ON THE COMPARATIVE BEHAVIOUR OF GEOGRIDS IN TENSION AND PULLOUT

by

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We accept this thesis as conforming to the required standard

THE UNIVERSITY OF BRITISH COLUMBIA

October, 1997

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Department of **CIVIL ENGINEERING**

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ABSTRACT

Two principle mechanisms of resistance to imposed loads in geosynthetic reinforced soil walls are the tensile strength of the geosynthetics and soil – geosynthetic interface bond. Although both are treated separately in design, they are intimately related. Through soil – geosynthetic interaction lateral loads from soil are transferred to the reinforcement putting it into tension. A central theme to this study is the investigation of geosynthetic reinforcement in the small, in - service strain range of 0.50 % to 1.0 % typically found in field structures. This will also be referred to in the text as the “strain range of interest.”

Geotextiles and geogrids are two types of geosynthetics used for soil reinforcement; this study investigates the behaviour of geogrids exclusively. Three separate apparatuses were utilised to investigate the tensile and pullout characteristics of three commercially available geogrids. The geosynthetics represent two different types of geogrid: those of low and high junction strength. Unconfined tensile tests were carried out with an industrial (Instron) testing machine. Long-term creep pullout tests were performed with a modified pullout box, designed and built at the B.C. Ministry of Transportation and Highways’ laboratory in Victoria, B.C.. Constant rate of displacement pullout tests were carried out with a large scale pullout apparatus designed and built at the University of British Columbia in Vancouver, B.C..

Unconfined tensile tests investigated the load – strain characteristics of all three
geogrids and the load – time (relaxation) and strain – time (creep) characteristics of the high junction strength geogrid. Creep pullout tests, carried out at comparable loads to unconfined creep tests, allowed comparison of confined and unconfined creep behaviour.

Pullout tests were carried out to investigate the soil – geogrid interaction behaviour of the two geogrid types. The effects of increases in normal stress were also investigated. Results of unconfined tensile tests reveal that, in the strain range of interest, little significant difference exists in the load – strain behaviour of the different geogrid types, despite the dramatically different ultimate short term tensile strengths reported for the materials in specification guides. Unconfined creep tests reveal a limit of instability for a high density polyethylene geogrid of approximately 15 to 20 % strain. This finding is corroborated by other literature. Unconfined creep test data compare reasonably well with isochronous load – strain curves supplied by the manufacturer for the same material. Creep pullout tests carried out at comparable load levels to the in – isolation creep tests demonstrate that creep strain appears to be mitigated by confinement. Pullout tests reveal significant differences in the mobilisation of normalised pullout resistance.

There are two chief components to interaction between soil and geogrids: friction between the geogrid and soil and passive bearing of transverse members against the soil. Results of pullout tests point to significant differences in the mobilisation of interaction between the geogrids tested. These differences include the magnitude of normalised pullout and displacement necessary to mobilise it. Although under low confining stress these differences are pronounced, under relatively high normal stress they are small.
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LIST OF SYMBOLS

\[ A_R \] Planar area of a geogrid

\[ B \] Bearing member thickness

\[ C \] is the reinforcement effective unit perimeter; e.g., \( C = 2 \) for strips, grids, mesh and sheets; \( C = \pi \) for nails.

\[ c_u \] Coefficient of uniformity

\[ DI \] Degree of interference

\[ E_0 \] Original modulus of a polymer in tension

\[ E \] Stiffness of the polymer

\[ CRF \] Creep Reduction Factor

\[ F^* \] Pullout resistance (or friction-bearing interaction) factor

\[ f_m \] Partial material factor for the reinforcement

\[ f_{m1} \] Partial factor for geogrid material quality

\[ f_{m2} \] Partial factor for the extrapolation of long term creep data.

\[ f_{m3} \] Partial factor for susceptibility to damage

\[ f_{m4} \] Partial factor to take account of environmental conditions

\[ F_q \] Embedment (or surcharge) bearing capacity factor

\[ FC \] Construction damage factor of safety.

\[ FD \] Durability factor of safety.

\[ FS \] Overall factor of safety to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties and externally applied loads.
K  Ratio of the actual normal stress to the effective normal stress

Le  Embedment length in the resisting zone

LTDS  Long-Term Design Strength of a geogrid

M(t)  Relaxation modulus

p  Arbitrary load

p(t)  Time function of load

PB  Maximum pullout load for a grid with n bearing members

PR  Pullout resistance

Po  Maximum pullout load for an isolated bearing member

po  A constant load

RF  Combined reduction factor to account for potential long-term degradation due to installation damage, creep, and chemical/biological aging.

RFID  Strength reduction factor to account for installation damage to the reinforcement.

RFCR  Strength reduction factor to prevent long-term creep rupture of the reinforcement.

RFD  Strength reduction factor to prevent rupture of the reinforcement due to chemical and biological degradation

S  Spacing of bearing members

t  Time

T  Measured tensile strength

Ta  Long-term tensile strength which will not result in rupture of the reinforcement during the required design life.

Ta  Allowable tensile force per unit width of reinforcement, (kN/m)

TCR  Peak tensile creep rupture strength at an appropriate temperature

xix
T_{CS}  Average tensile strength based on creep strain considerations at an appropriate temperature

T_{D}  Design strength of the reinforcement temperature

T_{L}  Creep limit strength obtained from creep test results

T_{R}  Rupture strength of geogrid from manufacturer's literature (ASTM D4595)

T_s  Long-term tension capacity of the geosynthetic at a selected design strain (usually 5% or less).

T_{ult}  Ultimate tensile strength (MARV) of the reinforcement determined from wide width tensile tests

z  Height of soil above a geogrid specimen

\alpha_b  Structural geometric factor for passive resistance

\alpha_f  A scale effect correction factor in pullout (average shear/peak shear)

\delta  Angle of friction between the geogrid and soil

\epsilon  Strain

\phi  Angle of shearing friction in the soil at large strain

\phi_{ps}  Plain strain friction angle

\phi_{ds}  Direct shear friction angles

\gamma  Unit weight of a soil

\sigma  Standard deviation

\sigma'_b  Passive bearing stress acting on the geogrid

\sigma'_n  Vertical normal stress acting on the geogrid

\psi  Angle of dilation for a soil

\tau_{rel}  Relaxation time
\( \tau_o \) Retardation time

\( \tau_{av} \) Average shear stress developed along a geogrid during pullout

\( \tau_p \) Peak shear stress developed along a geogrid during pullout

\( \mu \) Mean of the reinforcement base strength

\( \mu^* \) Apparent friction coefficient for the specific reinforcement
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This work is for Shawn and Colleen

who, after all, pointed me down this long road.
INTRODUCTION

1.1 Use of Geosynthetics in Transportation

Geosynthetics are used in a broad range of applications for transportation infrastructure. Both temporary and permanent works have been constructed with the aid of geosynthetics. Their economy is demonstrated in ease of construction and their effectiveness in reinforcing soil. Base reinforcement of roads, reinforced soil over piles in poor ground and retaining walls are examples of the applications of geosynthetics for transportation. Geosynthetics are of primary importance in transportation infrastructure where the desire for low maintenance costs and long design lives combine to make reinforced soil a feasible option.

Since geosynthetics are relatively new compared to more traditional means of soil reinforcement and, because of the uncertainty associated with their long-term behaviour, considerable caution has been exercised by transportation regulators in constructing geosynthetic reinforced structures for high load applications. Examples of such applications are bridge abutments or retaining walls above a certain height. This has lead to height limitation policies for many transportation departments, both in Canada and the United States. The need is apparent for a rational approach to the determination of long-term strength and reliability of geosynthetic reinforced soil structures. This need has been met in part by a considerable body of research and by the movement towards refined estimates of the long-term behaviour of geosynthetics by regulatory bodies in North America.
Introduction

1.2 Current Design Practice

Design of geosynthetic reinforced soil requires an appreciation of a number of parameters, including time dependency of polymer stresses and strains, the effect of confinement on the load - strain properties of geosynthetics and soil - geosynthetic interaction. Geotextiles and geogrids are two types of geosynthetics used for soil reinforcement. They can be distinguished by the differences in their structure - geotextiles are planar sheets, whereas geogrids are a network of ribs and bars - and in how they interact with the soil. This work will focus on geogrids only.

The above parameters can be classified into two chief categories: tensile capacity of the geogrid and interaction capacity of the soil - geogrid composite. Current practice in North America and Europe characterises geogrid tensile capacity by a long-term design strength (LTDS). The long-term design strength is the allowable tensile capacity of the geogrid at the end of the design life of the structure. Geogrid strength is susceptible to a number of deleterious agents including environmental degradation, construction induced damage, time and material variability. The LTDS is computed from the ultimate short term strength which is then reduced by partial safety factors. There are differences between North American and European practices in the identification of partial factors. Both recognise the importance of environmental degradation and construction damage as strength reducers. Both also recognise the creep susceptibility of polymers. The philosophy behind the two approaches is different, however. Traditionally, in North America the long-term allowable design strength has been computed by applying a series of reduction factors accounting for creep and environmental factors to a short-term ultimate tensile strength. European practice relies on isochronous load-strain...
curves to estimate a long-term creep strength, based on the design life of the structure. This long-term creep strength is then used as a base strength to which reduction factors reflecting environmental and material factors as well as the uncertainty of extrapolating isochronous load-strain data. The extrapolation of graphical data from the test duration to the design life is applied. Partial factors in the United Kingdom (UK) are distinguished between those concerned with the geogrid material and those with the environment. A partial factor reflecting the variability of the material is also applied. There has been a movement in North America towards a more rational approach to the estimation of long-term design strength. The National Concrete and Masonry Association (NCMA) has published a design manual in 1995 that adopts elements of the European philosophy in assigning uncertainty to data extrapolation. Moreover, it also speaks to several methods of accelerating creep tests through high temperatures to garner a better understanding of the anticipated creep strain likely to occur in the field.

Soil-geogrid interaction is important in transferring the lateral stresses of the soil mass to tensile stress in the geosynthetic. In design of reinforced soil walls a failure wedge is assumed to pass through the reinforced soil, creating two zones, one active and another resistive. The geogrid layers pass through the failure surface into the resistive zone and anchor the failure wedge. Design practice must ensure that adequate embedment length of geosynthetic extends beyond the failure surface to ensure that the active zone is tied back.
1.3 Objectives

The objectives of this study are to:

1) Compare the load – strain behaviour of geogrids represented by three commercially available products.

2) Investigate and compare the load – strain – time behaviour of an HDPE geogrid in both confined and unconfined conditions.

3) Compare and contrast the pullout behaviour of three geogrids in the small strain range associated with field conditions.

In fulfilling these objectives the study will speak to two chief issues of concern within the BC Ministry of Transportation and Highways:

i) To investigate the feasibility of approving geogrid reinforced soil walls over 5 m in height.

ii) To characterise different geogrid types according to their mechanical properties and respective means of interacting with soil.
CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

In this chapter literature is reviewed with a particular emphasis on the load – strain – time behavior of polymeric geogrids and the pullout interaction characteristic of geogrids in cohesionless, granular media. An inventory of geosynthetic reinforced soil structures at or near 5 m in height in British Columbia (B.C.) as well as some in Washington State is presented in support of the feasibility of such structures. Current codes of practice in North America and the U.K. are also briefly summarized.

2.2 Geogrid Reinforced Soil Walls in B.C.

Geogrid reinforcement is used under a variety of circumstances. Transportation infrastructure often calls for steepened slopes, embankments, retaining walls and abutments. One of the aims of the current study is to investigate geogrid reinforced soil walls of substantial height and nearly vertical face inclination for the B.C. Ministry of Transportation and Highways (MOTH). Specifically, whether it is feasible, given the load – strain – time characteristics of polymeric geogrids, together with the design life of transportation infrastructure, to approve geogrid reinforced soil walls of substantial height. In lieu of a more rational approach the B.C. MOTH and many US Departments of Transportation (DOT) have placed a ceiling on the height of geogrid reinforced soil walls in order to limit the stress in the geogrid.
Chapter 2. Literature Review

To begin investigating this, an inventory was created of geogrid reinforced soil walls of 5 m height (the current height limitation for the B.C. MOTH) or greater and near vertical face inclination. Information was gathered from several sources including the B.C. MOTH, B.C. Ministry of Forests, geogrid manufacturers and an existing data base found in the literature, (McGown and Jarret, 1987). This inventory, presented in Tables 2.1a and b, demonstrates that walls greater than 5 m in height have been constructed in B.C. and elsewhere. The walls were indexed according to soil conditions of the foundation and reinforced soil, height, face inclination (batter), and age, all of which have an impact on the stress state in geogrid reinforcement. Three walls from Washington State were also included.

The tallest structure in the inventory is a wall located at Royal Roads Military College in Victoria, B.C. and is 15 m in total height, although it is not single vertical faced structure, but a series of 6 stepped walls. The tallest nearly vertical, single structure is a wall on Admirality Island which is 9.5 m high.

2.2.1 Geotechnical Properties

Geotechnical properties were subdivided into those of the backfill material, the retained soil and the native foundation. Largely granular backfill (75 mm select granular sub-base, granular pit run) was utilized as backfill, and in some cases angular rockfill or crushed basalt was used. These types of soils are appropriate for use in geogrid reinforced soil walls. The Canadian Foundation Engineering Manual (C.F.E.M.) recommends “borderline clean granular soils (less than 12% passing 75 microns).”
### Table 2.1a: Inventory of Selected Walls in British Columbia with Height Near or Over 5 m and Near Vertical Face. May, 1995

<table>
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<th>Age (years)</th>
<th>Height (m)</th>
<th>Face Inclination</th>
<th>Backslope Angle</th>
<th>Geotechnical Properties</th>
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<td></td>
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<td>backfill</td>
<td></td>
</tr>
<tr>
<td>2. Coalmont Rd.</td>
<td>5</td>
<td>5.8</td>
<td>1:8</td>
<td>6:1</td>
<td>sand poorly graded sand</td>
<td>Tensar SR1</td>
</tr>
<tr>
<td>3. Osoyoos</td>
<td>new</td>
<td>4.57</td>
<td>1:8</td>
<td>0.75:1</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>4. Deep Cove</td>
<td>1</td>
<td>4.5</td>
<td>1:6</td>
<td>NP</td>
<td>sand poorly graded sand</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>5. Kelowna, BC</td>
<td>new</td>
<td>9</td>
<td>1:8</td>
<td>NP</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>6. Tsawwassen</td>
<td>1</td>
<td>5</td>
<td>1:1</td>
<td>NP</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>7. Steve Lake</td>
<td>1</td>
<td>4.5</td>
<td>1:10</td>
<td>NP</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>8. Woods Rd.</td>
<td>1</td>
<td>5.25</td>
<td>vertical</td>
<td>horizontal</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>9. Deep Cove</td>
<td>1</td>
<td>4.5</td>
<td>1:6</td>
<td>NP</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>10. Kelowna</td>
<td>new</td>
<td>9</td>
<td>1:8</td>
<td>NP</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>11. Victoria</td>
<td>7</td>
<td>6</td>
<td>NP</td>
<td>NP</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>12. Victoria</td>
<td>5</td>
<td>15.5*</td>
<td>NP</td>
<td>NP</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>13. Victoria</td>
<td>5</td>
<td>6</td>
<td>NP</td>
<td>NP</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>14. Admiralty Is.</td>
<td>&gt;5</td>
<td>9.5</td>
<td>NP</td>
<td>NP</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>15. Hazelton</td>
<td>&gt;5</td>
<td>6</td>
<td>NP</td>
<td>NP</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>16. Kamloops</td>
<td>7</td>
<td>8</td>
<td>NP</td>
<td>NP</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>17. Vancouver</td>
<td>5</td>
<td>6</td>
<td>NP</td>
<td>NP</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>18. Victoria</td>
<td>7</td>
<td>5.3</td>
<td>NP</td>
<td>NP</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>19. Coquitlam</td>
<td>&gt;5</td>
<td>7.6</td>
<td>NP</td>
<td>NP</td>
<td>sand poorly graded</td>
<td>Tensar SR2</td>
</tr>
<tr>
<td>20. Slocan</td>
<td>4.5</td>
<td>4.5</td>
<td>6:1 horizontal</td>
<td>pit run</td>
<td>concrete pads</td>
<td>Tensar SR2</td>
</tr>
</tbody>
</table>

* Six stepped walls for a total of 15.5 m of height.

### Table 2.1b: Structures in Selected Places in Washington State

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>Age (years)</th>
<th>Height (m)</th>
<th>Face Inclination</th>
<th>Backslope Angle</th>
<th>Geotechnical Properties</th>
<th>Reinforcement Used</th>
</tr>
</thead>
<tbody>
<tr>
<td>21. Shelton</td>
<td>20</td>
<td>0.9 - 6.1</td>
<td>vertical</td>
<td>NP</td>
<td>uniform crushed basalt</td>
<td>Fibertex 200 and 400</td>
</tr>
<tr>
<td>22. Gilford</td>
<td>16</td>
<td>6.7</td>
<td>NP</td>
<td>NP</td>
<td>uniform crushed basalt</td>
<td>Fibertex 200</td>
</tr>
<tr>
<td>Pinchot Nat'L Park</td>
<td>8</td>
<td>4.6 - 6.1</td>
<td>1:1</td>
<td>NP</td>
<td>uniform crushed basalt</td>
<td>Fibertex 200</td>
</tr>
<tr>
<td>23. Sultan</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>weathered rock</td>
<td>Fibertex 200</td>
</tr>
</tbody>
</table>
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The Canadian Foundation Engineering Manual (C.F.E.M.) also states that the “cost advantage of using on-site fill materials has led to the successful use of soils with large percentage of fines (e.g., > 50%, Simac et al. 1991).” The retained soil varied with location - sands, silts and clays are reported - since it was usually a native soil.

The foundation materials varied considerably from native ground to concrete starter pads. Since geogrid reinforced soil walls are internally stabilized gravity structures resisting retained earth pressures by their composite mass, the foundation below must be competent enough to resist the bearing stresses imposed by the walls. Further, if there are surcharges carried by the wall, these too, must be accommodated.

2.2.2 Age of walls

The age of walls in the inventory ranged from new (constructed in 1995) to 20 years old (geotextile reinforcement). The oldest geogrid reinforced wall was a one built in Kelowna, B.C. in 1987 using Tensar SR1 and SR2 reinforcement. Age is a significant parameter because of the time dependency of strength in polymeric materials. Also provided in the inventory for at least some of the walls was the purpose of the structure. This information, where it was available was given in the application column, referring to the application of the geosynthetic technology. Uses ranging from reinforced soil walls and steepened embankments to modifications on road ways were reported.

2.3 Tensile Behaviour of Polymers and Geogrids

Geogrids are made from polymers which are thermo - viscoelastic and as such their behavior is time, temperature, strain magnitude and strain rate dependent. Depending on the specific polymer used the effect of these factors will vary from geogrid to geogrid.
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When speaking of polymers in general the word stress will be used, whereas when geogrids are spoken of, stress will be replaced with load, referring to the load per meter width of geogrid. This causes polymers to exhibit a range of stress-strain behaviours. Depending on the type of material, a polymer may exhibit low stiffness and yield strength, but substantial strain-hardening, resulting in a tough material. Another type is a plastic response, characterized by a hyperbolic stress - strain curve with some strain-hardening. The latter is found with high density polyethylene geogrids. All of these combinations will be affected by rate of straining. Winding and Hiatt, 1961, characterize various stress-strain profiles for several types of polymeric materials. Many types of polymeric materials, including the ones used in this study, exhibit a decreasing rate of load increase with strain to a peak value followed by different post-peak behaviours, depending on the geogrid, under constant rate of displacement loading.

Relating the behaviour of polymers to the behaviour of polymeric geogrids has been investigated by some researchers (e.g. Baudonnel et al., 1982). For instance, in woven polyester geosynthetics slip between the fibers accounts for some of the strain noted in tensile tests. This suggests that the structure of geosynthetics has influence as much as the parent polymer. Related to this is a study (Austin et al., 1993) that found a difference in the calculated tensile modulus of polypropylene geogrids depending on the location of the extensometer, across junctions or between junctions. Another finding that is investigated in this study and which was found by Austin et al. (1993), may also relate to the structure of geogrids and that influence on measured strain. Depending on the gauge length over which extension is measured there have been found differences in the
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computed strains. It appears that the modulus of high junction strength polypropylene geogrids decreases over increasing gauge lengths.

2.3.1 Time effects

Elastic solids are characterized by completely and instantaneously recoverable strains imposed by loads. Conversely, perfectly plastic materials are characterized by the completely non recoverable strain they undergo. In both of these cases the time rate of change of the imposed stresses and strains is not considered. Viscoelastic materials, however, exhibit some of the properties of elastic and plastic materials. Moreover, the viscous nature of polymers implies that there exists a time dependent component to the resulting strains. Thus, the strain, $\varepsilon$, at any point in time, $t$, under the action of a stress, $p$ can be described in general as:

$$\varepsilon = \phi(p, t)$$  \hspace{1cm} [2.1]

where $\phi$ is some function.

The time dependent behavior of polymers has different ramifications. If a polymer is loaded with an increasing strain rate, $d\varepsilon/dt$, then it is generally found that the higher the strain rate the stiffer the response and the lower the ultimate rupture load (McGown et al., 1984; Van Krevelen, 1972). This sort of behavior is modeled with a velocity sensitive element such as a dashpot. Another type of time dependence is relaxation. This is the reduction of stiffness of a polymer under a constant strain with time. This can be described mathematically as:

$$p(t) = M(t)e$$  \hspace{1cm} [2.2]
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where

\[ p(t) \text{ is some function of stress} \]

\[ M(t) \text{ is the relaxation modulus} \]

\( \varepsilon \) is the strain.

The Maxwell model of viscoelastic behavior consisting of a spring and a dashpot in series is an approximation to relaxation behavior and can be expressed mathematically as:

\[ \sigma(t) = E_0 \exp(-t/\tau_1) \]  \[2.3\]

where

\( E_0 \) is the original modulus,

\( \sigma(t) \) is stress as a function of time,

\( \tau_1 \) is the relaxation time in which \( \sigma \) is reduced to a fraction of its original value.

A third type of time dependent behavior is creep. This is characterized by a constant load resulting in a steady accumulation of strain with time at a decreasing rate (provided no rupture occurs). It is described by the Voigt model, which consists of a spring and dashpot in parallel. Expressed mathematically,

\[ \varepsilon = \frac{P_0}{E} \left[ 1 - \exp\left( -\frac{t}{\tau_0} \right) \right] \]  \[2.4\]

where

\( P_0 \) is a constant stress

\( E \) is the stiffness of the material

\( \tau_0 \) is the retardation time.
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Both creep and relaxation happen simultaneously in polymers and for linear viscoelastic materials they can be related mathematically as inverses of each other (Williams, 1980).

Creep of polymers is characterized by three distinct regions: primary creep, secondary creep and tertiary creep, or creep rupture. Primary creep is associated with initial loading and is characterized by rapidly decreasing strain rate with time. Secondary creep is a plateau region with continuing strain rate decrease to a stable or asymptotic behavior if insufficient load to generate rupture is applied or, a gradual segue into tertiary creep which is associated with increasing strain rate with time. Strain versus time plotted in semi – log space demonstrate the three regions. It has been found, (Andrawes et al., 1986) that there exists a unique strain at which at least one polymer – HDPE - becomes unstable and begins the onset of tertiary creep.

It is convenient to represent the load – strain – time behavior of polymeric geogrids graphically in what is referred to as isochronous load – strain curves. Such a plot presents information on the accumulated strain under a given load with time. The result is a family of curves which trace the tensile load – strain relationships of geogrids over time. These were presented for Tensar geogrids by McGown et al. in 1984 and are commonly generated by manufacturers. Figure 2.1 shows isochronous load – strain curves for Tensar UX 1500 from the manufacturer.

2.4 Soil Geogrid Interaction

Soil geogrid interaction and the resultant bond set up between the two can be investigated effectively by the pullout test. Pullout resistance of a geosynthetic refers to the displacement, elongation and concurrent force development of a geogrid within a
surrounding soil mass under an imposed tensile force. Several factors influence pullout behavior both in the laboratory and the field. Among the most significant of these are the frictional and passive bearing stresses developed through relative displacement between the geosynthetic and soil. These stresses are described in terms of soil-geosynthetic interaction relationships which attempt to account for the geometry and surface characteristics of a geosynthetic. Other factors which influence pullout behavior are the tensile stiffness, or extensibility of a geosynthetic, the type of soil and the normal stress under which a geogrid rests. Each of these factors is described below along with their influence on pullout behavior.

2.4.1 Soil - Geosynthetic Interaction Relationships

Soil - geosynthetic interaction can be described in terms of two contributing factors: interface friction and passive bearing. Interface friction refers to the frictional bond set up between the planar surfaces of the geogrid and the soil immediately surrounding the grid. It should also be noted that interlock of soil particles and a rough textured surface of a geosynthetic is also identified as a potential means of resistance to pullout. Passive bearing occurs at the transverse bars of the grid (see figure 2.2) on the surfaces orthogonal to the direction of pullout. The frictional contribution to pullout can be described by the following expression derived by Jewell et al., 1984:

$$P_R = 2A_R \alpha_s \gamma z \tan \delta$$  \[2.5\]

where $P_R$ is the pullout resistance, $A_R$ is the area of the geogrid, $\alpha_s$ is the fraction of the grid area which is solid, $\gamma z$ is the unit weight of the soil multiplied by the height of soil
above the geogrid specimen (the normal stress on the surface of the geogrid) and $\delta$ is the angle of friction between the geogrid and soil. The equation is multiplied by 2 to account for the two sides of the grid, each of which develops similar bonds.

The passive bearing component of pullout bond was first described in terms of the ratio of the bearing stress on a lateral bearing member, $\sigma_b^\prime$, to the normal (vertical) stress, $\sigma_n^\prime$, acting on the geogrid specimen (see figure 2.3). Traditionally, the bearing component has been thought of as analogous to the bearing stress conditions in deep foundations. Alternatively, it can be thought of as analogous to the conditions on embedded anchors. Jewell et al., 1984 have derived an expression which provides a lower bound to the results of stress ratio on embedded anchors by Rowe and Davis (1982):

$$\frac{\sigma_b^\prime}{\sigma_n^\prime} = e^{\left(\frac{\pi}{2} + \phi\right)\tan \phi} \tan \left(\frac{\pi}{4} + \frac{\phi}{2}\right)$$

[2.6]

where $\phi$ is the friction angle of the soil.

An upper limit to the stress ratio is ratio found by considering the stress field created by a footing (Prandtl, 1921; Reissner, 1923) and reported by Jewell in his 1984 paper. Thus:

$$\frac{\sigma_b^\prime}{\sigma_n^\prime} = e^{\pi \tan \phi} \tan \left(\frac{\pi}{4} + \frac{\phi}{2}\right)$$

[2.7]

Jewell et al. then present their expression for bearing stress ratio as:
where $\alpha_b$ is the fraction of the geogrid area available for bearing, $B$ is the bearing member thickness and $S$ is the spacing of bearing members. The combined influences of friction and bearing on pullout resistance can be expressed by an expression derived by Jewell et al., 1984:

$$f_{bearing} = \left(\sigma'_b/\sigma'_n\right)(B/S)(\alpha_b/2 \tan \phi)$$

[2.8]

$$f_b = \alpha_s \cdot \frac{\tan \delta}{\tan \phi} + \frac{\sigma'_b}{\sigma'_n} \frac{B}{S} \frac{\alpha_b}{2 \tan \phi}$$

[2.9]

where

$f_b$ is the combined friction and bearing factor

$\alpha_s$ is the fraction of reinforcement surface area that resists direct shear.

$\sigma'_b$ is the passive bearing stress acting on the geogrid

$\sigma'_n$ is the vertical normal stress acting on the geogrid

$\alpha_b$ is a structural geometric factor for passive resistance

$B$ is the thickness of bearing element

$S$ is the spacing of bearing elements

$\delta$ is the angle of interface friction between geogrid and soil

$\phi$ is the angle of shearing friction in the soil at large strain.
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Equation 2.9 has combined both the frictional and passive bearing factors into a single interaction factor, \( f_{B} \), while also accounting for the influence of geometry (see figure 2.3). The first term on the right side is the friction factor and the second the bearing factor.

An important finding was that transverse bar movement through soil could lead to an "interference" with the development of passive stresses (Palmeira, 1987). A theoretical maximum value for the contribution of passive bearing to pullout, based on the contribution of one transverse member multiplied by the number of transverse members in a given sample, was derived by Palmeira. The theoretical value is compared with the actual value of pullout resistance, thus:

\[
P_{p} = nP_{0}
\]

[2.10]

where

\( DI = 1 - \frac{P_{B}}{nP_{0}} \)

Christopher et. al., 1990 derived an expression for pullout resistance which takes into account a unit perimeter

\[
P_{r} = F^{*} \alpha_{f} \sigma'_{n} L_{c} C
\]

[2.11]

where
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$L_e$ is the embedment or adherence length in the resisting zone behind the failure surface.

$C$ is the reinforcement effective unit perimeter; e.g., $C = 2$ for strips, grids, mesh and sheets; $C = \pi$ for nails.

$F^*$ is the pullout resistance (or friction-bearing interaction) factor

$\alpha_f$ is a scale effect correction factor $\tau_{av}/\tau_p$ which is the ratio of the average to peak interface lateral shear stresses mobilized along the reinforcement.

$\sigma'_n$ is the effective vertical stress at the soil reinforcement interfaces.

Moreover, $F^*$ can be estimated using the general equation:

$$F^* = \text{Passive Resistance} + \text{Frictional Resistance}$$

$$= F_q \alpha_B + K \mu^* \alpha_f \quad [2.12]$$

where

$F_q$ = the embedment (or surcharge) bearing capacity factor

$\alpha_b$ = a structural geometric factor for passive resistance

$K$ = the ratio of the actual normal stress to the effective normal stress

$\mu^*$ = an apparent friction coefficient for the specific reinforcement

$\alpha_f$ = a structural geometric factor for frictional resistance
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2.4.2 Factors Effecting Pullout

A number of factors affect the behaviour of geogrids in pullout. The relationships above demonstrate the influence of confining stresses, friction, and the structure of the geogrid. In addition, soil type and particle size also have an influence.

2.4.2.1 Confining Stress

The observation that pullout resistance tends to increase with increasing confining stress has been stated directly in the literature study, (e.g. Lentz and Pyatt, 1987; Chang et. al. 1995). Chang found that the effect of increasing confining stress (in this case normal stress) was dependent on the gradation of the soil. Ottawa sand of very uniform gradation did not experience a significant increase in pullout resistance. Conversely, soil that was well-graded (25% fines) did exhibit an increase in proportion to the increase in confining stress. The reason given for the increase was a greater ability for the soil to mobilize passive bearing owing to a denser configuration of the soil as a result of the presence of fine particles. Two other studies have demonstrated the effect of normal stress on pullout resistance (Fannin and Raju, 1992 and Raju, 1995). Fannin and Raju carried out pullout tests on both geogrids and geomembranes under varying vertical stresses. A characteristic response of the pullout resistance was for it to reach a peak and then either reduce to, or approach, a constant. At higher stresses the resistance approaches a peak asymptotically at very large displacements. For these studies the soil used was a uniformly graded ($C_u = 1.5$) and a grain diameter varying from 0.1 to 2.0 mm, however an increase in pullout resistance was still noted.
2.4.2.2 Soil Type and Particle Size

Sarsby, (1985) made a valuable contribution to the literature in investigating the relative influence of aperture size to particle size in geogrid reinforcement. He concluded that the optimum ratio of aperture diameter to average particle size should be approximately 3.5.

Mobilized pullout resistance is dependent on the internal angle of friction for both the soil-geosynthetic and the bearing components of pullout resistance. Bauer et. al., (1991) compared the performance of a uniaxial geogrid with three soil types: a coarse sand with rounded particles, a well-graded crushed limestone aggregate and a silty clay. Also, a test was carried out with the coarse sand below and the clay above the geogrid. The geogrid was a Tensar TJX 1600. It was found that the crushed aggregate yielded the highest resistance followed by the sand and sand/clay samples. The lowest values were for the clay. These results agree with the discussion above on the influence of the internal angle of friction, \( \phi \), on pullout capacity. The authors suggest that most of the resistance provided by the clay resulted from surface adhesion and friction between the clay particles and the geogrid.

Another study which examined the effect of soil type on pullout resistance was carried out by Chang et. al. (1995). In this study two high density polyethylene geogrids were tested in pullout in three different soil types: a coarse uniformly graded sand, a well-graded sand and a weathered mudstone (clay). Similar findings to Bauer et. al. (1991) were reported where higher ultimate resistance was observed in the sands over the clay. Chang et. al. seem observed that the well-graded soil yielded lower initial resistance than the uniform sand up to about 17 mm of pullout displacement (displacement of front or
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clamped end) and even lower than the clay soil up to 2 to 4 mm of displacement. The authors contend that the well-graded coarse sand had a lower initial resistance because the coarse particles "... may slide easily when pressed by the transverse ribs." (Chang, 1995). However, once enough displacement took place the soil would assume a denser configuration when the larger particles were packed against the smaller ones, thus resulting in a higher ultimate resistance.

2.4.2.3 Shear Stress Distribution

The stress distribution along a geosynthetic under pullout load is not uniform. This result is reported in at least three recent studies, however, it shown quite dramatically in one (Chan et. al., 1993). Using a finite element method of analysis the mobilized shear stress was found to be a maximum at the front of the soil sample - closest to the clamped end - decaying to a nearly constant finite value at the embedded end. Shear stress distribution was also seen to vary with displacement approaching a relatively uniform value with very large displacements. The authors caution that the use of a constant coefficient of interaction poorly models the actual behavior of the reinforcement under pullout. The degree to which the actual behavior and that calculated assuming uniform shear stress depends on the relative stiffness of the soil - geogrid interface to the geogrid stiffness itself. The lower the stiffness of the reinforcement the greater the discrepancy. For a perfectly rigid strip, the shear stress distribution would be uniform (assuming uniform soil conditions) since the displacement at the any point along the strip must be the same.
2.5 Confined Properties of Geosynthetics

There has been discussion in the literature as to whether the load-strain properties of geosynthetics vary with confinement (e.g. Min et. al., 1995; Juran et. al., 1989). It is evident that confinement within granular media increases the tensile stiffness of nonwoven geotextiles (Boyle, et. al., 1996; Gourc, 1990). This is due to the fact that confining stress increases the fiber to fiber friction in the nonwoven geotextile resulting in a higher tensile stiffness. The increase in tensile modulus in this case is due to the structure of the material. However, Boyle, et. al., 1996 also found that woven geotextiles, different from nonwoven in that they are more dimensionally stable, did not exhibit any increase in tensile modulus with confinement. Ballegeer et al. (1993), found significant increases in strength and stiffness for nonwoven and mild increases for woven geotextiles with confinement. Min et. al., 1995 found that strain rate in confined and unconfined creep tests was an intrinsic property of geogrids, not effected significantly by confinement.

2.6 Review of Current Design Practice

Current design practice for geosynthetic reinforced soil structures is briefly reviewed as pertains to the calculation of the Long-Term Design Strength (LTDS) of the geogrid reinforcement, the allowable post construction strains for structures and pullout resistance of geosynthetics. Two approaches to the calculation of LTDS are presented. Both account for creep, installation damage, environmental damage (chemical and biological degradation) and material uncertainty, the overall uncertainties associated with construction as well as the ramifications of failure. Differences between the two methods
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arise in the method of calculation of the long-term design strength of the geogrid. There are also differences in the approach to pullout resistance computation. The two approaches differ in their origin as well; one is based on FHWA 1990 guidelines originating in North America while the other is based on the European practice found in the British Standard BS 8006.

2.6.1 FHWA, 1990

The Federal Highways Administration produced a two-volume design guideline titled: Reinforced Soil Structures. Volume I is subtitled Design and Construction Guidelines. Within this volume guidelines for the design of various reinforced soil structures are laid out along with a suggested method for calculating the LTDS.

In both the North American and European codes the long-term design strength is arrived at by starting with a “base Strength” from a standard tensile test and then reducing this by a series of reduction factors accounting for the effects of creep, construction or installation damage and material variability. Thus,

\[
\text{LTDS} = \frac{\text{Base Strength}}{\text{Product of Safety Factors}} \quad [2.13]
\]

Specifically, in the FHWA guidelines, the long-term design strength is based on a short-term index tensile test which is reduced by partial factors of safety to account for various strength reducing effects. The LTDS is the minimum strength the geogrid must have at the end of the design life of the structure in order to preclude geogrid rupture or significant strain. Expressed mathematically,
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\[ T_a = \frac{T_{ult}}{FD} \cdot \left( \frac{CRF}{FC} \right) \cdot FS \leq T_s \]  

where:

- \( T_a \) = allowable tensile force per unit width of reinforcement, (kN/m).
- \( T_s \) = long-term tension capacity of the geosynthetic at a selected design strain (usually 5% or less).
- \( T_{ult} \) = ultimate geosynthetic tensile strength (kN/m).
- \( FD \) = Durability factor of safety.
- \( FC \) = Construction damage factor of safety.
- \( FS \) = Overall factor of safety to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties and externally applied loads.
- \( CRF \) = Creep Reduction Factor

The long-term tension capacity is an upper bound placed on the available strength of the geosynthetic to limit the strain in the reinforced structure to serviceable levels. The creep behavior of geosynthetics and the resultant reduction in strength is accounted for by the Creep Reduction Factor. This factor is the ratio of the ultimate tensile strength to the creep limit strength, thus:

\[ CRF = \frac{T_L}{T_{ult}} \]  

[2.15]

\( T_{ult} \) is the ultimate short term tensile strength as outlined above and \( T_L \) is the creep limit strength obtained from creep test results. \( T_{ult} \) is obtained from the American Society for...
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Materials and Testing (ASTM) standard test method D - 4595: Tensile Properties of Geotextiles by the Wide-Width Strip Method\(^1\). Within North America geosynthetic strength is based on a Minimum Average Roll Value which is defined as the strength above which at least 95 % of the specimens tested achieve. The FHWA guidelines do not provide details as to limitations or stipulations surrounding \(T_L\), however, they do defer to Task Force 27 guidelines in providing suggested minimum CRF values in lieu of product specific values (29). Moreover, as a preface to the calculation of LTDS it is mentioned that a more complex method of calculation, which considers the allowable elongation, creep potential and possible strength degradation is available (Bonaparte and Berg).

FHWA guidelines also provide direction on soil geosynthetic interaction. It is required that the design be adequate to resist pullout of the reinforcement at working tensile force with a specified factor of safety. Moreover, the relative soil geosynthetic displacement necessary to mobilize tensile force must not be greater than the allowable post construction displacements. Finally, the maximum pullout load must be less than the long-term creep strength of the geosynthetic.

2.6.2 UK Practice

Practice in the UK is somewhat different than that in North America as outlined in the FHWA guidelines. The British Standard (BS) 8006, *Code of Practice for Strengthened/Reinforced soils and Other Fills*, outlines the recommended practice for the design of mechanically stabilized soil structures. The approach in BS 8006 to the design of reinforced structures is through limit states design. Loads imposed on structures are

\(^1\) This method may also be used for geogrids.
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factored up, while the resistances, or material strengths, are factored down, similar to structural engineering codes in Canada. The result is that if the appropriate factors are applied to the loads and the material strengths a limit state should be precluded. A serviceability limit state, characterized by such strains in a structure that undue maintenance is required, is also avoided by placing tighter limitations on the strengths. The factors of safety are combined in a partial material factor, $f_m$, which accounts for the material and environmental effects, plus the uncertainty associated with test data.

BS 8006 takes as an unfactored base strength the lesser of either the peak tensile creep rupture strength at an appropriate temperature, $T_{CR}$, or the average tensile strength based on creep strain considerations at an appropriate temperature, $T_{CS}$. Expressed mathematically:

$$T_D = \frac{T_{CR}}{f_m} \text{ or } \frac{T_{CS}}{f_m}$$

[2.16]

where

$T_D$ is the design strength of the reinforcement

$f_m$ is the partial material factor for the reinforcement.

$T_{CR}$ is the peak tensile creep rupture strength at an appropriate temperature

$T_{CS}$ is the average tensile strength based on creep strain considerations at an appropriate temperature.

The partial material factor $f_m$ is comprised of two sub-factors, $f_{m1}$ and $f_{m2}$, which in turn are further subdivided into additional categories. Sub-factor $f_{m1}$ relates to the intrinsic properties of the geosynthetic and $f_{m2}$ to the constructions and environmental effects on the strength. Table 2.2 defines the factors. Material factors $f_{m21}$ and $f_{m22}$ are directly
analogous to the FD and FC factors in the FHWA guidelines. The factor $f_{m1}$ is intended to account for the uncertainty associated with the manufacturing process. If no standard for specification or manufacture control on the strength of polymeric reinforcement exists for a given geosynthetic then BS 806 assigns one. Either a “characteristic” base strength – the 95th percentile value – or, a mean value can be used. The characteristic base strength in the BS 806 is directly analogous to the Minimum Average Roll Value specified in FHWA guidelines. If a mean value is specified to define the strength then the partial material factor $f_{m11}$ is computed as:

$$f_{m11} = 1 + \frac{1.64 \sigma}{\mu - 1.64 \sigma}$$

[2.17]

where $\sigma$ is the standard deviation and $\mu$ the mean of the reinforcement base strength.

**Table 2.2. Partial material safety factors (BS-8006).**

<table>
<thead>
<tr>
<th>Principal Factor</th>
<th>Component Factor</th>
<th>Intended Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{m1}$</td>
<td>$f_{m11}$</td>
<td>Manufacture - to cover possible reductions in the capacity of the material as a whole compared with the characteristic value deduced from the control test specimen and possible inaccuracies in the assessment of the resistance of a structural element resulting from modeling errors. Minimum value: 1.05</td>
</tr>
<tr>
<td>$f_{m12}$</td>
<td></td>
<td>Extrapolation of test data - to take account of the confidence of the long-term capacity assessment. This factor may vary with the required service life of the structure.</td>
</tr>
</tbody>
</table>
BS 8006 also accounts for soil geosynthetic interaction in the design of reinforced soil structures. To characterize the bond strength between soil and geosynthetic reinforcement, the British Standard recommends direct shear tests be carried out. It also accepts pullout tests for characterizing soil geosynthetic bond.

2.6.3 Washington State DOT Approval Requirements

The Washington State Department of Transportation (WASDOT) has published (1996) submission and approval requirements guidelines for the purpose of determining the fitness of geosynthetic products in reinforced soil structures.(57) The publication provides guidelines in determining the long-term design strength of geosynthetics as well as specifications for required laboratory data. The procedure for calculating the LTDS along with the laboratory data required are reviewed.
2.6.3.1 Long-Term Design Strength

The computation of LTDS is based on ensuring that the geosynthetic is capable of resisting the imposed loads over the design life without rupturing. The long-term tensile strength of the geosynthetic necessary to preclude rupture is calculated as follows:

\[ T_{ul} = \frac{T_{ult}}{RF} \]  

[2.18]

where,

- \( RF \) = RF_{ID} \times RF_{CR} \times RF_{D}
- \( T_{ul} \) = The long-term tensile strength which will not result in rupture of the reinforcement during the required design life.
- \( T_{ult} \) = the ultimate tensile strength (MARV) of the reinforcement determined from wide width tensile tests.
- \( RF \) = a combined reduction factor to account for potential long-term degradation due to installation damage, creep, and chemical/biological aging.
- \( RF_{ID} \) = a strength reduction factor to account for installation damage to the reinforcement.
- \( RF_{CR} \) = a strength reduction factor to prevent long-term creep rupture of the reinforcement.
- \( RF_{D} \) = a strength reduction factor to prevent rupture of the reinforcement due to chemical and biological degradation.

The similarities to the FHWA guidelines can be seen in that the approach is based on applying reduction factors to a short-term tensile strength. However, the WASDOT
Chapter 2. Literature Review

guidelines also require test data to demonstrate the long-term creep behaviour of prospective geosynthetics in a way that embraces some of the philosophy behind European practice. Two ways of assessing the long-term creep behaviour of geosynthetics is through creep-rupture and creep strain data. The former utilizes plots of rupture strength at varying load levels against time. In log-log space such a plot will be approximately linear with a negative slope, signifying the loss in strength of the geosynthetic with time. Such a plot may also contain a “knee”, a sudden down-turn in the plot signifying a ductile to brittle transition. The result is that the geosynthetic may fail at a load less than anticipated by an early trend in the data. Creep strain data utilizes data in the form of creep strain versus time in semi-log space. The requirements for serviceability limit state design require that the geosynthetic not strain more that a prescribed amount in the design life of the structure. This procedure requires the data to be extrapolated from the test duration to the design life of the structure. Such a procedure is adopted in Europe. WASDOT guidelines allow both procedures.

2.7 Summary and Research Needs

Literature has been selectively reviewed in a number of areas relevant to the current study. A brief inventory of field structures has been compiled to demonstrate that over a range of geometrical and geotechnical conditions walls of 5 m in height or greater have been constructed in British Columbia and Washington State. There is also a wide range in the age of the structures. Literature concerning the stress - strain properties of polymers and load - strain properties of geosynthetics was reviewed. Moreover, the relationship between the behaviour of a geosynthetic and that of the parent polymer was raised. The long-term behaviour of polymers and polymeric geosynthetics was reviewed
in the literature and mathematical relationships presented. Research on soil-geogrid interaction was presented including several factors which influence this interaction. Finally, two codes of practice – the FHWA 1990 and the BS 8006 guidelines – representing North American and European practice respectively were reviewed with respect to the Long - Term Design Strength (LTDS), and soil – geosynthetic interaction. From this review of literature a number of research needs have arisen:

• investigate the long-term load – strain behaviour of polymeric geogrids confined in soil through creep pullout tests. Compare this behaviour with unconfined creep behaviour at comparable load levels.

• investigate the pullout load – displacement and strain development behaviour of geogrids at the small in – service strain levels. This will contribute to the understanding of the interaction mechanism and progressive strain development of geogrids under working stresses.

• investigate the effect of relatively high confining stress on the load – displacement behaviour, strain development and soil – geogrid interaction in pullout tests

• determine the utility in terms of providing reliable pullout data efficiently of a different method of testing geogrids in pullout in.

• compare the load – strain and load – time behaviour of geogrids to create a framework within which different geogrids may be described generically.

• local measured strains are compared with average computed strains to investigate their differences.

These research needs expand on areas already investigated by earlier studies and probe areas which have had comparatively little attention.
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Figure 2.1: Isochronous load-strain curves for Tensar UX 1500 (from manufacturer).

Figure 2.2: Schematic representation of geogrid specimen in pullout test showing bearing elements.
Fig. 2.3. Profile of a geogrid with bearing elements of thickness B, spaced at intervals S. Directions of normal and bearing stress are also indicated.
3.1 Introduction

The purpose of this study is to investigate the load-strain, strain-time, and pullout behaviour of polymeric geogrids. Three separate apparatuses were used in the present study: a standard industrial (Instron) testing machine, a modified pullout box, and a large pullout box. Identical Instron machines were used in both the Ministry of Transportation and Highways (MOTH) laboratories in Victoria and in the UBC lab in Vancouver. The modified pullout box was used in Victoria and was designed and built by MOTH. The large pullout box was designed and built at the University of British Columbia in a previous study (Raju, 1995).

The use of a range of apparatuses allowed a broad characterization of geogrid behavior in both tension and pullout. Described below are each of the experimental apparatuses as well as the instrumentation and data acquisition systems.

3.2 Tensile Test Apparatus

Tensile tests were carried out on both Instron tensile testing machines and a modified pullout box. Two distinct classes of tensile tests were performed: load controlled and displacement controlled. Short and long term tests were performed.
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3.2.1 Instron Tensile Testing Machine

An Instron, model 4206, part of the 4200 series, is an electromechanical belt driven unit with two lead screws and a cross head applying the load to the specimen. Specimens are gripped in two friction clamps. For all tests a 150 kN load cell was used which had a resolution of 0.005 kN.

Control of tensile tests was maintained through an integrated electronic console which acted as a data acquisition system and a control panel for specifying loading rates and cycling limits. A loading regimen could be specified electronically by keying in the tensile load, or strain, limits and selecting a cycling or monotonic mode for the cross head motion. Rupture load, and maximum strain and displacement were also retained in memory for any tensile test. All operations on the Instron are directed by a crystal controlled microprocessor-based CPU. The console was used in conjunction with a Hewlett Packard 7090A measurement plotting system which recorded the load and strain in real time. It also had a buffer which stored the data and reproduced it on demand for long term tests.

3.2.2 Creep Pullout Tests

A pullout box used in previous research at the M.O.T.H. (Harrison, 1993) was modified for the current study for performing creep pullout tests on geogrid specimens. The apparatus was also used to perform one unconfined creep test. Confinement simulated the types of conditions likely to be found in walls of approximately 5 m in height with typical MOTH backfill. Figures 3.2 and 3.3 show side and front views of the testing machine respectively. There are four principal components to the apparatus:
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1) An aluminum soil box

2) A normal load application system

3) A pullout load pulley assembly

4) A reaction frame

3.2.2.1 Aluminum Soil Box

The aluminum box is constructed of 19 mm plate aluminum and is assembled with steel bolts. It measures 1.22 m in total length, 0.53 m in width, and 0.36 m in height (figure 3.2). Bisecting the box is a 13 mm aluminum plate yielding two approximately equal chambers. At mid height of the front wall of the box is a 25.4 mm slot to accommodate the geosynthetic specimens and instrumentation. The soil sample is contained along with the geogrid in the front chamber. The soil chamber can accommodate a sample measuring 0.58 m long, 0.495 m wide, and 0.56 m deep with a 2.54 cm seat above the soil for the normal load assembly.

3.2.2.2 Normal Load Application

The apparatus was required to apply a constant, uninterrupted normal load over an extended period of time. Given the duration of the long term tests and thereby the chance of electrical outage or leakage problems with hydraulic systems, a mechanical means of administering the normal load was chosen. Figure 3.3 shows the entire normal load assembly.

The assembly consisted of a loading platform, a ball bearing, load cell, inclined plane press and reaction beam. Load was applied with a mechanical jack, actually a California
Chapter 3. Test Apparatus

Bearing tester, model CF 410 manufactured by Soil Test. Structural steel - 350W - was used for the reaction beam to which the jack was bolted. The jack was connected at the bottom end to a load cell. Below the load cell a 32 mm ball bearing was inserting to insure a purely normal load was applied to the system with no transfer of moment, and resulting in shear stress, imparted on the soil. The loading platform consisted of a 25 mm steel plate atop a series of successively larger plywood shims which spread the load over the total area of the soil sample. The resulting system provided 95 kPa of normal stress.

3.2.2.3 Tensile Load Assembly

A tensile load assembly was designed and constructed at the MOTH laboratories in Victoria. The design was based on applying a constant, cost-effective and space-efficient load over a long period of time. The assembly consists of 3 separate pulley and boom stations each of which in turn is comprised of dead weights, a steel boom, two pulleys and airforce cable. The section in figure 3.2 shows the centre station. Each boom was constructed of two 350W mild steel flats, 0.84 m long, 1 cm thick and 4 cm deep joined together with steel pins. At the end of each boom was attached a 1 inch diameter round steel cross bar which passed through both flats. Steel weights with pre-bored holes were carefully loaded onto the cross bars. At the opposite end of the boom the flats were machined into a pulley mounted on journal bearings. The bearings were in turn mounted onto stations which were part of the frame. Geogrid specimens were gripped with sand blasted steel angles and 3/8” stainless steel bolts (figure 3.4) Clamps were located a distance of 2.5 cm from the aluminum box at the start of test. Tensile load was transmitted between the geogrid specimens and the pulley booms by means of aircraft cable. This cable has a very high tensile modulus and is very strong. It was selected
Chapter 3. Test Apparatus

because of the need for long term stability and because of its dimensional stability. The cable was connected to the clamp at one end and to the pulley-boom assembly at the other.

3.2.2.4 Reaction Frame

The reaction frame for the vertical loads was constructed of welded sections of rectangular steel - 350W structural grade. The reaction beam to which the normal load jack was mounted was fully adjustable by means of a series of drilled holes with bearing pins. The resulting mechanism was a Vernier type adjustment which allowed a broad range of vertical movement for the frame. The aluminum box was supported on the underside by three transverse sections of steel channel which minimized the stress and deflection for the aluminum box (figure 3.3).

3.2.3 Instrumentation

The geogrid specimen and soil samples were instrumented with load cells and LVDTs to continuously monitor force and extension with time. Figure 3.5 shows a schematic of the instrumentation layout. Three LVDTs were positioned at the front clamps. Two were Selabs SE 373-150 and one was a Hewlett Packard 24 DCDT-3000. Rear LVDTs were also installed to measure any displacement of the geogrid; each was an HP 24 DCDT-500 series. Sensotec Model 75/1508-05-01 load cells were placed in line between the airforce cable and the clamps to measure tensile load for each sample. The normal load was measured with an Intertechnology model T24-10K-10PI load cell with an output of 10 mV/V at 10 kN. All instruments were powered by a GFC Hammond linear power supply, model GFOF 1-12. Signal input from the normal force load cell and the rear
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LVDTs was through Fluke 8000A digital multimeters. The front LVDTs were read with a Fluke 23 multimeter. Tensile force load cells connected to the samples were read by a MOTH in-house designed multimeter meter. Finally, temperature was monitored with a Fluke 2175A digital thermometer. Steps were taken to minimize the fluctuations of temperature over the months of testing. All measurements were recorded manually by this researcher and technicians at the MOTH.

3.3 Pullout Test Apparatus

The large pullout apparatus used here was designed and constructed for a previous study at the University of British Columbia (Raju, 1995). The pullout assembly was used to investigate the behavior of geogrids in pullout under a constant rate of displacement. Measurements of force, displacement, strain and pressure were made.

The apparatus is comprised of 6 systems

- pullout box
- reaction frame
- hopper
- normal stress application system
- clamp
- pullout control assembly

The instrumentation and data acquisition systems are described in sections 3.4 and 3.5 respectively.
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3.3.1 Pullout Box

The pullout box was designed on the basis of 3 criteria. It was to accommodate a geogrid specimen representative of the material used in field structures, accept specimens of length to width ratio of 2:1, and minimize boundary influence. The first two criteria pertain to the geogrid specimen size which must be large enough to be representative of geogrid material. Moreover, a minimum reinforcement anchorage length beyond a failure surface is typically taken to be 0.91 m (Chrisopher et. al., 1990). From this, a specimen of approximately 1 m in length and 0.50 m in width was selected for tests. The third condition dictates that the top, side and bottom boundaries of the pullout box not influence the geogrid specimen during pullout. An ASTM draft for a proposed pullout test standard stipulates that the side walls of a box be a minimum of 15 cm from the edge of the specimen if measures are not taken to minimize friction along the side walls, or 7.5 cm if friction is minimized. Friction was minimized in this box by lining it with glass. The resulting dimensions of the box were 1.3 m long, 0.64 m wide and 0.63 m deep, accommodating a soil sample 1.3 m long, 0.64 m wide and 0.60 m deep. A space of 0.03 m between the surface of the soil sample and the lid of the box allowed for placement of a polyvinylchloride (PVC) bag used to apply normal pressure. These dimensions satisfy the above criteria, and it was concluded that the box was sufficient for the needs of the current study. The box is shown in figure 3.6 (a).

The box is constructed of 2 side plates, 2 front and back plates and a bottom plate all of which are rigid. The top of the box is a flexible, stress-controlled boundary detailed below. Each of the side plates is a built up section of an outer 2.5 cm thick Plexiglas plate glued to an inner 0.30 cm thick glass plate. The Plexiglas is connected to the
Chapter 3. Test Apparatus

reaction frame described in section 3.3.2. The front boundary is made of two 13 mm thick aluminum plates separated vertically by a 12 mm slot through which the geogrid specimen is pulled, (figure 3.6 (b)). The slot is lined top and bottom with foam strips to keep the sand from exiting the box with the sample during pullout. The rear boundary is formed from a similar 13 mm thick aluminum section but has a 16 mm diameter hole in the centre through which the instrumentation wires and tell-tale cable exit the box, (figure 3.6 (b)). The bottom boundary is 13 mm aluminum plate bolted to the reaction frame. The top boundary as mentioned above is not rigid but flexible and the confining stress it imposes is prearranged and constant. For the 4 kPa tests the top of the pullout box is left open with no constraint, the standing height of the soil from the geogrid specimen level to the top of the box providing the necessary 4 kPa of normal stress. For the 50 kPa tests a PVC bag filled with water and pressurized furnishes the boundary. A flexible top boundary helps prevent unwanted additional normal stress on the geogrid specimen due to any dilation of the sand during pullout.

3.3.2 Reaction Frame

The pullout box is supported by a steel frame which provides a reaction to the pullout loads developed by the hydraulic actuator and the normal load system. The frame is constructed of mild steel plates, bars and tubes which gird the box horizontally. All rectangular sections are 76 mm by 54 mm spaced vertically at 250 mm centre on centre along the side, and 105 mm centre on centre over the front. The pullout box is supported on the same rectangular sections. The top of the box is a 1 inch thick steel plate measuring 1485 mm by 760 mm. It is secured to the frame by built up I - beams bolted
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to two 1 inch diameter steel bars (figure 3.7). The bars are threaded into the bottom of
the frame.

3.3.3 Hopper

A hopper was used to pluviate the sand sample through air to the pullout box (see
figure 3.8). Reference was made to a study by Vaid and Negussey (1988) which
investigated various factors effecting the density of Ottawa sand pluviated through water
and air. Both drop height and rate of pouring were identified as important factors in
resulting initial values of density after deposition. Drop height influences the velocity of
sand particles at deposition which is related to initial density. At the point of terminal
velocity, when no further increase in kinetic energy is realized by the particles, drop
height no longer influences the value of density of the deposit. Rate of pouring
influences the interference between falling particles; a greater rate of pouring results in
lower densities. Based on these factors, the hopper was designed to pluviate the sand
through a range of heights from 1.4 m to 0.80 m over the full 0.60 m depth of the sand
sample and achieve a targeted relative density of 85% - 90%. The bottom of the hopper
chamber was fitted with two thin, mild steel plates overlying each other with circular
apertures of 6 mm diameter spaced by 13 mm. The resulting configuration gave
approximately 50 % openings. This effectively controlled the rate of deposition and
interference between the falling particles. Pluviation of the sand was initiated by the
sudden opening of pneumatically controlled trap doors located below the perforated steel
plates. The hopper was supported by an aluminum frame which fitted on top of the
pullout box with thin Plexiglas side walls to contained an dust generated by pluviation.
3.3.4 Normal Stress Application

For certain tests, normal stress in addition to the weight of the sand, was applied by a pressurized PVC water bag (figure 3.6 (a)). The bag is 1300 mm long, 640 mm wide and 25.4 mm thick. It was placed above the sand sample just beneath the 1 inch steel plate lid of the box. Water from the laboratory supply was introduced to the bag through a tube, while air exited through a similar venting tube. Once the bag was full, the water was pressurized by connecting the two tubes of the bag to a modified triaxial testing cell located adjacent to the pullout box. A regulator connected to the laboratory supply of compressed air applied the pressure. A pressure transducer was located at the base of the triaxial cell in line with the top of the sand layer to measure pressure (figure 3.9).

3.3.5 Clamp

A clamp was used to grip the geogrid specimens, providing a load transfer between the hydraulic actuator and the specimen (figure 3.10). The clamp measured 645 mm across, gripping the full 0.5 m width of the specimen. The clamp has three components: a lower jaw, an upper jaw and a central insert. The lower jaw is connected to the hydraulic actuator by a swivel pin connection which transfers no moment to the specimen. The lower jaw is supported by a wooden mantle which runs the width of the pullout box. A mild steel plate replete with ball bearings spaced at 25.4 mm centres serves as a low resistance support for the jaw assembly. Arborite surfaces are glued to the lower jaw and the mantle which facilitates ease of sliding of the jaw.

The upper jaw attaches to the lower with bolts spaced at 190 mm centres. Additionally, four C-clamps were attached to the jaws to prevent any slippage of the specimen. The central insert is a triangular and wedge interfacing between the upper and lower jaws. Its
lower surface beared directly on the geogrid specimen, while the top surface pressed
gainst a stainless steel bar imbedded in the upper jaw, shown in the detail of section A-A
of figure 3.10. This configuration provides an efficient mechanical advantage for
clamping the specimen. The bottom surface of the central insert and the top edge of the
lower jaw are both serrated to better grip the geogrid specimen.

### 3.3.6 Pullout Control Assembly

Pullout resistance is mobilized using a servo-controlled hydraulic actuator. The pullout
control system, designed at UBC, comprises a hydraulic pump, an actuator, an electro-
mechanical control box and a 386-SX digital computer. The system acts as a self
correcting feedback mechanism. A schematic for the system is shown in figure 3.11.
Each component of the system is described below.

The hydraulic pump was manufactured by MTS systems corporation. It delivers 3
gallons (11.4 L) per minute within a range of 3000 to 5000 psi (20.7 to 34.5 MPa). The
pump is run electrically from the control box. Hydraulic oil lines connect the pump to
the pullout actuator.

Pullout force was imposed by a double acting hydraulic actuator connected to the clamp
at the fore end and mounted to a cross member of the reaction frame at the aft end. The
piston is aligned vertically with the slot in the front plate of the pullout box where the
geogrid specimen exits. The cylinder was manufactured by the Cunningham Cylinder
Co. and has a rod diameter of 33 mm, a stroke of 152.4 mm and an internal LVDT to
measure displacement. The servo valve, model 760-912A manufactured by Moog
Hydraulics, interfaced between the electro and mechanical systems. It mounts to the side
of the actuator.
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The hydraulic pump was operated through a control box which liaison between the computer and the servo valve. The box has two components: a controller and a control unit. The controller, model 406 by MTS, is an electric sub-system containing the principal servo control, fail-safe, and readout functions. It takes signals directly from the computer digital - analogue (D/A) board, connected to the computer, and sends them to the servo valve. It also reads the position of the actuator from the LVDT mounted within the hydraulic cylinder and compares this returning signal with the new incoming signal from the computer. The difference between the two is termed the error and is used to correct the imbalance through advancement of the actuator.

The control unit, model 436.11 also by MTS, centrally administers the electrical power, hydraulic pressure, program run-stop and program event counting. The control unit contains interlock circuits which automatically stop the test in the event of malfunction.

3.4 Instrumentation

Instrumentation was installed to collect data and also to maintain control of the tests. Measurements of force, displacement, pressure, time, temperature and strain were taken. Data were collected from 10 channels. Displacement was the control in all pullout tests through the use of a constant rate of pullout displacement. Moreover, tests were carried out at approximately the same time each day, thus keeping temperature fairly consistent between and throughout tests. A schematic illustration of the layout of instrumentation is shown in figure 3.9.
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3.4.1 Pullout Force

Pullout force was measured with a load cell located between the clamp and the hydraulic ram. An Interface, Inc. model 1210AF load cell with a capacity of 44.5 kN (10 000 lb) was selected on the basis of the anticipated loads to be encountered during the tests. The instrument was powered with a 10 V DC supply. Calibrations were carried out on the load cell to determine the linearity of response over the anticipated load range.

3.4.2 Pullout Displacement

Pullout displacement was measured with LVDTs and checked with dial gauges (figure 3.10). Two LVDTs were mounted on the clamp support mantle described in section 3.3.5. Since the clamp was pin-connected, the easiest way to measure the average clamp displacement (corresponding to the actual movement of the hydraulic actuator), was to position a LVDT at either side of the clamp. This also gave a measure of how much distortion of the geogrid took place during a test. Both front LVDTs were DC-DC types, model SE 373/100, manufactured by SE LABS. They each had 100 mm strokes. Two dial gauges were mounted beside the LVDTs to check the accuracy of them and to also check the consistency of the displacement rate throughout the test. Movement of the rear of the geogrid specimen was measured also with a LVDT mounted outside the 16 mm hole in the back plate of the pullout box and connected to the sample with a tell-tale cable. The cable was bonded to the geogrid specimen with two part epoxy and screwed into LVDT rod. The instrument was DC-DC type and was manufactured by Transtek. It had a similar stroke of 100 mm.

As a check of the accuracy of the LVDTs and of the consistency of the machine displacement rate, two dial gauges were positioned on either side of the clamp next to the
Chapter 3. Test Apparatus

LVDTs. The gauges were accurate to 0.01 mm. It was found that the LVDTs were accurate and the machine consistent.

3.4.3 Water Pressure Transducer

A water pressure transducer was used to measure the pressure in the PVC water bag throughout the test (figure 3.9). The transducer was a type SP100-15G by Magnetek Transducer Products and had a range of 0 to 100 kPa. As is standard practice, the transducer and connecting lines were fully saturated with water prior to each test. Checks for system compliance were also made. The transducer was calibrated with a standing head of water. Prior to each test a digital pressure indicator, model DPI 601 by Druck Inc. was used to check the calibration of the transducer.

3.4.4 Strain Gauges

Strain was measured over the length of the geogrid specimen for all tests. This was accomplished by attaching strain gauges to the geogrid specimen by an established procedure (Bathurst, 1990). Details of the procedure used are presented in Appendix A. The high extensibility of geogrids coupled with the fact that four of the strain gauges were to be embedded within soil were chief in determining which gauges would be used. Ultimately, gauges type EP-08-250BF-350 OPTION E, manufactured by the Micro-Measurements Division of Measurements Group Inc. were selected as they were high extension (10 to 15 % strain) and they were encapsulated for protection. Further protection was afforded by gauge coat compound and Teflon tape (see Appendix A). Bonded gauges were connected to the data acquisition system via a completion circuit that contained dummy gauges, forming full Wheatstone bridges for each gauge. Figure
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3.9 shows the layout of gauges for each test. Gauge positions were referenced with respect to the inside of the front wall of the pullout box and normalized by the total initial embedded length.

3.4 Data Acquisition System

A data acquisition system was set up to acquire and record data from tests. A software program was also developed (Raju, 1995) to control the tests. The system consists of a 386 SX personal computer, a DAS-16 board, a signal conditioning unit and the program. The DAS-16 board is a high-speed, multi-function digital to analogue converter and input-output expansion board. It is manufactured by Metrabyte Corp. and includes a 12-bit successive approximation converter. Gains can be selected by the user on all channels. There were either 16 single ended channels or 8 double ended (differential channels) available for use.

A signal conditioning unit was designed and built at UBC specifically for this study to filter the incoming signal from all instruments. The conditioner filters noise out of the incoming signal from instruments. It also provides DC input to transducers and amplifies the output from the instruments using the selected gain on each channel. The signal from the conditioner is converted from an analogue voltage to an equivalent digital value in the DAS-16. The same translation is used in the opposite direction to send a demand displacement signal to the servo-valve (figure 3.11).

A data acquisition program was written to collect and store data from the tests, and also generate the demand displacement signal, (Raju, 1995). The principal components of the program are a pretest channel scan routine for baseline values, a discrete channel scan routine to record data throughout the test, and a real-time, on-screen test monitoring
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routine. The latter was achieved by creating an initialization file containing calibration factors for all instruments, the width and embedment length of the geogrid sample and displacement rate of the test. The engineering quantities of force per meter width of geogrid, strain in percent, pressure in kPa and displacement in mm were all displayed on screen.

Figure 3.1 Instron Tensile Testing Machine
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![Diagram of Test Apparatus]

Figure 3.2. Side view of the load-controlled pullout and C tests apparatus (all dimensions in cm).
Figure 3.3. Front view of load-controlled pullout and C tests apparatus. Figure does not show the pulley and boom station.
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Figure 3.4 Clamp arrangement for load-controlled pullout and C tests. All dimensions in mm.

Figure 3.5. Layout of instrumentation for the load-controlled pullout tests.
Chapter 3. Test Apparatus

Figure 3.6 (a) Side view of CRD pullout assembly with internal dimensions of box. All dimensions in mm.

Figure 3.6 (b). Detail of front and rear boundaries of CRD pullout box. All dimensions in mm.
Figure 3.7: Top and side views of reaction frame for CRD pullout apparatus.
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Figure 3.8. Side view of hopper. All dimensions in mm.

Figure 3.9. Instrumentation layout for pullout assembly showing the positions of strain gauges along specimen.
Figure 3.10 Clamp assembly for CRD pullout apparatus. All dimensions in mm.

Figure 3.11 Schematic illustration of the closed-loop control system for CRD pullout apparatus, (After Raju, 1995).
CHAPTER 4
MATERIAL PROPERTIES

4.1 Introduction

Materials used in the current study are soil and geogrids. Two soil samples and three geogrid specimens were utilized for the research. Each soil was used for different aspects of the overall study and not for comparative purposes, whereas the geogrids were used in a comparative manner. Each material is described with respect to index tests and manufacturers’ literature.

4.2 Soil Samples

Two soils were used for separate aspects of the study. A well-graded sandy gravel was used in the confined tensile tests in the MOTH laboratories in Victoria, BC. A poorly-graded sand was used for the UBC pullout tests. The well-graded gravelly sand allowed the test results to be applicable to MOTH operational needs, whereas the poorly-graded sand used in the UBC pullout tests yielded consistent and repeatable results suited to a more fundamental interpretation.

4.2.1 Well-Graded Gravelly Sand

The well-graded sandy gravel used in the MOTH confined tensile tests is referred to in the MOTH as Select Granular Sub Base or SGSB. It is used extensively in road building and where a granular, free draining material is needed for construction. Sieve analyses were carried out for the material and it was found to have a $D_{50}$ of about 0.7 mm and a coefficient uniformity of approximately 7. This corresponds to a well-graded gravelly
Chapter 4. Material Properties

sand (GW-SW). There was only about 0.7% of material passing the #200 sieve. It was found to be a 25 mm minus material. On inspection the material particles are sub-angular to angular. The sand had a maximum dry density of 1971 kg/m$^3$ at 8.2% moisture content. The grain size distribution curves for the two analyses that were performed - one as a duplicate - are found in figure 4.1.

4.2.2 Poorly-Graded Sand

A poorly-graded sand manufactured by the Badger Mining Corporation was used for the UBC pullout tests. The soil was selected on the basis of its grain size and shape, and high crush strength. A grain size distribution is presented in figure 4.2 showing a $D_{50}$ of between 0.8 and 0.9 mm. The soil has a $C_u$ of 1.5 with little to no fines, classifying it as a poorly-graded sand (SP). Particles are rounded to sub rounded on inspection and range in size from 0.1 mm to 2 mm. The minimum and maximum void ratio according to ASTM D 4253-93 and ASTM D 4254-91 are 0.49 and 0.62 respectively.

Direct shear tests were carried out on the sand in a previous study, (Raju, 1995) and it was found that with a slight stress dependency the plain strain friction angle, $\phi_{ps}$, varied between 29° and 36°. The following relationship between plane strain and direct shear friction angles, $\phi_{ds}$ is presented (Jewell and Wroth, 1987),

$$\sin \phi_{ps} = \frac{\tan \phi_{ds}}{\cos \psi (1 + \tan \phi_{ds} \tan \psi)}$$

[4.1]

where

$\psi$ is the angle of dilation for the soil.
Chapter 4. Material Properties

4.3 Geogrid Test Specimens

Geogrids are polymeric materials manufactured from base polymers and additives. Typical polymers used for geogrids are polyethylene, polyester and polyvinylchloride. Polymers are thermo-viscoelastic materials and their behavior is temperature, strain, and strain rate dependent. Depending on the base polymer and the manufacturing process, various geogrids exhibit somewhat different behaviours in tension and interaction with soil. For this, and other studies, important features of geogrids are their surface roughness, the size and shape of junctions and the long term creep strain behavior. Table 4.1 presents the properties of all three geogrids used in this study.

There are a number of ways of classifying geogrids in order to compare their performance. Geogrids are either uni-directional or bi-directional depending on whether they exhibit significant tensile strength in one or two directions. All geogrids used in this study were uni-directional. Geogrids can be further differentiated on the basis of whether they are stiff or flexible according to ASTM D1388. A stiff geogrid is one that exhibits a flexural rigidity of more than 1000 g-cm. Geogrids may also be classified on the basis of junction strength, referring to the strength of the connection between the longitudinal and transverse members. A grid is either high or low junction strength. There is evidence to suggest that the shape of junctions is at least as important as the strength (Palmeira, 1987). Two flexible, low junction strength geogrids and one stiff, high junction strength geogrid were used in this study.

4.3.1 Tensar UX1500

The UX1500 geogrid is manufactured by the Tensar Earth Technologies Corporation of Atlanta, Georgia. It is a stiff, unidirectional geogrid of high junction strength. Figure 4.3
shows the Tensar UX1500 geogrid in plan and side view. UX1500 is made of high density polyethylene (HDPE) with 2 % (by weight) of carbon black added to protect the polymer from ultraviolet light degradation. UX1500 is a punched sheet drawn geogrid wherein a sheet of polyethylene is punched with holes which are then drawn into an oblong shape. The oblong holes are called apertures. This process creates a geogrid with a high tensile strength in the direction it is drawn - referred to as the machine direction - and a comparatively low tensile strength in the transverse direction, or the cross machine direction. The resulting structure is a series ribs in the machine direction and bars in the transverse direction. At the intersection of the ribs and bars are the junctions. As seen in figure 4.3 (b) the UX1500 profile shows prominent nodes at the junctions; these also enhance interaction with the soil.

4.3.2 Miragrid 12XT

Miragrid 12XT is manufactured by the Nicolon Mirafi Group. It is a flexible, low junction strength, woven geogrid made from high tenacity polyester yarn. The yarn is woven into high strength ribs which are then coated with polyvinylchloride (PVC) to protect them from ultraviolet degradation. Connecting bars of the same polyester yarn are attached to the longitudinal ribs at intervals. Figure 4.4 show the plan and side views of the Miragrid 12XT. It is noted that the apertures are rectangular in contrast to the oblong shape of the Tensar geogrid.

4.3.3 Stratagrid 700

Stratagrid 700 is the other flexible, low junction strength geogrid used in this study. It is manufactured by Strata Systems in Atlanta. This grid is also made of polyester yarn
which is knitted together and impregnated with PVC. Figure 4.5 detail the plan and side views of the grid. The apertures for Stratagrid 700 are a slightly different rectangular shape than the Miragrid 12XT and in contrast again to the oblong Tensar apertures.

![Graph showing grain size distribution curves for well-graded gravelly sand (GW-S) and poorly graded sand (SP).](image)

**Figure 4.1.** Grain size distribution curves for well-graded gravelly sand (GW-S) poorly graded sand (SP).
### Table 4.1. Properties of geogrids used in the testing program.

<table>
<thead>
<tr>
<th></th>
<th>Units</th>
<th>Tensar UX1500</th>
<th>Miragrid 12XT</th>
<th>Stratagrid 700</th>
</tr>
</thead>
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<tr>
<td><strong>Material Code</strong></td>
<td></td>
<td>T</td>
<td>M</td>
<td>S</td>
</tr>
<tr>
<td><strong>Dimensions</strong></td>
<td></td>
<td></td>
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<tr>
<td>Apertures</td>
<td>(mm)</td>
<td>144.78</td>
<td>81.3</td>
<td>59.4</td>
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<td>Machine Direction</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Cross Machine Direction</td>
<td></td>
<td>16.76</td>
<td>12.7</td>
<td>22.9</td>
</tr>
<tr>
<td>Open Area</td>
<td>%</td>
<td>60</td>
<td>60</td>
<td>46</td>
</tr>
<tr>
<td>Thickness</td>
<td>(mm)</td>
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<td>1.3</td>
<td>1.78</td>
</tr>
<tr>
<td>Ribs</td>
<td>(mm)</td>
<td>4.32</td>
<td>1.62</td>
<td>1.98</td>
</tr>
<tr>
<td>Junctions</td>
<td>(mm)</td>
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<td></td>
<td></td>
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<td><strong>Tensile Strength</strong></td>
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<td>Wide width strip tensile strength (ASTM D4595-86)</td>
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<td></td>
</tr>
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<td>@5% strain</td>
<td>(kN/m)</td>
<td>52.4</td>
<td>51.7</td>
<td>32</td>
</tr>
<tr>
<td>Ultimate strength</td>
<td>(kN/m)</td>
<td>86</td>
<td>92.8</td>
<td>146</td>
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<td>(kN/m)</td>
<td>31.8</td>
<td>55.6</td>
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<tr>
<td>Long term design load in MD</td>
<td>(kN/m)</td>
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<td>45.9</td>
<td>74.9</td>
</tr>
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<td><strong>Material</strong></td>
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<td>HDPE with 2% carbon black</td>
<td>PET with PVC coating</td>
<td>PET with PVC impregnation</td>
</tr>
<tr>
<td>Creep Reduction Factor</td>
<td></td>
<td>0.35 - 0.39</td>
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<td>0.62</td>
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<tr>
<td></td>
<td></td>
<td>0.2</td>
<td>0.4</td>
<td>0.4</td>
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<tr>
<td><strong>Dimensions of Roll</strong></td>
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<td>1.83</td>
</tr>
<tr>
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<td>(kN/m²)</td>
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<td>0.00656</td>
<td>0.00511</td>
</tr>
</tbody>
</table>
Chapter 4. Material Properties

Figure 4.2. Physical dimensions of the Tensar UX1500 geogrid.

Figure 4.3. Physical dimensions of the Miragrid 12XT geogrid. All dimensions in mm.

Figure 4.4. Physical dimensions of the Stratagrid 700 geogrid. All dimensions in mm.
CHAPTER 5
TEST PROCEDURES

5.1 Introduction

Presented in this chapter are the procedures and conditions for all tensile and pullout tests. Also described in this chapter is the sample and apparatus preparation for each test. Two classes of tests were carried out: tensile and pullout tests. A further distinction is made within the two types of tests between load- and displacement-controlled tests. The philosophy behind the testing program is to characterize the behaviour of geogrids in tension and pullout under a variety of conditions. Load-controlled tests in the form of creep and creep pullout tests are performed to characterize the long-term behaviour of geogrids and to investigate the effect of confinement on creep strain. The load levels used in the load-controlled tests are admittedly high, however they are intended to provide a full characterization of the creep response in a relatively short time period. Procedures for the preparation of the two soil samples and the three geogrid specimens are detailed below.

Table 5.1 provides an overview of the tests carried out. At the bottom are given the test codes for each class of test. In the presentation of results and analysis individual tests will be referenced to the codes in table 5.1.
5.2 Tensile Tests

Tensile tests were carried out on three different geogrids. The objectives of the tests were to investigate the load-strain, strain-time and load-time characteristics of geogrids. Moreover, comparison was made between the tensile and pullout tests with respect to load-strain and strain-time characteristics of geogrids.

Four different types of tensile tests were carried out: Constant Rate of Extension, Load Relaxation, Cyclic Pseudo-Static and Creep. Each test type will be described below under separate sections.

![Schematic chart of testing program.](image)

**Figure 5.1: Schematic chart of testing program.**

5.2.1 Displacement Controlled Tensile Tests

Displacement controlled tests were carried out to investigate the effects of time on the load-strain characteristics of geogrids. Two types of displacement controlled tests were
Chapter 5. Test Procedures

carried out: constant rate of extension (CRE) and load-relaxation (LR). Tests were carried out either on 4- or 5-rib specimens of Tensar UX1500 (approximately 0.08 m in width) or single rib specimens of Miragrid and Stratagrid geogrids. Most Tensar specimens were cut to lengths of 0.51 m, resulting in gauge lengths of approximately 0.375 m however, the 5 rib specimens had gauge lengths of 0.475 m. The reason for the different specimen widths and lengths for the Tensar UX1500 was to accommodate different clamping arrangements between laboratories. All strength load values are made on per meter width bases. Stratagrid and Miragrid specimens were all single ribs with gauge lengths of 0.32 to 0.35 m. Miragrid and Stratagrid specimens conformed to Geosynthetics Research Institute (GRI GG1) standards.(32) Where noted, Tensar specimens conformed to ASTM D 4595 specifications.

5.2.1.1 Constant Rate of Extension (CRE) Tests

CRE tests were performed to investigate rate-of-loading effects and to compare local and average strain in geogrids. In constant rate of extension tests geogrid specimens underwent a constant rate of extension until rupture at different rates of extension. Specimens were held in two friction clamps gripping their entire width. Care was taken to prevent slippage of the geogrid specimen in the clamp; this was monitored by marking the sample at the boundary of the clamp with a clearly visible hash mark. No significant slippage occurred in any of the tests. Once the specimen was secured a rate of extension was keyed in to an electronic console as a specified amount of displacement of the test machine cross-head per minute, for example 10 mm/min. Rates of 0.10, 2, 5, 10 and 50 mm/min were specified. A 150 kN load cell, standard for Instron machines, was used in all tests giving a resolution of 0.005 kN. Some specimens were instrumented with strain
Chapter 5. Test Procedures

gauges for comparison between the local and average strain as well as with pullout load-strain behaviour. Other specimens were only instrumented with displacement transducers. Test data were recorded in real-time on a Hewlett Packard plotter and with a data acquisition program.

5.2.1.2 Load Relaxation Tests (LR)

Two load relaxation tests were performed to investigate the influence of time on geogrids under load. Specimens were clamped similar to the CRE tests and brought to a target load corresponding to a certain displacement of the cross-head of the Instron machine. Once the target load was reached the cross-head motion of the testing machine was ceased. Measurement of load was recorded over time with the HP plotter. Tests were carried out at 25 and 75 kN/m and in both cases 20 hours of load and time data were collected.

5.2.2 Load Controlled Tensile Tests

Load controlled tensile tests were performed to investigate the strain-time characteristics of HDPE geogrids and to provide comparison between confined and unconfined conditions with respect to creep strain. HDPE geogrids have a greater tendency for creep strain than Polyester grids thus efforts were confined to testing them. Two types of load controlled tensile test were carried out: Cyclic Pseudo-Static and Creep. All specimens were 4 rib Tensar UX 1500 geogrids described above.

5.2.2.1 Cyclic Pseudo-Static
Chapter 5. Test Procedures

Cyclic - Pseudo Static refers to the manner in which the geogrid specimens were loaded (see figure 5.2). An Instron machine was used to cycle geogrid specimens in tension between two very close load limits and maintain this cycling until failure, allowing strain to increase with time. The result of cycling between two very close load limits is that a load-controlled (LC) condition arises, approximating a constant dead load test. Three CPS tests were performed at 50, 56.3 and 62.5 kN/m. The limits of cycling were maintained at ± 0.005 kN or 0.0625 kN/m of the target load. This translates into a deviation from the true target load of .125 %, .11 % and .10 % for the 50, 56.25 and 62.5 kN/m tests respectively. Measurements of extension and time were taken continuously with an HP plotter for all tests.

5.2.2.2 Creep Test

A single dead load creep test was carried out on the modified pullout box described in chapter 3. This was a true load-controlled test, as opposed to cyclic pseudo-static, that provided a qualitative comparison between confined (creep pullout) and unconfined creep behaviour.

A Tensar UX1500 4 rib sample was clamped in air and loaded in tension using the pulley and dead weight system of the modified pullout box. The load level was 45 kN/m, equal to one of the creep pullout tests. Measurements of force and extension were taken for the duration of the test at intervals. The intervals of measurement varied with time starting at 30 s immediately after load application followed by 1 minute, 10 minute, 1 hour and 8 hour. Eventually 1 and 2 week intervals were sufficient after little change was noted from day to day. Temperature measurements were taken approximately daily, followed by weekly and biweekly, coinciding with other measurements.
5.3 Pullout Tests

Two types of pullout test were performed: load controlled and displacement controlled. The former was a creep pullout (CP), the latter a constant rate of displacement (CRD) pullout test. The two types of tests both have different objectives. The aim of the creep pullout tests was to examine the effect of confinement on the strain-time behaviour of HDPE geogrids. The objective of the CRD pullout tests was to compare and contrast the behaviour of three geogrids with respect to soil-geogrid interaction and strain development. The creep pullout tests examined the large strain, limit state condition whereas the CRD pullout tests addressed the small in service strain range. Soil sample and geogrid specimen preparations as well as testing procedures are detailed in the following sections.

5.3.1 Creep Pullout (CP) Tests

Creep pullout tests were performed on a modified pullout box described in section 3.2.2. Preparation of the tests centred around maintaining a state of long-term equilibrium between the imposed tensile load and the resistance of the geogrid embedded in soil. This meant that the normal load had to be great enough to ensure that pullout would not occur and that the load levels not be so great as to rupture the geogrid too quickly. Measurements of displacement, load, time and temperature were taken for the duration of all tests in a regimen similar to the creep test (see section 5.2.2.2).
Chapter 5. Test Procedures

5.3.1.1 Soil Sample Preparation

Soil for the creep pullout tests was taken from MOTH borrow pits in Victoria, BC. The material was hand placed in the modified box and densified to maximum Proctor dry density in 2 inch (undensified) layers. Densification was achieved by dropping a 3.1 kg, 15 cm diameter steel tamper from a height of 150 mm 100 times, followed by 2 minutes of vibration using a rivet gun over a piece of 19 mm plywood. Since the modified pullout box was 0.36 m deep, in order to accommodate a 25 mm clearance at the top of the box for seating of the normal load apparatus, 0.33 m of soil was placed.

5.3.1.2 Placement of Geogrid Specimens

Three geogrid specimens were tested in the modified pullout box at once. Each specimen was cut to 4 bars (0.08 m) wide and 4 ribs (0.66 m) long. Soil was brought to the level of the pullout slot in the front of the box and the specimens were laid such that they were each separated by at least 7.5 cm with the two side specimens were 7.5 cm from the side walls of the apparatus (see figure 5.1). This spacing between the boundary and the geogrid helps minimize the boundary effects of the side wall. Specimens were placed such that the in-air portion between the front of the box and the clamp was no more than 25 mm (1 inch), thus reducing the potential for in-air creep. The rear of the specimens passed through a slot in the back of the box and extended 8 cm beyond that. The rear of the specimens were monitored continuously for movement with LVDTs.

5.2.1.3 Application of Loads

Tensile and normal loads were applied to geogrid specimens. Normal load was applied first by advancing the California Bearing gear box until the desired normal force, which
Chapter 5. Test Procedures

would result in 95 kPa, was reached. The load was monitored electronically and, over the course of testing, was adjusted as necessary to compensate for settlement of the sand. Tensile loads were applied by means of dead weights and pulleys as was discussed in chapter 3. The steel weights were added with as little shock as possible, but quickly enough as to approximate an immediate loading of the geogrid. The boom part of the pulley-boom apparatus was held up with a block until all the necessary loads were in place. Once the loads were ready the block was carefully removed.

Over time and with extension of the geogrid specimens, the pulley booms would rotate. This flattened the angle of the booms to the horizontal and slightly increased the lever arm length. As this occurred weights were taken off to adjust the loading to a constant value.

All measurements were measured electronically with volt meters which were recorded manually on data sheets. Measurements of extension were also taken manually with tape measures to check for consistency of the electronic measurements over extended periods of time.

5.3.2 CRD Pullout Tests

In this section the program for CRD pullout tests is described along with the preparation of the geogrid specimen and the sand sample. The testing procedure presented here is similar to the one used in a previous study (Raju, 1995).

In the current study a displacement-controlled procedure was used. The objectives of this work were to examine the effect of confining stress on pullout behavior of 3 different geogrids and study their behavior at small strains representative of working conditions in
Chapter 5. Test Procedures

the field. Moreover, the effects of staged tests, wherein a single pullout test is comprised of three stages of successive changes in normal stress application, were also examined.

Movement of the clamp during pullout tests was maintained by the servo-controlled hydraulic actuator described in chapter 3. The servo controller is programmed by software to displace at a given constant rate by a demand displacement signal. This demand displacement rate is a ramp function that varies linearly with time.

Normal stress was varied for each stage with 4 kPa applied for the first stage, 50 kPa applied to the second and 4 kPa again applied to the third. A specified displacement rate of 0.10 mm/min was used for all tests in the interest of capturing as much of the small strain behavior as possible. Tests were carried to either a limiting pullout resistance in the 4 kPa tests, or to sufficient displacement to define the small strain behavior in the 50 kPa tests.

5.3.2.1 Test Preparation

Each test involved the trimming and instrumentation of the geogrid specimen, pluviation of the sand sample and, for the 50 kPa tests, arrangement of surcharge loading apparatus. Each task is detailed below including checking procedures to ensure accuracy of the instruments.

5.3.2.2 Preparation of Test Specimen

Typically each geogrid specimen was trimmed to 0.50 m wide by 1.15 m long resulting in an embedment length of 0.93 m. Approximately 0.2 m of geogrid specimen length protruded from the pullout box for clamping purposes. Prior to placement in the pullout box a geogrid specimen was instrumented with strain gauges (see Appendix A). Five
Chapter 5. Test Procedures

gauges were placed at approximately equal intervals along the centre rib of the test specimen such that one gauge was left outside the pullout box while the other four were confined. After the specimen was placed the rear LVDT was attached with a tell-tale cable to the end of the specimen with epoxy. When the test specimen was placed properly within the pullout box there was typically a clearance of about 8 cm between its edges and the inside side walls of the pullout box.

5.3.2.3 Preparation of Test Apparatus

Prior to each test the pullout box was cleaned of all residual sand using a heavy duty vacuum cleaner. Care was taken not to contaminate the sand in any way since it was reused for each test. Once the box was empty the pluviation hopper was lifted onto the pullout box with an overhead crane and seated on the frame. Plastic drop sheet was placed over the instrumentation bay to prevent damage to the equipment during pluviation.

5.3.2.4 Sand Sample Pluviation and Geogrid Placement

Sand was delivered to the pullout box by means of the hopper described in chapter 3. With the hopper seated on the pullout apparatus, the trap doors were secured and the perforated sieve installed. Sand was placed in the top of the hopper by pouring it from barrels using a drum lifter and an overhead crane. Approximately 10 cm of sand was placed in the top of the hopper which, when pluviated generally resulted in about 7 to 8 cm of sand in the box. The sand in the hopper was leveled with a wooden straight edge that saddled the top of the hopper. The trap doors of the hopper were released by pneumatically-controlled stops which supported the doors. Four layers of sand were
Chapter 5. Test Procedures

placed in this manner which brought the sand surface up to the mid-height level of the opening in the front wall. Occasionally it was necessary to level off the surface of the sand in parts or to target certain low laying areas by selectively pluviating sand. At this point the geogrid specimen was placed.

The instrumented geogrid specimen was carefully lifted from the working platform and placed in the pullout box on top of the sand. The strain gauge wires for gauges 2 to 5 were sheathed in nylon tubes protecting the wires and gauges from damage and allowed for unencumbered movement of the wires throughout the test. The nylon tubes were passed through a 16 mm hole in the back of the box and the wires connected to a Wheatstone bridge completion circuit. The specimen was then passed through the front slot and brought to the clamp. The first strain gauge is located on that portion of the geogrid outside the box and, therefore, its wires were taken directly to the completion circuit. Once the sample was in place, the remaining sand was pluviated. It should also be noted that sand density was checked by placing open tins in the pullout box during pluviation. It was found that the density varied by 1.2% on average between the top and bottom layers and by 0.17% between tests. Placement of sand by air pluviation in this manner is an established method (see Vaid and Negussey, 1988).

5.3.2.5 Application of Surcharge Load

The first and third stages of each test, under a normal stress of 4 kPa, were performed with simply the overburden of soil above the geogrid specimen to the top of the soil sample. The second stages of each test were under 50 kPa surcharge, applied by a PVC bag filled with water. The bag was placed carefully on top of the soil sample and tucked in around the edges. The lid of the pullout box was placed on the box and care was taken
Chapter 5. Test Procedures

not to damage the bag during this process. The reaction beams were then placed over the lid and tightened. Once the bag was in place and the reaction frame secured water was introduced to the bag by means of a connecting tube from the laboratory water supply. Filling continued until water appeared in the venting tube at the level of the lid. When the bag was filled it was connected to a triaxial cell with a pressure transducer at the same height as the top of the soil. The triaxial cell was also fitted with a regulator which fed off the laboratory air supply. Prior to each test the transducer was calibrated and the target voltages corresponding to 50 kPa and atmospheric pressure were determined. These voltages were used to determine the pressure during the test. Before the test was commenced the water bag was pressurized by advancing the regulator until 50 kPa was read both from the regulator and an independent volt meter. During the test the pressure in the bag typically changed due to volume changes in the sand and therefore the pressure was monitored and adjusted as necessary throughout the test using the regulator.

5.3.2.6 Clamping of the Test Specimen

Once the geogrid specimen and the sand sample were in place and, in stage 2 tests, the PVC bag pressurized, the geogrid was clamped. This was achieved by advancing the lower jaw of the clamp to the position of the geogrid specimen, attaching the central insert and upper jaw and bolting the assembly together. Four C - clamps were also attached to secure the clamp assembly and prevent slipping of the geogrid specimen.

5.3.2.7 Test Procedure

Pullout tests were operated with a computer controlled electro-mechanical interface. Software was designed (Raju, 1995) to specify displacement rates and continually
monitor and record digital input from 16 instrument channels. For the current study 10 channels were monitored and recorded: 1 load cell, 2 front clamp LVDTs, 1 back LVDT connected to the geogrid specimen by a tell-tale cable, 5 strain gauges and 1 pressure transducer connected to the PVC bag.

The first stages of each test were carried out until a relatively constant, residual pullout resistance was achieved, typically after 7 to 8 mm of front clamp displacement. At this point the test was halted. The second stage, under a 50 kPa surcharge, was carried out usually the following day, typically to a total displacement of about 15 mm, sufficient to characterize the small strain behavior of the geogrid. The third stage began once the PVC bag had been drained and removed. It was carried to a similar displacement as the first stage. Time was left between tests to allow residual strains within the geogrid specimens to dissipate from previous tests. All displacement-controlled tests started with specifying a demand displacement rate, \( r_d \), which was 0.10 mm/min (see figure 5.3). Tests typically involved approximately 75 minutes of testing for 4 kPa tests and 90 minutes for the 50 kPa tests.

5.3.2.8 Post-Test Procedure

After each stage the hydraulic pump was shut off, the data acquisition system stopped and, in the case of the 50 kPa tests, the water pressure released and reaction frame dismantled. After all 3 stages of each pullout test were completed the sand sample was removed using the heavy duty vacuum cleaner to the level of the geogrid specimen. The specimen was then carefully exhumed from the instrumentation wires and inspected for damage, particularly to the strain gauge sites. The balance of the sand was removed and the box cleaned. Tests typically took 6 days to complete: 1 day to clean the box and
pluviate the sand to mid-height of the box, 1 day to instrument the geogrid specimen, 1 day to connect the instruments and pluviate the balance of the sand and 1 day for each stage to carry out the test.

Figure 5.2: Top view of soil sample showing geogrid specimen placement in load-controlled pullout box.
Chapter 5. Test Procedures

Figure 5.3: Schematic of demand load signal for CPS tests.

Figure 5.4: Schematic demand signal in displacement-controlled pullout tests.
CHAPTER 6
TEST RESULTS

6.1 Introduction

Results of all tests are presented in this chapter. A variety of comparisons were made possible in this study. In some tests different geogrid types were comparatively tested, while in others a single geogrid type was tested in a variety of ways. The testing program can be divided into two categories of tests: Tensile and Pullout. Furthermore these categories can be subdivided into displacement or load-controlled. The chapter is also organized along the same divisions. Tests are referenced in plots with respect to unique alpha numeric codes. Tables 6.1 to 6.4 give the codes for all tests. Each test is accounted for in the tables and thus, each entry represents a single test. A capital “R” signifies a repeated test. Table 6.5, located at the end of the chapter, is a schematic of each test showing the loading conditions, variables and constants, and fixity conditions.

6.2 Tensile Tests

Tensile tests focused on three issues: load – extension response (CRE), load-time response (LR) and extension-time response (CPS and C). In all, 16 tensile tests were carried out.

6.2.1 Displacement-Controlled Tensile Tests

Displacement controlled tensile tests include constant rate of extension, and load-relaxation tests. Each is discussed below with respect to the quantities measured and results observed.
6.2.1.1 Constant Rate of Extension (CRE)

A total of 10 Constant Rate of Extension tests were carried out on specimens of Tensar UX1500, Miragrid 12XT and Stratagrid 700 geogrids. Tensar specimens, with a single exception, were all 4 ribs wide (0.08 m), the Miragrid and Stratagrid specimens were single rib. One Tensar sample was 0.105 m wide for the sake of clamping constraints. The Tensar geogrid is described in terms of load - extension and load - strain behavior.

<table>
<thead>
<tr>
<th>Constant Rate of Extension (CRE)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Test Code</strong></td>
</tr>
<tr>
<td>---------------</td>
</tr>
<tr>
<td>T01</td>
</tr>
<tr>
<td>T01R</td>
</tr>
<tr>
<td>T2</td>
</tr>
<tr>
<td>T5</td>
</tr>
<tr>
<td>T10</td>
</tr>
<tr>
<td>T50</td>
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<tr>
<td>S01</td>
</tr>
<tr>
<td>S50</td>
</tr>
<tr>
<td>M01</td>
</tr>
<tr>
<td>M50</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Load Relaxation (LR)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Test Code</strong></td>
</tr>
<tr>
<td>----------------------</td>
</tr>
<tr>
<td>LR2</td>
</tr>
<tr>
<td>LR6</td>
</tr>
</tbody>
</table>

Table 6.1: Test codes and parameters for displacement controlled tensile tests.

The Miragrid and Stratagrid specimens are described by load-strain response. The effects of geogrid type, and rates of displacement are also presented.

Figure 6.1 shows the results of 4 CRE tests each at different rates on Tensar UX 1500. A distinctly hyperbolic load extension response is noted in all tests, consistent with literature on geosynthetics (Giroud, 1994; Ling, 1992). All specimens demonstrate an
Chapter 6. Test Results

initial linearity, followed by a change in slope. This behavior continues until a marked reduction in slope is noted to a nearly constant load. The behavior is distinctly sensitive to rate of displacement, with a ranking of initial slopes and ultimate strengths. Successively higher initial moduli and ultimate strength are noted with increasing rate of displacement in all tests. Moreover, it appears that the magnitude of ultimate extension at failure is inversely proportional to the displacement rate. The 10 mm/min test (T10) failed after approximately 130 mm of extension whereas the 0.10 mm/min test (T01) failed after nearly 175 mm. A similar trend is noted for T2 and T5. All specimens except the 0.10 mm/min failed at approximately the ultimate tensile strength of the material as reported by the manufacturer from ASTM D 4595.

Additional CRE tests were performed on all three geogrids. Specimens were instrumented with strain gauges and a load-strain response was generated. Tests were performed at 0.10 and 50 mm/min for a total of 6 tests. Figures 6.2, 6.3 and 6.4 show the results. The Tensar UX 1500 specimen tested at 50 mm/min (T50) displayed a three-stage response. From zero strain to approximately 1.5 % the relationship is almost linear after which a flattening of the load-strain response is seen. This trend continued to ~12.5 % strain whereupon the response becomes progressively flatter until failure. T01R displayed a distinctly three-stage response. The T50 test failed within the clamp at approximately the ultimate tensile strength as reported by the manufacturers. Stratagrid 700 response is shown in Figure 6.3. Distinct curvature is noted throughout both tests. The 50 mm/min test displays 3 regions. The slope is initially steep but decreasing up to approximately 28 kN/m at 3 % strain where it begins to stiffen. The slope is only slightly concave up for most of the test until about 8 to 9 % strain after
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which the slope steadily increases to rupture, signifying a strain-hardening response. The load at rupture appears to approach the ultimate tensile load of the material as reported by the manufacturer. The 0.10 mm/min test shows the same initial steep and concave down behavior as the higher rate. The behavior continues until approximately 2.5 % strain where the slope flattens. The slope is slightly less than the 50 mm/min test. The test was not carried to rupture.

Miragrid 12XT displays only slight curvature with an initially steeper modulus up to approximately 15 kN/m at 1 % strain (Figure 6.4). Between 1 and 3.5 % strain a very slight downward concavity is perceptible. Beyond 3.5 % strain a slight strain stiffening is noted. Miragrid 12XT is only slightly sensitive to displacement rate. The specimen did not fail at the reported ultimate tensile strength of 92.8 kN/m, as it failed in the clamp before reaching ultimate material strength.

6.2.1.2 Load-Relaxation (LR) Tests

The load-time response of geogrids was investigated through load-relaxation (LR) tests. Polymeric geogrids exhibit a stress response that is time-dependent as a direct result of being viscoelastic materials (Van Krevelen, 1972). Time-dependent stress manifests itself in load-relaxation behavior observed when a polymer is kept at a constant total strain and measurements of force are made over time. Load-relaxation tests (LR2 and LR6) were carried out on Tensar UX 1500 specimens in the Instron testing apparatus. LR2 was brought to a tensile load of 2 kN (25 kN/m), LR6 to 6 kN (75 kN/m). These loads corresponded to strains of approximately 2.48 % and 10.68 % respectively. Figure 6.5 shows the results of the two tests as tensile load versus time. Both specimens display significant reduction in tensile load with time. Moreover, both curves appear to approach
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an asymptotic limiting value of load. Original loads on the samples were 64.5 kN/m and 21.5 kN/m for LR2 and LR6 respectively. Loads of 45 and 15 kN/m after 20 hours were observed for LR6 and LR2 respectively.

6.2.2 Load-Controlled Tensile Tests

Polymeric geogrids display a time-dependent strain response in addition to the stress response discussed above. This response is referred to as creep and is characterized by increasing strain with time under constant stress. To investigate the time-dependent nature of geogrid strains a total of 4 load-controlled tensile tests were carried out on specimens of Tensar UX 1500. Two types of test were carried out: cyclic pseudo-static (CPS) and creep (C). All specimens were 4-ribs wide.

<table>
<thead>
<tr>
<th>Cyclic Pseudo-Static (CPS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Code</td>
</tr>
<tr>
<td>-----------</td>
</tr>
<tr>
<td>CPS1</td>
</tr>
<tr>
<td>CPS2</td>
</tr>
<tr>
<td>CPS3</td>
</tr>
<tr>
<td>Creep (C)</td>
</tr>
<tr>
<td>C45</td>
</tr>
</tbody>
</table>

Table 6.2: Test codes and parameters for load-controlled tensile tests.

6.2.2.1 Cyclic Pseudo-Static (CPS) Tests

Figure 6.6 shows the results of 4 CPS tests on Tensar UX 1500 specimens. The plots are extension versus time in semi-log space. The test codes for the tests are CPS1, CPS2 and CPS3 (see table 6.3). All tests exhibit an initial region of mostly elastic strain, occurring during loading, wherein the slopes of the curves are steep. Following this is a region characterized by sharply decreasing slopes of all test data. This behavior is
consistent with *secondary creep*, characterized by a steadily decreasing rate of strain. The second region is followed by a third, termed *tertiary creep*, where the slopes steadily increase until rupture. In semi-log space an upward concavity of the curve signifies an accelerated creep failure (Juran et. al., 1991). It is readily seen that there is a distinct ranking of slope in all tests which is proportional to the applied loads. Moreover, the onset of tertiary creep is also ranked according to load level, with higher loads resulting in earlier onset and higher accumulated extension than lower loads. CPS1 failed prematurely in the clamp and so did not display tertiary creep. CPS2 and CPS3 both ruptured within the third stage.

### 6.2.2.2 Creep (C) Test

A single dead-load creep test was performed on a 4-rib specimen of Tensar UX 1500. The conditions of the test are presented in table 6.2 along with the test code. The test was carried out to compare the strain-time behaviour of HDPE geogrids unconfined with that of creep pullout behaviour (confined condition). Moreover, the test allowed comparison with manufacturer’s data for the same material. The results of extension versus time data are presented in table 6.6 together with CPS test plots. It can be seen that in semi-log space there is a slightly perceptible upward concavity in the plot between 100 and 1000 hours, although the trend seems to lessen thereafter.

### 6.3 Pullout Tests

Two types of pullout test were carried out on geogrids: Creep Pullout (CP) and Constant Rate of Displacement pullout (CRD). The two tests addressed different aspects of geogrid confined behaviour. Creep pullout tests provided a comparison of the strain-
Chapter 6. Test Results

time behaviour of HDPE geogrids. CRD pullout tests addressed soil geogrid interaction and strain and resistance mobilization of three geogrids. Tables 6.3 and 6.4 give the test codes and parameters.

6.3.1 Creep Pullout (CP) Tests

Creep pullout tests were performed on three 4-rib specimens of Tensar UX 1500 (HDPE) geogrids. The objectives of the tests were to examine the effects of in-soil confinement on creep behavior of geogrids, characterize the long-term behavior of geogrids and compare the data with that from unconfined creep tests (CPS and C) in the current study and from manufacturer’s data. The tensile loads imposed on the geogrid specimens were 45 kN/m, 50 kN/m and 62.5 kN/m. While these are in no way representative of loads typically found in reinforced soil structures, they allow characterization of the limit states of strength of the material. Moreover, high loads also enabled the specimens to progress through all three stages of creep in relatively short order. It is standard industry practice to carry out creep tests to 10 000 hours (~ 1.1 years). This was accomplished for one unconfined specimen. Measurements of load, displacement, time and temperature were taken over a period of several months, yielding extension time plots. Table 6.3 presents the test codes and parameters for all CP tests.

<table>
<thead>
<tr>
<th>Creep Pullout (CP)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Code</td>
</tr>
<tr>
<td>-----------</td>
</tr>
<tr>
<td>CP1</td>
</tr>
<tr>
<td>CP2</td>
</tr>
<tr>
<td>CP3</td>
</tr>
</tbody>
</table>

L - Load
E - Extension

Table 6.3: Test codes and parameters for creep pullout tests.
Chapter 6. Test Results

Figure 6.7 is a plot of extension versus time in semi-log space for all specimens. Extension was measured at the clamp located originally within 2.5 cm of the front of the box. The plots show a distinct ranking of cumulative displacement according to the magnitude of initial load. Three distinct regions are visible, identical in form to the unconfined creep cases, though this is less pronounced for the 62.5 kN/m test which was dominated by consistently increasing extension rate (see Figure 6.7). There is also a ranking in the slopes of the plots according to magnitude of initial load which implies that the rate of extension is sensitive to initial applied load. The tests exhibit varying degrees of upward concavity from just perceptible (CP1) to marked (CP3). This, too, is sensitive to the initial applied load, as is evident from the increasing time for the onset of tertiary creep with decrease in initial load. The CP3 and CP2 specimens failed after 8 and 2739 hours respectively, while CP1 continued straining for more than 10 000 hours.

Failure patterns for the two ruptured specimens (CP2 and CP3) were noted. Rupture boundaries did not occur cleanly across the specimen but rather varied along the length of the specimens. For both CP2 and CP3, 75 % of rupture (according to cross-sectional area) took place within the soil with the remaining occurring outside. This suggests that some strain of the geogrid occurred within the soil. Moreover, small but measurable displacements were recorded by rear LVDTs of the back ends of specimens CP2 and CP3. These results are desirable as they lend strong support to the assertion that the tests were in fact creep pullout. Temperature throughout the test duration was monitored. The average temperature was 20.5° C with a standard deviation of 1.9° C.
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6.3.2 Constant Rate of Displacement (CRD) Pullout

Constant rate of displacement (CRD) pullout tests were carried out on specimens of Tensar UX 1500, Stratagrid 700 and Miragrid 12XT. The objectives of the tests were to describe geogrid pullout behaviour through changes in force, displacement and strain under a constant rate of pullout displacement and a given vertical stress. Between tests vertical stress was changed for a given geogrid to observe its impact on geogrid force, displacement and strain. Finally, the testing program was designed to investigate the merits of staging pullout tests as described in section 5.3.2.

Tests were performed at 0.10 mm/min which is slower than a proposed ASTM standard test method for pullout and previous studies (Raju, 1995; Bergado et. al., 1993), however an aim of the tests was to characterize the small strain behavior of geogrids buried in sand. The displacements and strains examined are comparable to those found in field structures (Fannin and Hermann, 1990). Table 6.4 presents the test codes referred to in the text along with the test parameters.

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Geogrid</th>
<th>Displacement Rate (mm/min)</th>
<th>Normal Stress (kPa)</th>
<th>Test Stage</th>
<th>Constants</th>
<th>Variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>T4011</td>
<td>Tensar UX1500</td>
<td>0.1</td>
<td>4</td>
<td>1</td>
<td>dD/dt</td>
<td>L,D,ε</td>
</tr>
<tr>
<td>T50011</td>
<td>Tensar UX1500</td>
<td>0.1</td>
<td>50</td>
<td>2</td>
<td>dD/dt</td>
<td>L,D,ε</td>
</tr>
<tr>
<td>T4012</td>
<td>Tensar UX1500</td>
<td>0.1</td>
<td>4</td>
<td>3</td>
<td>dD/dt</td>
<td>L,D,ε</td>
</tr>
<tr>
<td>T4011R*</td>
<td>Tensar UX1500</td>
<td>0.5</td>
<td>4</td>
<td>N/A</td>
<td>dD/dt</td>
<td>L,D,ε</td>
</tr>
<tr>
<td>S4011</td>
<td>Stratagrid 700</td>
<td>0.1</td>
<td>4</td>
<td>1</td>
<td>dD/dt</td>
<td>L,D,ε</td>
</tr>
<tr>
<td>S50011</td>
<td>Stratagrid 700</td>
<td>0.1</td>
<td>50</td>
<td>2</td>
<td>dD/dt</td>
<td>L,D,ε</td>
</tr>
<tr>
<td>S4012</td>
<td>Stratagrid 700</td>
<td>0.1</td>
<td>4</td>
<td>3</td>
<td>dD/dt</td>
<td>L,D,ε</td>
</tr>
<tr>
<td>M4011</td>
<td>Miragrid 12XT</td>
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<td>4</td>
<td>1</td>
<td>dD/dt</td>
<td>L,D,ε</td>
</tr>
<tr>
<td>M50011</td>
<td>Miragrid 12XT</td>
<td>0.1</td>
<td>50</td>
<td>2</td>
<td>dD/dt</td>
<td>L,D,ε</td>
</tr>
<tr>
<td>M4012</td>
<td>Miragrid 12XT</td>
<td>0.1</td>
<td>4</td>
<td>3</td>
<td>dD/dt</td>
<td>L,D,ε</td>
</tr>
</tbody>
</table>

* Test carried out to verify the agreement of results with previous study (Raju, 1995)
L - Load, D - Displacements, ε - strain

Table 6.4 Test codes and parameters for constant rate of displacement pullout tests.
Chapter 6. Test Results

The first letter in the code signifies the proprietary name of the geogrid (T, Tensar; M, Miragrid; S, Stratagrid). The second part of the code refers to the normal stress on the geogrid and is either 4 or 50. Note that the normal stress change corresponds to the different stages of the tests. The third part of the code, the number 01, refers to the displacement rate of the test which, for most tests, was 0.1 mm/min. A duplicate test were carried out at 0.5 mm/min to ensure repeatability with the literature results. The final digit is an index which signifies the first or second run with a test configuration. Thus, the test M4012 describes the second pullout test on Miragrid 12XT under 4 kPa normal stress, displaced at a rate of 0.10 mm/min.

6.4.1 Pullout Displacements

Displacements were measured at the clamped end \( (d_c) \) and the embedded end \( (d_e) \). Results are presented as plots of clamped end displacement versus embedded end displacement. As geogrids undergo displacement of the clamped end, resistance is mobilized along the geogrid such that less relative displacement between the soil and the geogrid occurs with distance along the geogrid. This results in progressive displacement from the front to the back of the geogrid inclusion.

Figures 6.8, 6.9 and 6.10 are plots of \( d_e \) versus \( d_c \) for all tests on all three geogrids. There is a clear distinction between the behavior of geogrids in tests under 4 kPa and 50 kPa. Without exception all 50 kPa test specimens experienced no embedded end movement corresponding to the flat line plots. In contrast tests under 4 kPa show a three part response. Initially, no embedded end displacement is noted, corresponding to the horizontal plot for all Figures. At some point embedded end movement is initiated, whereupon a non-linear relationship between clamped and embedded end displacement is
set up. The final stage, after continued clamp displacement, is a progressively more linear relationship between clamped and embedded end displacement. There exist differences between the geogrid responses. Test T4011 showed initial embedded end response at about 2 mm of clamp displacement. Miragrid showed a similar response with movement of the embedded end starting around 2.3 mm. In contrast the Stratagrid showed no embedded end displacement until just over 4 mm, although there is a blip in the data as far back as 3 mm.

Stage 3 responses were similar in appearance to stage 1 however, all tests exhibited earlier initiation of embedded end displacements. The Tensar grid showed embedded end movement as early as 0.6 mm. S4012 began moving at about 2 mm, while the Miragrid exhibited the least change, moving at about 2.5 mm. Stage 3 tests also appeared to exhibit slightly elongated non-linear portions of the response.

The extensibility of the geogrid is responsible for the three part behavior. It has been shown that very stiff inclusions, such as aluminum sheets, do not show any progressive displacement (Raju, 1995). A possible explanation for the earlier embedded end displacement of all stage 3 tests are that the grids were rendered stiffer by virtue of locked in strains accrued during the previous two stages.

6.4.2 Pullout Resistance

Pullout resistance, $P_r$, was measured at the clamped end for all tests and was plotted against clamp displacement. All geogrids displayed increasing pullout resistance with increasing clamp displacement. Pullout resistance of geogrids was also seen to increase with applied normal stress. Resistance versus displacement relationships for tests under 4 kPa showed a distinctly three-part response: an initial linear region where the increase in
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pullout resistance is proportional to clamp displacement; a non-linear region with a reduction in the increase of pullout resistance with clamp displacement; a residual condition following a peak pullout resistance and characterized by a limiting resistance. The response of geogrids is also discussed with respect to an initial modulus which is the ratio of pullout resistance per meter width divided by the displacement at that point and corresponding to the linear region. Figures 6.11, 6.12 and 6.13 show plots of pullout resistance versus clamp displacement for all geogrids and all tests.

The Tensar geogrid reached a maximum pullout resistance of approximately 4 kN/m at $d_c = 4$ to 6 mm for both tests under 4 kPa normal stress. The response appears to be, again, three part with an initial linear portion up to approximately 1 mm at 2.8 kN/m, corresponding to an initial ratio of pullout resistance to displacement of about 2.8 MPa. Following the initial linear response was a non-linear region, between $d_c = 1$ and 3 mm, characterized by steadily decreasing pullout resistance with displacement. This non-linear region blended into a limiting pullout resistance at large displacement (Figure 6.11). Test T4012 was the stage 3 test of the Tensar series carried out on the same specimen as test T4011. A similar response is noted for this test. The test T4011R is a separate test carried out under 4 kPa on Tensar UX 1500 to assess the repeatability of the data. The data show good repeatability.

Test T50011 was the Tensar stage 2 test carried out under 50 kPa of normal stress. This test is characterized by a stiffer initial linear region terminating at just over 0.5 mm and corresponding to a pullout resistance of 3 kN/m. The resulting initial modulus was 6 MPa. In contrast to the 4 kPa tests, T50011 exhibited a much longer period of non-
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linearity and at test termination still had not realized a limiting pullout resistance (Figure 6.11).

Stratagrid tests exhibited similar responses to Tensar. Test S4011 showed an initial linear response to about 1 mm, followed by a long period of non-linearity to approximately 5 mm. A peak resistance of 6.5 kN/m was realized at 5 - 6 mm of displacement. S4012 showed a slight stiffening in its linear region over that of S4011 with an initial modulus of about 3.3 MPa compared to 2.2 MPa for S4011. The linear portion also appears to be slightly longer than for S4011, blending into the non-linear region at just over 1 mm of displacement. The non-linear response for S4012 occurs more rapidly than that of S4011, characterized by a very sharp decline in pullout resistance with displacement. A peak resistance of 5.8 kN/m is achieved by S4012 at \( d_c = 3 \) mm. Beyond this, a reduction in pullout resistance with displacement is clearly visible. Resistance appears to be approaching a residual value.

Resistance versus displacement for S50011, the Stratagrid test under 50 kPa normal stress, is plotted on Figure 6.12. A much greater pullout resistance is achieved in this test than both S4011 and S4012. The test appears to display a much longer region of linearity than T50011, (Figure 6.11), degrading at about 3.5 mm into the beginning of the non-linear region. The corresponding pullout resistance at this point is approximately 13 kN/m, resulting in an initial pullout resistance to displacement ratio of 3.7 MPa.

The Miragrid 12XT tests, M4011, M4012 and M50011 showed similar responses to the Stratagrid series. A peak pullout resistance of around 5.8 kN/m was reached by M4011, just slightly greater than M4012. A similar stiffening of the pullout response in the stage 3 test (M4012) is noted with Miragrid as well. Both tests reach a peak pullout resistance
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followed by a reduction and eventual residual response. M50011 shows a response similar to S50011 with a significant linear response to about 2 mm, in contrast to T50011. The initial pullout to displacement ratio is 3.7 MPa.

Relative displacement mobilizes the interaction between the geogrid and soil. In this study the force measured at the clamped end was observed to increase with displacement to a peak and then approach a limiting pullout resistance. Embedded end displacement, $d_e$, was observed only after a certain clamp displacement, depending on the grid type. It is worth noting that in all stage 1 tests, regardless of geogrid type, the point of "lift-off" of the $d_e$ versus $d_c$ plot (Figures 6.8, 6.9 and 6.10), where initiation of embedded end displacement occurred, corresponded approximately with the end of the non-linear force-displacement region in Figures 6.11, 6.12 and 6.13.

6.4.3 Geogrid Strain

Strain along the geogrid specimens was measured during pullout tests. Five strain gauges were attached to each specimen according to the bonding procedure outlined in Appendix A. It was possible, by placing five gauges along the length of the specimen, to monitor the strain from the clamped end through to the embedded end. This provided a complete characterization of strain along the specimen throughout the test. Data are presented for all stages as strain against displacement of the clamp (Figures 6.14 to 6.22). In these plots the quantity $x/L_{ei}$ is referenced. $X/L_{ei}$ is the ratio of the distance of the strain gauge from the inside of the front boundary at the start of the test ($x$) with the initial embedded length of the geogrid ($L_{ei}$). The origin of the frame of reference is the inside boundary of the soil box. A negative $x/L_{ei}$ implies that the gauge is located outside the pullout box.
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Common to all pullout tests is a distinct ranking of the strain magnitude from gauge 1 to gauge 5; progressively less strain is developed with distance along the geogrid specimen at all points in a test. With stage 1 tests, under 4 kPa, strain was ultimately developed at all gauges. Strain, like pullout resistance, was progressive with clamp displacement and in the 4 kPa tests reached a limiting condition. The Miragrid 12XT specimen reached a limiting strain condition at about 4.5 to 5 mm of clamp displacement. Stratagrid 700 exhibits a similar behavior reaching a limiting condition between 5 and 6 mm. T4011 was only taken to 3.5 mm of clamp and appears still to be developing strain, however, T4012 reached a constant strain condition at approximately 4 to 5 mm displacement (Figure 6.20). Ultimate strains in T4011 ranged from 0.02 to 0.14 % over all gauges, while S4011 and M4011 ranges were 0.02 to 0.115 % and 0.02 to 0.11 % respectively. The condition of limiting strains is indicative of a pullout failure since the geogrid is displacing and no longer continuing to accumulate strain. Comparing the stage 1 \( d_e \) versus \( d_c \) plots (Figures 6.8 to 6.10) with the stage 1 strain versus \( d_c \) plots (Figures 6.14 to 6.16) it can be observed that the onset of limiting strains approximately corresponds with the point of lift-off. The exception to this was the strain in gauge 1, located outside the pullout box, which continued to strain in all stage 1 plots.

Development of strain in the Tensar 4 kPa tests was characterized by a concave down curvature throughout the test for all gauges (Figures 6.14 and 6.20). This is in contrast to the Miragrid (Figures 6.16, 6.22) and Stratagrid 4 kPa (Figures 6.15, 6.21) tests where a downward concavity is seen in only SG 1 and SG 2 throughout the tests. Strain gauges 3 to 5 exhibit a reversal of curvature with a concave up response to approximately 1 to 2.5 mm for Miragrid 12XT and 2 to 4 mm for Stratagrid 700 (Figures 6.15 and 6.20). It
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should be noted that the inflection point on the plot of SG 5 versus clamp displacement for Miragrid (Figure 6.16) and Stratagrid (Figure 6.15) corresponds closely with the initiation of embedded end displacement (Figures 6.10 and 6.9) for both grids.

Tests under 50 kPa (Figures 6.17, 6.18 and 6.19) all developed higher strains than the 4 kPa tests. Maximum strains were 0.04 to 0.375 % for T50011, while again Stratagrid was lower at 0.025 to 0.26 %. M50011 appears to respond only to about 2 mm at which the data give probably erroneous results (Figure 6.20) as no embedded end displacement was realized in M50011. This is probably due to spalling of gauges, although, M4012 results seem to be correct (Figure 6.22). Up to 2 mm displacement the strains were comparable to T50011. Also noted among the 50 kPa tests was a condition of zero strain for SG5 corresponding to no embedded end displacement for these tests. The responses appear to be more linear in the 50 kPa tests than 4 kPa tests over the range of displacements.

Strains in all stage 3 tests were less than stage 1 tests. This could be due to “locked-in” strains accrued during stage 1 and particularly stage 2 tests. Alternatively, could have arisen due to a local densification of soil in front lateral bearing members during stages 1 and 2.
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### TENSILE TESTS

<table>
<thead>
<tr>
<th>Test type</th>
<th>Displacement Controlled</th>
<th>Load Controlled</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant rate of extension @ 0.1, 2, 5, 10, 50 mm/min</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### LOAD CONTROLLED PULLOUT TESTS

- **Constant** rate of displacement @ 0.1, 2, 5, 10, 50 mm/min
- **Load relaxation test** @ 2.48, 10.68% strain
- **Cyclic Psuedo static** @ 50, 56.3, 62.5 kN/m
- **Creep test** @ 45 kN/m
- **Creep pullout tests** variable normal stress @ 45, 50, 62.5 kN/m

### NOTES:
- dE/dt, e and L are extension rate, strain and load respectively.
- dx/dt is displacement rate, CRD is constant rate of displacement.

**Table 6.5:** Schematic of test conditions for all tests.
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Figure 6.1. Relationship between tensile load and clamp displacement for Tensar UX1500. Initial specimen length = 375 mm.

Figure 6.2: Relationship between tensile load and average strain for Tensar UX1500 by ASTM D 4595.
Figure 6.3: Relationship between tensile load and average strain for Stratagrid 700 using the GRI GG1 single rib test method.

Figure 6.4: Relationship between tensile load and average strain for Miragrid 12XT using the GRI GG1 single rib test method.
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Figure 6.5: Relaxation of tensile load with time for Tensar UX 1500.

Figure 6.6: Creep extension versus time for Tensar UX 1500 from CPS and C tests.
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Figure 6.7: Creep extension versus time for creep pullout tests on Tensar UX 1500.

Figure 6.8: Relationship between $d_e$ and $d_c$ for Tensar UX 1500 at 4 and 50 kPa normal stress
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Figure 6.9: Relationship between $d_e$ and $d_c$ for Stratagrid 700 under 4 and 50 kPa

Figure 6.10: Relationship between $d_e$ and $d_c$ for Miragrid 12XT under 4 and 50 kPa.
Figure 6.11: Pullout resistance with displacement for Tensar UX 1500 under 4 and 50 kPa normal stress.

Figure 6.12: Pullout resistance with displacement for Stratagrid 700 under 4 and 50 kPa normal stress.
Figure 6.13: Pullout resistance with displacement for Miragrid 12XT under 4 and 50 kPa normal stress.

Figure 6.14: Mobilization of strain with $d_c$ for the Tensar UX 1500 geogrid at $\sigma = 4$ kPa.
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Figure 6.15: Mobilization of strain with $d_c$ for Stratagrid 700 under $\sigma = 4$ kPa.

Figure 6.16: Mobilization of strain with $d_c$ for Miragrid 12XT under $\sigma = 4$ kPa.
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Figure 6.17: Mobilization of strain with $d_c$ for Tensar UX 1500 under $\sigma = 50$ kPa.

Figure 6.18: Mobilization of strain with $d_c$ for Stratagrid 700 under $\sigma = 50$ kPa.
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Figure 6.19: Mobilization of strain with $d_c$ for Miragrid 12XT under $\sigma = 50$ kPa.

Figure 6.20: Mobilization of strain with $d_c$ for Tensar UX1500 under $\sigma = 4$ kPa (stage 3).
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Figure 6.21: Mobilization of strain with $d_c$ for Stratagrid 700 under $\sigma = 4$ kPa (stage 3).

Figure 6.22: Mobilization of strain with $d_c$ for Miragrid 12XT under $\sigma = 4$ kPa (stage 3).
CHAPTER 7
ANALYSIS OF TEST RESULTS

7.1 Introduction

In this chapter test results are interpreted with respect to load-strain-time relationships, tensile stiffness and soil-geogrid interaction. Load-strain and tensile stiffness at large strain are examined with regard to constant rate of extension tensile tests and a comparison of the three geogrids tested is presented. Further, geogrids are compared on the basis of the ratio of the measured tensile strength and the manufacturer's reported ultimate short-term tensile strength (ASTM D 4595). Load-controlled tensile (CPS and C) tests are interpreted within the framework of an ultimate limit state. Isochronous load-strain – time and rupture load – time curves will be used as well.

Creep pullout tests are discussed with respect to in-soil creep behavior, and a comparison between confined and unconfined creep strain is made. Geogrid CRD pullout behavior is analyzed comparatively between three geogrids with respect to resistance, strain and displacement. A comparison between CRD pullout and CRE load-strain responses is used to further understand the progressive nature of soil geogrid interaction. The influences of normal stress and geogrid structure are also used to explain pullout behaviour. Load-strain and soil-geogrid interaction are discussed within the framework of a strain range of interest, comparable to that found in field structures.
7.2 Tensile Tests

7.2.1 Displacement-Controlled Tensile Tests

Displacement-controlled tensile tests have provided an understanding of the short-term load-strain and long-term load-time behavior of polymeric geogrids. Interpretation is presented with respect to load-strain behaviour from polymeric theory. Also, the apparent difference between local and average strain is discussed and some possible interpretation given. Long-term strain - time relationships developed for Tensar UX1500 demonstrate the influence of time and strain magnitude on the creep behaviour of HDPE geogrids.

7.2.1.1 Load - Strain Relationships

Figure 7.1 presents plots of tensile load versus average strain for specimens of Tensar UX 1500 at 0.1, 2, 5, 10 and 50 mm/min. Data are presented from tests T01, T2, T5, T10 and T50. The initial slopes of the curves was computed from the data. These varied from about 1350 kN/m to 800 kN/m for the 50 mm/min and 0.10 mm/min tests respectively. Although this is not a true modulus of elasticity, it does serve to describe the behavior of the geogrids. Well known models of viscoelasticity such as the Maxwell or Voigt model predict time dependency with a dashpot element coupled with a spring. The dashpot yields greater resistance with increased rate of strain, thus the stiffness should increase in kind. From this it is seen that the geogrid load-strain behaviour reflects the presence of the polymer.

Also observed in the plots is the increasing strain at failure with decreasing strain rate. This behaviour follows from the decrease in stiffness associated with smaller strain rates.
A specimen with a lower modulus will require more strain to achieve the same load at rupture. Also apparent in the figure, with the exception of T50, is a reduction in the rupture loads with decreasing strain rate. This can be explained by relaxation behavior of polymers. Relaxation results in a reduction of tensile strength of the geogrid with no coincident reduction in strain, another property of viscoelastic materials (see Figure 6.5). The 0.10 mm/min test (T01) took approximately 32 hours to rupture the sample. Relaxation tests carried out over 20 hour periods (Figure 6.5) show that significant relaxation in load takes place even within 1 hour of a sustained tensile load.

CRE tests were also carried out on Stratagrid 700 and Miragrid 12XT at different rates. Figures 7.2 to 7.4 show plots of tensile load versus average strain for all three geogrids at rates of 0.10 and 50 mm/min over a strain range of 0 % to 2 %. The Tensar plots are from tests T01R and T50, carried out at UBC (Figure 7.2). It can be seen from these plots that Miragrid 12XT and Stratagrid 700 exhibit slightly less rate effect than Tensar UX 1500 (Figure 7.2) in this strain range. Stratagrid (Figure 7.3) also exhibited rate dependency although to a lesser degree than Tensar. Initial moduli of Stratagrid 700 were 1400 kN/m and 1800 kN/m for the 0.10 (S01) and 50 mm/min (S50) tests respectively (Figure 7.4). Miragrid 12XT displayed almost no rate effect with both M01 and M50 reporting approximately 1600 kN/m.

7.2.1.2 Normalized Tensile Load, $T/T_R$

In chapter 6 (Figures 6.2, 6.3 and 6.4) it was noted that a ranking existed between the load versus clamp displacement curves for the three different geogrids. Stratagrid 700 reached a higher tensile load at a given strain than Tensar UX 1500 which in turn realized a higher load than Miragrid 12XT. Figures 7.5 and 7.6 show normalized tensile load
versus average strain. \( \frac{T}{T_R} \) is the measured load divided by the reported “ultimate wide width tensile load” for all three materials according to ASTM D-4595. Average strain is computed knowing the initial distance between the clamps and using displacement measurements. In the range of strain observed there is only a slight difference in normalized load at any specific strain between the geogrids for the tests T01, S01 and M01 at 0.10 mm/min displacement rate. The 50 mm/min tests (T50, S50 and M50) also show small differences. If, however, comparison is made on the basis of strain levels in each geogrid for a given normalized load then some differences exist. Recall that the LTDS (Long-Term Design Load) is the minimum allowable strength a geogrid must have at the end of the design life of a reinforced soil structure. Conservative values of LTDS were computed for each geogrid according to FHWA suggested guidelines. The range of LTDS values for all three was between 10\% and 20\% of the ultimate short-term tensile load reported by the manufacturers. The horizontal line plotted on Figure 7.6 represents the approximate average range of LTDS values for all three geogrids. Apparent differences are seen between the three geogrids tested. The approximate strains required for Miragrid 12XT, Stratagrid 700 and Tensar UX 1500 to achieve 15\% of their ultimate short-term strengths are 1.3\%, 1.5\%, and 2.2\% respectively.

### 7.2.1.3 Local Strain versus Average Strain

As noted in table 6.1 some CRE specimens were instrumented with strain gauges allowing a comparison to be made between local strain, measured by a strain gauge, and average strain computed over the gauge length of the specimen. Figures 7.7, 7.8 and 7.9 show comparisons of average and local strain for all three geogrids. Without exception, all geogrids showed the same behavior: local strain was always less than average strain at
any given load. Moreover, the relationships between tensile load and local strain was distinctly linear for all load levels, in contrast to the average strain plots which show a hyperbolic relationship. These findings are similar to another study (Austin et al., 1993), that found similar specimen length effects. They reported reductions in tensile modulus of 11% when increasing the gauge length by 25%. It appears that smaller specimen gauge lengths - that is, the length over which strain is measured - yields stiffer responses. One possible explanation for this in the case of Tensar geogrids is that a flowing of material from the junctions into the longitudinal ribs is occurring during tensile testing. Since average strain is measured across junctions larger strains would be expected. It is not clear why Stratagrid and Miragrid which do not have the same structure as Tensar might be exhibiting the same behaviour.

7.2.2 Load-Controlled Tensile Tests

7.2.2.1 Strain - Time Response

Long-term unconfined tensile tests yielded a characterization of the creep response of the Tensar UX 1500 geogrid. Results are summarized in plots of strain versus time in semi-log space, strain rate versus strain magnitude (Sherby - Dorn plot) and isochronous load-strain curves (Figures 7.10 to 7.12, respectively)

Creep plots in semi log strain - time space yield information on the development of strain over a long period of sustained load. The rate of creep strain was also evaluated, qualitatively, from this plot. Figure 7.10 is a plot of total (average) strain versus time for the Tensar UX 1500 CPS and C tests. A diagnostic feature of strain time plots is the point of onset of tertiary creep, characterized by an increasing upward concavity of the
Chapter 7. Analysis of Test Results

data, corresponding to an increasing strain rate with time. This behavior signifies imminent creep rupture and is considered the commencement of instability. The point in time at which specimens become unstable varies according to the applied load. Test CPS3 showed almost immediate instability. CPS2 appears to begin exhibiting instability at around 20 hours. The lone creep test (C), carried out at 45 kN/m (52.3 % of the reported ASTM D - 4595 strength) may be showing instability anywhere from 500 to 1000 hours. Alternatively, it may be seen that for all specimens exhibiting a marked instability, the onset was after 15 % to 20 % strain. This finding is in good agreement with literature suggesting that there exists a "limit of instability" of strain for HDPE, at 20 % strain, beyond which failure is imminent (Gourc, 1987).

Figure 7.11 is a plot of strain rate versus strain magnitude in semi log space. Presented this way, the data show that instability occurs at different strain rates corresponding to different load magnitudes. However, the strain magnitude of between 15 and 20 % is still observed to be a limit of stability. Load levels of 56.3 kN/m (CPS2) and 62.5 kN/m (CPS3) show reversal in the values of strain rate with strain signifying first a slowing in the reduction of strain followed by an increase in strain rate with strain corresponding to imminent creep rupture. The 50 kN/m test (CPS1) which was not carried to rupture appears to be approaching a limiting strain rate for the limited data available. The 45 kN/m test (C) appears also to be approaching a limiting condition.

The normalized tensile load values (T/Tₚ) for the creep tests are 0.73, 0.65, 0.58 and 0.52 corresponding to 62.5, 56.25, 50 and 45 kN/m loads respectively. Tensar literature reports a creep reduction factor, defined as the creep limited strength - corresponding to a performance limit strain of 10 % (FWHA, 1990) - of the material (ASTM D - 5262) to
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the ultimate short-term tensile strength (ASTM D - 4595) of 0.35. The values of load in this study are admittedly very high and not likely to be found in actual field structures (Fannin and Hermann, 1990), however, these data do contribute to the understanding of creep behavior of geogrids.

Isochronous load - strain plots were generated with the above data and compared with the manufacturer’s data (Figure 7.12). The data show reasonably good agreement for shorter time periods. The higher loads used in this study complement the manufacturer’s data at lower loads.

Frequently creep data are presented in the form of rupture load versus time plots. This relationship demonstrates the reduction in strength accompanying prolonged periods of sustained load. Figure 7.13 shows the results of unconfined creep test data plotted in log - log space. The influence of time on polymer strength is clearly evident from the negative slope of the plot. A recent publication of the Washington State DOT utilizes such plots to help determine the anticipated reduction in strength from long-term sustained loading.(56)

7.3 Pullout Tests

In this chapter pullout test data are analyzed and interpreted. Load-controlled (creep) pullout tests (CP) are analyzed with respect to strain-time relationships. Comparison is made between creep pullout and CPS and C tests, the former addressing the issue of confined creep and the latter unconfined creep strain. Displacement-controlled pullout tests are analyzed with respect to the effects of confining stress and geogrid type. Soil-geogrid interaction, quantified by the interaction factor is also addressed.
Chapter 7. Analysis of Test Results

Load-controlled pullout tests address an ultimate limit states for geogrid reinforced soil, while displacement-controlled tests address a serviceability limit state.

7.3.1 Load-Controlled Pullout Tests

Creep pullout tests were performed to investigate the influence of confinement on the creep behavior of geogrids. Figure 7.14 shows the plots of creep pullout tests CP2 and CP3 and companion in - isolation tests CPS1 and CPS3 carried out at the same loads (see table 6.2).

In order to make a comparison of the data from creep pullout and unconfined creep tests it was necessary to recognize that the two tests were different. In the unconfined tensile tests the boundary conditions are fixed - fixed, whereas in the creep pullout tests, because there was freedom of movement at the back end of the specimen during the test the boundary conditions were fixed and free. Thus, the strain data calculated from the pullout tests represent half of the value of a confined tensile test. Through superposition the appropriate data can be calculated. These are represented by the dashed lines in Figure 7.14. The three tests on specimens of Tensar UX 1500 at three different load levels revealed that the magnitude of creep is lessened by confinement in granular media.

In an excellent study, Min et al. (1995), found that creep strain of geogrids was significantly effected by confinement. Although, they also found that the creep strain rate was not appreciably affected by confinement.

7.3.2 Displacement-Controlled Pullout Tests

All CRD pullout tests were carried out at a displacement rate of 0.10 mm/min in the interest of collecting as much information about the in-service strain range as possible.
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Geogrid pullout behavior is described in terms of the mobilization of strain, pullout resistance, and interaction factor with increasing pullout displacement.

7.3.2.1 Mobilization of Strain in Geogrids

Strain was measured continuously in all geogrids during pullout. Strain gauges were bonded, according to the procedure outlined in Appendix A, to pre-specified points along the length of the geogrid as shown in Figure 7.15. Strain gauges are referenced in the text as SG1 to SG5. SG1 is located outside the pullout box, while SG2-5 are within the box. From this arrangement strain profiles were generated for each geogrid and each test. Figures 7.16 to 7.21 show the resultant profiles for Tensar, Stratagrid and Miragrid. Strain magnitudes are plotted against the normalized distance from the front wall of the pullout apparatus, computed by dividing the distance, x, of the gauge from the inside front boundary of the pullout apparatus by the total initial embedment length, $L_e$. All gauges were bonded to the same longitudinal series of ribs along the centre line and so represent a strain profile. The profiles are progressive in that they are taken at several points during the pullout process according to increasing increments of clamp displacement.

Miragrid and Stratagrid profiles during stage 1 under 4 kPa, (Figures 7.18 and 7.20), show an initial non-linear response for which the plot is curved, tending towards a linear, straight line profile with increasing clamp displacement. There is immediate strain in strain gauge 1 (SG1) followed by measurable, but progressively less strain, in gauges 2 and 3. There is no significant strain reported in gauges 4 and 5 until approximately 3 mm of clamp displacement. A similar response is seen in the Tensar stage 1 (Figure 7.16). Strain is noted in all gauges at $d_c = 1$ mm. Greater strains were developed in Tensar and
Chapter 7. Analysis of Test Results

Stratagrid than in Miragrid. However a more linear, or flatter shape of the profile is apparent in Miragrid and Stratagrid than in Tensar which shows slightly more curvature in its profile. It would appear that all three geogrids are similar in their respective strain profiles, however strain magnitudes seem to be slightly less in Miragrid and Stratagrid than in Tensar. This is in agreement with the ranking of stiffness presented above.

Stage 2 tests, carried out under 50 kPa normal stress, show a dramatically non-linear strain profile (Figures 7.17, 7.19 and 7.21). Marked reversals of curvature are apparent in all profiles. In contrast to tests under 4 kPa normal stress, non-linearity remains throughout the test duration. Even at maximum displacement the majority of strains take place in the first three gauges for all geogrids. Negligible strains are noted for all tests in gauge 5, the most deeply embedded, consistent with plots of strain development (Figures 6.18, 6.19 and 6.20) and embedded end displacement (Figures 6.9, 6.10 and 6.11). Tensar and Miragrid both show similar magnitudes of strain at 1 mm of displacement, in contrast to Stratagrid which shows less at this point. Moreover, Tensar develops strain further back, to gauge 3, than does Stratagrid. This again is similar to the behavior noted above in stage 1 tests where the progression of strain along the specimen occurred earlier for the Tensar than the Stratagrid specimen.

7.4.3 Soil - Geogrid Interaction

Soil - geogrid interaction is characterized by the interaction factor, $F*\alpha$ which is the ratio of the shear resistance mobilized by the geogrid to the normal stress acting on the grid surface. In this study the interaction factor was computed as follows:
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\[ F \cdot \alpha = \frac{\tau_{\text{avg}}}{\sigma_n} = \frac{P_R}{2L_e W \sigma_n} \]  

where:

- \( \tau_{\text{avg}} \) is the average shear mobilized over the original imbedded area
- \( P_R \) is the pullout resistance measured by the load cell
- \( L_e \) is the embedment length
- \( W \) is the width of the specimen
- \( \sigma_n \) is the normal stress acting on the geogrid

Equation 7.1 is an average interaction factor. It does not account for the reduction in geogrid area with pullout displacement, nor does it account for the non-linear distribution of shear stress developed during the mobilization of pullout displacement. The latter is evident from the strain profiles discussed above and is a result of the extensibility of geogrids. The use of the average interaction factor, however, serves to show some of the relative differences between geogrid types. Moreover, the use of \( L_e \), the original embedded length is appropriate in this study as maximum displacements for all three geogrids were on order of 0.1 % of the total embedded length.

An improvement on average interaction factor will come from a discussion of the inter-relationships of interaction factor, strain and pullout displacement as well as tensile force distribution in geogrids during pullout.

7.4.3.1 Influence of Normal Stress on Interaction

It is clear that increasing the normal stress between two surfaces will increase the force required to overcome the friction between them. With extensible inclusions in-soil this
situation is complicated by the progressive straining of the inclusion. Due to this the mobilization of pullout resistance in extensible geogrids is progressive, highest at the clamped end and decreasing towards the embedded end. Under even high normal stresses a very stiff inclusion, such as a planar steel sheet, would transmit the tensile force from the clamp to the embedded end readily with clamp displacement, mobilizing a uniform interaction along its length. Conversely, a very flexible inclusion would exhibit a very rapid decay of strain with length, mobilizing most of its interaction in a narrow region towards the clamped end. Depending on the stiffness of the geogrid, its behavior will lie between these two extremes. The pullout tests carried out at two different normal stresses demonstrate this stiffness dependency.

Presented in Figure 7.22 are the computed interaction factors for all three geogrids under 4 kPa normal stress (stage 1 and 3 tests). It is apparent that staging generates repeatable results on the approach to a peak interaction factor. The mobilized interaction factor is very similar for all three geogrids up to about 2 mm clamp displacement. At this point the Tensar data begin diverging significantly from the others. It appears that a peak stress ratio of about 0.63 is reached by approximately 8 mm and a limit, or residual of about 0.6 by 10 mm clamp displacement. The Stratagrid and Miragrid plots continue on their initial slope beyond the point of divergence until a clamp displacement of 3 mm. Stratagrid and Miragrid reach peak interaction factors of approximately 0.7 to 0.85 at about 3 to 5 mm clamp displacement. Towards 6 to 7 mm of clamp displacement $\tau/\sigma$ for Stratagrid and Miragrid appear to be converging to a value of around 0.65 - 0.70.

Figure 7.23 shows the plots of interaction factor for all geogrids under 50 kPa normal stress. For comparison, stage 1 data are plotted as dashed lines. Evident in this plot is
the dramatic effect of normal stress on the mobilization of interaction in all geogrids. At comparable clamp displacements $t/\sigma$ is less than tests at $\sigma = 4$ kPa. This finding is corroborated by other studies (e.g. Raju, 1995; Farrag, et. al., 1993). Farrag et. al., testing a low junction strength geogrid similar to Stratagrid, did not find a convergence to a single value of stress ratio with clamp displacement. Raju, (1995), found that the interaction factor for varying normal stress plotted within a band, depending on the geogrid type, with large clamp displacements. Stage 2 tests, like stage 1 tests show no marked difference between the computed average interaction factor for all three geogrids to about 5 mm clamp displacement. Beyond this point Stratagrid is seen to plot consistently above the others. All curves are still increasing at test termination. Table 7.2 summarizes the computed stress ratios by the total area method, (Ochiai et al., 1992).

Relative displacement between geogrid and soil is necessary for the mobilization of shear stresses. Both geogrid - soil interface friction and passive bearing mechanisms require relative movement to be activated. Thus, strain profiles give a picture of the profile of relative displacement and therefore, interaction development along the geogrid. The tests at $\sigma = 4$ kPa all exhibited fairly uniform strain distributions (Figures 7.16, 7.18 and 7.20) and correspondingly, with little exception, most of the geogrid is utilized in developing resistance along its length. While this the mobilization of interaction is progressive in these tests, within 5 - 6 mm of clamp displacement a uniform condition is approached. In contrast, under 50 kPa normal stress, none of the geogrids tested achieved a uniform strain condition in the range of clamp displacements of interest. This signifies that there was a very non-uniform distribution of relative displacement and hence, interaction along the geogrid. Thus, resistance to the tensile force imposed on the
Chapter 7. Analysis of Test Results
/geogrid was confined to those areas of the grids - towards the front - where significant strain was developed.

7.4.3.2 Influence of Geogrid Type on Interaction Factor

The differences between computed interaction factors for all three geogrids at a single normal stress can be used to demonstrate the influence of geogrid structure on interaction. The three geogrids used in this study can be differentiated on the basis of the mechanisms they use to resist pullout force and displacement. The Tensar geogrid is a smooth HDPE geogrid with very strong, rigid bearing members connecting longitudinal members. In the direction normal to the plane of the geogrid, parallel to an applied normal stress, Tensar has raised ridges at the junction points (see Figure 4.3). This structure lends itself to a passive bearing based resistance mechanism by virtue of the available area and rigidity of members orthogonal to the direction of pullout. Some of the basic equations developed to analyze the passive bearing capacity of geogrids in general (Jewell, et. al., 1984) were based on a structure not unlike the Tensar geogrids used in this study. Moreover, tests carried out on smooth HDPE geomembranes, (Raju, 1995), were used in this study to calculate the contribution of skin friction to the average coefficient of interaction from the smooth surfaces of the Tensar geogrid. It was found to be on the order of 20 % to 25 % considering the available planar area of the Tensar geogrid. Therefore, the Tensar geogrid might be considered as predominantly a bearing capacity based resistance geogrid. Conversely, both Stratagrid 700 and Miragrid 12XT, considered "low junction strength" geogrids, have much less prominent features orthogonal to the direction of testing. However, Stratagrid has greater planar area available for frictional interaction than Tensar. Miragrid and Stratagrid also have rougher
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surface textures, promoting interlock with the surrounding sand. One study (Raju, 1995) examined the frictional contribution of Miragrid geogrids in pullout resistance. By cutting the bearing members out of the grid and testing the longitudinal ribs in pullout it was found that 65\% to 70\% of the total pullout resistance was due to friction. This structure lends itself to the development of a frictional based resistance mechanism. A study (Farrag et. al. 1993) demonstrated that the frictional contribution to interaction for a geogrid similar to Stratagrid 700 was on average 75\%.

The differences in structure reveal themselves in the computed $\tau/\sigma$ plots under 4 kPa normal stress (Figure 7.22). The Tensar response is hyperbolic in shape, requiring approximately 8 mm clamp displacement to mobilize a peak. This is likened to a classic plastic failure envelope for cohesionless soil. Stratagrid and Miragrid both reveal a strain softening behavior, with a peak and residual resistance. The bond for Stratagrid and Miragrid increases with small relative displacement owing to the friction and interlock between the soil and the grid until an ultimate resistance is achieved. Once the peak resistance is reached it reduces to a residual value. This reduction to a residual could be due to restrained dilation in the neighbourhood of the interface occurring during relative movement of the grid and soil. No measurements were made of $\sigma_n$ at the interface in this study, however, evidence exists in the literature that $\sigma_n$ increases locally due to restrained dilatancy (see e.g. Juran, et al., 1988). Moreover, ribbed or rough textured inclusions are known to mobilize resistance more efficiently than sheet inclusions (Ingold, 1983a). With the rough surface of Stratagrid and, to a lesser extent Miragrid, interlock is encouraged between the soil and the geogrid resulting in-soil to soil shearing. The Tensar geogrid, however, resists pullout predominantly with passive bearing which
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requires more relative movement between the geogrid and the soil to reach the full passive state than the frictional based resistance of Stratagrid and Miragrid. The surface texture of Tensar geogrids is much smoother than Stratagrid and Miragrid. Thus, dilation is not encouraged through skin friction as readily in Tensar than in the others. Although, upon inspection of the Tensar geogrid specimen after exhuming, considerable surface scratching and gouging was noted on the surface. Obviously, energy was spent in abrading the grid surface and it is probable that this was reflected in the pullout resistance.

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<th>4 kPa</th>
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<tr>
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<td>1 mm</td>
<td>Peak</td>
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<tr>
<td>Tensar UX1500</td>
<td>0.34</td>
<td>0.61</td>
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<td></td>
<td>(10 mm)</td>
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<tr>
<td>Stratagrid 700</td>
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<td>0.84</td>
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<td></td>
<td>(5.7 mm)</td>
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<tr>
<td>Miragrid 12XT</td>
<td>0.29</td>
<td>0.77</td>
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<td>(4.3 mm)</td>
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Table 7.1 Summary of computed interaction factors by the Total Area Method.

7.4.4 Interaction and Strain

As noted in section 7.4.3, the interaction coefficient computed was an average value. This formulation is referred to as the total area method of computing the interaction coefficient after Bonckewicz et al. (1986). It assumes the entire initially embedded area of the geogrid is utilized in resisting pullout force, that is, shear stress is assumed to act uniformly over the plane of the geogrid. However, since geogrids are extensible and furthermore, because the mechanisms of interaction require relative displacement between geogrid and soil, the total area method does not completely describe the interaction between soil and geogrid. The extensibility of geogrids, as revealed by the strain profiles in Figures (7.16 to 7.21), show that there is a non-uniform distribution of
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strain and therefore relative displacement along the geogrid. To complete the discussion of interaction between geogrids and soil in this study a comprehensive plot is developed showing the inter-relationship of interaction factor (total area method) with both displacement and strain. From such a plot the differences in normalized resistance help explain the differences in strain development. Moreover, the progressive strain development in geogrids will refine the understanding of normalized resistance.

Figure (7.24) shows a plot of interaction factor against clamp displacement on the upper portion and strain development with clamp displacement on the lower portion for Miragrid 12XT at $\sigma = 4$ kPa. It is evident that the strain development in the geogrid corresponds to increasing normalized resistance. The plot shows $\tau/\sigma$ increasing at an almost linear rate as strain is developed in successively deeper points along the geogrid. The limiting interaction coefficient realized between 4 and 6 mm clamp displacement corresponds to limiting strains in all gauges. This implies that the grid is no longer elongating and is effectively acting as a rigid inclusion. The entire length of the geogrid is contributing to resistance beyond this point.

In contrast, Figure 7.25 shows the same type of plot for Stratagrid 700 under 50 kPa. This plot shows that there is no strain in gauge 5 for the entire test and negligible strain in gauge 4 up to approximately 4 to 5 mm of clamp displacement. This suggests that the geogrid is mobilizing normalized resistance with only a portion of its length. Figure 7.26 shows the Tensar UX1500 data under 50 kPa. It can be seen that slightly more strain is developed in the more deeply embedded gauges for a given clamp displacement than for Stratagrid. Stratagrid 700 is stiffer than Tensar UX1500 in unconfined tension over the
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strain range of interest, (see section 7.2.1.1), which could account for the differences observed.

From figure 7.22 it appears that all the geogrids lie in a fairly narrow band of computed interactions factor up to approximately 1 to 1.5 mm of displacement. The strains mobilized in the geogrids at this point show a ranking in accordance with CRE test results with Tensar reporting slightly more strain than Stratagrid and Miragrid. Beyond this there is a significant difference between the computed factors of Stratagrid and Miragrid and those of Tensar. Stratagrid and Miragrid show a peak followed by a reduction to post peak behaviour, while Tensar shows a gradual approach to a peak over larger displacement. It appears that the curves of all three grids may be approaching each other. Under $\sigma = 50$ kPa the differences are much less pronounced with all geogrids plotting in a very narrow band up to approximately 5 mm, beyond which Stratagrid appears to report consistently higher interaction factors than Tensar and Miragrid.

To conclude the discussion on interaction factor, it seems that the differences between the geogrids' interaction factors is most significant at peak, reached at approximately 4 to 5 mm of displacement under 4 kPa normal stress and at large displacements ($> 5$ mm of displacement) under 50 kPa.
Chapter 7. Analysis of Test Results

Figure 7.1: Relationship between tensile load and rib strain (Tensar UX 1500)

Figure 7.2: Comparison of load versus average strain at different extension rates for Tensar UX 1500.
Figure 7.3: Comparison of load versus average strain at different extension rates for Stratagrid 700.

Figure 7.4: Comparison of load versus average strain at different extension rates for Miragrid 12XT.
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Figure 7.5: Normalized tensile load versus average strain for all geogrids at 50 mm/min clamp displacement.

Figure 7.6: Normalized tensile load versus average strain for all geogrids at 0.10 mm/min clamp displacement.
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Figure 7.7: Comparison of local and average strain for Tensar UX 1500 at 0.10 mm/min.

Figure 7.8: Comparison of local and average strain for Stratagrid 700 at 0.10 mm/min.
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Figure 7.9: Comparison of local and average strain for Miragrid 12XT at 0.10 mm/min.

Figure 7.10: Creep strain versus time for Tensar UX 1500 from C, CPS1,2 and 3.
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Figure 7.11 Sherby - Dorn plot for Tensar UX 1500. Data taken from CPS4, CPS45 and CPS5.
Figure 7.12: Comparison of creep data with isochronous load strain curves (from manufacturer).

Figure 7.13: Rupture load versus time for tests C, CPS1, CPS2, CPS3, CP1, CP2 and CP3 (see table 6.3 and 6.4).
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Figure 7.14: Comparison of confined (CP2, CP3) and unconfined (CPS1, CPS3) load-strain-time behavior.

Figure 7.15: Layout of strain gauges along a typical geogrid sample (plan view).
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Figure 7.16 Strain profile for Tensar UX 1500 with increasing clamp displacement under $\sigma = 4$ kPa (Stage 1).

Figure 7.17: Strain profile for Tensar UX 1500 with increasing clamp displacement under $\sigma = 50$ kPa (Stage 2).
Chapter 7. Analysis of Test Results

Figure 7.18: Strain profile for Stratagrid 700 with increasing clamp displacement under \( \sigma = 4 \) kPa (Stage 1).

Figure 7.19: Strain profile for Stratagrid 700 with increasing clamp displacement under \( \sigma = 50 \) kPa (Stage 2).
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Figure 7.20: Strain profile for Miragrid 12XT with increasing clamp displacement under $\sigma = 4$ kPa (Stage 1).

Figure 7.21: Strain profile for Miragrid 12XT with increasing clamp displacement under $\sigma = 50$ kPa (Stage 2).
Chapter 7. Analysis of Test Results

Figure 7.22: Computed interaction factor for all geogrids under 4 kPa (Stage 1 and 3 tests).

Figure 7.23: Computed interaction factor for all geogrids under 50 kPa (Stage 2 tests).
Figure 7.24: Interaction factor and corresponding rib strain versus clamp displacement for Miragrid 12XT under 4 kPa (Stage 1).
Figure 7.25: Interaction factor and corresponding rib strain versus clamp displacement for Stratagrid 700 under 50 kPa (Stage 2).
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Figure 7.26: Interaction factor and corresponding rib strain versus clamp displacement for Tensar UX 1500 under 50 kPa (Stage 2).
8.1 Summary

This study was carried out as part of a collaborative effort between the University of British Columbia and the BC Ministry of Transportation and Highways to investigate the behavior of geogrid reinforced soil walls. The main objectives of the study were to:

- Investigate the comparative behavior of different geogrid types in pullout and tension.
- Investigate, through a variety of testing procedures the short term unconfined load - strain behavior of geogrids and apply that knowledge in explaining pullout test results.
- Compare and contrast the long term load - strain - time behavior of HDPE geogrids in both confined and unconfined conditions.
- Put forward recommendations for the BC MoTH on the feasibility of building geogrid reinforced soil structures greater than 5 m in height.
8.2 On The Program of Testing

8.2.1 Testing Apparatuses and Instrumentation

- Tests were carried out on three different apparatuses located at UBC in Vancouver, and the BC MoTH laboratories in Victoria. Unconfined tensile tests were carried out on a standard Instron tensile testing machine. Confined creep pullout, as well as an unconfined creep test, were performed using a modified pullout apparatus designed and built at the BC MoTH laboratories in Victoria. Pullout tests were carried out on a large scale pullout apparatus designed and built at the University of BC (see Raju, 1995).

- Geogrid specimens were tested in a variety of ways to obtain the load - strain, strain - time and load - time characteristics of HDPE geogrids. The apparatus was capable of testing specimens in tension under a range of strain rates. Unconfined creep tests were also performed. Measurements of load and extension were taken continuously with an integrated data acquisition system.

- An existing pullout box was modified with a constant tensile loading apparatus to carry out long term creep pullout tests on HDPE geogrid specimens. Tests were performed at three load levels and acted as companion tests to the unconfined creep tests carried out with the Instron. Measurements of tensile load, normal load and front and back displacement were taken. In those specimens exhibiting back end displacement, an average strain was computed. Temperature was monitored over the duration of the tests.
Chapter 8. Summary and Conclusions

- A large scale pullout test apparatus was adopted for use in the current study to carry out pullout tests. The apparatus had all rigid boundaries excepting the top which had a stress controlled boundary. The geogrid specimen was pulled through a slot in the front wall of the box. The apparatus was equipped with a servo-controlled, electro-hydraulic system that was capable of pulling a geogrid specimen from the pullout box at a constant rate of displacement. Instrumentation measured pullout force, front clamp displacement, geogrid embedded end displacement, normal stress acting on the soil sample and strain along the geogrid specimen throughout the test.

8.2.2 Materials and Test Procedure

- Two different soils were used in the program of testing: a uniformly graded medium sand, used in the UBC pullout tests, and a well graded gravelly sand used in the creep pullout tests. A means of delivering sand to the UBC pullout box yielded consistent samples at between 85 to 90% relative density. The efficacy of the method was demonstrated by reproducible results. The gravelly sand was placed by hand and densified by vibration to 100% of Standard Proctor density.

- The geogrids used in this work represent two types of geogrids commonly used in practice: those of high junction strength (Tensar UX 1500) and low junction strength (Stratagrid 700 and Miragrid 12XT).

- Two principal types of tests were carried out: tensile tests (CRE, LR, CPS and C series) and pullout tests (CP and CRD series). Tensile tests included confined and unconfined conditions. Unconfined tensile consisted of industry standard index tests (ASTM D-4595 and GRI GG1) at varying rates of displacement, long term unconfined creep tests and load relaxation tests. Confined tests consisted of long
term creep pullout tests at three different load levels which acted as companion tests
to the unconfined creep tests. Pullout tests were carried out at a constant rate of
displacement of 0.10 mm/min. The rationale was to capture the behavior of geogrids
in pullout at the small, in-service range of strain. Each test consisted of stages of
different normal stress. The first stage was carried out at 4 kPa, the second at 50 kPa
and the third again at 4 kPa. The purpose of this was to collect information about the
effect of normal stress variation on the pullout behavior of geogrids in sand while
investigating whether staging is a feasible means of performing many tests in
relatively short order.

8.2.3 Test Results and Interpretation

8.2.3.1 Constant Rate Extension (CRE)

- Tensile tests carried out on three geogrids revealed information about the effects of
  extension rate, polymer type and geogrid design on the load-strain characteristics. CRE
tests reveal a ranking in ultimate short term strength between the geogrids. The tests also
investigated the effects of different rates of extension on load-strain behaviour. It was
found that there exists rate effects in all grids to a varying degree. Tensar UX 1500
showed the most marked effects, followed by Stratagrid 700 and Miragrid 12XT. Miragrid showed very little rate effect.

- Tensile test data were normalized by the manufacturers' reported ultimate short-term
tensile strength for all three geogrids. There exists a reasonably narrow band of ranking
between the three geogrids with respect to normalized tensile strength at a given strain for
tests carried out at 0.10 mm/min and a significantly wider range for tests performed at 50
mm/min. Approximate long term design strengths (LTDS) were computed according to FHWA guidelines for all three geogrids. Differences were found to exist between the geogrids with respect to the strain required to achieve the computed LTDS (Figure 7.6). Tensar UX 1500 and Stratagrid behaved similarly, requiring approximately 0.75 % more strain than Miragrid 12XT to achieve the computed LTDS.

8.2.3.2 Cyclic Pseudo-Static, Creep and Creep Pullout Tests

- Long-term creep tests, both confined and unconfined, have demonstrated the creep potential of HDPE geogrids. CPS data agree reasonably well with isochronous load – strain curves provided by the manufacturer for the same product.
- Rupture load versus time data from tests carried out exclusively on HDPE geogrids reveal a decided reduction in strength with time for HDPE geogrids.
- Comparison between creep and creep-pullout tests show that confinement significantly reduces the potential for creep strain for HDPE geogrids. This finding is in agreement with other research (e.g. Min et al., 1995) and has implications for design.

8.2.3.3 CRD Pullout Tests

- CRD pullout tests carried out on three commercially available geogrids have yielded information on the progressive strain and interaction of geogrids buried in sand.
- Pullout test data demonstrate the usefulness of staging pullout tests, subject to an understanding of the potential for possible residual strains in the geogrid and/or local densification of soil in front of transverse bearing members. Results have confirmed the progressive mobilization of strain and resistance of geogrids in pullout found in the previous study, (Raju, 1995), and recognized by others (Ochiai, et al., 1992; Juran
et al., 1991). Results show that differences in geogrid design and structure yield differences in mobilized pullout resistance. The results of tests under low confining stress (4 kPa), taken together with the unconfined tensile tests at 0.10 mm/min, imply that the differences in mobilization of pullout resistance are due mainly to the differences between the geogrid structures as opposed to the load – strain behavior of the material. It is believed that the number of bearing elements per length of geogrid has an influence on pullout behaviour. This could explain the observed differences in interaction factor for all three geogrids. Tests under high confining stress (50 kPa) show considerably less difference between geogrids with respect to interaction at small displacements.

8.3 Implications for Design

The results of this study have implications for generic geogrid specification. From the results of creep and creep pullout tests, together with the survey of British codes of practice and recent publications from Washington State some suggestions can be made regarding height limitations of reinforced soil walls.

8.3.1 Geogrid Characterization

Tests carried out have allowed comparison between geogrids on the basis of polymer type, geogrid design and structure, and confining stress.

8.3.1.1 Polymer Type

- Results of CRE tests at different extension rates have yielded differences in the load-strain behaviour of geogrids. The limited data suggest that HDPE geogrids appear to be more sensitive to rate effects than the polyester geogrids tested.
8.3.1.2 Geogrid Design

Geogrid design refers to the structure of the geogrid and how it was manufactured. Knitted or woven and punched-drawn are two manufacturing processes which yield different geogrid structures.

- Pullout behaviour appears to be strongly influenced by geogrid structure. Both Stratagrid and Miragrid achieved higher peak interaction factors than Tensar. Also, the displacements necessary for Stratagrid and Miragrid were less than for Tensar. For small clamp displacements \( d_c \leq 1 - 2 \text{ mm} \) the computed interaction factors of the three geogrids lie in a fairly narrow band. However, differences become significant at larger displacements \( d_c \geq 2 - 3 \text{ mm} \). The strains associated with the larger clamp displacements appear to encompass the strain range of interest (0.5 % to 1.0 %).

8.3.1.3 Effect of Confinement

- Results of CRD and creep pullout tests have demonstrated that confinement effects the strain time and soil-geogrid interaction behaviour. Differences between the computed interaction factors for all three geogrids were negligible for tests carried out at 50 kPa for displacements up to 5 mm.

- Creep pullout test results show that confinement significantly reduces creep strain compared to unconfined creep tests. No creep pullout tests were carried out on Stratagrid or Miragrid.

In conclusion the significant differences in the geogrids tested appear to be

- the respective strains required to achieve a given normalized tensile strength (Figure 7.6);
Chapter 8. Summary and Conclusions

- the mobilization of interaction, including the peak values of interaction factor and the displacements and strains required to achieve them under low confining stress.

8.3.2 Wall Height Limitation

The significant issues surrounding walls of substantial height are geogrid strength and deflections. Implicit in polymeric reinforcement is the distinction between short and long term cases for deflections and strength.

8.3.2.1 Short-Term Strength

- Short term stability is concerned with the construction period and immediately thereafter. The data in this study seem to suggest that, in the strain range of interest for field structures, differences between geogrids in stiffness and strength are more apparent under high strain rates. By normalizing the tensile strength with respect to the ultimate short-term tensile strength, the strain required to achieve the anticipated LTDS is readily seen. The data seem to show differences exist in the strain necessary to mobilize the anticipated LTDS between geogrids. These differences and the resultant ranking of normalized strengths are not in agreement with the ranking of ultimate short-term tensile strengths from the manufacturers (Figure 7.6, Table 4.1). Index tests such as the ASTM D-4595 and GRI GG1 which compare geogrids on the basis of rupture strength may not reflect the strength of the material available at small strains.

- The data in this study appear to suggest that there exist differences in the amount of displacement necessary to mobilize peak interaction factors for different geogrids.
Some geogrids may be more efficient at mobilizing peak interaction in terms of the amount of displacement necessary.

- The effects of normal stress on strain and interaction were also investigated by staging the CRD pullout tests. It was found that confining stress reduced the observed differences between the geogrids with respect to mobilized coefficient of interaction.

8.3.2.2 Long-Term Strength

- Long term stability concerns the creep potential of polymeric reinforcement. Both ultimate limit states (ULS) and serviceability limit states (SLS) are of concern. Practice in North America has been to approach long term strength through the adoption of safety factors. FHWA (1990) guidelines suggest the use of a creep reduction factor, which is the ratio of the creep limited strength determined through ASTM D 5262 to the ultimate short term strength (ASTM D 4595). The British Standard Code of Practice for Strengthened/Reinforced Soils and Other Fills adopts the use of more elaborate strain estimation. Partial factors of safety for the extrapolation of data strongly encourage reliable data for creep strain estimates. Isochronous load - strain curves are utilized for the estimation of design life strains both in Britain. The test results on HDPE geogrids have stressed the importance of reliable long-term creep data. Moreover, the adoption of extrapolation methods similar to those used in Britain and recently implemented in Washington State provides a more rational approach to long-term design.
Chapter 8. Summary and Conclusions

In summary, these test data (displacement- and load-controlled tensile and creep and displacement-controlled pullout), all suggest that quality test data together with isochronous curves are excellent ways to describe the load-strain-time behaviour of geogrids. The good agreement in isochronous load-strain curves with manufacturer’s data gives confidence in predicting long-term strength. Thus, there would appear to be justification in re-evaluating conservativeness in design with respect to:

i) creep reduction factors applied to $T_{ULT}$,

ii) height limitations of geogrid reinforced soil walls.
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APPENDIX A

TECHNIQUE FOR STRAIN GAUGING PLASTICS

A.1 Introduction

The development of a strain gauging technique for plastics requires that consideration be given to the mechanical, thermal and chemical properties of polymeric materials. Plastics are considerably less stiff than steel and the ranges of strain encountered are frequently on the order of 15 to 20%. Consequently, a strain gauge with a high capacity for elongation is required in order to avoid damage to the gauge. Moreover, the adhesive used to bond the gauge must be compatible with the anticipated strains otherwise debonding may occur. Regarding thermal properties, polymers have thermal coefficients approximately 5 to 10 times greater than those of metals or concrete. Thermal properties influence the selection of the gauge size and excitation voltage necessary to achieve an acceptable power dissipation per unit of geogrid area. The chemical properties of gauges requires the researcher to take caution in not exposing the gauge to aggressive reagents.

Considering all of the above it is important to select a strain gauge and bonding agent that are compatible with each other and the anticipated strains. Use of a high elongation gauge is strongly suggested for strain gauging of plastics. With regard to the sensitivity of gauges to impurities it is vital that a scrupulous surface cleaning technique be applied. Finally, since there is a strong potential for debonding of gauges bonded to plastics it is vital that an effective surface preparation technique be employed.
Appendix A

A.2 Characteristics of the Strain Gauge

The strain gauge selected for this study was a type EP-08-250BF-350 Option E, manufactured by the Micro-Measurements Division of Measurements Group Inc. It is selected for the following reasons:

i) the EP series gauges are made of a special annealed contantan foil with a tough, high elongation polyamide backing that offers high elongation capacity;

ii) the geometry of the gauge, defined by the gauge pattern designation 250BF and reported in Table A-1.1, fits well on the ribs of the geogrid test specimens;

iii) a high resistance gauge minimizes heat dissipation, for which the 350 ohm is selected;

iv) encapsulation of the gauge, the option E, protects the gauge circuit from damage incurred by abrasion in sand.

<table>
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<tr>
<th>Gauge length (mm)</th>
<th>Overall length (mm)</th>
<th>Grid width (mm)</th>
<th>Overall width (mm)</th>
<th>Matrix size (LxW)</th>
</tr>
</thead>
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<td>9.53</td>
<td>3.18</td>
<td>3.18</td>
<td>13.2 x 5.6</td>
</tr>
</tbody>
</table>

A.3 Strain Gauging Procedure

A.3.1 Chemicals for Surface Preparation

A 1-1-1 Trichloro-ethane solvent is used to degrease the surface of the test specimen at the selected gauge site due to its inertness to polymers. The degreaser cleans the surface
of any contaminants and prevents the embedment of them in the surface of the geogrid. A number 400 grit sand paper or emery cloth is used to roughen the surface, thus promoting mechanical interlock of the specimen and the gauge. Following this the surface is treated with a mild ammonia solution, which leaves it with a slightly alkaline pH. Gauge installation is performed within a few minutes of completing the surface conditioning.

A.3.2 Adhesive Selection and Preparation

M-Bond AE 10/15 adhesive was selected to obtain a high elongation capability. Resin AE (10 gram unit) with Curing Agent 10 will cure in 6 hours giving approximately 6% elongation capabilities. By extending the curing time from 24 to 48 hours at 24° C, a higher elongation capability of 10% may be obtained.

A.3.3 Geosynthetic Surface Preparation

Supplies required: 1,1,1 Trichloro-ethane degreaser, No. 400 sand paper, some medical gauze or cheese cloth, compressed air supply, cotton swabs, M-Prep neutralizer.

Steps involved in surface preparation are:

1. Cut the geosynthetic specimen to the required dimensions. Place and secure it on a clean, flat surface and decide on the gauge locations, marking them if necessary. It is vital that the gauge be aligned precisely in the direction of straining. This is usually along the longitudinal ribs for geogrids, but at any rate in the direction of pullout. If a woven geogrid, such as a Stratagrid or Miragrid is used, then it is necessary to prepare an artificial flat surface onto which the gauge be bonded. This was achieved
Appendix A

2. by placing a layer of quick curing epoxy at the gauge site and allowing it to cure. After it was cured it was sanded flat and as thin as possible. Surface preparation was then carried out as above.

3. Spray the gauge location with 1,1,1 Trichloro-ethane degreaser and wipe clean using the cheese cloth. It is advised that when wiping clean the degreaser that one starts at the centre of the gauge site and applies an even stroke out past the edge of the gauge site. This is to ensure that the gauge site is not recontaminated by debris outside the gauge site.

4. Using no. 400 grit sand paper to roughen the surface, sanding first at a 45° angle to the direction of pullout and then at right angles to that get a pattern of cross hatches. Approximately 4 minutes of sanding is required.

5. Using a compressed air hose, clean the gauge location to remove any debris.

6. Neutralize the surface by wiping, again in the same fashion as above, with M-Prep Neutralizer 5.

7. The gauge should be applied with 2 to 3 minutes of completing the surface preparation.

A.3.4 Gauge Preparation

**Supplies required:** A Plexiglas frame, 1,1,1 Trichloro-ethane degreaser, tweezers, eraser, MGJ-2 tape (a special cellophane tape), strain gauges. The Plexiglas frame was a small rectangular frame measuring approximately 3” by 5”. The dimensions of the frame were ½” by ¾”. The purpose of the frame was to allow for easy, and non destructive transfer of the gauge to the geogrid specimen.
Appendix A

Steps involved in the preparation are:

1. Clean the Plexiglas frame with 1,1,1 Trichloro-ethane degreaser, wiping with cheese cloth.
2. Take a small length of MJG-2 tape and place it across the top of the Plexiglas frame with the sticky side down. Do this for as many gauges as you have.
3. Remove the strain gauge from its package, ensuring it is held on the edge using tweezers.
4. Turn over the frame and place the gauge on to the sticky portion of the tape, taking care to align it parallel with the edge of the tape. Apply low air pressure from the compressed air supply to affix the gauge firmly to the tape.
5. The gauge is now ready for transfer to the geogynthetic test specimen.

A.3.5 Application of the Gauge

**Supplies required:** AE 10/15 adhesive kit, cheese cloth, TFE-1 sheet, silicone pads, aluminum blocks, and MJG-2 tape.

TFE-1 sheet is a mylar film used to cover and protect the gauges.

Steps involved in the adhesive preparation are:

1. To prepare the adhesive mix, fill one of the calibrated droppers, provided with the adhesive kit, with Curing Agent 10 exactly to the 10 mark on the side of the dropper. Dispense this into the jar of Resin AE. Immediately recap the bottle of Curing Agent to avoid moisture absorption.
2. Thoroughly mix for 5 minutes using the plastic stirring rods provided.
3. The working time after mixing is 15 to 20 minutes. Perform application of the gauge with in this time otherwise it flash hardens and gets very hot.
4. Discard the dropper, stirring rod and the adhesive mix after the gauge application

Steps involved in the gauge application are:

1. Lift the tape off the Plexiglas frame along with the gauge and attach it to the geosynthetic at the desired gauge location, aligning the gauge in the direction of testing. The tape on the terminal side of the gauge should not be pressed firmly, but the opposite side should be.

2. Peel back the tape from the terminal side at an acute angle so that the tape lifts off with the gauge. Pull back the tape 3 mm further than the edge of the gauge.

3. Apply two drops of prepared adhesive (M Bond AE 10) to the geosynthetic test specimen at the gauge location and quickly lower the gauge to make contact with the adhesive and geosynthetic.

4. Using the cheese cloth, apply a uniform pressure to the gauge with your thumb. Be very careful not to come in contact with the adhesive.

5. Overlay the gauge with TFE-1 film, a silicone pad and an aluminum block, and apply pressure using a dead weight to obtain approximately 100 to 135 kPa.

6. Maintain the clamping pressure for 15 to 20 hours to obtain a reasonable elongation capability.

7. After 15 to 20 hours carefully peel off the tape from the terminal side, pulling back at an angle of more than 150° to discourage any debonding of the gauge.

8. The gauge is now ready for soldering.

A.3.6 Gauge Soldering

**Supplies required:** Rosin solvent, 3-strand wires, and soldering accessories

The steps involved are:
Appendix A

1. For the first test cut the 3-strand wire into desired lengths, and pass it through the stiff plastic tubing that is used to protect the wire from damage by the sand. For subsequent tests it will be preferable to simply cut the old gauges off and advance the 3-strand wire forward for the next test specimen. This avoids having to go through the process of feeding the wire and tubing repeatedly through the hole in the back of the apparatus.

2. Solder the ends of the wires (tinning) and trim to leave 2 mm exposed.

3. Tape down the stiff tubing to the geosynthetic test specimen and draw enough 3-strand wire through to make a loop of excess wire adjacent to the gauge. This will help prevent unnecessary pulling on the wires after they have been soldered by leaving some slack.

4. Brush the gauge surface with rosin solvent to remove dust particles.

5. Using flux and solder, and taking care not to apply excess heat that will damage the geosynthetic specimen, quickly place solder on the tabs of the gauges. The solder should end up smooth round and shiny. If it is dull or jagged it means that not enough flux was added. The trick is to apply some solder to the iron, then simultaneously place the iron (with the solder droplet on the tip) and additional solder on the terminal tab, feeding additional solder as needed.

6. Solder the prepared wires to the solder on the gauge tabs.

7. Check the resistance of the gauge and its connection using an ohm-meter.

8. Clean the surface with rosin solvent to remove flux.

9. The gauge assembly is now ready for protecting.
A.3.7 Gauge Protection

**Supplies required:** Cellophane tape, M – coat A, TFE-1 and MJG-2 tape.

The steps involved are:

1. Coat the gauge assembly with M – coat A, a polyurethane coating, placing three coats at an interval of 30 minutes.
2. Coat the exposed portions of the wires between the gauge and protective tubing.
3. Cover the gauge assembly with TFE – 1 film and tape it down firmly using cellophane tape or MJG – 2 tape.

A.3.8 Analysis of Strain Data

Corrections to be applied to the measured data are: transverse sensitivity; thermal output; gauge factor variation with temperature; Wheatstone bridge non – linearity; and gauge factor variation with strains. Considering all these factors, the measured percentage strain in a full bridge circuit is related to the change in electrical output recorded by the following expression:

\[ \% \varepsilon = \left( \frac{4E_0}{(F + \varepsilon)E - 2E_0(F + \varepsilon)} \right) \times 100\% \]  

(A.1)

where:

E₀ is the output of the bridge in mV,
E is the input to the bridge in mV,
F is the gauge factor supplied by the manufacturer.