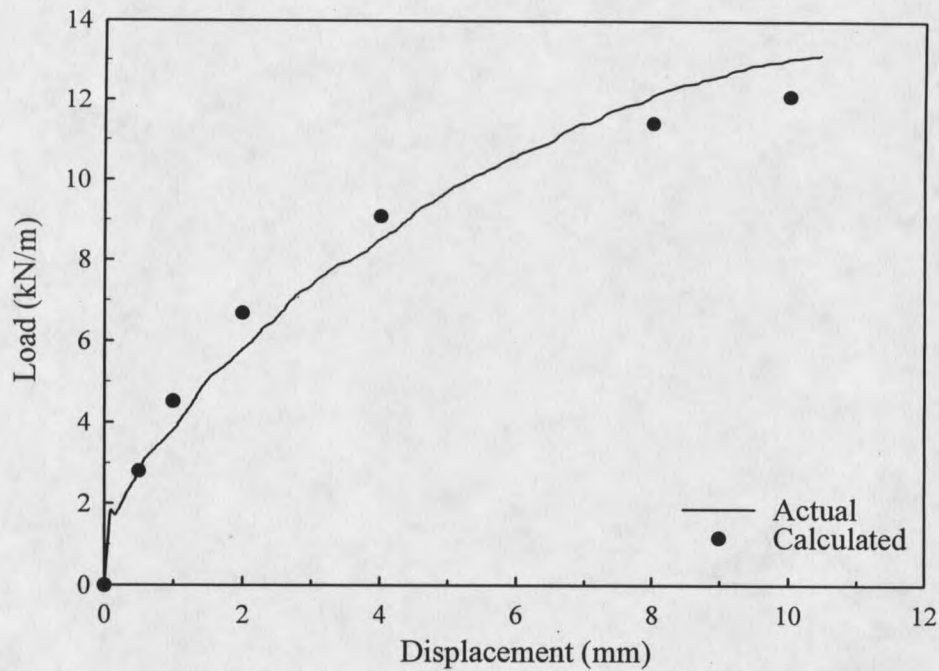


Figure C.5: Comparison between actual and predicted load/displacement curves for the geogrid pulled in the machine direction at $\sigma_n = 15$ kPa: ggmd-11.

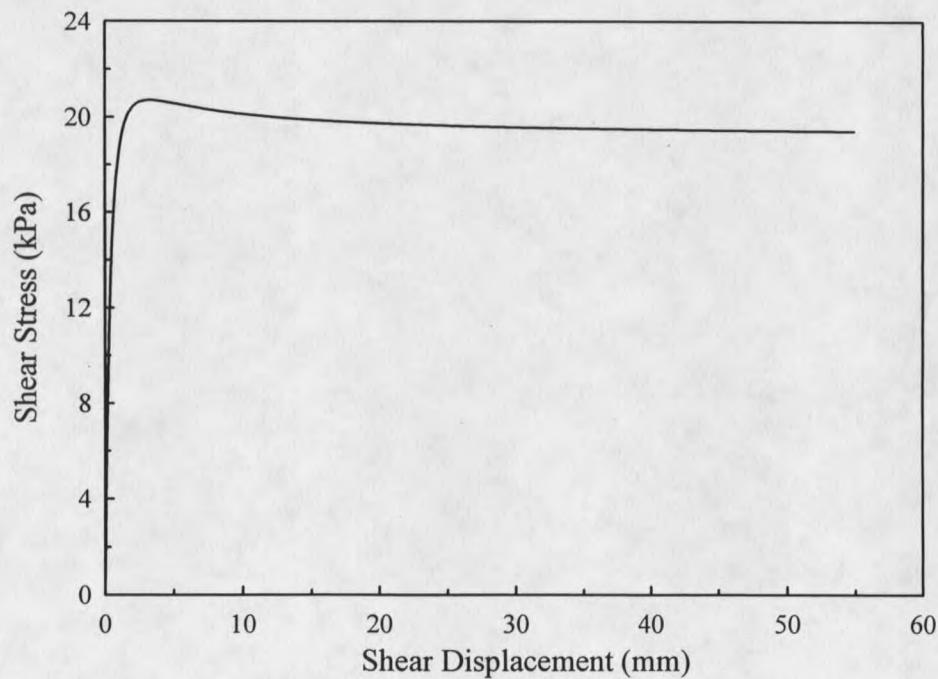


Figure C.6: Predicted shear stress versus shear displacement curve for the geogrid pulled in the machine direction at $\sigma_n = 15$ kPa: ggmd-11.

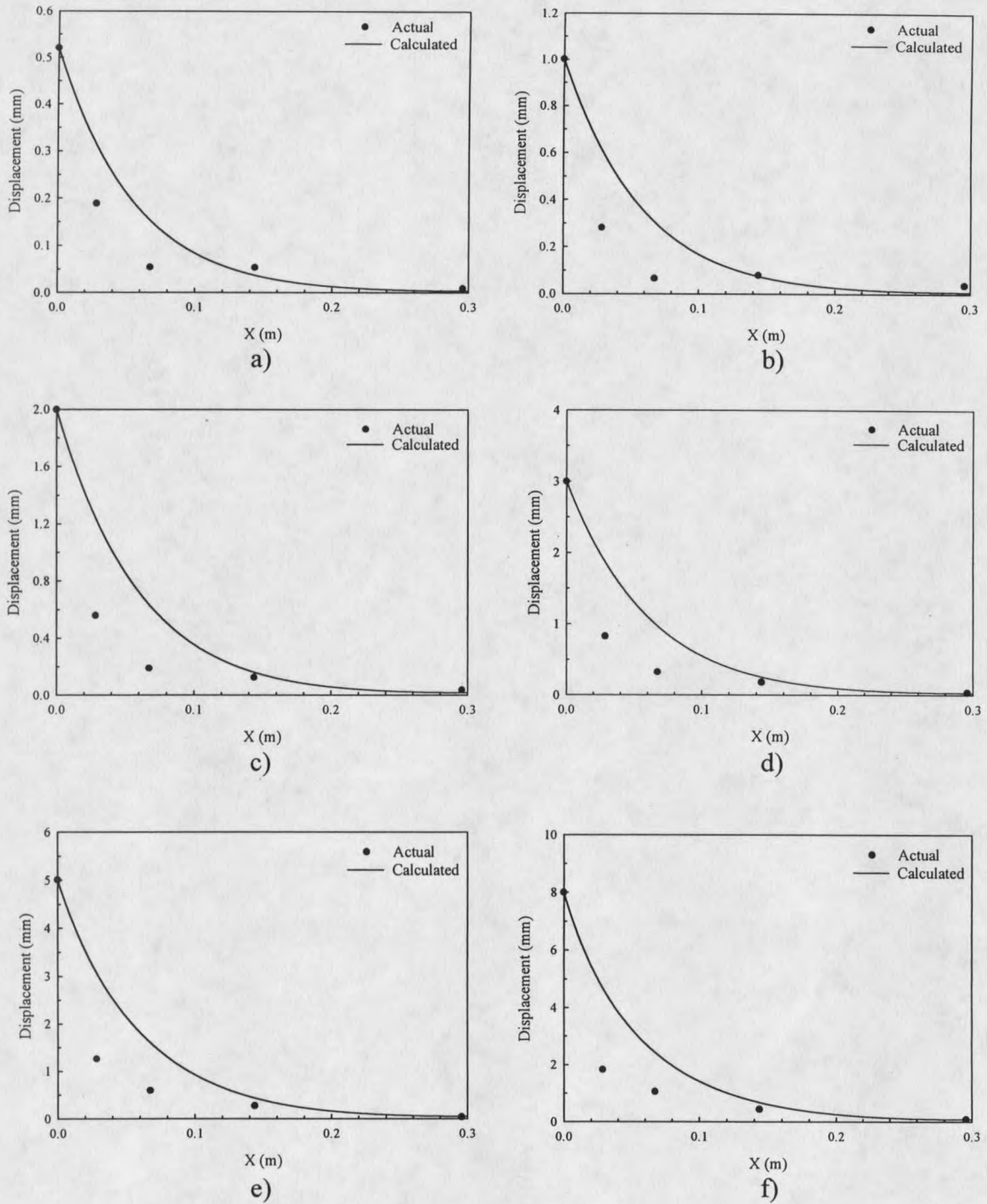


Figure C.7: Comparisons between actual and predicted displacement curves for the geogrid pulled in the machine direction at $\sigma_n = 35$ kPa: a) Level I, b) Level II, c) Level III, d) Level IV, e) Level V, f) Level VI: ggmd-10.

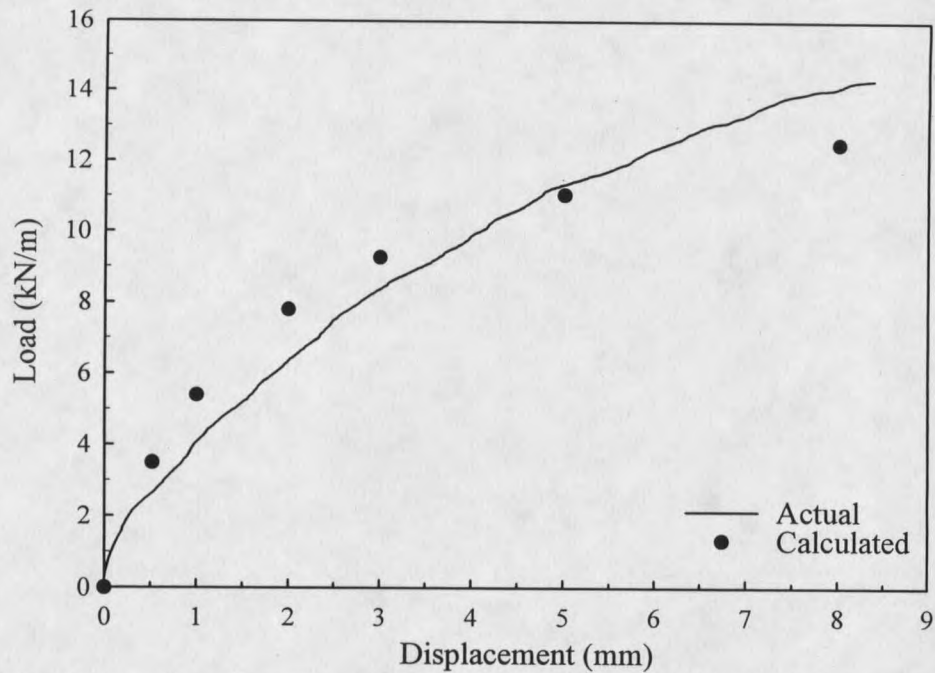


Figure C.8: Comparison between actual and predicted load/displacement curves for the geogrid pulled in the machine direction at $\sigma_n = 35$ kPa: ggmd-10.

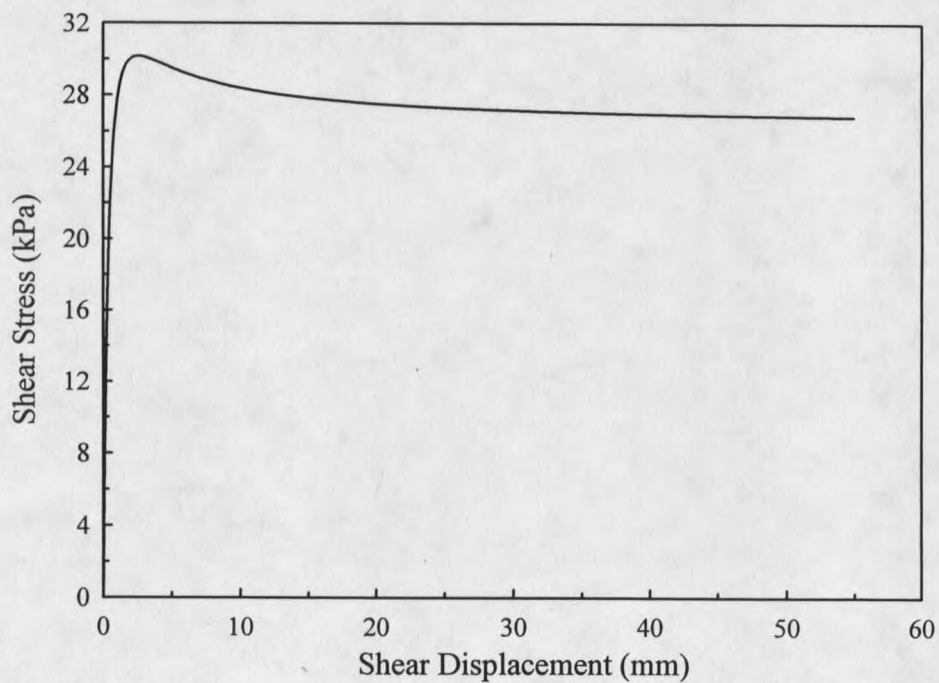


Figure C.9: Predicted shear stress versus shear displacement curve for the geogrid pulled in the machine direction at $\sigma_n = 35$ kPa: ggmd-10.

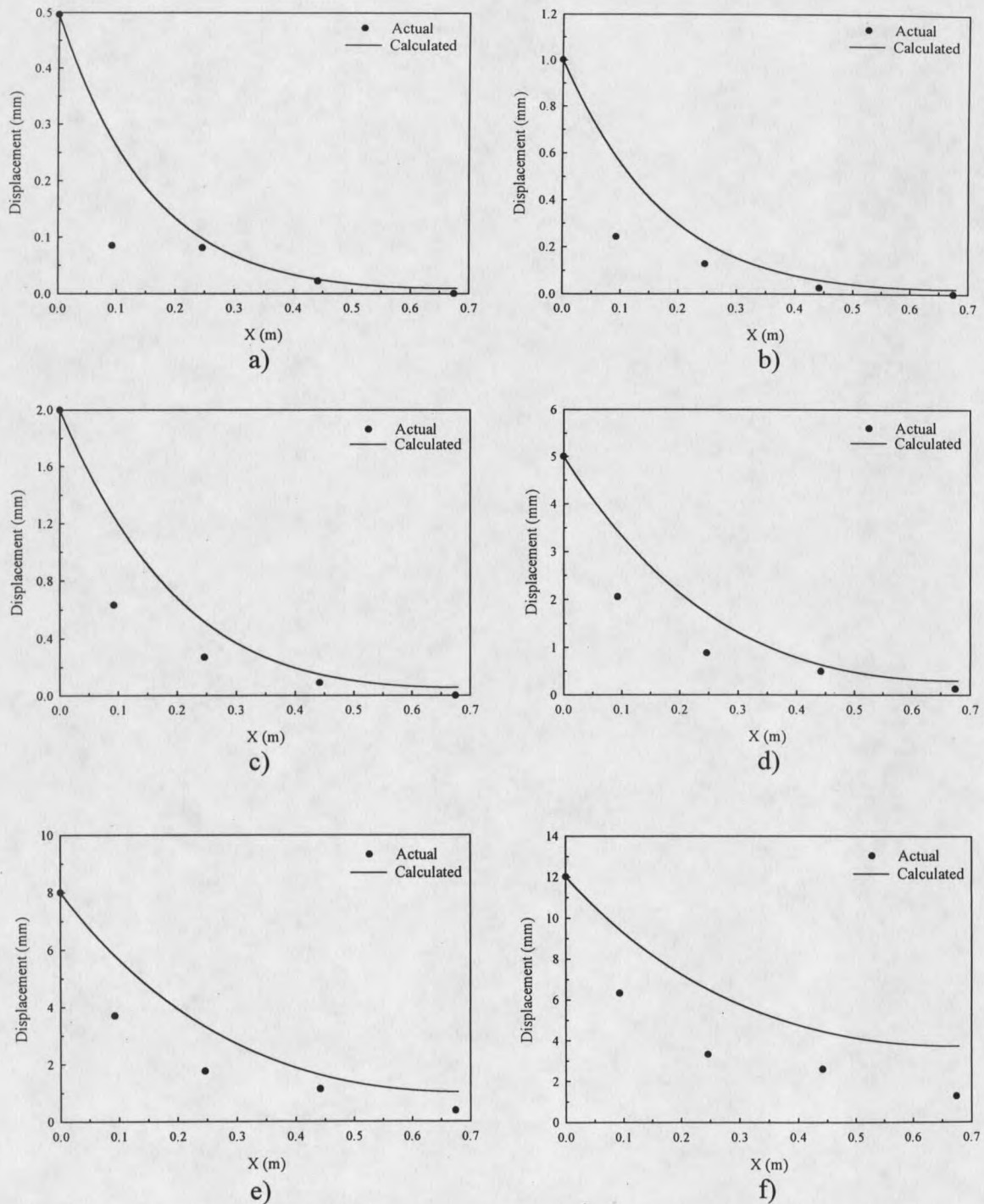


Figure C.10: Comparisons between actual and predicted displacement curves for the geogrid pulled in the cross-machine direction at $\sigma_n = 5$ kPa: a) Level I, b) Level II, c) Level III, d) Level IV, e) Level V, f) Level VI: ggxd-9.

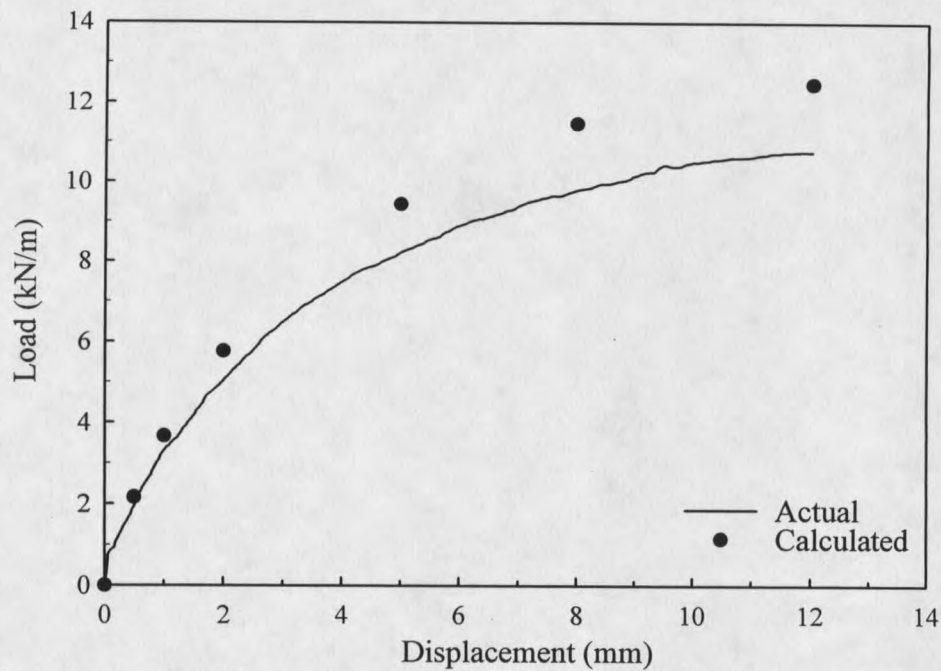


Figure C.11: Comparison between actual and predicted load/displacement curves for the geogrid pulled in the cross-machine direction at $\sigma_n = 5$ kPa: ggxd-9.

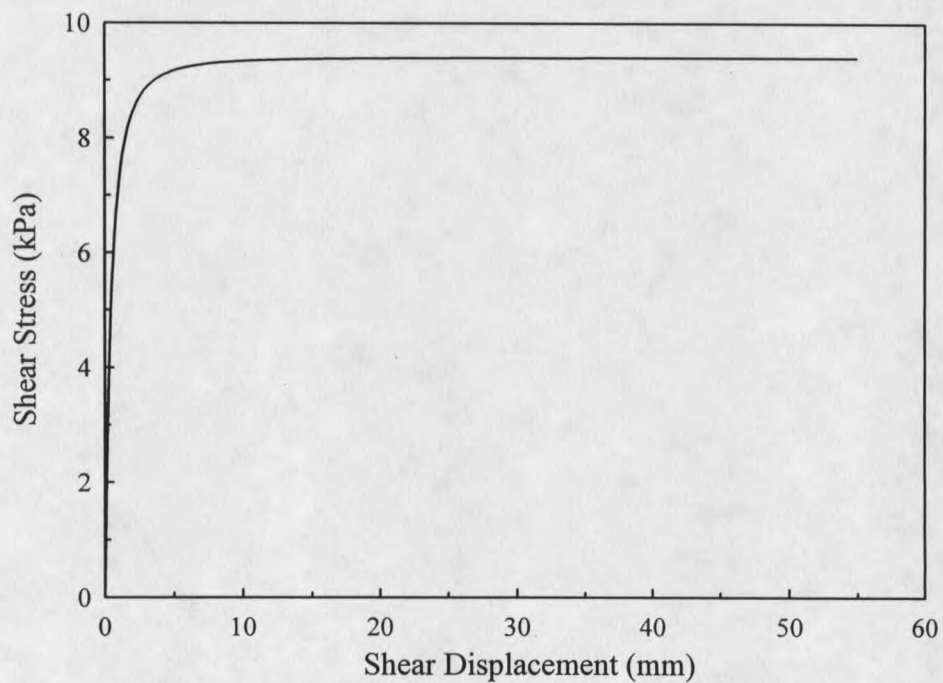


Figure C.12: Predicted shear stress versus shear displacement curve for the geogrid pulled in the cross-machine direction at $\sigma_n = 5$ kPa: ggxd-9.

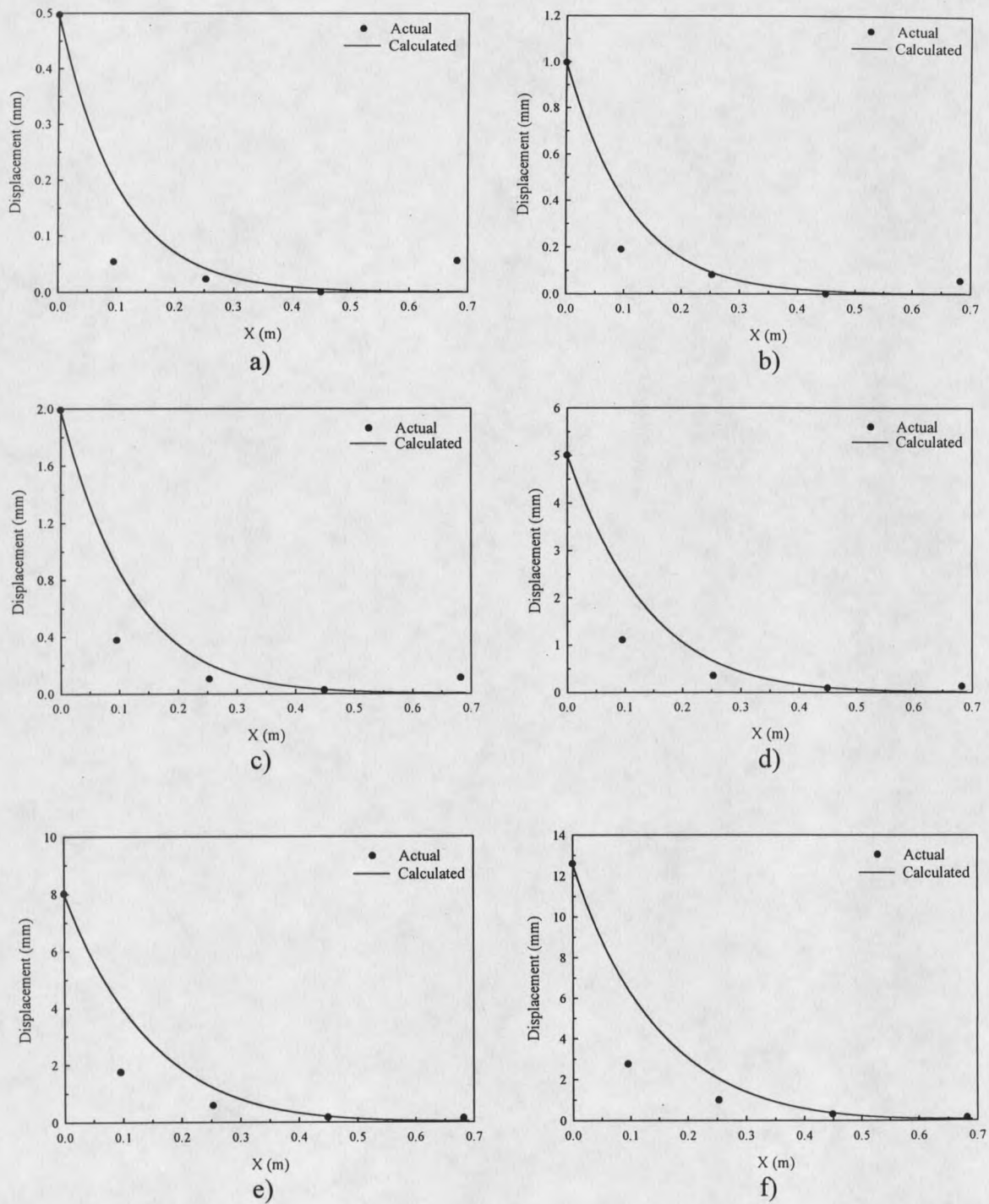


Figure C.13: Comparisons between actual and predicted displacement curves for the geogrid pulled in the cross-machine direction at $\sigma_n = 15$ kPa: a) Level I, b) Level II, c) Level III, d) Level IV, e) Level V, f) Level VI: ggxd-8.

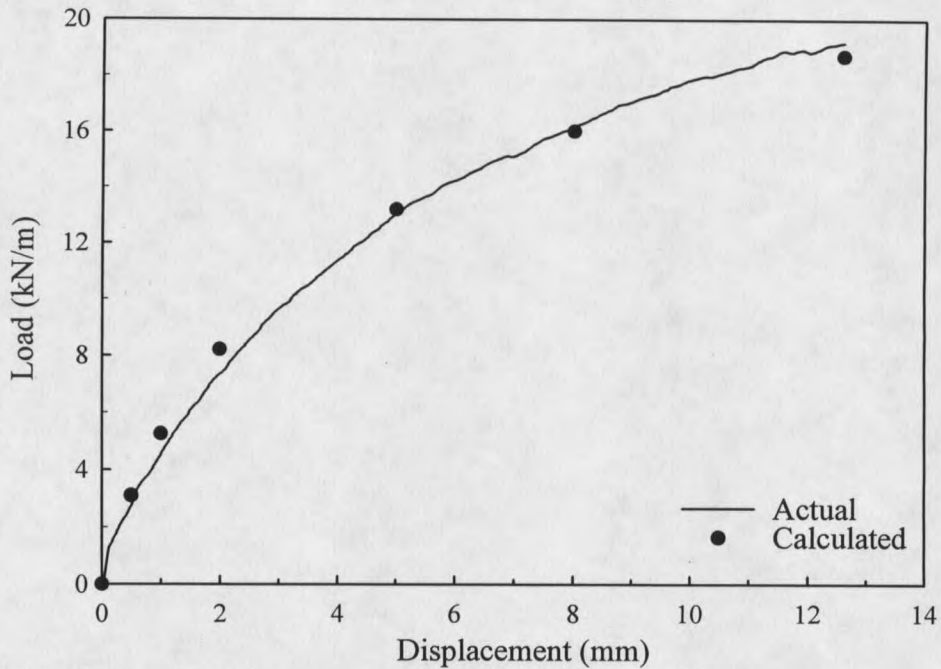


Figure C.14: Comparison between actual and predicted load/displacement curves for the geogrid pulled in the cross-machine direction at $\sigma_n = 15$ kPa: ggxd-8.

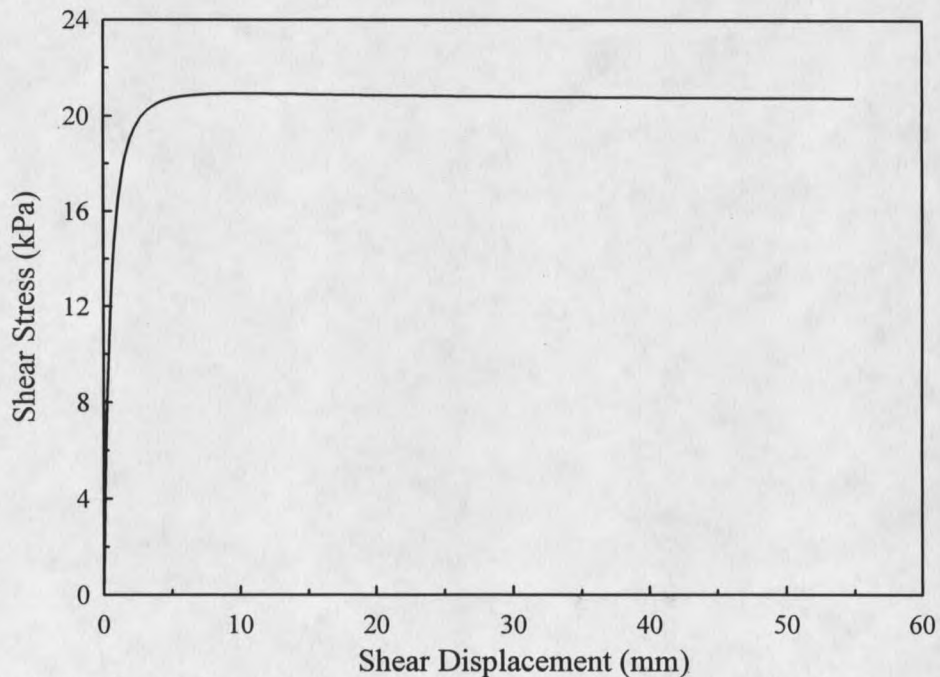


Figure C.15: Predicted shear stress versus shear displacement curve for the geogrid pulled in the cross-machine direction at $\sigma_n = 15$ kPa: ggxd-8.

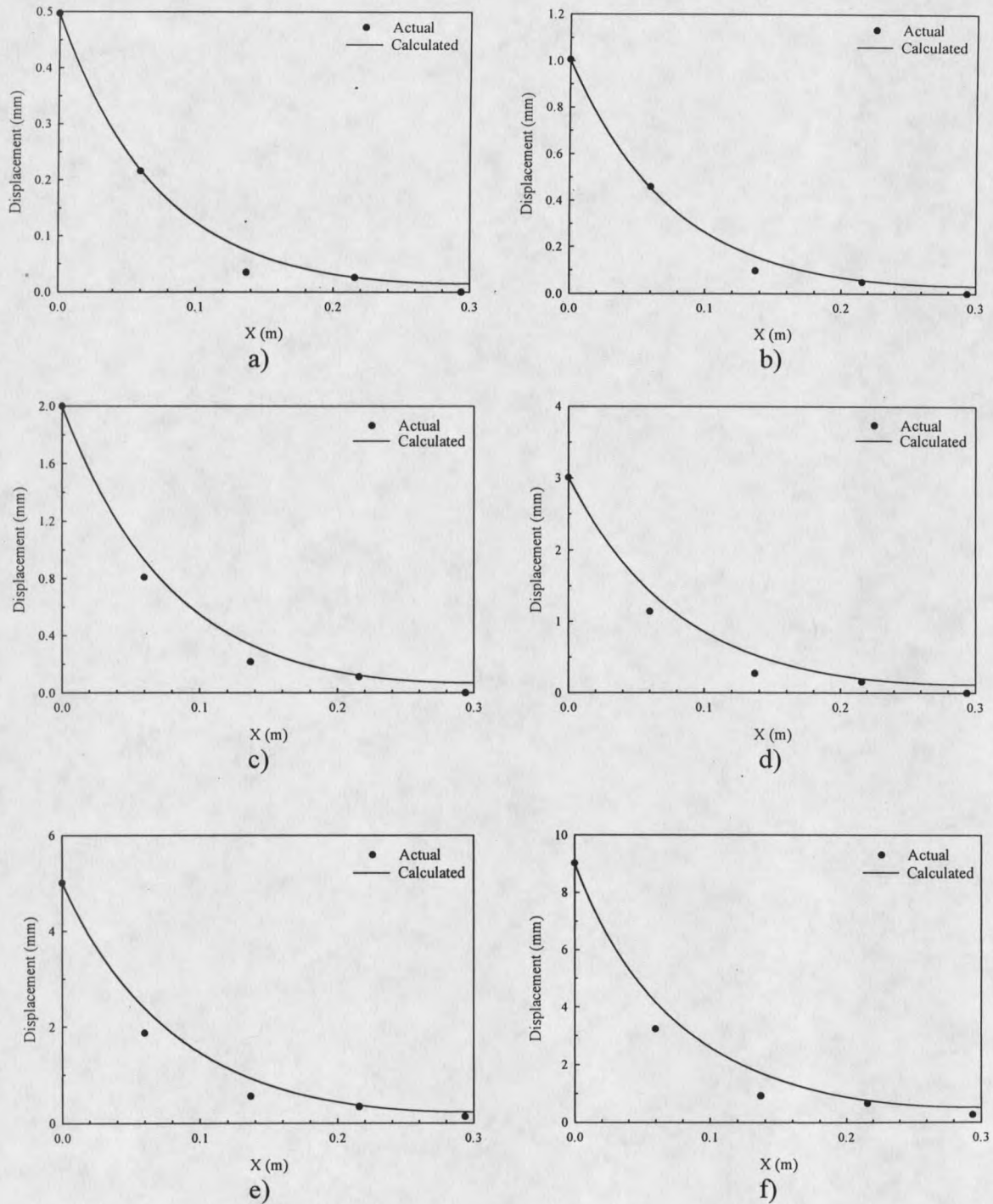


Figure C.16: Comparisons between actual and predicted displacement curves for the geogrid pulled in the cross-machine direction at $\sigma_n = 35$ kPa: a) Level I, b) Level II, c) Level III, d) Level IV, e) Level V, f) Level VI: ggxd-7.

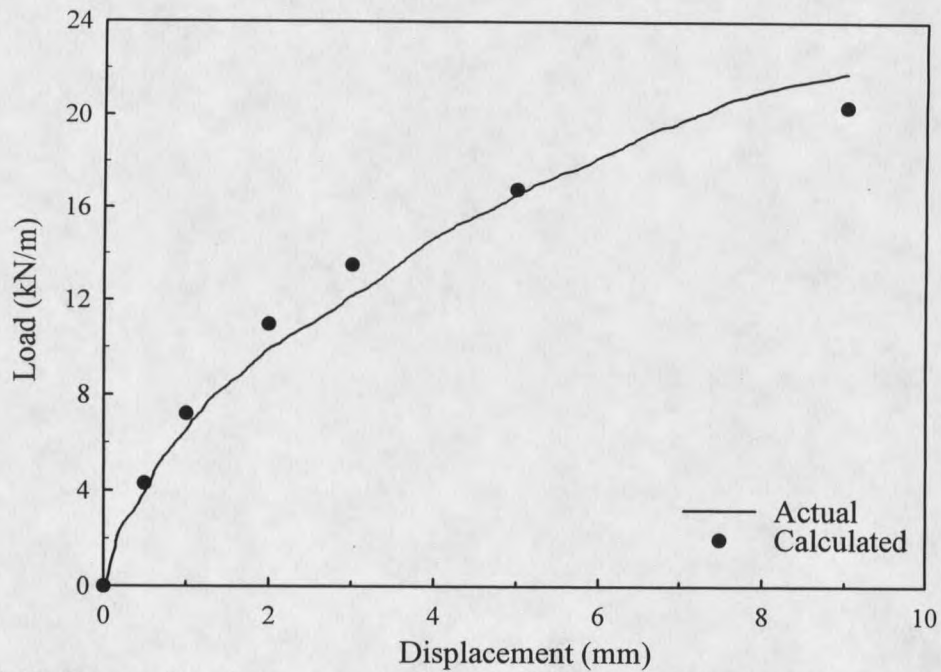


Figure C.17: Comparison between actual and predicted load/displacement curves for the geogrid pulled in the cross-machine direction at $\sigma_n = 35$ kPa: ggxd-7.

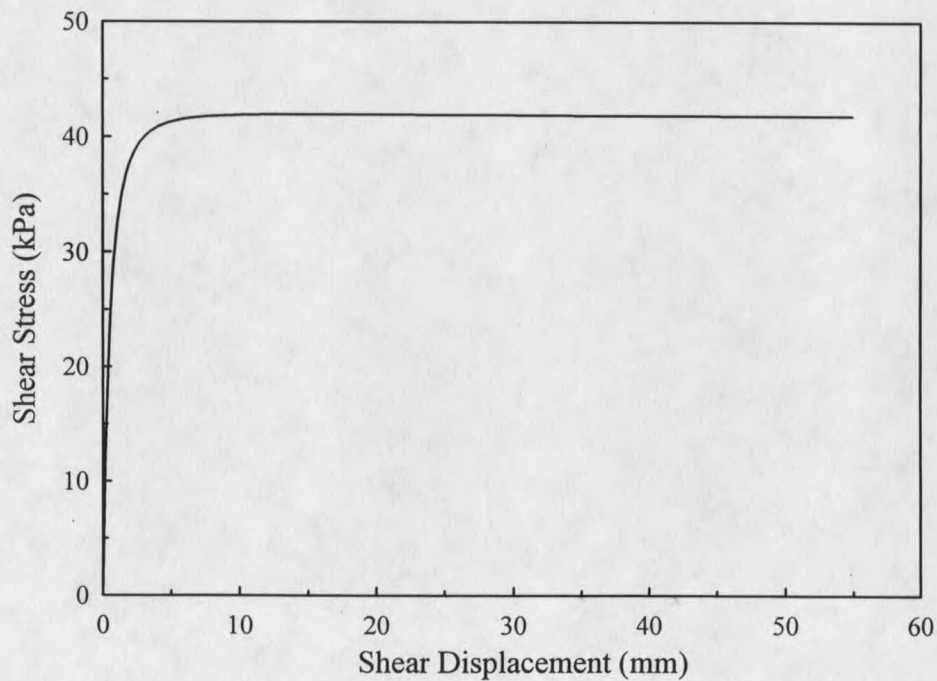


Figure C.18: Predicted shear stress versus shear displacement curve for the geogrid pulled in the cross-machine direction at $\sigma_n = 35$ kPa: ggxd-7.

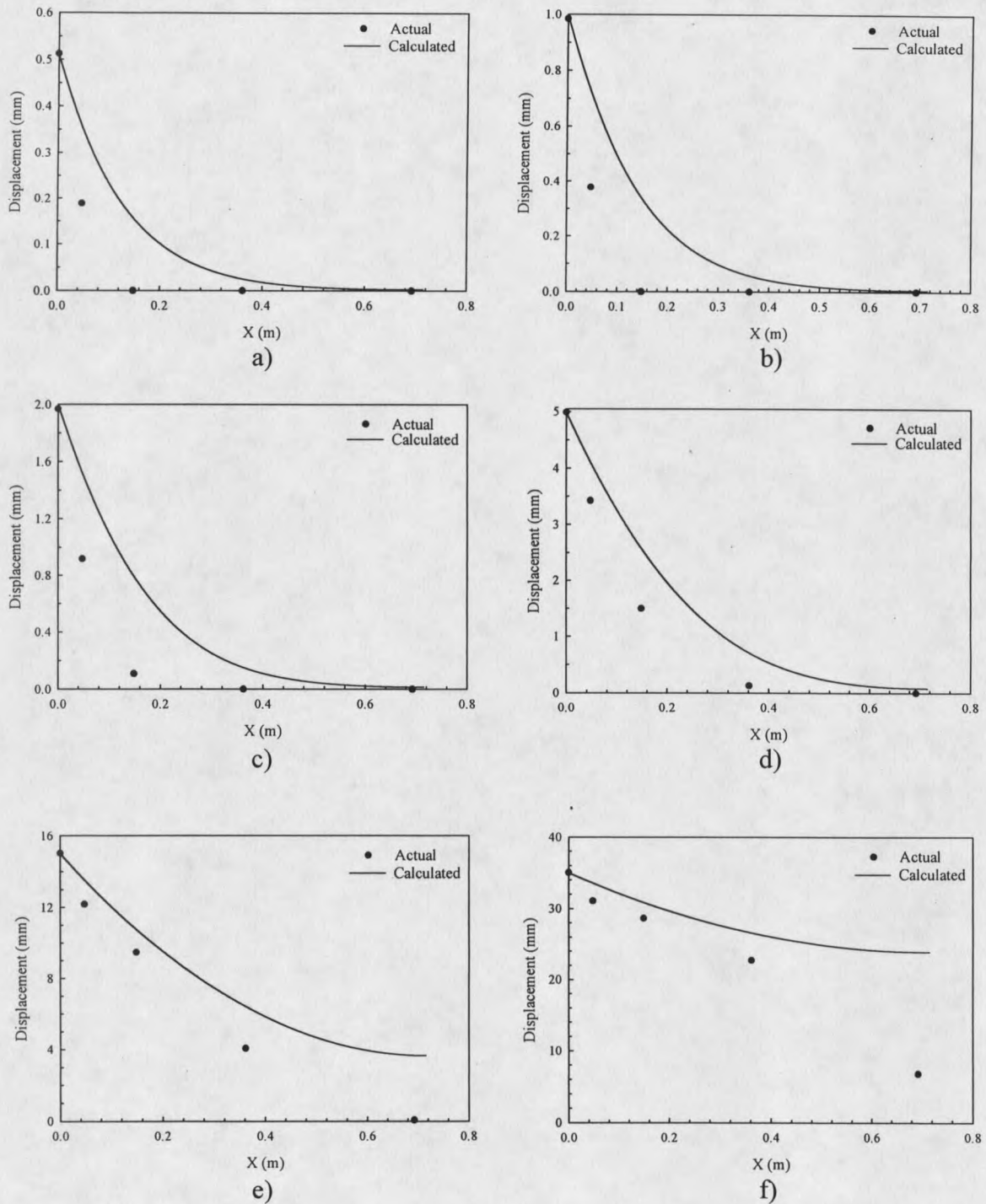


Figure C.19: Comparisons between actual and predicted displacement curves for the geotextile pulled in the machine direction at $\sigma_n = 5$ kPa: a) Level I, b) Level II, c) Level III, d) Level IV, e) Level V, f) Level VI: gtmd-14.

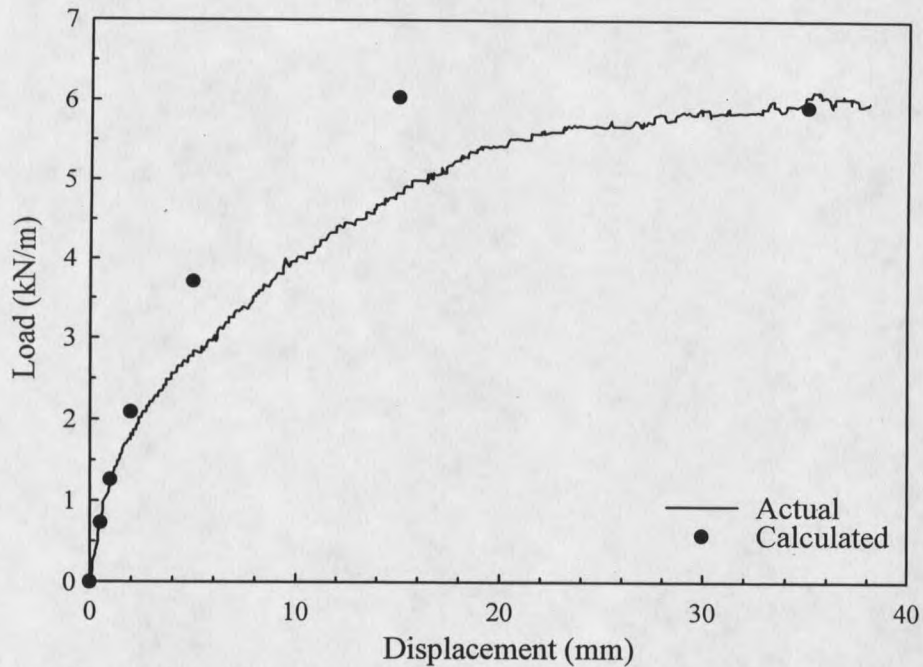


Figure C.20: Comparison between actual and predicted load/displacement curves for the geotextile pulled in the machine direction at $\sigma_n = 5$ kPa: gtmd-14.

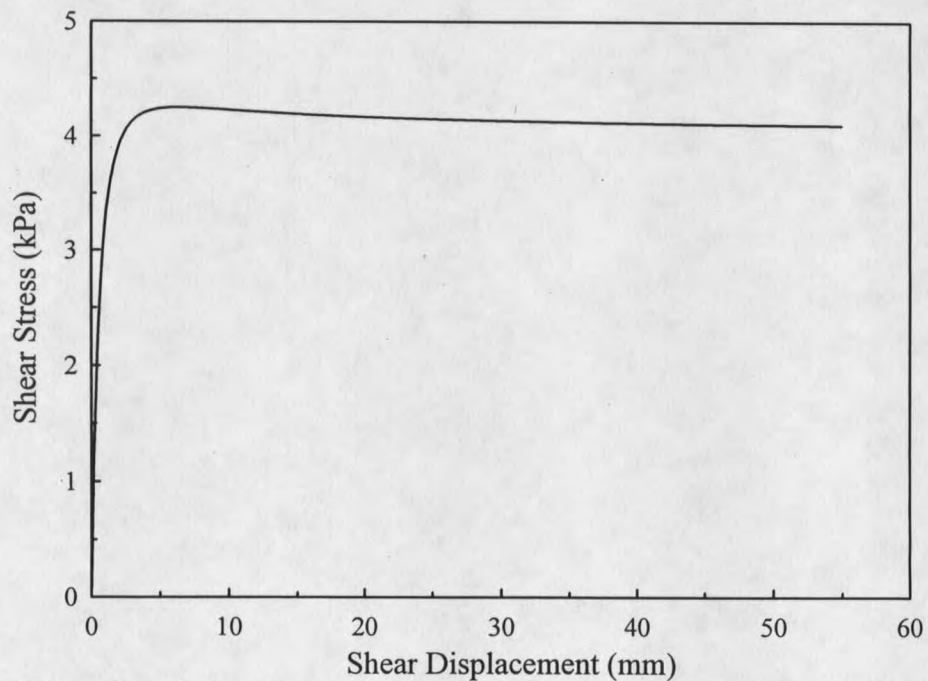


Figure C.21: Predicted shear stress versus shear displacement curve for the geotextile pulled in the machine direction at $\sigma_n = 5$ kPa: gtmd-14.

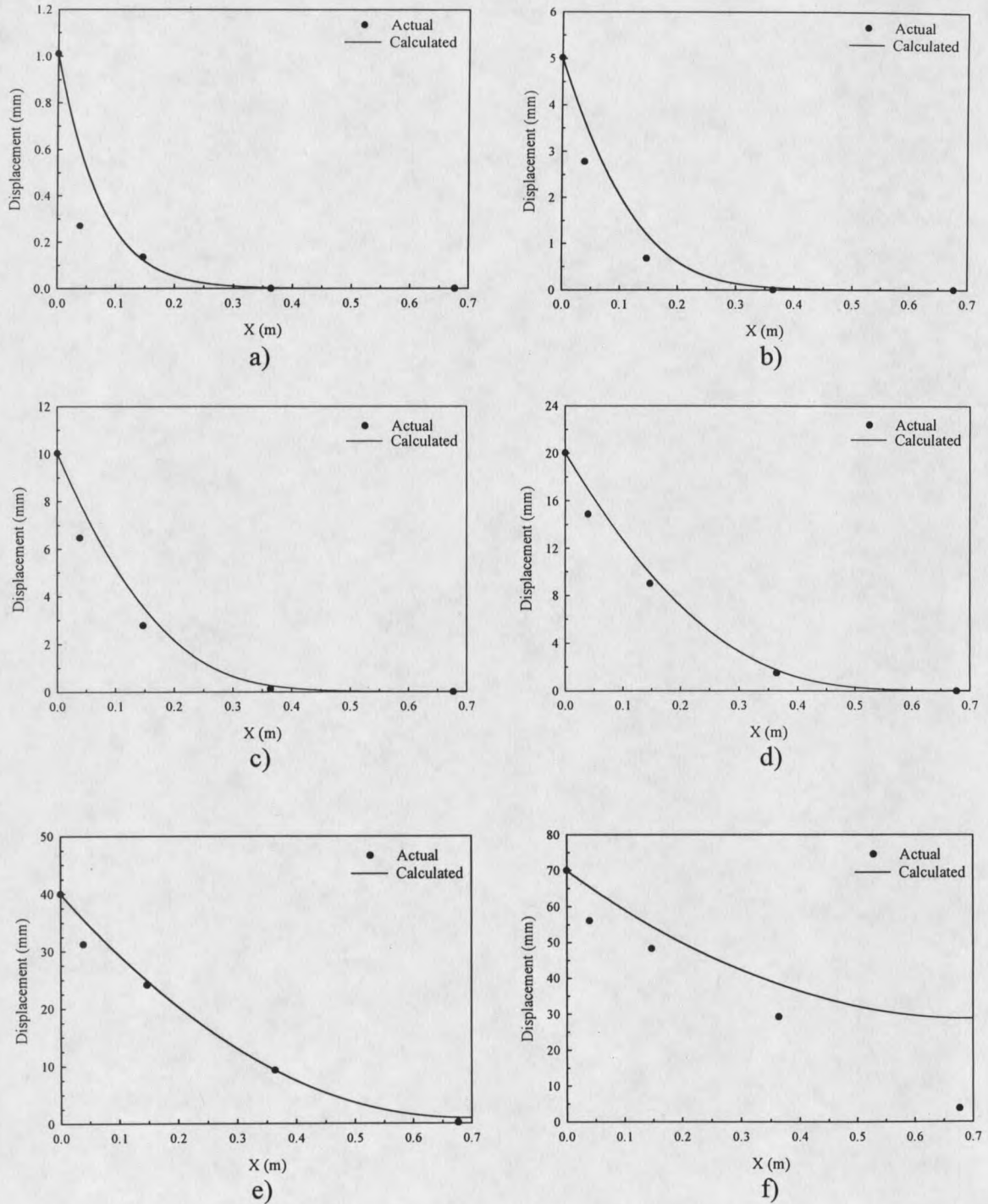


Figure C.22: Comparisons between actual and predicted displacement curves for the geotextile pulled in the machine direction at $\sigma_n = 15$ kPa: a) Level I, b) Level II, c) Level III, d) Level IV, e) Level V, f) Level VI: gtmd-12.

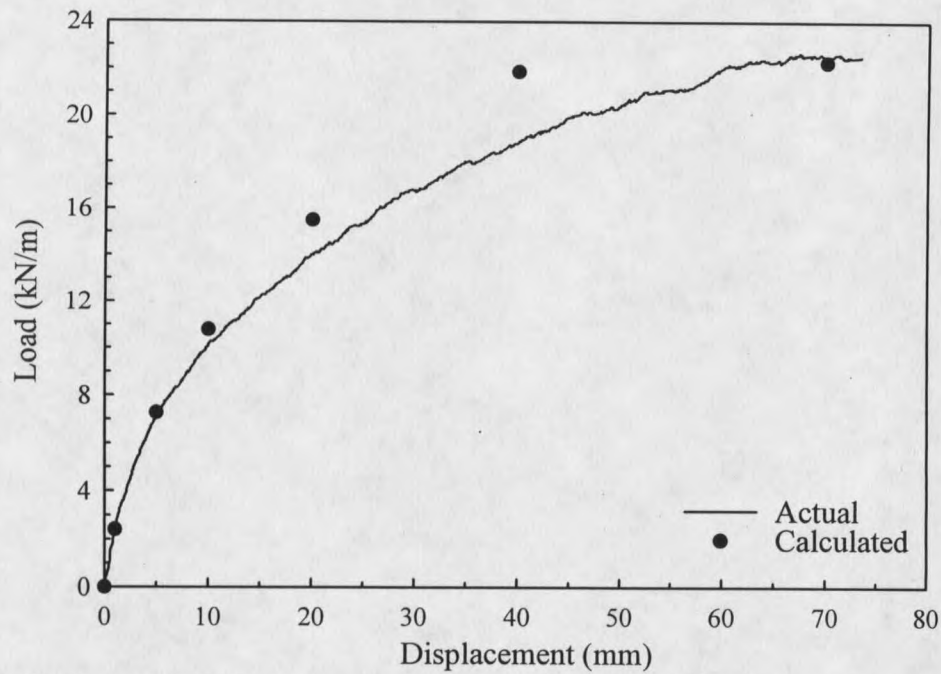


Figure C.23: Comparison between actual and predicted load/displacement curves for the geotextile pulled in the machine direction at $\sigma_n = 15$ kPa: gtmd-12.

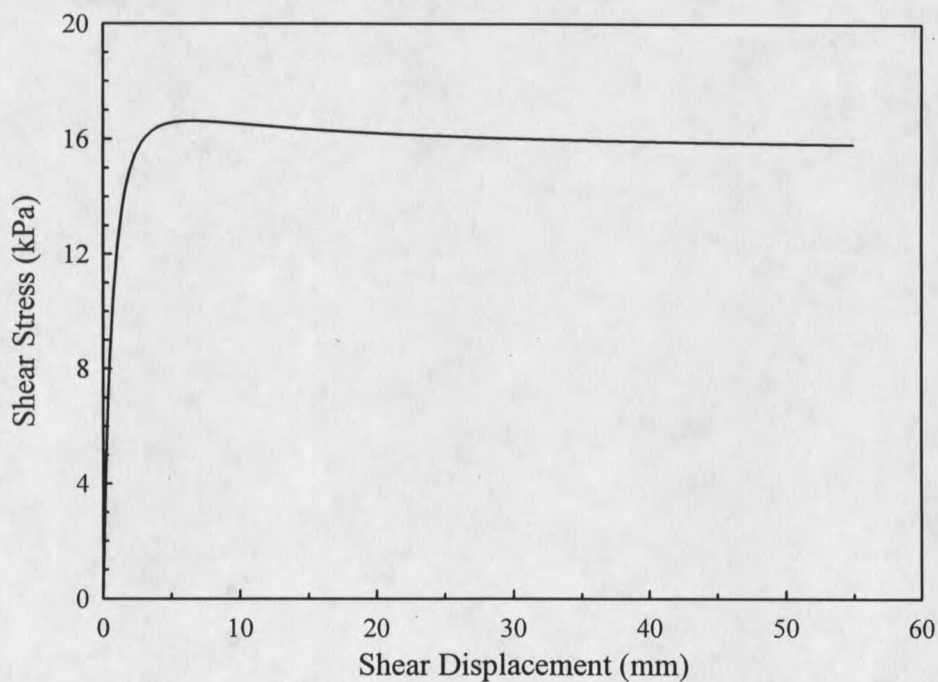


Figure C.24: Predicted shear stress versus shear displacement curve for the geotextile pulled in the machine direction at $\sigma_n = 15$ kPa: gtmd-12.

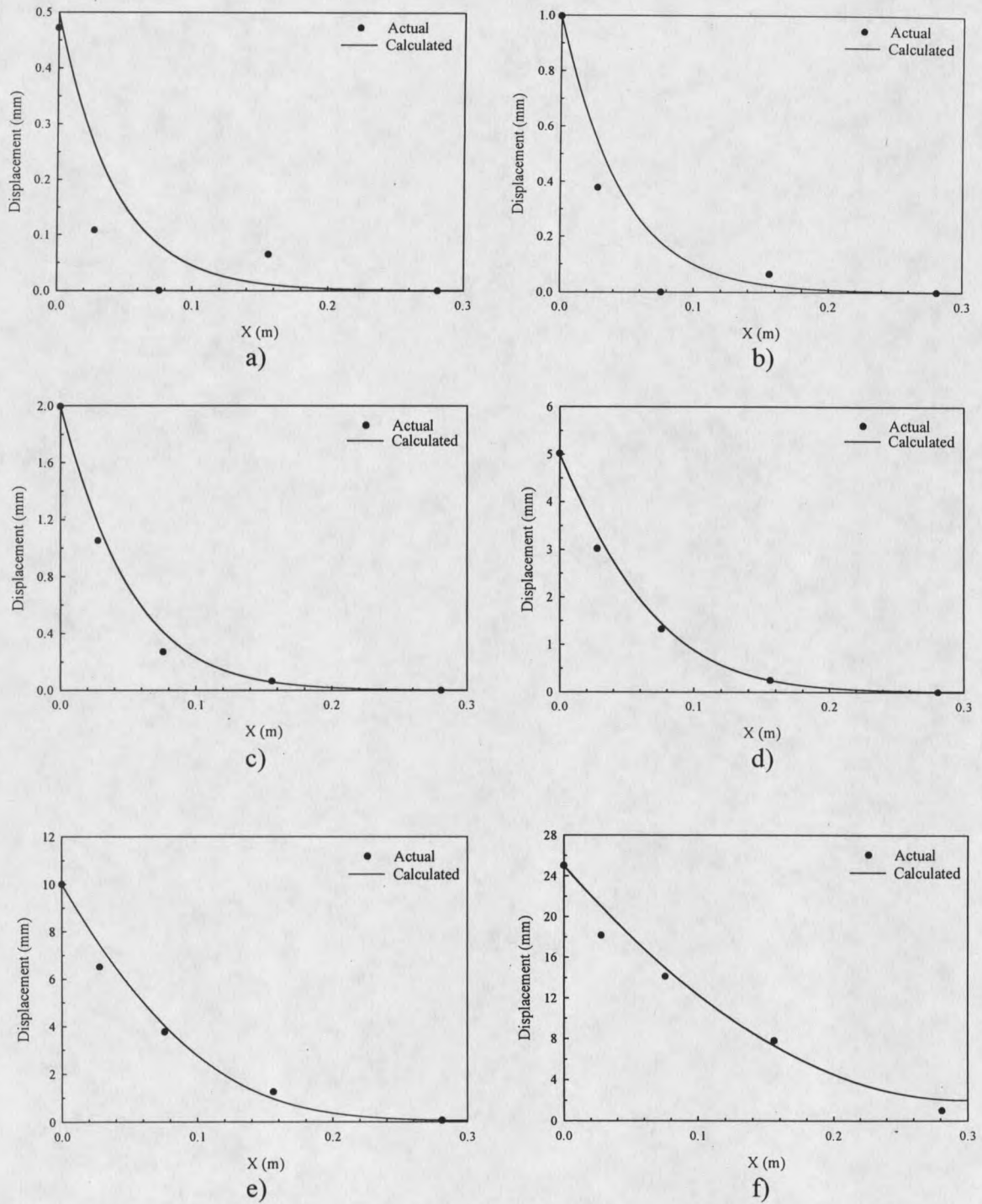


Figure C.25: Comparisons between actual and predicted displacement curves for the geotextile pulled in the machine direction at $\sigma_n = 35$ kPa: a) Level I, b) Level II, c) Level III, d) Level IV, e) Level V, f) Level VI: gtmd-13.

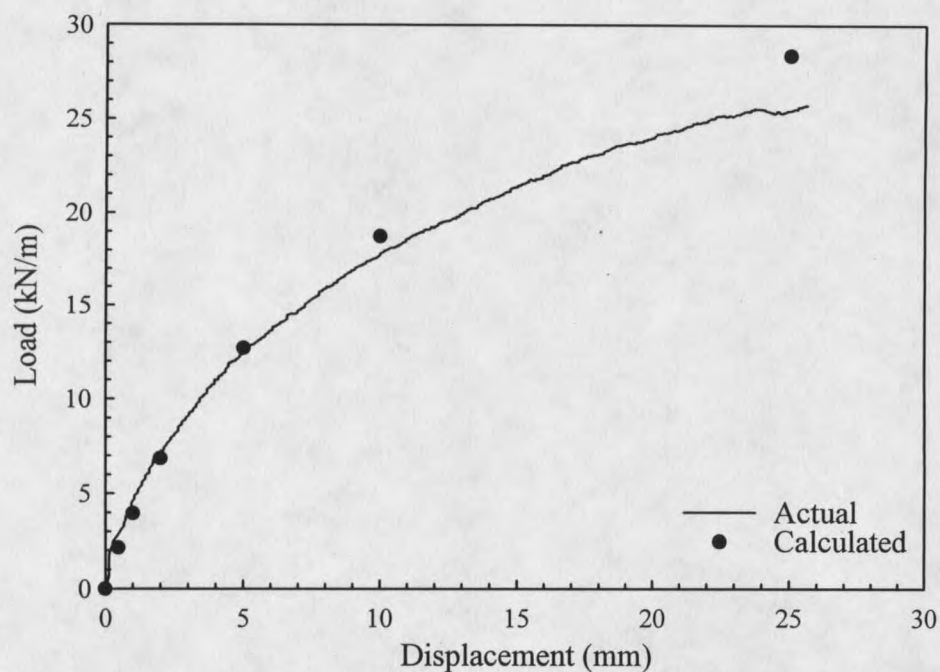


Figure C.26: Comparison between actual and predicted load/displacement curves for the geotextile pulled in the machine direction at $\sigma_n = 35$ kPa: gtmd-13.

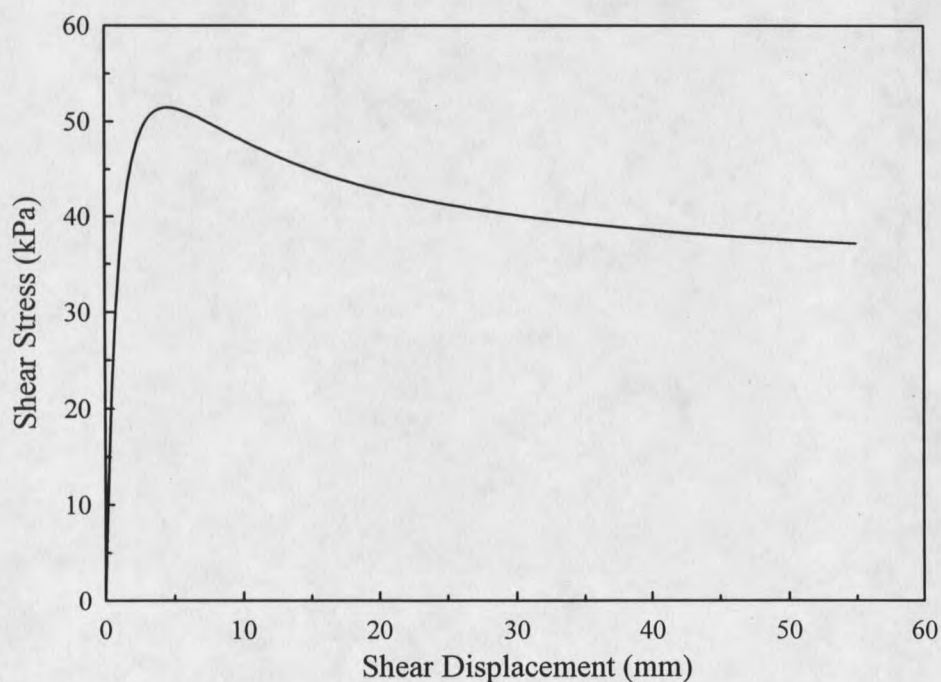


Figure C.27: Predicted shear stress versus shear displacement curve for the geotextile pulled in the machine direction at $\sigma_n = 35$ kPa: gtmd-13.

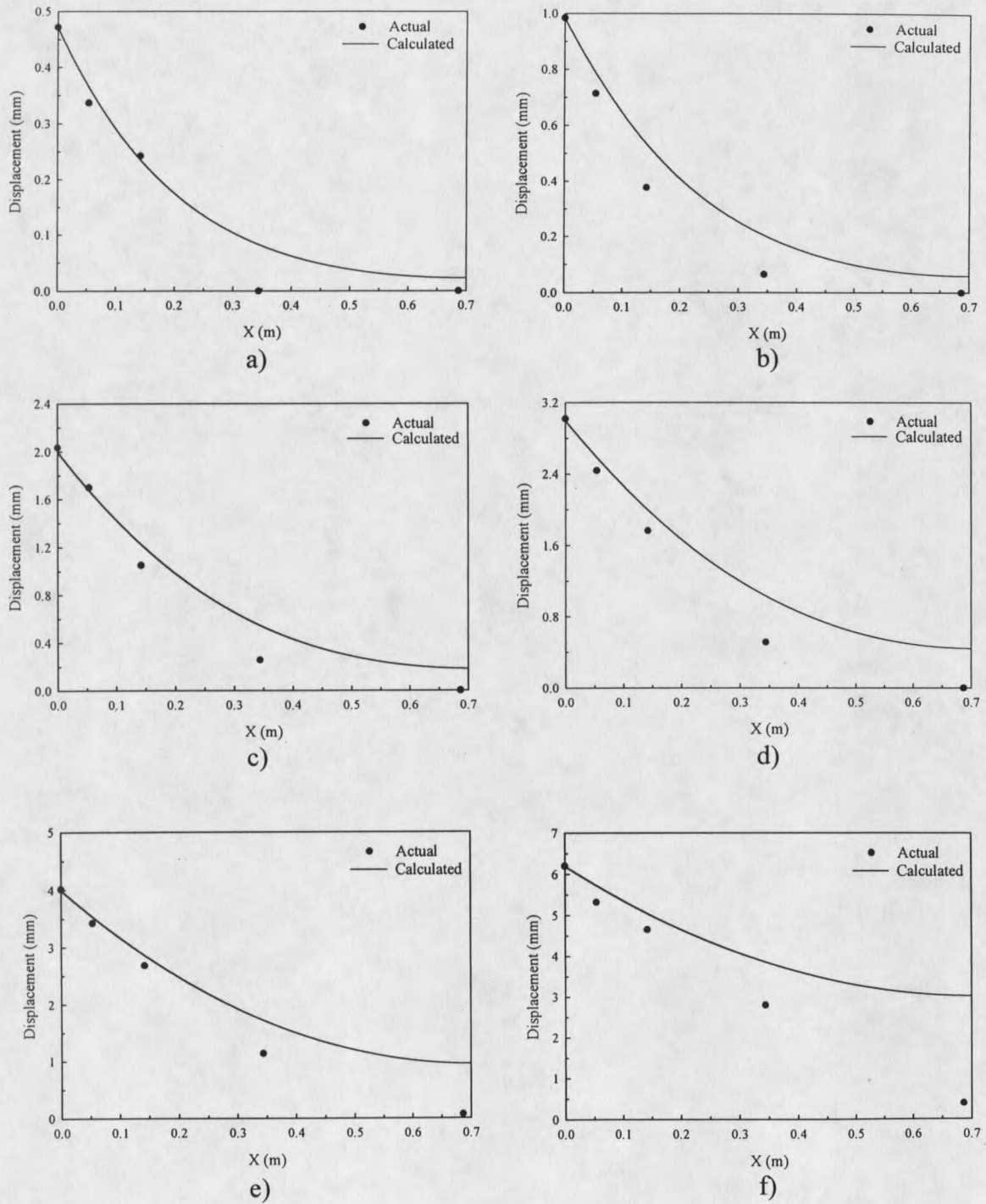


Figure C.28: Comparisons between actual and predicted displacement curves for the geotextile pulled in the cross-machine direction at $\sigma_n = 5$ kPa: a) Level I, b) Level II, c) Level III, d) Level IV, e) Level V, f) Level VI: gtxd-13.

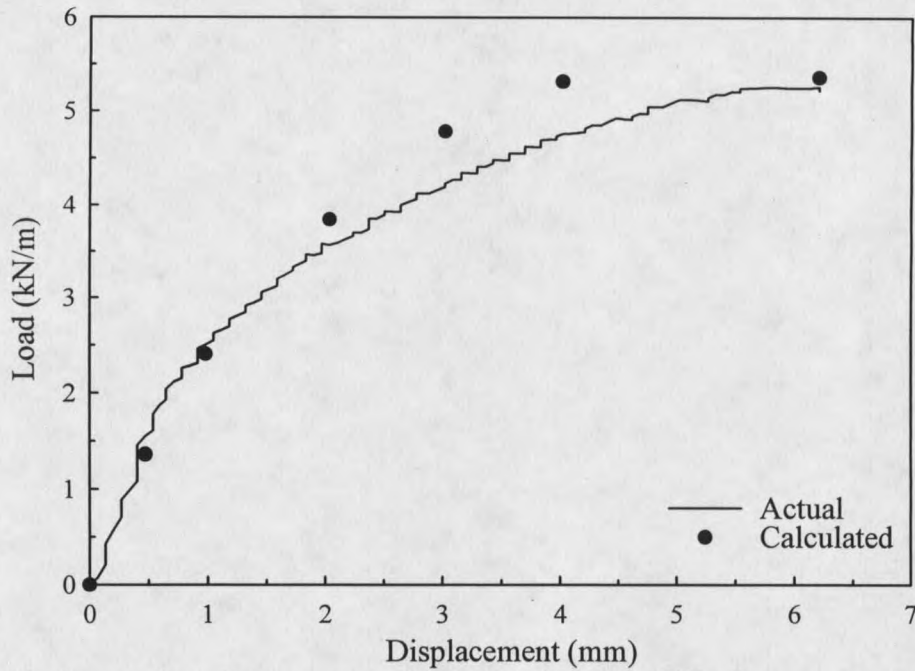


Figure C.29: Comparison between actual and predicted load/displacement curves for the geotextile pulled in the cross-machine direction at $\sigma_n = 5$ kPa: gtxd-13.

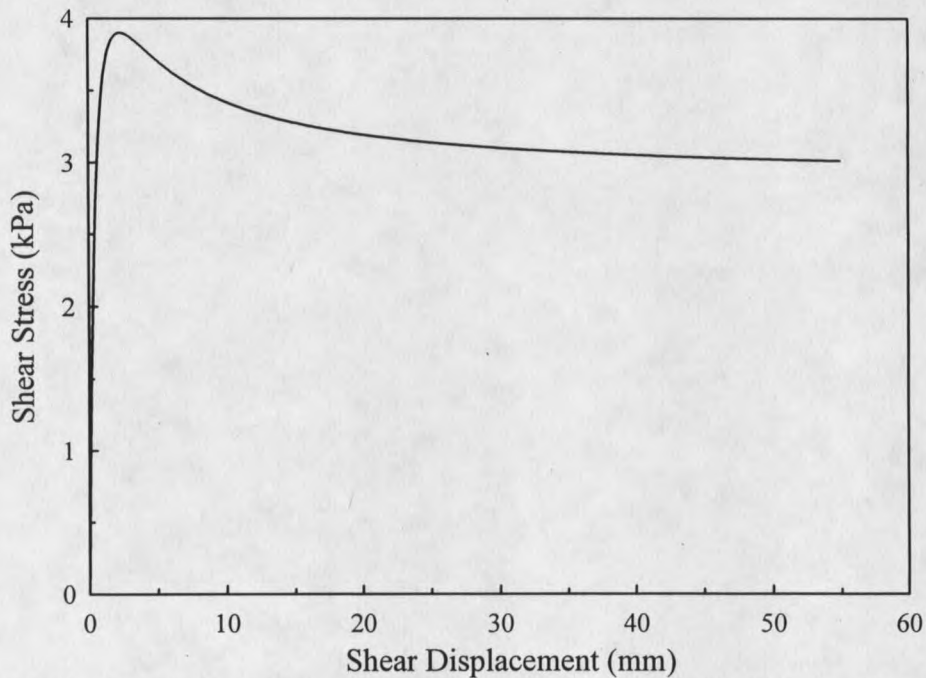


Figure C.30: Predicted shear stress versus shear displacement curve for the geotextile pulled in the cross-machine direction at $\sigma_n = 5$ kPa: gtxd-13.

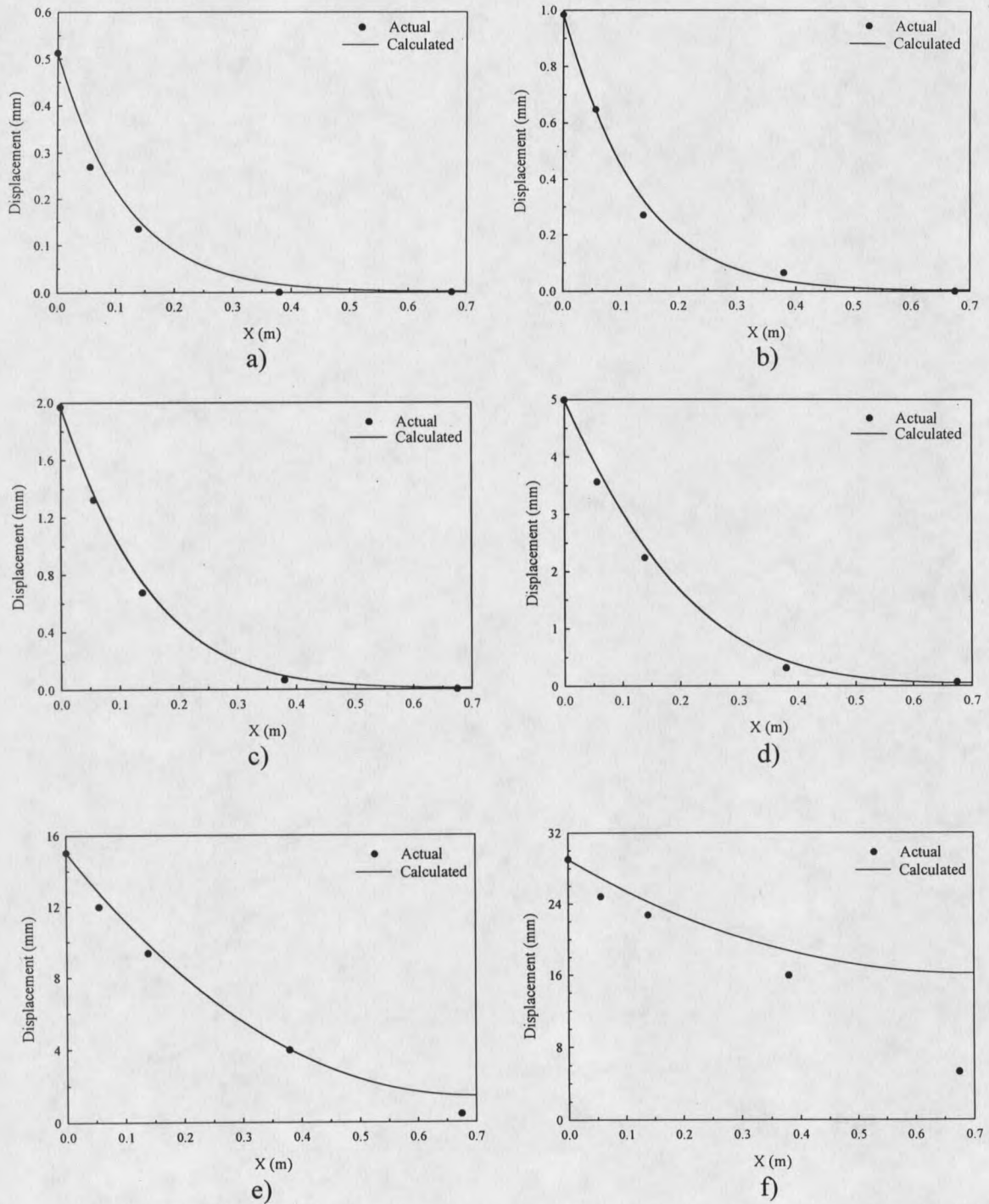


Figure C.31: Comparisons between actual and predicted displacement curves for the geotextile pulled in the cross-machine direction at $\sigma_n = 15$ kPa: a) Level I, b) Level II, c) Level III, d) Level IV, e) Level V, f) Level VI: gtxd-14.

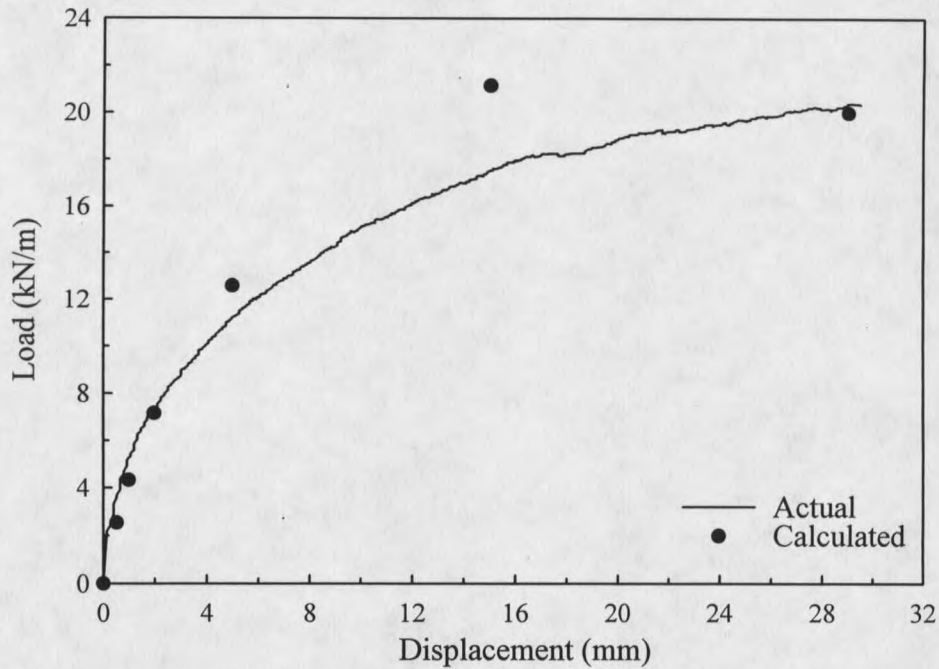


Figure C.32: Comparison between actual and predicted load/displacement curves for the geotextile pulled in the cross-machine direction at $\sigma_n = 15$ kPa: gtxd-14.

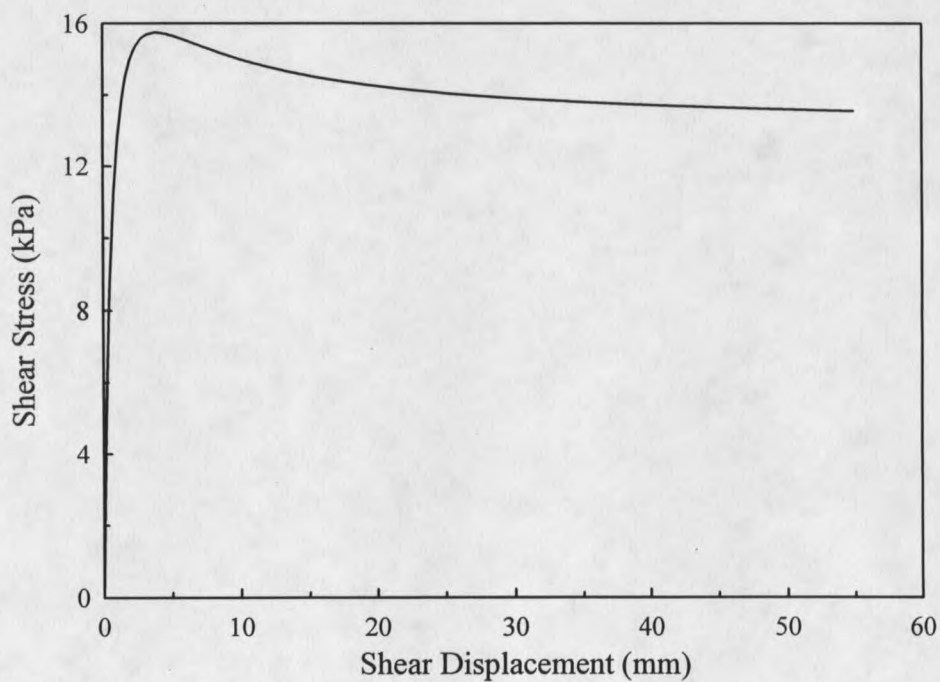


Figure C.33: Predicted shear stress versus shear displacement curve for the geotextile pulled in the cross-machine direction at $\sigma_n = 15$ kPa: gtxd-14.

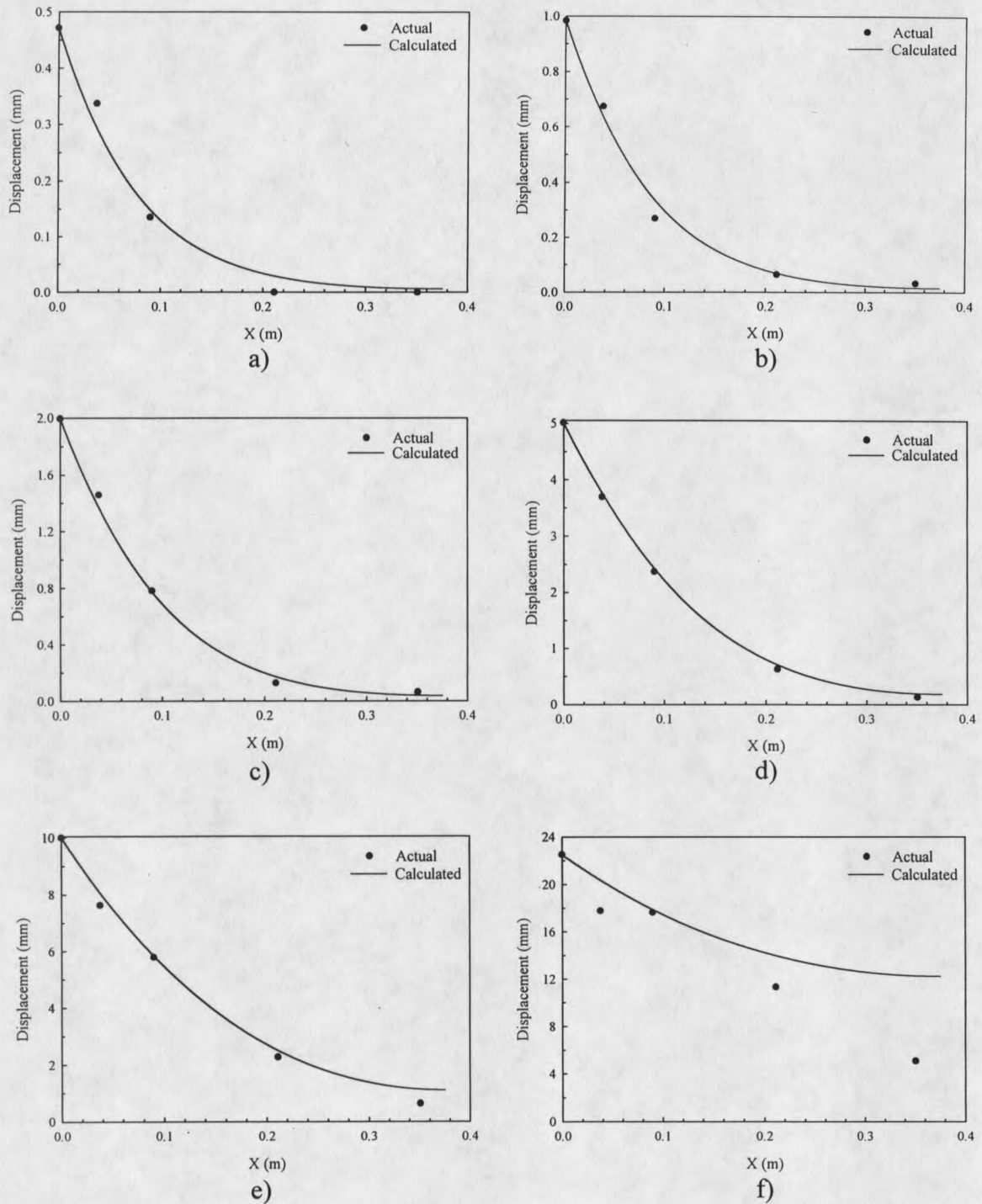


Figure C.34: Comparisons between actual and predicted displacement curves for the geotextile pulled in the cross-machine direction at $\sigma_n = 35$ kPa: a) Level I, b) Level II, c) Level III, d) Level IV, e) Level V, f) Level VI: gtxd-15.

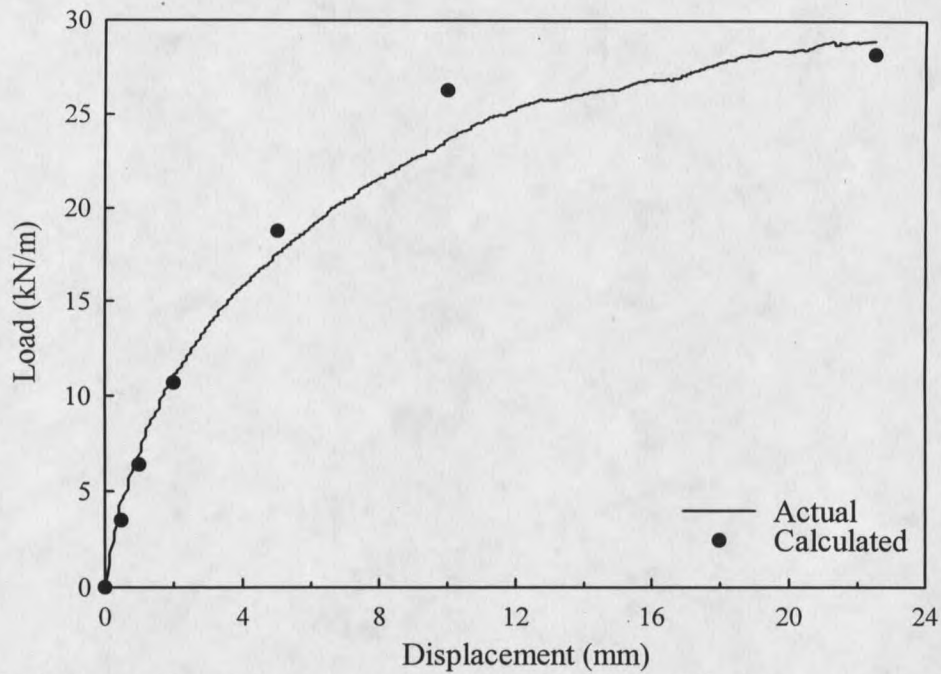


Figure C.35: Comparison between actual and predicted load/displacement curves for the geotextile pulled in the cross-machine direction at $\sigma_n = 35$ kPa: gtxd-15.

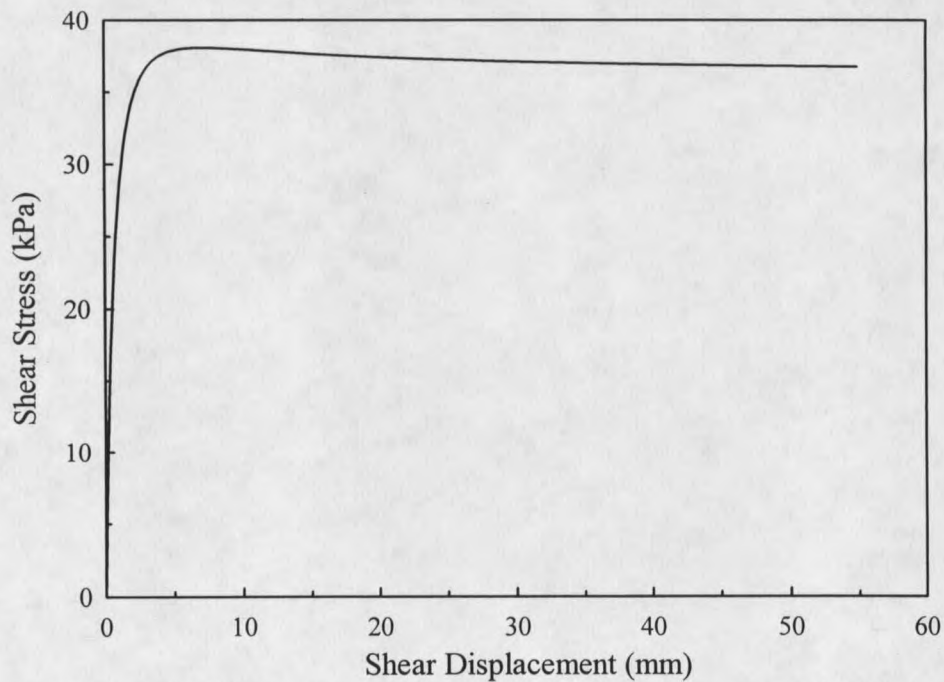


Figure C.36: Predicted shear stress versus shear displacement curve for the geotextile pulled in the cross-machine direction at $\sigma_n = 35$ kPa: gtxd-15.

APPENDIX D
LIS OF EQUIPMENT SOURCES

List of Equipment Sources

- Adhesive/Sealant -- Dow Corning 3145 RTV (used to bond strain gages to geotextile)
- Air pressure regulator -- Fairchild
- Automatic Binary Regulator (ABR™) -- Parker Pneumatic Products
- Biaxial Geogrid -- Tensar BX1100
- Biaxial Woven Geotextile -- Amoco 2006
- Double Reduction Worm Gear -- Hub City
- Extensometers -- Celesco Transducer Products Inc.
- Five-ton, machine screw actuator -- Duff Norton
- General-purpose Silicone Grease -- AGS
- High accuracy pressure gage -- Omega Engineering Inc.
- Loadcell -- Sensotec Transducers and Instruments
- Pneumatic Airbag (normal confinement) -- Aero Tech Laboratories Inc.
- Pneumatic Ball Vibrator -- ISKCO, Ltd.
- Pneumatic Cylinder -- Parker
- Scotch-Weld 2-part Epoxy/Adhesive #2216-- 3M (used to glue sample to sheet metal)
- System Controls/Data Acquisition -- LabVIEW

