Full-Scale Field Study of a Geogrid-Reinforced Unpaved Road System

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A thesis submitted in partial fulfillment of the requirements for the Doctor of Philosophy degree in Civil and Environmental Engineering
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ABSTRACT

Geogrids are widely used to improve the performance of unpaved roads constructed over weak subgrade. However, the behavior of geogrids under traffic loads and their reinforcing mechanisms, as well as the resulting benefits to the roadway performance are not well understood. A full-scale field study was conducted to quantify the effectiveness of geogrids in unpaved roads, evaluate their reinforcing mechanism, and identify which geogrid properties are most directly related to performance improvement. Ten full-scale unpaved road test sections were constructed with varying relevant parameters including geogrid aperture shape, geogrid tensile modulus, and thickness of the base course layer. Five of the test sections were constructed with a 200-mm nominal base course thickness and five were constructed with a 250-mm nominal base thickness. For each base course thickness, four test sections were reinforced with different geogrids while one test section was unreinforced in order to evaluate their performance. The geogrids were placed at the subgrade-base course interface on top of a non-woven geotextile separator. Trafficking was applied using a single-axle dump truck. Measurements of rut depth and surface deformation were taken to evaluate the respective performance of each test section. Additionally, the test sections were instrumented for measuring road response under traffic loading. Dynamic and permanent geogrid strains were measured using foil strain gauges attached to the geogrid ribs. Earth pressure cells were installed at the top of the subgrade layer to measure dynamic and permanent vertical stresses transferred to the subgrade soil. The measured performance indicated that the geogrids effectively reduced surface rutting and reduced the dynamic vertical stresses transferred to the top of subgrade. Traffic benefit ratio of up to 3.6 and reductions in base layer thickness between 6 and 25% were achieved, with higher benefit values observed for stiffer geogrid and smaller base layer thickness. Improvement in performance was related to the tensile strength at 2% and junction strength in the cross-machine direction. The geogrid strain data demonstrated that the geogrid was not under constant tension across the road. The field testing results clearly confirmed the validity of shear-resisting interface or lateral restraint mechanisms.
Keywords

Base reinforcement, Field testing, Geogrid, Geosynthetics, Instrumentation, Pressure cells, Reinforcement mechanisms, Road stabilization, Rut depth, Strain, Strain gauges, Subgrade stress, Traffic benefit ratio, Traffic loads, Unpaved road.
SUMMARY FOR LAY AUDIENCE

Geogrids are polymer materials used to improve the performance of unpaved roads constructed over weak soils. However, the behavior of geogrids under traffic loads and their reinforcing mechanisms are not well understood. A full-scale field study was conducted to quantify the effectiveness of geogrids in improving unpaved roads performance, evaluate their reinforcing mechanism, and identify which geogrid properties are most directly relate to performance improvement. To achieve these objectives, ten full-scale unpaved road test sections were constructed. Test variables included geogrid type, geogrid strength, and thickness of the fill layer. Two fill thicknesses were evaluated, each implemented in four reinforced test sections and one unreinforced test section. The geogrids were placed at the bottom of the fill layer on top of a fabric separator. Trafficking was provided by a dump truck. Measurements of surface deformation were taken to evaluate the respective performance of each test section. Additionally, the test sections were instrumented for measuring road response under traffic loading. Strains were measured in the geogrids. Vertical stresses were measured at the top of the subgrade layer to quantify the stresses transferred to the subgrade soil. Analysis of the measured performance data indicated that the geogrids effectively reduced surface deformation and reduced the vertical stresses transferred to the top of the subgrade. The geogrids resulted in extension of the road service life and reductions in fill layer thickness. The geogrid benefits were more pronounced when a stronger geogrid was used and were reduced by increasing the fill layer thickness. The geogrid strain data demonstrated that the geogrid was not under constant tension across the road. The results of the field testing clearly supported the existence of the mechanism of lateral restraint.
CO-AUTHORSHIP STATEMENT

This thesis is prepared in accordance with the regulation for a monograph format thesis stipulated by the school of graduate and post graduate studies at Western University. The work presented in this thesis was carried out by the author under the supervision of Dr. M. Hesham El Naggar, who is a co-author of all the work presented in this thesis and the papers submitted for publication.

A version of Chapter 4 has been published at the GeoVancouver 2016 Conference.

Parts of Chapters 3 and 4 along with limited part of Chapter 5 have been published in the Journal of Ground Improvement.

A version of chapter 2 will be submitted to the Journal of Innovative Infrastructure.

In addition, the results and conclusions obtained from this research and presented in Chapters 5 and 6 will be submitted for further journal publications.
Lovingly dedicated to my wonderful children

Hala, Yumna, Qusai, and Kinda

You are the light of my life
ACKNOWLEDGMENTS

All praise is due to Allah who guided us to this, and we would not have been guided, had He not guided us. My success can only come from Allah. I praise and thank Him for all the help, strength, patience and the many blessings He has provided me with throughout my life.

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<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a$</td>
<td>Regression function describes the effects of initial conditions (i.e., effects of geogrid and base layer thickness)</td>
</tr>
<tr>
<td>$a_2$</td>
<td>Structural coefficient for the base course layer in the AASHTO 1993 pavement design procedure</td>
</tr>
<tr>
<td><strong>AASHTO</strong></td>
<td>The American Association of State Highway and Transportation Officials</td>
</tr>
<tr>
<td><strong>ASTM</strong></td>
<td>The American Society For Testing And Materials</td>
</tr>
<tr>
<td><strong>BCR</strong></td>
<td>Base Course Reduction</td>
</tr>
<tr>
<td><strong>CBR</strong></td>
<td>California Bearing Ratio</td>
</tr>
<tr>
<td><strong>CBR_{bc}</strong></td>
<td>Base course California bearing ratio</td>
</tr>
<tr>
<td><strong>CBR_{sg}</strong></td>
<td>Subgrade California bearing ratio</td>
</tr>
<tr>
<td><strong>CPL</strong></td>
<td>Cyclic Plate Load Test</td>
</tr>
<tr>
<td>$D_2$</td>
<td>Base layers thicknesses in the AASHTO 1993 pavement design procedure</td>
</tr>
<tr>
<td><strong>CD</strong></td>
<td>Consolidated Drained Triaxial Test</td>
</tr>
<tr>
<td><strong>CSL</strong></td>
<td>Critical State Line</td>
</tr>
<tr>
<td>$d$</td>
<td>Plate diameter</td>
</tr>
<tr>
<td><strong>DAQ</strong></td>
<td>Data Acquisition System</td>
</tr>
<tr>
<td>$D_n$</td>
<td>Particle size in the gradation curve corresponding to n% finer</td>
</tr>
<tr>
<td>$E$</td>
<td>Elastic modulus</td>
</tr>
<tr>
<td>$E_{bc}$</td>
<td>Elastic modulus of the base course</td>
</tr>
<tr>
<td>$E_{bc,1}$</td>
<td>Initial elastic modulus of the base course</td>
</tr>
<tr>
<td>$E_{bc,N}$</td>
<td>Elastic modulus of base course after $N$ traffic passes</td>
</tr>
<tr>
<td>$E_{sg}$</td>
<td>Elastic modulus of the subgrade soil</td>
</tr>
<tr>
<td><strong>EDA</strong></td>
<td>Exploratory Data Analysis</td>
</tr>
<tr>
<td><strong>EPC</strong></td>
<td>Earth Pressure Cell</td>
</tr>
<tr>
<td><strong>ESAL</strong></td>
<td>Equivalent Single-Axle Load</td>
</tr>
<tr>
<td><strong>FWD</strong></td>
<td>Falling Weight Deflectometer Test</td>
</tr>
<tr>
<td><strong>FR(M)</strong></td>
<td>Geogrid flexural rigidity in the machine direction</td>
</tr>
<tr>
<td><strong>FR(X)</strong></td>
<td>Geogrid flexural rigidity in the cross-machine direction</td>
</tr>
<tr>
<td>$h$</td>
<td>Thickness of the base layer</td>
</tr>
<tr>
<td>$h_e$</td>
<td>Equivalent thickness of the base layer</td>
</tr>
</tbody>
</table>
\( IQR \)  
Interquartile Range in a box-and-whisker plot

\( J \)  
Geogrid aperture stability modulus

\( JS(M) \)  
Geogrid junction strength in the machine direction

\( JS(X) \)  
Geogrid junction strength in the cross-machine direction

\( LEF \)  
Load Equivalency Factor

\( LVDT \)  
Linear Variable Differential Transformer

\( M \)  
Slope of critical state line

\( m_2 \)  
Drainage coefficient of base course in the AASHTO 1993 pavement design procedure

\( MD \)  
Machine direction of the biaxial geogrid

\( MET \)  
Method of equivalent thickness

\( M_r \)  
Resilient modulus of the subgrade soil

\( MS \)  
Motion sensor

\( N \)  
Number of traffic passes

\( N_{60} \)  
Corrected SPT number for standardized 60% energy ratio

\( N_c \)  
Bearing capacity factor

\( N_{ESAL} \)  
Number of passes of the equivalent (reference) single-axle load

\( N_{ESAL(add)} \)  
additional number of ESAL passes carried by a reinforced section compared to a corresponding unreinforced section

\( OPSS \)  
Ontario Provincial Standard Specification

\( P \)  
Wheel load

\( PE \)  
Polyethylene

\( PET \)  
Polyester

\( PP \)  
Polypropylene

\( PPY \)  
Polypropylene yarns

\( HDPE \)  
High-density polyethylene

\( P \)  
Wheel load

\( p \)  
Tire contact pressure or tire inflation pressure

\( p' \)  
Mean effective stress

\( PSI \)  
Initial serviceability index
Terminal serviceability index (the lowest index that will be tolerated before requiring rehabilitation of the road)

Polyvinyl Chloride

Deviator stress

Radius of the equivalent tire contact area

Level of reliability of the design in the AASHTO 1993 procedure

The coefficient of determination

Limited modulus ratio of base course to subgrade soil

Initial modulus ratio of base course to subgrade soil

Modulus ratio of base course to subgrade soil after $N$ traffic passes

Geogrid radial stiffness at low strain

Allowable rut depth

Standard deviation

Single-Depth Deflectometer

Strain gauge

Structural number of the base layer

Signal-to-noise ratio

Standard Penetration Test

Self-Temperature-Compensation

Geogrid ultimate tensile strength

Geogrid tensile strength at 2% strain in the machine direction

Geogrid tensile strength at 2% strain in the cross-machine direction

Geogrid tensile strength at 5% strain in the machine direction

Geogrid tensile strength at 5% strain in the cross-machine direction

Traffic Benefit Ratio

Time-Domain Reflectometer

Load on a given single axle

Number of equivalent single-axle load applications that causes a tolerated reduction in serviceability index (Allowable number of ESAL applications during the design life of the road)

Cross-machine direction of the biaxial geogrid
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Z_R$</td>
<td>Standard normal deviate</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Stress distribution angle</td>
</tr>
<tr>
<td>$\alpha_I$</td>
<td>Initial stress distribution angle</td>
</tr>
<tr>
<td>$\alpha_N$</td>
<td>Stress distribution angle after $N$ traffic passes</td>
</tr>
<tr>
<td>$\Delta PSI$</td>
<td>Tolerated reduction in serviceability index</td>
</tr>
<tr>
<td>$\phi'$</td>
<td>Effective angle of internal friction</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>Vertical stress at the base-subgrade interface</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>$\nu_{bc}$</td>
<td>Poisson’s ratio of the base course</td>
</tr>
<tr>
<td>$\nu_{sg}$</td>
<td>Poisson’s ratio of the subgrade soil</td>
</tr>
</tbody>
</table>
Chapter 1

1 INTRODUCTION

1.1 Background

Unpaved roads such as haul roads, access roads, rural roads, and construction platforms are widely used to serve either temporary or permanent transportation purposes. Transport Canada (2018) reports that there are more than 1.13 million kilometers of (two-lane equivalent) public road in Canada. Approximately 60% of this road network is unpaved. Unpaved roads play a substantial role in rural economy, military transportation, and natural resource industries. Conventional unpaved roads consist of a layer of base course material placed directly over the subgrade soil. They are typically subject to low traffic volumes of heavy vehicles and hence are susceptible to excessive rutting, which may interrupt traffic service and increase maintenance cost. The common practice is to excavate the weak subgrade soil and replace it with a more suitable fill material. Other typical approaches used to minimize damage to the road are to increase the thickness of the base layer or stabilize the subgrade using a variety of methods such as mixing with lime or cement. In some situations, however, this can be laborious, time-consuming, and costly. Stabilization with geosynthetics can be used in these situations.

Geosynthetics are planar artificial materials manufactured from polymers. Since they were first introduced in the 1970’s (Koerner 2012), geosynthetics have been used successfully to improve the performance of unpaved roads constructed over weak subgrade. They have become popular construction materials in civil engineering due to their favorable characteristics such as lightness, non-corrosiveness, long-term durability, and simplicity of installation. The two main types of geosynthetics most widely used in roadway stabilization are geotextiles and geogrids. Geotextiles are flexible, porous fabrics made of either woven, nonwoven, or knitted synthetic fibers, while geogrids are polymers formed into a very open, gridlike configuration by weaving yarns into an open structure (woven geogrids), bonding straps together at their junctions (welded geogrids), or by extruding, punching and drawing polymer sheets (extruded geogrids) (Koerner, 2012). A more detailed description of these materials is presented in Chapter 2.
The inclusion of geosynthetics benefits the unpaved road through two functions: separation and reinforcement. Reinforcement is the primary function (especially for geogrids); however, separation (typically achieved by geotextiles) becomes an essential function when there is a potential for intermixing of aggregate and subgrade soil. Geotextiles are usually placed at the subgrade-base course interface, while the optimal placement location of geogrids depends on the strength of the subgrade, the thickness of the base layer and the traffic loads (Perkins and Ismeik 1997); nevertheless, placing the geogrids at the subgrade-base course interface is commonly accepted in the current state of practice.

This study focuses on the use of geogrids as reinforcement for unpaved roads. Geogrids provide tensile reinforcement to the unpaved structure mainly through shear interaction with the aggregate. This interaction minimizes lateral movement of aggregate particles and increases the modulus of the base layer, thereby resulting in distributing the applied vertical stress to a wider area of the subgrade. Consequently, vertical deformation of the subgrade caused by subgrade overstress decreases, which leads to improving the performance of the unpaved system. Therefore, inclusion of geogrids in unpaved roads can help increase the service life of the road for a given base layer thickness, or decrease the thickness of the base layer needed to achieve a given service life. In areas where quality aggregate resources are scarce and costly, the use of geogrids becomes more attractive.

1.2 Need for Research

Despite the considerable amount of research that has been carried out on the use of geogrids as road reinforcement, there are too many discrepancies between research studies, and there is a lack of comprehensive knowledge of geogrids applications in roadways that underscores the need for more research. Gaps in the current knowledge of reinforced unpaved roads are identified as follows:

- The majority of the experimental work on geogrid applications in roadways has been performed at reduced-scale in the laboratory. Despite the good quality control
achieved in laboratory testing, the actual traffic loading conditions cannot be replicated appropriately in most laboratory settings. Furthermore, reduced-scale tests cannot adequately model the interaction between geogrids and aggregate which occurs over the entire width of the road. Therefore, conclusions from laboratory studies cannot be reliably adopted in practice (Hufenus et al. 2006), and additional field testing is needed.

- Inconsistencies exist between studies on the amount and type of benefits to the roadway due to the inclusion of geogrids. While many studies showed significant improvement of the road performance (Cuelho et al., 2009; Fannin and Sigurdsson, 1996; Al-Qadi and Bhutta, 1999), others have shown little benefit from including the geogrids (Cox et al., 2010; Henry et al., 2011). These discrepancies indicate that more variables pertinent to the behavior of the reinforced structure, such as subgrade strength and base layer thickness, need to be investigated to understand the conditions under which the geogrids will be cost-effective as roadway reinforcements.

- The strain behavior of geogrids under traffic loads is not well understood. Measurement of geogrid strain in a full-scale field testing is needed to investigate the distribution of geogrid strains across the geogrid.

- The mechanisms through which the geogrids improve the performance of unpaved roads haven’t yet been fully understood. More experimental testing methods involving measurements of stress and strain responses are needed to provide information as to how the geogrids affect and benefit the road structure.

- Review of previous research revealed disagreement between researchers as to which geogrid properties are most pertinent in the application of road reinforcement. Additional work is needed to identify the geogrid parameters that are critical to their effectiveness.

- New geogrid products and manufacturing procedures are being regularly introduced to the market. Assessment of suitability of these products for the application of road reinforcement is critical to updating the specifications required for industrial and design purposes so that to encompass all suitable products. Berg et al. (2000) suggested that geogrid performance is specific to the product type.
• Few researchers have instrumented road sections reinforced geogrids (Brandon et al. 1996; Warren and Howard 2007), but limited results have been provided. Besides, most of these studies were conducted on flexible pavements. Instrumentation program in full-scale test sections is necessary in order to understand the behavior of the unpaved road.

1.3 Research Objectives

Based on the literature review, and to advance areas where further research is needed such as those listed above, the following research objectives were established:

• To quantify the performance benefits, if any, from using geogrid reinforcements in unpaved roads.
• To examine the effect of geogrid reinforcement in reducing the vertical stresses transferred to the subgrade.
• To investigate the strain behavior of geogrids under traffic loads.
• To evaluate the reinforcement mechanisms of geogrids in field traffic conditions using mechanical measurements from unpaved test sections reinforced with geogrids.
• To investigate the influence of base layer thickness, geogrid reinforcement type and stiffness, and traffic loading level and magnitude on the response of the unpaved structure.
• To determine geogrid properties most relevant to the performance of the reinforced unpaved roads.
• To expand the existing database in the literature related to the performance of geogrid reinforcements in roadways by providing additional data for geogrid products for which the performance in actual traffic conditions has never been assessed or has not been adequately investigated.
1.4 Research Methodology

In order to meet the research objectives a comprehensive research program was initiated that involved the following tasks:

- Performing a full-scale field testing program that included the construction and trafficking of unpaved test sections, reinforced with geogrids of different types and stiffnesses, and built with two base layer thicknesses.
- Incorporating, into the testing program, geogrid materials that have never been tested under field testing conditions and with this level of monitoring and data collection. Terrafix Geosynthetics Inc. is a leading distributor and manufacturer of geosynthetic products in Canada; however, Terrafix products have not been adequately investigated. Accordingly, evaluation of these products will help to improve design practices of geogrid materials in roadways.
- Monitoring the performance of the test sections through measurements of rut depth and surface deformation.
- Instrumenting the test sections in order to monitor the mechanical response of the unpaved structure.
- Analyzing the collected field data to investigate the effects of geogrids on the behavior and performance of the unpaved structure.

1.5 Original Contributions

This is the first comprehensive field study on unpaved roads to:

- Evaluate the performance of the never studied biaxial geogrid materials under consideration.
- Investigate strain anisotropy in the geogrid reinforcement under actual traffic loads; in other words, examine the geogrid strain behavior with respect to traffic direction (transverse versus longitudinal strain).
- Examine the geogrid strain distribution across the road width to validate the concepts of constant versus variable distribution of transverse geogrid strains.
• Investigate the main geogrid reinforcement mechanism governing the performance of unpaved roads using the geogrid strain response under actual traffic conditions.
• Identify the parameters influencing stress distribution by the base layer and describe the relationship between these parameters and the change in stress distribution angle under traffic loading.

1.6 Thesis Organization
This thesis is presented in a ‘monograph’ format and consists of seven main chapters as follows:

Chapter one provides an overview of unpaved roads and the use of geosynthetics for their stabilization. The motivations, objectives, and methodology of this research is also outlined. The original contributions are listed, and the organization of the thesis is described.

Chapter two presents a comprehensive literature review of previous research work pertaining to the application of geosynthetics as reinforcement in unpaved roads, as well as a synthesis and evaluation of results and findings from previous research studies that involve geosynthetic reinforcement of unpaved roads.

Chapter three describes the field experimental program and configuration of the full-scale unpaved test sections. This includes the description of the research site, the subgrade soil, the base course material, and the geosynthetics used. The instrumentation used to measure the response of the unpaved test sections during the field testing along with the data acquisition system used to collect response data are also detailed in this chapter.

Chapter four describes the construction and trafficking of the test sections. The methods used to install and protect the instrumentation in the field are also detailed in this chapter. In addition, the procedures used to collect, manage, and process the various types of field data are discussed.
Chapter five discusses the results of the field traffic testing on the unpaved road test sections. Discussed data includes measurements of rut depth and surface deformation at selected traffic levels, the measured mechanical response of the unpaved structure to the applied traffic loads in terms of vertical stresses on top of the subgrade and strains in the geogrids.

Chapter six presents the analyses performed on the data collected from the full-scale field traffic testing. The performance benefits of using geogrid reinforcements to stabilize unpaved road layers are quantified and evaluated in terms of both extending the service life of the road and reducing the required aggregate thickness. The degradation of the base layer is investigated through the changes in stress distribution angle and base-to-subgrade modulus ratio with cumulative traffic passes. The tensile forces mobilized in the geogrid reinforcements are computed and their role in stabilizing the unpaved layers is discussed. In addition, results from the field testing are utilized to investigate the reinforcement mechanisms through which the geogrids stabilized the unpaved test sections. Finally, geogrid properties most relevant to their effectiveness in improving the performance of unpaved structures are identified utilizing the measured reinforcement benefits.

Chapter seven includes a summary of the research work performed in this study, the main conclusions and findings from this research study, and suggests recommendations for future research in the area.
Chapter 2

2 LITERATURE REVIEW

2.1 Introduction

Unpaved roads are widely used for temporary and permanent transportation purposes. They consist of a layer of base course material placed directly over weak subgrade, which is usually incapable of supporting large traffic loads. These roads are prone to excessive rutting or complete collapse, resulting in interruption of traffic service and increase of maintenance costs. When excavating and replacing unsuitable subgrade materials is not cost effective and/or time consuming, stabilization with geosynthetics offers a good alternative. Inclusion of geosynthetics benefits the unpaved road through two main functions: separation and reinforcement. Reinforcement is the primary function and it has commonly been performed using geogrids. However, separation (which typically achieved by geotextiles) becomes an essential function when there is a potential for intermixing of aggregate and subgrade soil.

This study focuses on the reinforcement function of geogrids in unpaved roads. Geogrids provide tensile reinforcement to the unpaved structure mainly through shear interaction with the aggregate. This interaction minimizes lateral movement of aggregate particles and increases the modulus of the base layer, thereby resulting in distributing the applied vertical stress to a wider area of the subgrade as demonstrated in Figure 2.1. Consequently, vertical deformation of the subgrade caused by subgrade overstress decreases, which leads to improving the performance of the unpaved system. Therefore, inclusion of geogrids in unpaved roads can help increase the service life of the road for a given base layer thickness, or decrease the thickness of the base layer needed to achieve a given service life. In areas where quality aggregate resources are scarce and costly, the use of geogrids becomes more attractive. The geogrids have generally been placed at the base-subgrade interface or within the base course layer of the flexible pavement. The optimal placement location of geogrids depends on the strength of the subgrade, the thickness of the base layer and the traffic loads (Perkins and Ismeik 1997)
2.2 Geosynthetics

Geosynthetics are planar artificial materials manufactured from polymers used with soil, rock, earth, or other geotechnical engineering related material as an integral part of a man-made project, structure, or system (ASTM, 2020). The most polymers commonly used in the manufacturing of geosynthetic materials are (Koerner, 2012): polypropylene (PP), polyester (PET), polyethylene (PE), high-density polyethylene (HDPE), polyvinyl Chloride (PVC). The main functions of geosynthetics used in the construction of roadways are:

- Separation – Prevent intermixing of particles from two soil layers with different properties so that the integrity and functioning of both materials can remain intact.
- Reinforcement – The interactive improvement of the strength of a system created by the inclusion of a geosynthetic (that is good in tension) into a soil (that is good in compression but poor in tension).
- Filtration – The permission of adequate water flow with limited soil loss across the plane of the geosynthetic over a service lifetime compatible with the application under consideration.
- Drainage – The permission of adequate liquid flow with limited soil loss within the plane of the geosynthetic over a service lifetime compatible with the application under consideration.
• Containment – Acting as an impervious liquid or vapor barrier.

Available types of geosynthetics include geotextiles, geogrids, geonets, geomembranes, geosynthetic clay liners, geofoam, geocells, and geocomposites (Figure 2.2). Geotextiles and geogrids have been the two main types of geosynthetics typically used in pavement applications for reinforcement and stabilization purposes. Geotextiles and geogrids are the only geosynthetics used in this study, and they are described further in the following sections.

![Figure 2.2: The types of geosynthetics (Koerner, 2012)](image)

### 2.2.1 Geogrids

Geogrids are manufactured by punching holes into a heavy gauge sheet of the appropriate polymer, typically 4 to 6 mm-thick, on a regular pattern. The sheet is then drawn
typically in one or two directions. They have in-plane ribs with integral junctions or nodes and a uniformly distributed array of openings or apertures between their individual ribs in the machine and cross machine directions. The size of the apertures varies from 10 to 100mm and the ribs are often quite stiff compared to the fibers of geotextiles. Geogrids are formed into a very open, grid-like configuration by weaving yarns into an open structure (woven geogrids), bonding straps together at their junctions (welded geogrids), or by extruding, punching and drawing polymer sheets (extruded geogrids) (Koerner, 2012).

Geogrids are classified by whether they can provide resistance to in-plane loads in one direction (uniaxial) or multiple directions (biaxial and triaxial) as shown in Figure 2.3. Uniaxial geogrids are often used in walls and slopes, while biaxial geogrids are commonly used in base reinforcement. Geogrids are relatively high strength, high-modulus, and low-creep-sensitive polymers. The polymer materials used to manufacture geogrids are typically high-density polyethylene for the uniaxial types and polypropylene for the biaxial types. The key feature of geogrids is that the apertures are large enough to allow for strike-through from one side of the geogrid to the other which increases the interaction between the geogrid and the surrounding aggregate material in the unpaved structure (Koerner, 2012).

Geogrids are designed primarily to accomplish a reinforcement function. The use of geogrids to reinforce soft and/or weak subgrade soils for unpaved roads is a major application area. Some of the geogrid uses reported in the literature are (Koerner, 2012):

- Beneath or within aggregate in unpaved roads
- Beneath or within ballast in railroad construction
- Beneath or within surcharge fills or temporary construction sites
- As mechanically stabilized earth for a variety of wall facings
- Reinforcement of embankment fills and earth dams
- Repairing slope failures and landslides
- As basal reinforcement over soft soils or karst areas
- As basal reinforcement between pile and other deep foundation caps
- As lateral confinement to stone for constructing stone columns
• As a bridge over cracked or jointed rock
• To construct mattresses for fills over soft soils
• As sheet anchors for retaining-wall facing panels
• To reinforce disjointed rock sections
• As composite forms with nonwoven geotextiles
• As inserts between geotextiles, geomembranes, or a geotextile and a geomembrane
• To reinforce landfills to allow for vertical or lateral expansion
• To stabilize landfill cover soil as veneer reinforcement

Figure 2.3: Geogrid types (Das, 2016)
2.2.2 Geotextiles

Geotextiles are the oldest type among the different geosynthetic products. They are one of the two largest groups of geosynthetics, beside geogrids. Geotextiles are flexible, porous fabrics made of textile materials. The main polymers used to manufacture geotextiles are polypropylene and polyester. Geotextiles are indeed textiles, but they consist of synthetic rather than natural fibers. These synthetic fibers or filaments are produced by extruding melted polymers through a spinner. The fibers are then converted into a permeable fabric. Two main types of fabrics are typically formed, woven and nonwoven.

Woven geotextiles are made of two perpendicular sets of parallel filaments or yarns interlaced using traditional weaving methods and a variety of weave types. Non-woven geotextiles are manufactured by arranging the fiber filaments onto an oriented or random pattern to form a fabric mat, which is subsequently bonded using heat-bonding, chemical-bonding, or needle-punching process.

The terms machine direction and cross machine direction are commonly associated with geotextiles and geogrids. Machine direction refers to the direction in the plane of the fabric (or sheet in the case of geogrid) in line with the direction of manufacture. Cross machine direction refers to the direction perpendicular to the direction of manufacture. Geotextiles perform several functions in geotechnical engineering applications; however, the four primary functions of geotextiles are: separation, reinforcement, filtration, and drainage (Koerner, 2012).

2.3 Mechanisms of Reinforcement

A geosynthetic is used as a reinforcement in unpaved road applications to improve the load-carrying capacity or structural efficiency of the unpaved structure by transferring part of the load to the geosynthetic material. The mechanisms through which the geosynthetic reinforcement improves the performance of the unpaved road haven’t yet been fully understood. However, previous studies have attributed the improved performance due to geosynthetic reinforcement to the following reinforcement
mechanisms: tensioned membrane effect, subgrade vertical confinement, and lateral restraint (Giroud et al., 1985; Perkins and Ismeik, 1997; Giroud, 2009). These mechanisms are discussed in the following sections.

2.3.1 Lateral Restraint of Aggregate

Repeated traffic loads induce shear stresses at the bottom of the base layer and cause the aggregate to spread laterally. Geosynthetic reinforcement can restrain lateral spreading of aggregate (Figure 2.4a) and prevent shear failure of the base layer through shear interaction between geogrid and base aggregate. Giroud (2009) suggested that shear interaction mechanisms are different for the various types of geogrids. For extruded geogrids in particular, interaction is provided primarily by “interlocking” with aggregate particles. This mechanism is insignificant for geotextiles because of the relatively poor friction interaction between the aggregate and geotextiles (Webster, 1993; Giroud, 2009).

The interaction between the base aggregate and the geogrid transfers shear load from the base layer to a tensile load in the geogrid. As the geogrid is much stiffer in tension than the aggregate, it restrains lateral movement of the aggregate at the bottom of the base layer. Lateral restraints of aggregate can effectively minimize surface rutting of the unpaved road since lateral movement of base aggregate leads to vertical strain and plastic deformation. Lateral restrain increases the mean stress at the bottom of the base layer, which in turn causes the stiffness of the granular base course material to increase. The mechanism of lateral restraint does not require the development of large rut depths to be realized.

2.3.2 Tensioned Membrane Effect

The tensioned membrane effect (Figure 2.4b) develops as a result of vertical deformations under traffic loads causing a concave shape in the geosynthetic. This mechanism requires large rutting to create the concave shape in the geosynthetic and mobilize tension. The resultant of the geosynthetic tensions on each side of the concave
shape. The upward vertical components of the geosynthetic tensions on each side of the concave shape help support the wheel load and reduce the vertical stress on the subgrade. These components are balanced by downward components associated with the convex shape of the tensioned geosynthetic away from the wheels. Accordingly, this mechanism suggests that tension is mobilized laterally in the geogrid almost across the entire width of the road.

In the early attempts to identifying the reinforcing actions of unpaved roads, the tensioned membrane effect was thought to be the main mechanism governing the performance of unpaved roads. It is known now that it is negligible in geogrid-reinforced unpaved roads and supports only 10% of the load in geotextile-reinforced unpaved roads (Giroud, 2009).

### 2.3.3 Subgrade Vertical Confinement

The mechanism of subgrade vertical confinement or “inverted tensioned membrane” (Figure 2.4c) is based on the assumption of developing tension in the geogrid between the wheels to provide vertical confinement to the subgrade in the heave area away from the wheel loads. The vertical confinement will reduce the shear strain near the top of subgrade and limit subgrade rutting (under the wheel) and upheaval (away from the wheel).

As a result, the subgrade can be loaded near its ultimate bearing capacity and the mode of bearing failure of subgrade may change from punching failure to general failure with reinforcement. As with the tensioned membrane effect, this mechanism is also associated with large vertical deformations.
(a) Lateral restraint  
(b) Tensioned membrane  
(c) Subgrade confinement

Figure 2.4: Reinforcement mechanisms attributed to geogrids in roadways
2.4 Application of Geosynthetics in Unpaved Roads

The purpose of this section is to review and summarize literature pertaining to the application of geosynthetics as reinforcement in unpaved roads, as well as to synthesize and evaluate results from research studies that involve geosynthetic reinforcement of unpaved roads. This review focuses primarily on studies involving geogrid-reinforced unpaved roads. Studies involving an asphalt layer are cited only if information relevant to unpaved roads are involved. Research pertaining to flexible pavements have been reviewed and summarized by Perkins and Ismeik (1997) and Berg et al. (2000). Experimental features of pertinent studies that have important contribution to the current body of knowledge were summarized in a tabular form so that they are clearly highlighted.

2.4.1 Experimental Features

Details related to the experimental setup of the research studies is summarized in this section. Studies reviewed included full-scale field work involving controlled, random, or accelerated traffic, or monotonic loading, as well as model-scale laboratory work involving cyclic or monotonic plate loading. Experimental work is discussed with respect to testing facility and loading type, properties of the road structural layers, geosynthetics used and their placement location, and instrumentation used to measure mechanical response of the unpaved system.

2.4.1.1 Testing Facility and Loading Type

In general, two types of testing facility are used in investigating applications of geosynthetics in unpaved roads: full-scale test sections constructed in the field or model-scale laboratory facilities. A summary of the type of testing facilities employed in the studies is provided in Table 2.1, while Table 2.2 summarizes details regarding the utilized loading system. Unpaved test sections have been constructed in public roadways, access roads, or test roads (Fannin and Sigurdsson, 1996; Tingle and Jersey, 2009; Morris,
The length of the field test sections ranged from 6 to 20 m. These sections were typically loaded using channelized traffic of pre-weighted loaded trucks. The truck typically had a front axle of 25 to 35 kN and a dual-wheel, single rear axle of 80 kN was used to simulate traffic loading. Trucks with dual-wheel, tandem rear axle of 150 to 180 kN were also used. The wheels typically inflated to a tire pressure of 550-690 kPa in most studies. Field test sections have also been constructed on outdoor test-tracks (Tang et al., 2015), or indoor test-tracks (Watts et al., 2004) with varying lengths. The test tracks are characterized with shorter testing periods. They were typically loaded using 40 kN moving dual-wheel assembly that traffics forwards and backwards over the test sections in the same wheel-path (bidirectional traffic). On average, the speed of the truck or wheel assembly ranged between 10 and 15 km/h. Other studies involved the construction of model-scale unpaved test section in a small laboratory test tank or box (Leng and Gabr, 2002; Tingle and Jersey, 2005; Abu-Farsakh et al., 2016). In these studies, stationary rectangular or circular plates were typically loaded using either monotonic (hydraulic loading ram) or cyclic loading (load actuator). The response of unpaved structures have also been evaluated in large-scale covered test bays using monotonic plate load tests (Milligan et al., 1986).
<table>
<thead>
<tr>
<th>Testing type</th>
<th>Study</th>
<th>Testing facility</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Facility type</td>
<td>Facility dimensions</td>
</tr>
<tr>
<td></td>
<td>Facility dimensions (m)</td>
<td>(m)</td>
</tr>
<tr>
<td>Full-Scale tests</td>
<td>1. Chaddock (1988)</td>
<td>Concrete pit</td>
</tr>
<tr>
<td></td>
<td>2. Austin and Coleman (1993)</td>
<td>Test road</td>
</tr>
<tr>
<td></td>
<td>3. Fannin and Sigurdsson (1996)</td>
<td>Test road</td>
</tr>
<tr>
<td></td>
<td>4. Dawson and Little (1997)</td>
<td>Haul road</td>
</tr>
<tr>
<td></td>
<td>5. Santoni et al. (2001)</td>
<td>Test road</td>
</tr>
<tr>
<td></td>
<td>6. Tingle and Webster (2003)</td>
<td>Test road</td>
</tr>
<tr>
<td></td>
<td>7. Hufmus et al. (2006)</td>
<td>Test road</td>
</tr>
<tr>
<td></td>
<td>8. Cuelho and Perkins (2009)</td>
<td>Test road</td>
</tr>
<tr>
<td></td>
<td>(Phase I)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>9. Tingle and Jersey (2009)</td>
<td>Test road</td>
</tr>
<tr>
<td></td>
<td>10. White et al. (2011)</td>
<td>Test road</td>
</tr>
<tr>
<td></td>
<td>11. Morris (2013) (Phase II)</td>
<td>Test road</td>
</tr>
<tr>
<td>Accelerated pavement testing</td>
<td>12. Tang et al. (2015)</td>
<td>Outdoor test-track facility</td>
</tr>
<tr>
<td>Monotonic loading</td>
<td>13. Watts et al. (2004)</td>
<td>Indoor test-track facility</td>
</tr>
<tr>
<td></td>
<td>14. Milligan et al. (1986)</td>
<td>Covered test bay</td>
</tr>
</tbody>
</table>
Table 2.1 (Continued)

<table>
<thead>
<tr>
<th>Testing type</th>
<th>Study</th>
<th>Testing facility</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Facility type</td>
</tr>
<tr>
<td>Cyclic plate load test</td>
<td>15. Leng and Gabr (2002)</td>
<td>Laboratory test tank</td>
</tr>
<tr>
<td></td>
<td>16. Tingle and Jersey (2005)</td>
<td>Laboratory test box</td>
</tr>
<tr>
<td></td>
<td>17. Qian et al. (2013)</td>
<td>Laboratory test tank</td>
</tr>
<tr>
<td></td>
<td>18. Palmeira and Gongora (2016)</td>
<td>Cylindrical laboratory test tank</td>
</tr>
<tr>
<td></td>
<td>19. Bauer and Abdelhalim (1987)</td>
<td>Laboratory test box</td>
</tr>
<tr>
<td>Monotonic plate load test</td>
<td>20. Milligan and Love (1984)</td>
<td>Laboratory test tank</td>
</tr>
<tr>
<td></td>
<td>21. Abu-Farsakh et al. (2016)</td>
<td>Laboratory test tank</td>
</tr>
</tbody>
</table>

1 Length × Width × Depth, unless otherwise listed
2 Length × Width, unless otherwise listed
3 The facility generated a wheel path about 12 m
Table 2.2: Description of loading system

<table>
<thead>
<tr>
<th>Study</th>
<th>Loading description</th>
<th>Loading system</th>
<th>Width/diameter of loading area (mm)</th>
<th>Applied Load (kN)</th>
<th>Applied Pressure (kPa)</th>
<th>Load frequency or wheel speed</th>
<th>Maximum rut/settlement reached (cm)</th>
<th>Passes/cycles performed</th>
</tr>
</thead>
<tbody>
<tr>
<td>3. Fannin and Sigurdsson (1996)</td>
<td>Channelized traffic</td>
<td>Dual-wheel, single-rear axle truck</td>
<td>-</td>
<td>80 (rear axle), 110 (gross)</td>
<td>620</td>
<td>7 km/h</td>
<td>12 (thick)-20 (thin)</td>
<td>500</td>
</tr>
<tr>
<td>4. Dawson and Little (1997)</td>
<td>Channelized traffic</td>
<td>Dual-wheel, single-rear axle truck</td>
<td>195</td>
<td>80 (rear axle), 110-136 (gross); 126 (rear axle), 176 (gross)</td>
<td>490 (630, last 750 passes of)</td>
<td>32 km/h (measuring response: 10 km/h)</td>
<td>15 (unreinforced) 1.8 to 4.9 (reinforced)</td>
<td>Standard load: 1,000+115 Heavier load: 1,000</td>
</tr>
<tr>
<td>5. Santoni et al. (2001)</td>
<td>Channelized traffic</td>
<td>Dual-wheel, tandem-rear axle military truck</td>
<td>NR</td>
<td>185 (gross)</td>
<td>517</td>
<td>8-16 km/h</td>
<td>22</td>
<td>up to 2,000</td>
</tr>
<tr>
<td>6. Tingle and Webster (2003)</td>
<td>Channelized traffic</td>
<td>Dual-wheel, tandem-rear axle military truck</td>
<td>360 cm² (contact area)</td>
<td>147 (rear axle), 193.5 (gross)</td>
<td>517</td>
<td>16 km/h</td>
<td>7.6</td>
<td>2,000</td>
</tr>
<tr>
<td>7. Hufenus et al. (2006)</td>
<td>Channelized traffic</td>
<td>Dual-wheel, tandem-rear axle truck, tandem-steering axle</td>
<td>300</td>
<td>130 to 280 (variable, gross)</td>
<td>850</td>
<td>4 km/h</td>
<td>10</td>
<td>77</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Monotonic PLT, alternating with 20 cm lifts base compaction</td>
<td>300</td>
<td>-</td>
<td>500</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8. Cuelho and Perkins (2009)</td>
<td>Channelized traffic</td>
<td>Dual-wheel, tandem-rear axle dump truck</td>
<td>200</td>
<td>152.5 (tandem-rear axle), 204.6 (gross)</td>
<td>690</td>
<td>15 km/h</td>
<td>10</td>
<td>40</td>
</tr>
<tr>
<td>(Phase I)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9. Tingle and Jersey (2009)</td>
<td>Channelized traffic</td>
<td>Dual-wheel, tandem-rear axle truck</td>
<td>324 cm² (contact area)</td>
<td>194 (gross)</td>
<td>345</td>
<td>NR</td>
<td>7.5+ (up to 10)</td>
<td>10,000</td>
</tr>
<tr>
<td>10. White et al. (2011)</td>
<td>Channelized traffic</td>
<td>Dual-wheel, single-rear axle truck</td>
<td>NR</td>
<td>180 (rear axle), 200 (gross)</td>
<td>690</td>
<td>5 km/h</td>
<td>6.5</td>
<td>150</td>
</tr>
</tbody>
</table>
Table 2.2 (Continued)

<table>
<thead>
<tr>
<th>Study</th>
<th>Loading description</th>
<th>Loading system</th>
<th>Width/diameter of loading area (mm)</th>
<th>Applied Load (kN)</th>
<th>Applied Pressure (kPa)</th>
<th>Load frequency or wheel speed</th>
<th>Maximum rut/settlement reached (cm)</th>
<th>Passes/cycles performed</th>
</tr>
</thead>
<tbody>
<tr>
<td>11. Morris (2013) (Phase II)</td>
<td>Channelized traffic</td>
<td>Dual-wheel, tandem-rear axle dump truck</td>
<td>220</td>
<td>150.5 (tandem-rear axle)</td>
<td>620</td>
<td>8 km/h</td>
<td>7.5</td>
<td>740</td>
</tr>
<tr>
<td>12. Tang et al. (2015)</td>
<td>Accelerated traffic</td>
<td>Moving dual-wheel assembly</td>
<td>NR</td>
<td>43.4 (dual-wheel assembly)</td>
<td>725</td>
<td>16.8 km/h</td>
<td>2.5 (prerutting)</td>
<td>2,000 (reinforced) 400 (unreinforced)</td>
</tr>
<tr>
<td>13. Watts et al. (2004)</td>
<td>Accelerated traffic</td>
<td>Moving dual-wheel assembly</td>
<td>NR</td>
<td>40 (dual-wheel assembly)</td>
<td>NR</td>
<td>15 km/h</td>
<td>8</td>
<td>10,000</td>
</tr>
<tr>
<td>14. Milligan et al. (1986)</td>
<td>Monotonic</td>
<td>Stationary rectangular &amp; circular plates &amp; hydraulic loading ram</td>
<td>300, rectangular 300, circular</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>8</td>
<td>-</td>
</tr>
<tr>
<td>15. Leng and Gabr (2002)</td>
<td>Cyclic</td>
<td>Stationary circular plate &amp; servohydraulic load actuator</td>
<td>305</td>
<td>40</td>
<td>550</td>
<td>0.67 Hz</td>
<td>7.2</td>
<td>8,000 cycles</td>
</tr>
<tr>
<td>16. Tingle and Jersey (2005)</td>
<td>Cyclic (sinusoidal load pulse)</td>
<td>Stationary circular plate &amp; hydraulic actuator</td>
<td>305</td>
<td>40</td>
<td>550</td>
<td>1 Hz</td>
<td>7.5</td>
<td>10,000 cycles (1x10^6 ESALs)</td>
</tr>
<tr>
<td>17. Qian et al. (2013)</td>
<td>Cyclic</td>
<td>Stationary circular plate &amp; load actuator</td>
<td>300</td>
<td>40</td>
<td>566</td>
<td>0.77 Hz</td>
<td>7.5</td>
<td>up to 1,700 cycles</td>
</tr>
<tr>
<td>18. Palmeira and Gongora (2016)</td>
<td>Cyclic</td>
<td>Stationary circular plate &amp; hydraulic actuator</td>
<td>200</td>
<td>NR</td>
<td>560</td>
<td>1 Hz</td>
<td>7.5</td>
<td>up to 6,500 cycles</td>
</tr>
<tr>
<td>19. Bauer and Abdelhalim (1987)</td>
<td>Cyclic</td>
<td>Stationary circular plate &amp; hydraulic load actuator</td>
<td>305</td>
<td>40</td>
<td>550</td>
<td>3 Hz</td>
<td>2.8</td>
<td>up to 233,000 cycles (reinforced) 155,000 cycles, (unreinforced)</td>
</tr>
<tr>
<td>20. Milligan and Love (1984)</td>
<td>Monotonic (plane strain conditions)</td>
<td>Stationary strip Footing plate &amp; hydraulic loading ram</td>
<td>75 (covering tank width)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>5</td>
<td>-</td>
</tr>
<tr>
<td>21. Abu-Farsakh et al. (2016)</td>
<td>Monotonic</td>
<td>Stationary circular bearing plate &amp; hydraulic jack</td>
<td>190</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

1 Bidirectional traffic: the truck/wheel traffics forwards and backwards over the test sections in the same wheel-path

NA = Not Applicable; NR = Not Reported
2.4.1.2 Properties of Road Structural Layers

Table 2.3 lists a summary of the properties of the structural layers of the test sections in the main research studies examined in this review. The unpaved test sections consisted of a base course layer constructed over a subgrade. The subgrade was either native, modified (e.g., scarified) or artificial/imported soil ranging from a very weak subgrade (e.g., very soft high plasticity clay with CBR as low as 0.5%) to a strong subgrade (e.g., stiff clay or clayey sand with California Bearing Ratio, CBR, as high as 5-8%). Properties of the base course material used in each study along with the thickness of the base layer are listed in Table 2.3. In the majority of the studies examined, base layer consisted of crushed stone materials (Tingle and Jersey, 2005; Cuelho and Perkins, 2009; Tang et al., 2015). Other materials used included sand and gravel mixture (Fannin and Sigurdsson, 1996; Dawson and Little, 1997) recycled material (Santoni et al., 2001; Hufenus et al., 2006). Milligan and Love (1984) used scaled down crushed limestone to 12 mm maximum size in their small-scale monotonic plate load tests. The constructed thickness of the base layer varies widely between the studies (i.e., 150 to 900 mm), but the average thickness used for the base layer was around 250 mm. Several studies constructed test sections with base thicknesses that varied linearly with the length of the section (i.e., tapered) (Chaddock, 1988; Austin and Coleman, 1993; Fannin and Sigurdsson, 1996).
Table 2.3: Details of road layers

<table>
<thead>
<tr>
<th>Study</th>
<th>Layer material type</th>
<th>Structure layers</th>
<th>Layer thickness (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Material description</td>
<td>Dry unit weight</td>
<td>Description</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(Imported)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>overlying sandy clay (native)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1. Stiff</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2. Soft/firm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3. Very soft</td>
</tr>
<tr>
<td>1. Chaddock (1988)</td>
<td>Crushed limestone (50 mm maximum size)</td>
<td>NR</td>
<td>Gault clay</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(Imported)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>overlying sandy clay (native)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1. Stiff</td>
</tr>
<tr>
<td>2. Austin and Coleman (1993)</td>
<td>Well-graded crushed limestone (GW, 38 mm maximum size)</td>
<td>20 kN/m³</td>
<td>0.5-1%</td>
</tr>
<tr>
<td>3. Fannin and Sigurdssohn (1996)</td>
<td>Very sandy gravel (19 mm maximum size)</td>
<td>20.7 kN/m³</td>
<td>=1.3%</td>
</tr>
<tr>
<td>4. Dawson and Little (1997)</td>
<td>1. Main material: crushed rock/diorite</td>
<td>[20.2-22 (21.1 ave) kN/m³</td>
<td>75-80 kPa</td>
</tr>
<tr>
<td></td>
<td>2. Supplemental material: sand and gravel mixture</td>
<td></td>
<td>Mr = 360 &amp; 450 MPa at 20 &amp; 45 kPa conf. stress; φ = 54°</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>20.2-22 (20.7 ave) kN/m³</td>
</tr>
<tr>
<td>5. Santoni et al. (2001)</td>
<td>1. Crushed limestone (SM-SC, 19 mm maximum size)</td>
<td>10-20%</td>
<td>Very soft low-plasticity clayey silt² (CL-ML)</td>
</tr>
<tr>
<td></td>
<td>2. Sand (SP)</td>
<td>6%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3. Fibre-reinforced sand</td>
<td>10%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4. wood chips</td>
<td>7-14%</td>
<td></td>
</tr>
<tr>
<td>6. Tingle and Webster (2003)</td>
<td>Well-graded crushed limestone (GP-GC, 25 mm maximum size)</td>
<td>22.6 kN/m³</td>
<td>16.3 kN/m³ (max.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>50-75% (64% ave.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Soft high-plasticity clay² (CH)</td>
</tr>
<tr>
<td>7. Hufenus et al. (2006)</td>
<td>1. Bottom two layers: Loose recycled rubble (GP, 64 mm maximum size)</td>
<td>14.3-14.9 kN/m³</td>
<td>16.3 kN/m³ (max.)</td>
</tr>
<tr>
<td></td>
<td>2. Surface/top layer: Finer grained recycled material (GP, 32 mm maximum size)</td>
<td>16.7 kN/m³</td>
<td>0.7-1%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Medium plasticity silty clay (CM), very low bearing capacity</td>
</tr>
<tr>
<td>Study</td>
<td>Material description</td>
<td>Layer material type</td>
<td>Dry unit weight</td>
</tr>
<tr>
<td>-------------------------------------------</td>
<td>----------------------</td>
<td>---------------------</td>
<td>----------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>CBR</td>
</tr>
<tr>
<td>8. Cuelho and Perkins (2009) (Phase I)</td>
<td>Crushed gravel (GW-GM)</td>
<td>Base</td>
<td>21 kN/m³ (ave.)</td>
</tr>
<tr>
<td></td>
<td>1. Crushed limestone (SW-SM, higher-quality)</td>
<td>Base</td>
<td>20.7 kN/m³</td>
</tr>
<tr>
<td></td>
<td>2. Crushed chert aggregate (GW, more uniformly graded)</td>
<td>Base</td>
<td>15.77-18.26 (16.88 ave.) kN/m³</td>
</tr>
<tr>
<td>10. White et al. (2011)</td>
<td>Crushed limestone (GP-GM, 38 mm maximum size)</td>
<td>Base</td>
<td>13.5% (halfway through compaction), 45% (after trafficking)</td>
</tr>
<tr>
<td>11. Morris (2013) (Phase II)</td>
<td>Poorly graded gravel with clay and sand (GP-GC, 25 mm maximum size)</td>
<td>Base</td>
<td>21 kN/m³</td>
</tr>
<tr>
<td>12. Tang et al. (2015)</td>
<td>Dense-graded crushed limestone (GW)</td>
<td>Base</td>
<td>19.5-20.1 kN/m³</td>
</tr>
<tr>
<td>13. Watts et al. (2004)</td>
<td>Crushed granite</td>
<td>Base</td>
<td>Surface modulus/FWD:</td>
</tr>
<tr>
<td>14. Milligan et al. (1986)</td>
<td>Well graded Crushed limestone (37.5 mm maximum size)</td>
<td>Base</td>
<td>23.1 kN/m³</td>
</tr>
<tr>
<td>15. Leng and Gabr (2002)</td>
<td>Well graded gravel (GW, 30 mm maximum size)</td>
<td>Base</td>
<td>19.3-20.5 kN/m³</td>
</tr>
</tbody>
</table>

Note: 1. Dry unit weight | CBR | 1.7% (ave.) | 52-62 kPa |

2. Clayey sand² (85% Lillington Sand and 15% Kaolinite) | 15.2 (thin) |

3. Clayey sand² (85% Lillington Sand and 15% Kaolinite) | 15.2 (thin) |

4. Clayey sand² (85% Lillington Sand and 15% Kaolinite) | 15.2 (thin) |

5. Clayey sand² (85% Lillington Sand and 15% Kaolinite) | 15.2 (thin) |

6. Clayey sand² (85% Lillington Sand and 15% Kaolinite) | 15.2 (thin) |

7. Clayey sand² (85% Lillington Sand and 15% Kaolinite) | 15.2 (thin) |

8. Clayey sand² (85% Lillington Sand and 15% Kaolinite) | 15.2 (thin) |

9. Clayey sand² (85% Lillington Sand and 15% Kaolinite) | 15.2 (thin) |

10. Clayey sand² (85% Lillington Sand and 15% Kaolinite) | 15.2 (thin) |

11. Clayey sand² (85% Lillington Sand and 15% Kaolinite) | 15.2 (thin) |

12. Clayey sand² (85% Lillington Sand and 15% Kaolinite) | 15.2 (thin) |

13. Clayey sand² (85% Lillington Sand and 15% Kaolinite) | 15.2 (thin) |

14. Clayey sand² (85% Lillington Sand and 15% Kaolinite) | 15.2 (thin) |

15. Clayey sand² (85% Lillington Sand and 15% Kaolinite) | 15.2 (thin) |
<table>
<thead>
<tr>
<th>Study</th>
<th>Structure layers</th>
<th>Layer thickness (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Base</td>
<td>Subgrade</td>
</tr>
<tr>
<td><strong>Material description</strong></td>
<td>**Dry unit weight</td>
<td>CBR**</td>
</tr>
<tr>
<td>16. Tingle and Jersey (2005)</td>
<td>crushed limestone, with nonplastic fines (SW-SM, 38 mm maximum size)</td>
<td>20.1 kN/m³</td>
</tr>
<tr>
<td>17. Qian et al. (2013)</td>
<td>AB-3 well graded aggregate (20 mm maximum size)</td>
<td>20.4 kN/m³ (max.)</td>
</tr>
<tr>
<td>18. Palmeira and Gongora (2016)</td>
<td>Gravel (NR) (21 mm maximum size)</td>
<td>17.3 kN/m³ (RD = 83%)</td>
</tr>
<tr>
<td>19. Bauer and Abdelhalim (1987)</td>
<td>Well graded crushed limestone (13-mm maximum size)</td>
<td>23 kN/m³ (max.)</td>
</tr>
<tr>
<td>20. Milligan and Love (1984)</td>
<td>Sand and gravel (scaled down crushed limestone to 12 mm maximum size)</td>
<td>17.5 kN/m³</td>
</tr>
<tr>
<td>21. Abu-Farsakh et al. (2016)</td>
<td>Crushed limestone</td>
<td>22.7 kN/m³ (ave.)</td>
</tr>
</tbody>
</table>

1 Undrained shear strength unless otherwise specified
2 Artificial, prepared or modified subgrade
2.4.1.3 Geosynthetic Reinforcement Type, Properties and Placement Location

The influence of geosynthetic type, structure, stiffness, placement location, and layering on the reinforcement benefit has been investigated in the literature. Geogrids, geotextiles, or combinations of both were typically used. The physical and mechanical properties of the geogrids and geotextiles used in some studies are listed in Table 2.4 and Table 2.5, respectively. In general, the geogrids used varied in terms of manufacturing process or “structure” (e.g., punched sheet-drawn, woven, welded, extruded to a triplanar shaped grid, or continuously extruded and orientated) and polymer composition (e.g., polypropylene, polyester, or polymer aramid fibers). The geogrids also differed with respect to stiffness and aperture size to examine their effect on reinforcement benefit. Some researchers have assembled geogrids in the laboratory to allow variations of specific geogrid properties while keeping others constant (Palmeira and Gongora, 2016). Others used scaled versions of geogrids or “miniature geogrids” with small openings (Milligan and Love, 1984). Similarly, the geotextiles used in previous studies differed in structure (e.g., woven, nonwoven, needle-punched, or multifilament) and polymer composition (e.g., polypropylene, polyester, or polyethylene). Researchers used geotextiles of widely varied stiffness. The purpose of using these geotextiles varied from only separation (nonwoven material with stiffness that is typically lower than that of a medium stiff geogrid) to reinforcement (woven fabric with stiffness as high as three times that of a light-duty geogrid). The type of geosynthetics used in each study and their placement location in the unpaved structure are summarized in Table 2.6. Geosynthetics, especially geotextiles, were typically placed at the base–subgrade interface. However, other placement locations of geogrids at different depths within the base layer were examined by Abu-Farsakh et al. (2016). Several studies have examined the effect of using additional layers of geogrids within the base layer on performance (Milligan et al., 1986; Watts et al., 2004; Tang et al., 2015; Abu-Farsakh et al., 2016). All of the studies investigated different placement locations and layering were model-scale laboratory studies. In some studies, geosynthetics were not anchored, restrained or pre-tensioned (Milligan et al., 1986; Dawson and Little, 1997; Tingle and Jersey, 2005). Chaddock (1988) placed geogrids across only half the subgrade width (5 m) to be compared with the other unreinforced half of the test section.
<table>
<thead>
<tr>
<th>Geogrid</th>
<th>Manufacturer, product name</th>
<th>Structure / Polymer composition</th>
<th>Mass/unit area (g/m²)</th>
<th>Aperture size MD/XD (mm)</th>
<th>Secant modulus MD/XD (kN/m)</th>
<th>Tensile strength MD/XD (kN/m)</th>
<th>Ultimate Aperture stability modulus (mN/deg)</th>
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<td>12.4/19</td>
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<td>220/400</td>
<td>6/9</td>
<td>11.8/19/19.6</td>
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<td>220/400</td>
<td>6/9</td>
<td>11.8/19/19.6</td>
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<td>—</td>
<td>7/12</td>
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</tr>
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<td>—</td>
<td>11/12</td>
</tr>
<tr>
<td>9</td>
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<td>B-K PET/PVC-C</td>
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<tr>
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<td>8/8</td>
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<td>7.3/7.3</td>
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<td>15/15</td>
<td>400/650</td>
<td>400/540</td>
<td>8/13</td>
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<td>300 ⁴</td>
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<td>Secant modulus MD/XD (kN/m)</td>
<td>Tensile strength MD/XD (kN/m)</td>
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<td>PP</td>
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<td>PP</td>
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<td>PP</td>
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<td>205/330</td>
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<td>PET</td>
<td>--</td>
<td>23/35</td>
<td>811</td>
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Table 2.4 (Continued)
Table 2.4 (Continued)

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<tr>
<th>Geogrid</th>
<th>Manufacturer, product name</th>
<th>Structure ¹</th>
<th>Polymer composition ²</th>
<th>Mass/unit area (g/m²)</th>
<th>Aperture size MD/XD (mm)</th>
<th>Secant modulus MD/XD (kN/m)</th>
<th>Tensile strength MD/XD (kN/m)</th>
<th>Aperture stability modulus (m.N/deg)</th>
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<td>NR</td>
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<td>PP</td>
<td>–</td>
<td>26/40</td>
<td>–</td>
<td>474</td>
<td>–</td>
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<td>54</td>
<td>NR</td>
<td>B-(NR)</td>
<td>PP</td>
<td>–</td>
<td>11/15</td>
<td>–</td>
<td>474</td>
<td>–</td>
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<td>PP</td>
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<td>59</td>
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<td>PP</td>
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<td>60/60</td>
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<td>225⁴</td>
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<td>PP</td>
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<td>300⁴</td>
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<td>PP</td>
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<td>365⁴</td>
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<td>PP</td>
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<td>50/60</td>
<td>–</td>
<td>4.4/8.6</td>
<td>9.9/16.8</td>
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¹ B = Biaxial; T = Triaxial; U = Uniaxial; PSD = Punched, sheet drawn (integrally-formed geogrids); W = Woven; K = Knitted; CEO = Continuous extrusion and orientation (integrally-formed geogrids); MCEO = Multilayer, continuous extrusion and orientation (integrally-formed geogrids); ETP = Extruded to a triplanar shaped grid; HWFR = Heat welded flat ribs; LWe = Laser welded; VWe = Vibratory welded; GC = Geogrid composite
² PP = Polypropylene, PET = Polyester, PVA = Polyvinyl alcohol, PVC-C = Polyvinyl chloride coated, P-C = Polymer coated; PET-PA = geogrid composite formed with polymer aramid (Para-Aramid) fibers "Twaron®" embedded in a polyester nonwoven "Colback®"
³ Reported as “rib pitch” in manufacturer’s specification sheet
⁴ Radial stiffness (tensile modulus) at 0.5% strain (ASTM D6637-10), kN/m
⁵ Tensile strength (at 0.5% strain) in radial direction, kN/m
⁶ Tested a single layer, and multiplied by 3 (three layer material) based on Kinney (2000) (not published by manufacturer)
⁷ Tested using a torque of 4.34 in-lb. (5 kg-cm). The standard is 17.70 lb-in (20.4 kg-cm); may result in stability moduli less than those from the procedure outlined by Kinney (2000)
NR = Not Reported
Table 2.5: Properties of geotextiles used in test studies

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<th>Geotextile</th>
<th>Manufacturer, product name</th>
<th>Structure</th>
<th>Polymer composition</th>
<th>Mass/unit area (g/m²)</th>
<th>Apparent opening size (mm)</th>
<th>Secant modulus MD/ XD (kN/m)</th>
<th>Tensile strength MD/ XD (kN/m)</th>
<th>Grab tensile strength (N)</th>
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<td>PP</td>
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<td>7/7</td>
<td>711</td>
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<td>NW-NP</td>
<td>PP</td>
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<td>2%</td>
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<td>933</td>
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<td>Polyfelt, TS 700</td>
<td>NW-NP</td>
<td>PP</td>
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<td>2%</td>
<td>15.8/14</td>
<td>933</td>
</tr>
<tr>
<td>4</td>
<td>Rhone Poulenc, BIDIM B2</td>
<td>NW-NP</td>
<td>PET</td>
<td>144</td>
<td>—</td>
<td>2%</td>
<td>0.74/0.32^1</td>
<td>6.8/7.9^1</td>
</tr>
<tr>
<td>5</td>
<td>DuPont, 3407</td>
<td>NW-HB</td>
<td>PP</td>
<td>134</td>
<td>—</td>
<td>2%</td>
<td>3.5/3.4^1</td>
<td>7/7</td>
</tr>
<tr>
<td>6</td>
<td>Polyfelt, TS500</td>
<td>NW-NP</td>
<td>PP</td>
<td>142</td>
<td>—</td>
<td>3%</td>
<td>0.7/1.9^1</td>
<td>9/9.8^1</td>
</tr>
<tr>
<td>7</td>
<td>NR, 40/45kN</td>
<td>W-SF</td>
<td>PP</td>
<td>265</td>
<td>—</td>
<td>2%</td>
<td>8.9/32.2^2</td>
<td>54.8/48.3^2</td>
</tr>
<tr>
<td>8</td>
<td>NR</td>
<td>W-ST</td>
<td>PP</td>
<td>NR (330)</td>
<td>—</td>
<td>2%</td>
<td>12/12</td>
<td>65/65</td>
</tr>
<tr>
<td>9</td>
<td>NR</td>
<td>W-ST</td>
<td>PP</td>
<td>NR</td>
<td>—</td>
<td>2%</td>
<td>8/30</td>
<td>30/30</td>
</tr>
<tr>
<td>10</td>
<td>NR</td>
<td>NW-NP</td>
<td>PP</td>
<td>NR</td>
<td>—</td>
<td>3%</td>
<td>0.4/0.3</td>
<td>20/20</td>
</tr>
<tr>
<td>11</td>
<td>NR</td>
<td>NW-NP</td>
<td>PP</td>
<td>NR</td>
<td>—</td>
<td>3%</td>
<td>0.3/0.2</td>
<td>10/10</td>
</tr>
<tr>
<td>12</td>
<td>NR</td>
<td>NW-MF SRPY</td>
<td>PP and PET</td>
<td>280</td>
<td>8.5</td>
<td>2%</td>
<td>7.5/7.5</td>
<td>50/50</td>
</tr>
<tr>
<td>13</td>
<td>LINQ, 180EX</td>
<td>NW-NP</td>
<td>PP</td>
<td>285</td>
<td>0.18</td>
<td>3%</td>
<td>8.8/8.8</td>
<td>21.9/21.9</td>
</tr>
<tr>
<td>14</td>
<td>Contech, C-80NW</td>
<td>NW-NP</td>
<td>PP</td>
<td>271</td>
<td>0.18</td>
<td>3%</td>
<td>8.8/8.8</td>
<td>21.9/21.9</td>
</tr>
<tr>
<td>15</td>
<td>NR</td>
<td>W</td>
<td>PP</td>
<td>342</td>
<td>0.425</td>
<td>3%</td>
<td>8.8/8.8</td>
<td>21.9/21.9</td>
</tr>
<tr>
<td>16</td>
<td>Propex, Geotex801</td>
<td>NW-NP</td>
<td>PP</td>
<td>271</td>
<td>0.18</td>
<td>3%</td>
<td>8.8/8.8</td>
<td>21.9/21.9</td>
</tr>
<tr>
<td>17</td>
<td>NR</td>
<td>NW</td>
<td>PP</td>
<td>150</td>
<td>NR</td>
<td>3%</td>
<td>8.8/8.8</td>
<td>21.9/21.9</td>
</tr>
<tr>
<td>18</td>
<td>TenCate, Mirafi RS580i</td>
<td>W-MF</td>
<td>PP</td>
<td>417</td>
<td>0.425</td>
<td>3%</td>
<td>7/26.3</td>
<td>70/70</td>
</tr>
<tr>
<td>19</td>
<td>NR</td>
<td>W</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>3%</td>
<td>39/344.5</td>
<td>—</td>
</tr>
<tr>
<td>20</td>
<td>NR</td>
<td>NW</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>3%</td>
<td>50.6</td>
<td>—</td>
</tr>
<tr>
<td>21</td>
<td>NR</td>
<td>W</td>
<td>PP</td>
<td>—</td>
<td>0.425</td>
<td>3%</td>
<td>1112</td>
<td>—</td>
</tr>
<tr>
<td>22</td>
<td>NR</td>
<td>NW-NP</td>
<td>PP</td>
<td>—</td>
<td>0.21</td>
<td>3%</td>
<td>578</td>
<td>—</td>
</tr>
<tr>
<td>23</td>
<td>NR, W-PP-GT</td>
<td>W</td>
<td>PP</td>
<td>—</td>
<td>14/14</td>
<td>3%</td>
<td>70/70</td>
<td>—</td>
</tr>
</tbody>
</table>
Table 2.5 (Continued)

<table>
<thead>
<tr>
<th>Geotextile</th>
<th>Manufacturer, product name</th>
<th>Structure¹</th>
<th>Polymer composition²</th>
<th>Mass/unit area (g/m²)</th>
<th>Apparent opening size (mm)</th>
<th>Secant modulus MD/XD (kN/m)</th>
<th>Tensile strength MD/XD (kN/m)</th>
<th>Grab tensile strength (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24</td>
<td>NR</td>
<td>NW-NP</td>
<td>PP</td>
<td>NR</td>
<td>0.212</td>
<td>2%</td>
<td>2%</td>
<td>2%</td>
</tr>
<tr>
<td>25</td>
<td>NR, geotextile separator</td>
<td>NW-NP</td>
<td>NR</td>
<td>—</td>
<td>—</td>
<td>2%</td>
<td>2%</td>
<td>2%</td>
</tr>
<tr>
<td>26</td>
<td>Lotrak 40</td>
<td>W</td>
<td>NR</td>
<td>—</td>
<td>—</td>
<td>2%</td>
<td>2%</td>
<td>2%</td>
</tr>
<tr>
<td>27</td>
<td>NR</td>
<td>NR</td>
<td>PP</td>
<td>—</td>
<td>—</td>
<td>2%</td>
<td>2%</td>
<td>2%</td>
</tr>
<tr>
<td>28</td>
<td>NR, comparable with TenCate, Mirafi RS580i)</td>
<td>NR</td>
<td>PP</td>
<td>—</td>
<td>—</td>
<td>2%</td>
<td>2%</td>
<td>2%</td>
</tr>
<tr>
<td>29</td>
<td>NR</td>
<td>W-(NR)</td>
<td>PET</td>
<td>—</td>
<td>—</td>
<td>2%</td>
<td>2%</td>
<td>2%</td>
</tr>
</tbody>
</table>

¹ W = Woven, NW = Nonwoven, NP = Needlepunched, HB = Heat Bonded, MF = Multifilament, SRPY = substrate reinforced with polyester yarn, SF = Split Film, ST = Slit Tape

² PP = Polypropylene, PET = Polyester, PE = Polyethylene

PET-PA = geogrid composite formed with polymer aramid (Para-Aramid) fibers "Twaron®" embedded in a polyester nonwoven "Colback®"

³ Based on BS 6906, in kN

NR = Not Reported
Table 2.6: Type and location of geosynthetic used in each study

<table>
<thead>
<tr>
<th>Study</th>
<th>Geosynthetic type</th>
<th>Geosynthetic location and layering</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Chaddock (1988)</td>
<td>Geogrid 4</td>
<td>Interface</td>
<td>Geogrid was laid across only half the subgrade width to be compared with adjacent unreinforced half of the section</td>
</tr>
<tr>
<td>2. Austin and Coleman (1993)</td>
<td>Geogrids 22 to 25, Geotextile 19, 20</td>
<td>Interface</td>
<td>Effect of placing geogrid over a NW geotextile separator was investigated in one section</td>
</tr>
<tr>
<td>3. Fannin and Sigurdsson (1996)</td>
<td>Geogrid 1, Geotextiles 1 to 3</td>
<td>Interface</td>
<td></td>
</tr>
<tr>
<td>4. Dawson and Little (1997)</td>
<td>Geogrid 4, 5, Geotextiles 4 to 7</td>
<td>Interface</td>
<td>No pre-tension or end restraining to geosynthetics</td>
</tr>
<tr>
<td>5. Santoni et al. (2001)</td>
<td>Geogrid 2, 12, Geotextile 13, 14</td>
<td>Interface</td>
<td></td>
</tr>
<tr>
<td>6. Tingle and Webster (2003)</td>
<td>Geogrid 26 / Geotextile 21, 22</td>
<td>Interface</td>
<td>Geogrid was used combined with NW geotextile separator</td>
</tr>
<tr>
<td>7. Hufenus et al. (2006)</td>
<td>Geogrids 7 to 11, Geotextiles 8 to 12</td>
<td>Interface</td>
<td>Geogrids 8, 10 &amp; 11 were used with and without NW geotextile separator</td>
</tr>
<tr>
<td>8. Cuelho and Perkins (2009) (Phase I)</td>
<td>Geogrids 1, 2, 13 to 17, Geotextiles 15 to 17</td>
<td>Interface</td>
<td></td>
</tr>
<tr>
<td>10. White et al. (2011)</td>
<td>Geogrid 2, 21, Geotextile 23</td>
<td>Interface</td>
<td></td>
</tr>
<tr>
<td>11. Morris (2013) (Phase II)</td>
<td>Geogrid 3, 6, 13, 14, 16 to 21, Geotextile 16, 18</td>
<td>Interface</td>
<td></td>
</tr>
<tr>
<td>12. Tang et al. (2015)</td>
<td>Geogrid 28, Geotextile 18, 25</td>
<td>Interface (all) / interface + at the upper one-third of the base layer (geogrid 28)</td>
<td>NW geotextile separator (No 25) was placed at the interface of the unreinforced and geogrid-reinforced sections</td>
</tr>
<tr>
<td>13. Watts et al. (2004)</td>
<td>Geogrids 29 to 39, Geotextile 26</td>
<td>Interface (all) / interface + at the mid-height of the base layer (geogrid 31)</td>
<td></td>
</tr>
<tr>
<td>14. Milligan et al. (1986)</td>
<td>Geogrid 5</td>
<td>Interface / interface + at 85 and 160 mm below the top of base layer (supplemental tests)</td>
<td>Without restraint or artificial tensioning</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Effect of geotextile separator was investigated in one test</td>
</tr>
<tr>
<td>Study</td>
<td>Geosynthetic type</td>
<td>Geosynthetic location and layering</td>
<td>Notes</td>
</tr>
<tr>
<td>-------</td>
<td>------------------</td>
<td>-----------------------------------</td>
<td>-------</td>
</tr>
<tr>
<td>15. Leng and Gabr (2002)</td>
<td>Geogrid 1, 2</td>
<td>Interface</td>
<td></td>
</tr>
<tr>
<td>16. Tingle and Jersey (2005)</td>
<td>Geogrid 27</td>
<td>Interface</td>
<td>Geosynthetics were not anchored Geogrid was used with and without NW geotextile separator</td>
</tr>
<tr>
<td>17. Qian et al. (2013)</td>
<td>Geogrids 61 to 63</td>
<td>Interface</td>
<td></td>
</tr>
<tr>
<td>18. Palmeira and Gongora (2016)</td>
<td>Geogrids 49 to 60</td>
<td>Interface</td>
<td>Geogrids 55 to 60 were lab assembled to allow variations of specific properties while keeping others constant.</td>
</tr>
<tr>
<td>19. Bauer and Abdelhalim (1987)</td>
<td>Geogrid 64</td>
<td>Interface</td>
<td>Other locations were used but not discussed in the study</td>
</tr>
<tr>
<td>20. Milligan and Love (1984)</td>
<td>A miniature Geogrid 4</td>
<td>Interface</td>
<td>scaled version of SS1, 8 x 10 mm openings</td>
</tr>
<tr>
<td>21. Abu-Farsakh et al. (2016)</td>
<td>Geogrids 40 to 48</td>
<td>Interface (all) / middle of the base (geogrids 44 &amp; 48) / upper one-third of the base (geogrids 41, 44 &amp; 48) / interface + upper one-third of the base (geogrids 41, 46, 48 and geogrid 44 + geotextiles 27 &amp; 28)</td>
<td></td>
</tr>
</tbody>
</table>
2.4.1.4 Measurements and Instrumentation

The instrumentation used to measure the response of the test sections and the measurements performed during the field and laboratory tests can help evaluate the performance and explain the performance mechanism. The measured parameters and instrumentation used in the primary studies reviewed are summarized in Table 2.7. Measurements performed on the surface of the unpaved sections included rut depth, surface deformation (i.e., elevation rut), and transverse surface profiles. Laboratory studies used Linear Variable Differential Transformers (LVDTs) or dial gauges to measure displacement of the loading plate and lateral profiles at varying distances from the center of the plate. The applied load was typically measured using load cells.

The instrumentation programs used focused on measuring basic layer parameters such as layer thickness, deformation, strain, stress, pore pressure, moisture content, or temperature at different locations within the road layers. The instrumentation used in the base and subgrade layers included inductive strain coils, LVDTs, extensometers, potentiometers, time-domain reflectometers, piezometers, and earth pressure cells.

The literature review of strain measurement in geosynthetics for roadway applications showed that foil strain gauges were widely used (Bauer and Abdelhalim, 1987; Hufenus et al., 2006; Tingle and Jersey, 2009). Several studies have determined strain in geosynthetics based on displacement measurement using lead wire attached to LVDTs or marker studs (Fannin and Sigurdsson, 1996; Cuelho and Perkins, 2009; Morris, 2013). Fannin and Sigurdsson (1996) determined the strain in geotextiles from displacements of marker studs. Dawson and Little (1997) used inductive strain coils to measure permanent and transient transverse strain on the geosynthetics.

While instrumentation can be useful, it is usually restricted by the additional cost resulting from the cost of the instruments, installation, and data processing. In addition, instrumentation survivability is a major consideration. For example, the reported instrument survivability in literature indicated that survivability rates for foil strain gauges were as low as 30% (Table 2.7). Meanwhile, survivability rates as low as 6% were reported in pavement studies (Brandon et al., 1996).
Failure of the unpaved test sections was generally defined as a specific amount of rut depth or surface deformation. Parameters used as performance criteria included rut depth or surface deformation (Austin and Coleman, 1993; Tingle and Webster, 2003), the response of the test section to a falling weight deflectometer (FWD) test (Watts et al. 2004; Tingle and Jersey, 2009), compaction measurements (White et al., 2011), bearing capacity or bearing capacity ratio (Milligan et al., 1986; Abu-Farsakh et al. 2016), and vertical stress transferred to the subgrade (Leng and Gabr, 2002; Tingle and Jersey, 2005).
<table>
<thead>
<tr>
<th>Study</th>
<th>Surface</th>
<th>Base</th>
<th>Subgrade</th>
<th>Geosynthetic</th>
<th>Survivability</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>2. Austin and Coleman (1993)</td>
<td>Elevation rut (rod &amp; level)</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>Failure criterion: 75 mm rut depth</td>
</tr>
<tr>
<td>4. Dawson and Little (1997)</td>
<td>Rut depth (straightedge)</td>
<td>Layer thickness</td>
<td>Transient &amp; permanent vertical strains at the top (inductive strain coil)</td>
<td>Permanent &amp; transient transverse strain (inductive strain coil)</td>
<td>Strain coils: 80% survived construction</td>
<td>Failure criterion: 150 mm rut depth</td>
</tr>
<tr>
<td>5. Santoni et al. (2001)</td>
<td>Elevation rut</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>6. Tingle and Webster (2003)</td>
<td>Rut depth (straightedge)</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>7. Hufenus et al. (2006)</td>
<td>Elevation rut &amp; transverse surface profiles (cross bar)</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>32 strain gauges: 100% (no comment on functionality)</td>
</tr>
<tr>
<td>8. Cuelho and Perkins (2009) (Phase I)</td>
<td>Elevation rut &amp; transverse surface profiles (total station)</td>
<td>--</td>
<td>Displacement (LVDT)</td>
<td>Transverse strain based on displacement measurements (lead wire attached to LVDT)</td>
<td>--</td>
<td>Displacement and pore water pressure was collected at 200 Hz</td>
</tr>
<tr>
<td>Study</td>
<td>Surface</td>
<td>Base</td>
<td>Subgrade</td>
<td>Geosynthetic</td>
<td>Survivability</td>
<td>Notes</td>
</tr>
<tr>
<td>------------------------</td>
<td>-------------------------------------------------------------------------</td>
<td>-----------------------------</td>
<td>--------------------------------------------------------------------------</td>
<td>----------------------------------------------------------------------------</td>
<td>--------------------------------------------</td>
<td>----------------------------------------------------------------------</td>
</tr>
<tr>
<td>9. Tingle and Jersey (2009)</td>
<td>Rut depth (straightedge)</td>
<td>Surface deflection (Dynatest FWD)</td>
<td>Vertical stress at the top (Geokon 229-mm EPC)</td>
<td>Pore pressure near the top (Geokon vibrating wire pore pressure transducers)</td>
<td>Transverse &amp; longitudinal strain in the wheel-path &amp; 457 mm from the wheel-path (foil strain gauge, two perpendicular gauges in half-bridge)</td>
<td>Data from geosynthetic foil strain gauges was not reported</td>
</tr>
<tr>
<td>10. White et al. (2011)</td>
<td>Rut depth (straight edge)</td>
<td>Elevation rut &amp; transverse surface profiles (level &amp; RTK-GPS)</td>
<td>Vertical &amp; horizontal stresses at the top (1000 kPa Geokon 3500 piezoelectric EPC)</td>
<td>--</td>
<td>--</td>
<td>Measurements collected at 1613 Hz</td>
</tr>
<tr>
<td>11. Morris (2013) (Phase II)</td>
<td>Elevation rut &amp; transverse surface profiles (total station)</td>
<td>--</td>
<td>Pore pressure (200 kPa pressure transducer)</td>
<td>Permanent &amp; transient transverse strain &quot;on top and bottom&quot; (Micro Measurements strain gauge, 20% EP series)</td>
<td>28 strain gauges: 100% (construction); 32% (traffic); water infiltration</td>
<td>Data collected at 25 Hz.</td>
</tr>
<tr>
<td>12. Tang et al. (2015)</td>
<td>Elevation rut &amp; transverse surface profiles (wireless laser profilometer)</td>
<td>Permanent vertical deformation of the layer (potentiometer, Honeywell MLT)</td>
<td>Vertical stress at top (Geokon 3500 EPC)</td>
<td>Pore pressure (Geokon 3400 piezometer)</td>
<td>Transverse strain &quot;on top and bottom&quot; (Micro- Measurements strain gauges, 5% EA series for geogrids, 10% EP series for geotextiles)</td>
<td>Strain gauges: At least 20% damage</td>
</tr>
<tr>
<td>13. Watts et al. (2004)</td>
<td>Elevation rut (optical level)</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>Failure defined as 80mm surface deformation</td>
</tr>
<tr>
<td>14. Milligan et al. (1986)</td>
<td>Footing displacement (dial gauge)</td>
<td>Lateral profiles of surface displacement (graduated sleeved guided rod)</td>
<td>Permanent vertical &amp; lateral strains (Bison Inductive strain coil)</td>
<td>--</td>
<td>--</td>
<td>Failure criterion: footing penetration of 80mm</td>
</tr>
<tr>
<td>15. Leng and Gabr (2002)</td>
<td>Plate displacement (dial gauge)</td>
<td>--</td>
<td>Vertical stress at the top at varying distances from plate cente (r = 0, 152, 305 &amp; 457-mm) (EPC)</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
</tbody>
</table>
Table 2.7 (Continued)

<table>
<thead>
<tr>
<th>Study</th>
<th>Surface</th>
<th>Base</th>
<th>Subgrade</th>
<th>Geosynthetic</th>
<th>Survivability</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>16. Tingle and Jersey (2005)</td>
<td>Transient &amp; permanent plate displacement &amp; lateral profiles at offsets of 180, 457, &amp; 762 mm from plate center (LVDT)</td>
<td>–</td>
<td>Vertical stress under the plate center at the top &amp; near the bottom of the subgrade (Geokon EPC)</td>
<td>–</td>
<td>–</td>
<td>Dynamic data collected at a sampling rate of 500 Hz at selected intervals</td>
</tr>
<tr>
<td>17. Qian et al. (2013)</td>
<td>Plate displacement &amp; lateral profiles at offsets of 250, 500, &amp; 750 mm from plate center (displacement transducer)</td>
<td>–</td>
<td>Vertical stress at the top at varying distances from plate center (r = 0, 250, 500 &amp; 750 mm) (500 kPa strain gauge-type Tokyo Sokki EPCs, 50 mm diameter, 11.3 mm thick)</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>18. Palmeira and Gongora (2016)</td>
<td>Plate displacement &amp; lateral profiles at offsets of 0.5, 1, &amp; 1.5d from plate center (LVDT)</td>
<td>–</td>
<td>Vertical stress under the plate center at two different depths in the subgrade (EPC, 55 mm diameter, 6 mm thick)</td>
<td>–</td>
<td>–</td>
<td>Failure criterion: vertical plate displacement of 75 mm</td>
</tr>
<tr>
<td>19. Bauer and Abdelhalim (1987)</td>
<td>Plate displacement (Built-in/internal LVDT)</td>
<td>–</td>
<td>–</td>
<td>Strain (foil strain gauge)</td>
<td>–</td>
<td>Failure criterion: plate displacement of 28 mm</td>
</tr>
<tr>
<td>20. Milligan and Love (1984)</td>
<td>Footing displacement and lateral profiles of surface displacement (LVDT)</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>Failure criterion: footing displacement of 50mm (equivalent to 200 mm at full scale)</td>
</tr>
<tr>
<td>21. Abu-Farsakh et al. (2016)</td>
<td>Plate displacement (dial gauge)</td>
<td>–</td>
<td>Vertical stress distribution at the top (Geokon 4800 VW EPC, 102mm diameter)</td>
<td>Strain distribution in geosynthetics at interface, 5 gauges placed at 0.5d (plate diameter) distance apart, first gauge below the center (20% Micro-Measurements strain gauge, EP series)</td>
<td>–</td>
<td>Only limited data from geosynthetic foil strain gauges in one section was discussed</td>
</tr>
</tbody>
</table>

EPC = earth pressure cell; TDR = time-domain reflectometer; SDD = Single-depth deflectometer; d = plate diameter
2.4.2 Summary and Implications of Results from Previous Research

This section summarizes the main results and the important conclusions and findings from the reviewed studies and other relevant research work on flexible pavements. An attempt to synthesize the major findings from literature is also presented. Particular focus is given to the reinforcement benefits of geosynthetics and the variables and conditions that influence these benefits.

2.4.2.1 Benefits of Geosynthetic Reinforcement

Field and laboratory studies showed that the inclusion of geosynthetics in unpaved roads can improve the performance by extending the service life or saving base course thickness with the same performance. Extension of service life can be directly measured by a Traffic Benefit Ratio (TBR), defined as the number of equivalent single-axle load (ESAL) passes sustained by a reinforced section divided by that of an equivalent unreinforced section at a given level of rut. The base course reduction (BCR) or equivalency is defined as a percentage savings of the unreinforced base thickness.

Several studies have shown that geosynthetics can improve trafficability and extend the service life of unpaved roads (Fannin and Sigurdsson, 1996; Cuelho and Perkins, 2009; Tingle and Jersey, 2009; Palmeira and Gongora, 2016). Overall, reported TBR values ranged from 1 (no benefit; Watts et al., 2004) to 217 (Tingle and Jersey, 2009). For test sections containing geogrids, TBRs ranging from 1 to 161 were reported, while TBRs ranging from 1.8 to 36.5 were obtained for test sections containing geotextiles. TBRs ranging from 10 to 217 were obtained for test sections containing geogrid-geotextile composite.

It was also revealed that geogrids and geotextiles can reduce base course thickness for a given service life (Chaddock, 1988; Tingle and Webster, 2003; Watts et al., 2004). Test sections reinforced with geogrids produced a base course reduction of 3 to 35%. Base course savings of 16 to 26.9% were obtained with geotextiles, and up to 50% savings were obtained with geogrid-geotextile combinations.
Leng and Gabr (2002) showed that the geogrids improved the deformation and stress aspects as they resulted in decreased surface deformation, improved distribution of stresses transferred to the subgrade, and inhibited degradation of the base layer. Qian et al (2013) used triangular aperture geogrids and concluded that the geogrids reduced permanent deformation and maximum vertical stress at the interface and resulted in a more-uniform stress distribution compared with the unreinforced sections. Geosynthetics tested by Abu-Farsakh et al (2016) resulted in appreciable reduction of surface displacement and increase of bearing capacity.

However, some studies observed little or no improvement due to the geosynthetic inclusion. Dawson and Little (1997) reported that possible savings of base course by including geosynthetics are small and hence their benefit for the test sections constructed was, in this instance, marginal due to the firm condition at the subgrade surface.

2.4.2.2 Variables that Influence the Reinforcement Benefit of Geosynthetics

The benefits of geosynthetics in unpaved road construction are well-recognized by many researchers. However, the relative improvement in performance due to the inclusion of geosynthetics varies depending on structural considerations such as material properties, geometries and loading conditions (Sigurdsson, 1993). The review of previous studies indicated a range of performance from no improvement to a significant increase in service life or reduction in base course thickness. The variables or conditions believed to influence the reinforcement effect or benefit of geosynthetics and thereby control the performance of the reinforced test section have been discussed in the literature. Evaluation of the impact of an individual variable is difficult, however, due to the interfering effects of other variables and conditions in the test study under consideration.

i. Subgrade Strength and Base Course Layer Thickness:

The test results reported by Chaddock (1988) suggest that the influence of base thickness on the reinforcement effect of geogrid was affected by the strength of the subgrade. The geogrid clearly improved the performance for the stronger subgrades (with a relatively...
thin base), but there was little improvement for the very soft subgrade (with a thicker base) where subgrade intrusion was observed (reduces interlocking). Improvement by geogrids was observed to be less for increasing layer thickness and decreasing subgrade strength.

The results by Fannin and Sigurdsson (1996) showed that improvement was greatest on the thinner bases (250 mm and 300 mm) and diminished with increasing base layer thickness. The authors speculated that separation appears to be the primary function of the geosynthetic for thin bases over soft subgrades with the primary function shifting to reinforcement as the base thickness increased.

Unexpectedly, some of the thicker bases tested by Dawson and Little (1997) (> 350mm) showed higher amounts of rut than their thinner counterparts. The authors attribute this to the occurrence of maximum shear forces within the aggregate layer in the thick base layers while the geosynthetic was placed too low at the interface.

The full-scale sections tested by Hufenus et al. (2006) showed that improvement in bearing capacity and compactability by geosynthetics were observed only on a weak subgrade (CBR < 3%) and for thin bases (h ≤ 500 mm). The influence on the bearing capacity on stiffer and stronger subgrade and for thicker fill layers was marginal.

For geogrid-reinforced sections, Morris (2013) observed that increasing the subgrade CBR by 20% resulted in reduced rut depths by approximately 7.5% starting at +300 passes, while reducing the subgrade CBR by 8–9% resulted in more dramatic rut behavior and increased rut depths by approximately 54% at 300 passes.

Leng and Gabr (2002) concluded that the benefit from geogrids decreased as the thickness of base layer was increased from 152 mm to 254 mm. The 152-mm base showed more uniformly distributed vertical stress on the subgrade. Qian et al (2013) observed that the benefit of using a heavy-duty geogrid over a light-duty geogrid in the 150-mm-thick base is not that significant as that in the 230- or 300-mm-thick bases (majority of the permanent deformation for the tests with a thin base came from the subgrade). In their large-scale monotonic loading tests, Milligan et al. (1986) observed little or no improvement in bearing capacity for the thicker base layer as well as the weaker subgrade. Improvement was greatest with the thinner base layers and with the
stronger subgrade. Milligan and Love (1984) observed that the performance of unpaved systems incorporating a geogrid under monotonic loading clearly improved with increasing subgrade strength.

**ii. Base Material Type:**

Dawson and Little (1997) used two base course materials of different quality in their full-scale test sections. The main material was crushed diorite and the supplemental material was sand and gravel mixture. The results suggest that the type of base course did not appear to have much influence on the rate of rut development. However, it is noteworthy to mention that these results were based on one pair of unreinforced test sections (effect on reinforcement was not investigated).

Santoni et al. (2001) used four base course materials of different quality in their full-scale test sections. The main material was crushed limestone and the supplemental materials were sand, fiber-reinforced sand, and wood chips. The crushed limestone placed over geogrid and geotextile provided excellent performance.

Tingle and Jersey (2009) constructed geosynthetic-reinforced unpaved test sections with different base materials: crushed limestone (SW-SM, higher-quality), crushed chert aggregate (GW, more uniformly graded), and clay gravel base (GP-GM, marginal). The clay gravel base performed the best (mostly attributed to natural cementation, presumably because of its high plastic fines content; moisture susceptibility was not evaluated "would have performed worst"), followed by the crushed limestone, and then the crushed aggregate (because of low binder content). Performance differences between base materials were more distinguishable in the unreinforced bases. The clay gravel base has the best performance in the unreinforced condition. The effect of reinforcement inclusion (in terms of rutting resistance) was significant for high-quality bases (especially for the crushed limestone) but little or none for marginal base materials. Likewise, the results reported by Milligan et al. (1986) suggest that a geogrid would be much more effective in a less stiff granular material (better match with the weak subgrade).
iii. Geosynthetic Type and Structure:

Several studies examined whether geogrids or geotextiles provide more improvement to unpaved structure. The effect of the structure or the process used in manufacturing the geosynthetic and the polymer composition of the geosynthetic have also been investigated.

Austin and Coleman (1993) evaluated the effectiveness of various geosynthetics as the primary reinforcement in unpaved roads over very soft subgrades. Rut measurements performed in this study indicated that the extruded geogrid provided the best performance, followed by the woven geotextile then the sheet-punched geogrid.

Fannin and Sigurdsson (1996) suggested that the geotextile outperformed the geogrid for the thin base course layers (i.e., separation is more important). However, the geogrid outperformed the geotextile for the thicker base course layers (reinforcement is more important).

Tingle and Webster (2003) observed that more stabilization benefits were provided by the geogrid-geotextile combined section, while the woven and nonwoven geotextiles provided similar benefits. Hufenus et al. (2006) reported that stiff flat rib and extruded geogrids had greater effect on compactability than the geotextiles and the other geogrids used in the study (knitted and heat-welded).

The field study conducted by Cuelho and Perkins (2009) revealed that welded geogrids, woven geogrids and the stronger integrally-formed geogrids provided significantly more stabilization benefit than geotextiles and the weaker integrally-formed geogrid.

Tingle and Jersey (2009) installed an integrally-formed geogrid and a nonwoven geotextile at the interface of 150 mm-thick base layers. The results showed that the geogrid performed better than the geotextile, whereas the combined geotextile-geogrid section performed the best.

White et al. (2011) compared the performance of two geogrids with different aperture shapes and a woven geotextile placed at the base-subgrade interface. Based on compaction and rut depth measurements, the triaxial geogrid section performed better than test sections reinforced with biaxial geogrid and woven geotextile. Performance
improvement in the triaxial geogrid section was attributed to higher lateral restraint of the base layer (greater “locked-in” horizontal stress in the base) and reduced “locked-in” horizontal stresses in the subgrade under loading, compared to the other sections. However, a higher relative density was achieved in the triaxial geogrid section (98% versus 90% for the biaxial geogrid and control sections), which could have contributed to the improved performance. According to the authors, variations in relative density cannot be attributed to improvement in compaction due to the geogrid inclusion.

Morris (2013) constructed full-scale test sections to describe the transverse behavior of various geosynthetics. A woven geotextile performed the best, followed by an extruded biaxial geogrid, a vibratory welded geogrid and a nonwoven geotextile. The poorest performance was observed in a knitted, woven geogrid, followed by a triaxial geogrid. Watts et al. (2004) assessed the ability of various geosynthetics to reduce surface deformation of unpaved road. The results showed that sections incorporating integral geogrids (punched, sheet drawn geogrids and extruded geogrids) demonstrated by far the best performance over the woven, welded, and composite geogrids and the woven geotextile. Palmeira and Gongora (2016) evaluated the performance of various geosynthetics including commercial geogrids, laboratory assembled geogrids, and a woven geotextile under cyclic load tests. In general, the commercial geogrids showed better performance than the laboratory assembled geogrids. The performance of the woven geotextile was quite poor compared with that of most of the commercial geogrids. Finally, Abu-Farsakh et al (2016) evaluated the benefits of using various geogrids and geotextiles to stabilize unpaved test sections using monotonic loading. The geotextiles showed slightly more improvement in bearing capacity than geogrids. The triaxial geogrids and polypropylene biaxial geogrids performed better than the polyester biaxial geogrids.

**iv. Geosynthetic Layering and Location:**

The optimal placement location of geosynthetics within the base layer has been investigated by many researchers. It depends on the subgrade, the thickness of the base layer and the magnitude of the applied loads (Hufenus et al., 2006). The effectiveness of
using multiple geogrid layers in the pavement structure has also been examined. For soft subgrades and thin base layers, many studies suggested that the optimal placement location for geosynthetics is at the interface (Cancelli and Montanelli, 1999; Haas et al., 1988; Miura et al., 1990). For stronger subgrades and thicker bases, the placement location should be closer to the top of the base layer (Haas et al., 1988). Little (1993) also recommended that geosynthetic be placed higher in the unpaved structure for thicker bases (> 350 mm) to pick-up internal shear forces induced in the base layer. The author recommended that geosynthetic inclusions within the base layer should be considered if the design requires a thickness larger than a maximum value for singly reinforced pavements.

Tang et al. (2015) observed that placing a second layer of geogrid (in addition to a first layer at the interface) did not lead to significant improvement in performance. Watts et al. (2004) reported that a test section contained a second layer of geogrid at the mid-height of the base layer performed less than a section with only one geogrid at the interface. They attributed this result to the relatively small thickness of the base layer (300 mm). They suggested that thicker base layer (> 300 mm) should be used when incorporating a second layer of geogrid at the mid-height of the base layer.

Tingle and Jersey (2005) hypothesized that the maximum placement depth of the geogrid is a function of the shape of the induced horizontal stress bulb because the mechanism of lateral restraint is based on the reduction of horizontal stress applied to the subgrade surface.

Milligan et al. (1986) reported that the optimum location of the geogrid could be within the base layer. They observed that the geogrid was more effective when placed at a depth of about 160 mm below the surface (approximately at mid-height of the base) than at the interface. Abu-Farsakh et al (2016) noted that, under static loading conditions, the upper one-third location yielded the highest improvements for a single layer of geogrid. Placing two layers of geogrid consistently yielded the greatest improvement. Al-Qadi et al. (2012) examined the effectiveness of geogrid and identify the optimal locations for its placement. The study concluded that for thick-base layers, a single geogrid layer placed
in the upper one-third of the base layer would improve the performance; the addition of a geosynthetic layer at the subgrade–base interface may be needed for stability.

v. Strain Required to Mobilize Geosynthetic Reinforcing Action:

The majority of reviewed studies on geosynthetic-reinforced unpaved roads indicated that some level of surface deformation is required before the reinforcement effects of the geosynthetic can be realized (Perkins and Ismeik, 1997).

McCartney and Cox (2013) proposed a new experimental methodology to assess the influence of strain level on the geosynthetic contribution to resisting surface deformation. Data collected in the study indicated that increasing surface deformation is necessary to mobilize the geosynthetic contribution in road layers with an increasing depth of geosynthetic placement from the surface. The review of literature conducted in this study indicated that this behavior appears to hold for larger base layer thicknesses \( h >> 250 \) mm, where geosynthetics placed at the interface are typically located at greater depths, resulting in less reinforcement effects from the geosynthetics (i.e., higher surface rutting is needed to mobilize the reinforcement effect of geosynthetics as the base thickness increases). The data reported by Tingle and Jersey (2005) indicated that the maximum placement depth of the geosynthetic depends on the shape of the induced horizontal stress bulb. The authors observed that the horizontal stress approaches zero at a depth of 400 mm below the surface.

For thin base layers \( 150 \) mm \( \leq h \leq 250 \) mm), Leng and Gabr (2002) reported that rut needed to mobilize reinforcement effects increased with further decrease in the base layer thickness below 250 mm. For thinner base layers, geosynthetics are typically located within the horizontal stress zone, so the influence of the geosynthetic placement depth is less significant. However, both reinforced and unreinforced sections will generally experience greater magnitudes of surface deformation as the base layer thickness decreases further. Therefore, for shallow base layers, the smaller the base layer thicknesses \( h << 250 \) mm), the greater the overall magnitude of surface deformation in the unpaved system would be and the greater the surface deformation level at which a
performance difference could be observed between reinforced and unreinforced layers (surface deformation needed to mobilize the reinforcement effect of geosynthetics).

In addition, for the range of relatively thin base thicknesses used in the study by Leng and Gabr (2002), rut needed to mobilize reinforcement effects appeared to be slightly higher for geosynthetics with lower stiffness. They concluded that geosynthetic stiffness plays a greater role in thin base layers because it correlates to reducing the surface deformation, which influence the mobilization of reinforcement effects of geosynthetics in thin bases as previously discussed.

vi. Geosynthetic Properties:

The relevant properties of geosynthetics in paved and unpaved road applications have been examined in previous research studies. In particular, a variety of geogrid types are being used, which resulted in a wide variety of properties and modes of interaction with road layers (Giroud, 2009). Although much of the experimental work has shown that the geosynthetics improve rutting performance, there does not seem to be a consistent relationship between improved performance and any of the geosynthetic properties such as tensile stiffness or junction strength (Cuelho et al., 2014).

Austin and Coleman (1993) suggested that performance of geogrid-reinforced roads is independent of the tensile strength and the structure of the products. Data from their tests was insufficient to support a correlation between the geosynthetic tensile strength and the performance.

Fannin and Sigurdsson (1996) correlated improved trafficability to the tensile stiffness of the geotextile. Several studies reported similar conclusions that performance improvement due to geogrid inclusion was more pronounced for the higher modulus geogrid (Leng and Gabr, (2002); Qian et al. 2013)). Morris (2013) conducted field trafficking on unpaved test sections with thick base layers (275 mm) to provide insight into which material properties are most relevant to field performance. The data indicated that junction strength/stiffness and tensile strength (at 2 and 5% strain, in the cross-machine direction) correlated well with performance. Cuelho and Perkins (2009) tested
thinner base layers (200 mm) and concluded that the benefit from the geogrids is likely directly related to the geogrid tensile strength at 2% strain in the cross-machine direction (and to a lesser extent at 5% strain).

Hufenus et al. (2006) correlated improvement in bearing capacity and compactability with the tensile stiffness of the geogrid at 2% strain. They concluded that the achievable degree of reinforcement depends on the stiffness of the geosynthetic and that it is not necessary to use extremely stiff geosynthetic because tensions of only 6–10 kN/m were mobilized for base thicknesses between 200 and 500 mm. Abu-Farsakh et al (2016) concluded that higher tensile modulus geosynthetic usually yields a higher stress distribution angle and an increased bearing capacity ratio and reloading elastic modulus.

Dawson and Little (1997) concluded that geosynthetic properties required for improved performance are not limited to stiffness, but includes also creep resistance. Palmeira and Gongora (2016) suggested that the performance of a given geogrid was markedly influenced by a combination of its mechanical and physical properties such as tensile stiffness, aperture–aggregate particle diameter ratio, rib thickness, and fraction of geogrid area available for bearing. No correlation was observed between geogrid aperture stability modulus and unpaved system performance.

2.5 **Summary of Gaps in the Current Knowledge of Geogrid-reinforced Unpaved Roads**

Geogrids have been commonly used as reinforcement materials to improve the performance of unpaved roads. Laboratory and field studies have been conducted to evaluate the performance improvement of the unpaved structure due to the inclusion of geogrids. Based on review of literature, gaps in the current knowledge of reinforced unpaved roads are identified as follows:

The majority of the experimental work on the use of geogrids in road applications has been performed at reduced-scale in the laboratory. Despite the good quality control achieved in laboratory testing, the actual traffic loading conditions cannot be replicated appropriately in most laboratory settings. In addition, reduced-scale tests cannot
adequately model the interaction between geogrids and aggregate which occurs over the entire width of the road. Therefore, conclusions from laboratory studies cannot be reliably adopted in practice (Hufenus et al. 2006), and additional field testing is needed.

Inconsistencies exist between studies on the amount and type of benefits to the roadway due to the inclusion of geogrids. While many studies showed significant improvement of the road performance (Cuelho et al., 2009; Fannin and Sigurdsson, 1996; Al-Qadi and Bhutta, 1999), others have shown little benefit from including the geogrids (Cox et al., 2010; Henry et al., 2011). These discrepancies indicate that more variables pertinent to the behavior of the reinforced structure, such as subgrade strength and base layer thickness, need to be investigated to understand the conditions under which the geogrids will be cost-effective as roadway reinforcements.

The strain behavior of geogrids under traffic loads has not received much attention in the literature. Measurement of geogrid strain in a full-scale field testing is needed to investigate the nature and distribution of strains in the geogrid reinforcement. There is also disagreement between the reviewed studies as to which geogrid properties are most pertinent in the application of road reinforcement. Additional work is needed to identify the geogrid parameters that are critical to their effectiveness.

The mechanisms through which the geogrids improve the performance of unpaved roads haven’t yet been fully understood. Early work on geosynthetic reinforcement of unpaved roads suggested that the principal reinforcing mechanism is the tensioned membrane effect, which requires a large amount of rut to develop. Recent work, more related to flexible pavements, presented other mechanisms for conditions of moderate rut depths. This disagreement resulted in the development of two different design approaches; one is based on large deformations (Giroud and Noiray 1981), and the other considers small strains (Houlsby and Jewell 1990; Douglas 1993). A better understanding of the reinforcing role of geosynthetics will help to develop a more generic design approach.

The traffic benefit ratio (TBR) and base course reduction (BCR) factor are commonly used in design procedures. Since the mechanism in which the geosynthetics affect these benefits is not fully understood, TBR and BCR values are considered product-specific (Berg et al., 2000). New geogrid products and manufacturing procedures are being
regularly introduced to the market. Assessment of suitability of these products for the application of road reinforcement is critical to updating the specifications required for industrial and design purposes so that to encompass all suitable products.

Few researchers have attempted to instrument road sections reinforced with geogrids (Brandon et al. 1996; Warren and Howard 2007), but limited results have been provided. Besides, most of these studies were conducted on flexible pavements. Instrumentation program in full-scale test sections is necessary in order to understand the behavior of the reinforced unpaved structure. In addition, lack of proper instrumentation in some of the instrumented studies has influenced the amount of benefit gained from their results. Rigorous sensor selection and installation techniques must be followed in order to achieve decent survivability and collect valuable data.

Further research is needed to investigate the strain behavior of geogrids under traffic loads, evaluate the reinforcement mechanisms of geogrids in field testing conditions using mechanical measurements from unpaved test sections reinforced with different geogrids. The influence of important parameters such as base layer thickness, geogrid reinforcement type and stiffness, and traffic loading level and magnitude on the response of the unpaved structure need to be further investigated using full-scale field tests. Furthermore, geogrid products currently available in the market with little or no performance data need to be tested under actual traffic conditions in the field to expand the existing database in literature related to the performance of geogrid reinforcements in roadways.
Chapter 3

3 FIELD EXPERIMENTAL SETUP

3.1 Test Site and Material Properties

3.1.1 Site Location

Several sites were considered as possible locations for the test road. Eventually, an unpaved parking lot inside the Blue-Con Construction Company’s site was selected, which is located 9.3 km northeast of the city of London, Ontario, near the London International Airport. Figure 3.1 displays a plan view of the research site prior to construction. It is located approximately 300 m northwest of the intersection of Oxford Street East and Crumlin Road. The Blue-Con site provides a realistic setup for the field testing program. The site was selected based on the following factors:

- The site provides an adequate area to allow for the construction of ten individual test sections of at least 14 m long each.
- The subgrade at the site has a low California Bearing Ratio (CBR) value.
- Proximity to the Western University campus to facilitate the commute of the research team to and from the site during construction and testing.
- Availability of the site to start construction and testing activities once preliminary preparation is completed.
- Blue-Con was the primary contractor for the construction of the test road. Cooperation from the contractor (the owner of the test site) to allow for downtime during construction to install the instruments.
- Unlike the other sites, the Blue-Con site offers a completely traffic free space during the entire research period, which provides the needed flexibility for scheduling instrument installation and road construction, and the ability to control field trafficking of the test sections.
- The site features a low risk of vandalism.
3.1.2 Subgrade Soil Characterization

A preliminary site investigation was conducted in order to establish the subgrade soil stratigraphy and to determine the engineering properties of the subgrade soil and their variability with depth and along the road. The site investigation program included borehole drilling, soil sampling, and field and laboratory testing.

3.1.2.1 Subsurface Exploration and In-Situ Testing

A series of 10 boreholes were drilled. The boreholes were terminated at a depth of 3 m below grade. Boring was performed by Aardvark Drilling Inc. using a CME 55 truck mounted drill rig. Hollow-stem continuous flight augers were used to drill the boreholes.
One borehole was drilled at the center of each test section of the proposed road as will be described in the next section. Figure 3.2 illustrates the locations of boreholes in the test site. Borehole logs are presented in Appendix A.

**Sampling:** Samples collected from the site included disturbed split-spoon and auger samples as well as undisturbed Shelby tube samples. *Split-spoon samples* were obtained from the entire depth of the borings. The split-spoon samples were used for subgrade soil index testing and classification purposes. In order to avoid moisture loss, the water content of the samples was determined at the earliest opportunity. Collected *auger samples* were also used for identification purposes and for performing CBR tests on reconstituted specimens. *Undisturbed subgrade soil samples* were retrieved from the first 2 m below grade level for use in consolidated drained (CD) triaxial testing. Thin-walled sampling tubes with 76 mm outside diameter were used to obtain the soil samples. Once a soil sample was recovered, the ends of the tube were sealed with sealing wax, covered with plastic caps and wrapped using a duct tape to prevent moisture loss. The tubes were then transferred to the laboratory and stored in a cooled, high humidity room awaiting testing.

**Standard Penetration Tests:** The hollow-stem auger used to drill the Boreholes had an inside diameter of 152.4 mm. After the desired depth was reached in the borehole, a Standard Penetration Test (SPT) was performed in accordance with ASTM D1586 by driving a 50.8 mm split-spoon sampler using a 63.5 kg hammer connected via steel rods. The hammer was allowed to fall freely from a height of 760 mm, and the process was repeated until the sample penetrated a depth of 450 mm. The number of blows was recorded for each 150 mm interval. A profile of the SPT blow count for the test site is shown in Figure 3.3.

**Soil Stratigraphy:** Based on examination of the samples retrieved from the boreholes, the soils at the research site consisted mainly of loose to medium-dense light brown and gray sand and silt, with a trace of gravel topped by a surface layer of fill material. The upper fill layer (approximately 1 m thick) was excavated to the grade level prior to borehole drilling and construction. The water table at the research site varied around 0.9 m below grade level.
Density Tests: The in-situ density of the subgrade soil was measured using the sand-cone test method. The sand-cone test is an accurate and reliable test method that has long been used to measure the in-place density of soils. The tests were performed in accordance with ASTM D1556. The in-situ dry unit weight and natural moisture content of the subgrade soil ranged from 16.7 to 17.4 kN/m³ and 10 to 14%, respectively.

Figure 3.2: Location of boreholes
3.1.2.2 Laboratory Testing

Several laboratory tests were carried out on disturbed and undisturbed samples retrieved from different depths of the boreholes. Material properties of the subgrade soil were determined from index and physical property tests. Strength characteristics of the subgrade soil were also evaluated. Tests performed included:

- Moisture Content (ASTM D2216)
- Laboratory Determination of Density (ASTM D7263)
- Particle Size Distribution (ASTM D422)
- Atterberg Limits (ASTM D4318)
- Classification of Soils for Engineering Purposes (ASTM D2487)
- California Bearing Ratio (ASTM D1883) [Reconstituted Samples]
- Consolidated Drained Triaxial Compression Test (ASTM D7181)
**Moisture content:** Moisture content of the samples was determined in accordance with ASTM D 2216. The samples were transported from the site to the laboratory in zipped plastic bags to avoid moisture loss. The test results showed that the moisture content of the tested soil samples varied between 9 and 15%.

**Particle Size Distribution:** Nine particle size distribution tests were performed on the subgrade soil in accordance with ASTM D422. The particle size distribution was determined from split-spoon soil samples representing the soil profile. The laser diffraction method was used to obtain the particle size distribution of the fine-grained fraction of the subgrade soil passing the No. 200 (75-μm) sieve. Figure 3.4 shows the range of particle size distributions for the subgrade soil. Average percentages of gravel, sand, silt, and clay-size particles present in the tested soil samples were 3, 60, 32 and 5%, respectively. The results of the particle size analysis are presented in Appendix A.

![Grading curve limits](image)

**Figure 3.4: Particle size distribution for the subgrade soil**

**Atterberg Limits:** Atterberg limit tests were performed on eight samples of the subgrade soil in accordance with ASTM D4318. Liquid and plastic limits were determined for the split-spoon samples. The results indicated a liquid limit in the range of 17% to 21.5%, and a plasticity index in the range of 3.8% to 6%.
Soil Classification: The subgrade soil below grade was reasonably uniform across the research site. It was found that approximately 32 to 40% by dry mass of the test specimens used for particle size analysis passed No. 200 sieve (i.e., between 12% and 50%) and that more than 50% of the coarse fraction (plus No. 200 sieve) passed No. 4 (4.75-mm) sieve. The fines plot as CL-ML in the plasticity chart (Figure 3.5). Accordingly, the subgrade soil is classified as a silty, clayey sand (SC-SM) according to the Unified Soil Classification System (ASTM D 2487).

![Plasticity chart](image)

**Figure 3.5: Plasticity chart**

California Bearing Ratio: California Bearing Ratio (CBR) tests were performed on laboratory-compacted subgrade soil specimens in accordance with ASTM D 1883. Average values of the previously determined in-situ water content and dry density were used to obtain the CBR value of the subgrade soil. CBR test specimens were compacted at the average in-situ moisture content using different compaction efforts to achieve dry densities below and above the average in-situ dry density. The CBR value for the desired in-situ dry density is then interpolated from a plot of CBR values against molded dry density. CBR value of approximately 2.3% was obtained for the subgrade soil.
**Triaxial Testing:** The shear strength parameters of the subgrade soil were determined from undisturbed soil samples recovered from the boreholes using thin-walled Shelby tubes. Consolidated drained (CD) triaxial compression tests were performed on saturated soil specimens in accordance with ASTM D7181. A fully automated GDS Triaxial Testing System was employed in conducting the tests. The system is capable of achieving a maximum cell and back pressure of 1 MPa by means of pneumatic and hydraulic controllers, respectively. Stresses as low as 0.1 kPa and sample volume changes of 1 mm$^3$ can be measured by the system. Cell pressure, back pressure and pore-pressure are measured using three pressure transducers of 1 MPa capacity. The accuracy of measurements of these pressure transducers is 0.25% of their full range. The system is equipped with a submersible load cell, with a capacity of 4 kN and resolution of 1 N, to monitor the axial load. The load cell has a measurement accuracy of 0.1% of its full range.

The tests were performed on 50 mm diameter specimens. The specimens were first back-pressure saturated and were next isotropically consolidated at confining pressures of 25, 50, 100, and 200 kPa. Drained shearing was subsequently conducted on the specimens in a very slow rate (0.1% mm/min) in order to allow a full dissipation of any developed excess pore pressure. Shearing continued until the test reached 20% strain. Figure 3.6 displays a test specimen installed in the GDS triaxial machine ready for testing. The stress-strain plots obtained from the CD triaxial tests are shown in Figure 3.7. The shape of the stress-strain plots in Figure 3.7 indicates hardening behavior. Specimens reached critical state at an axial strain of about 15%.

Examination of the specimens after shearing indicated that the tested specimens displayed distinct failure planes during shearing with an average inclination angle, $\alpha$, of approximately 61°. The angle of inclination of failure plane is related to the effective friction angle, $\phi'$, by (Das, 2010):

$$\alpha = 45 + \frac{\phi'}{2}$$  \hspace{1cm} (3-1)

Therefore, the effective friction angle can be estimated as:

$$\phi' = 2 \times (61 - 45) = 32°$$
Figure 3.6: A subgrade soil specimen ready for testing in a GDS triaxial machine

Figure 3.7: Plots of deviator stress vs. axial strain from CD triaxial tests on subgrade soil specimens
A small amount of bulging (barreling) was observed in the tested specimens. The bulging of the triaxial specimen occurs due to end friction and it indicates that the portions of the specimen near the ends are not displacing tangentially (laterally) as much as the part near the middle.

The critical state line (CSL) from the CD triaxial tests is plotted in $q - p'$ space (deviator stress and mean effective stress) as shown in Figure 3.8. The specimens exhibited higher strength as confining pressure increased, and the stress paths for the various confining pressures reached the same critical state line. For triaxial compression tests, the effective friction angle is related to the slope, $M$, of critical state line as follows (Muir Wood, 1990):

$$ M = \frac{6 \sin \phi'}{3 - \sin \phi'} $$  \hspace{1cm} (3-2)

From Figure 3.8 we get:

$$ M = \frac{\Delta q_{cs}}{\Delta p'_{cs}} = \frac{528 - 0}{400 - 0} \approx 1.32 $$

$$ \therefore \phi' = \sin^{-1} \left( \frac{3M}{6 + M} \right) = \sin^{-1} \left( \frac{3 \times 1.32}{6 + 1.32} \right) \approx 33^\circ $$

This value of $\phi'$ compares relatively well with that estimated from the inclination of the failure plane. Accordingly, the angle of internal friction for the subgrade soil was determined to be $33^\circ$ and the soil had no cohesion.
Figure 3.8: Critical state line and stress paths from CD triaxial tests on subgrade soil specimens (q – p’ space)

3.1.3 Base Course Material Properties

The base course material used in the construction of the test sections was Granular B type I aggregate as per the Ontario Provincial Standard Specification (OPSS 1010). The material was produced by crushing quarried bedrock.

Sampling: Base course aggregate samples were collected during construction of the test sections in accordance with OPSS 1010 for subsequent laboratory testing in order to determine the material properties of the aggregate. Samples were collected on a time basis during aggregate placement rather than on test section basis since base course loads were placed on multiple test sections at once as it is being supplied to the site. Samples were taken from each truck-load of base course material to ensure representative sampling.
**Particle Size Distribution and classification:** Sieve analysis was conducted on the base course aggregate samples in accordance with ASTM C136 and LS-602 (Ontario Ministry of Transportation). The results of the particle size analysis of the base course material are plotted in Figure 3.9. The aggregate had a maximum particle size of 50 mm. The results of the sieve analysis indicated that more than 50% of the coarse fraction [plus No. 200 sieve] was retained on the No. 4 (4.75-mm) sieve. The base course material was classified as GW-GM (well-graded gravel with silt and sand) according to the Unified Soil Classification System (ASTM D 2487).

![Particle size distribution](image)

**Figure 3.9: Particle size distribution for the base course material**

**Laboratory Compaction Characteristics:** The relationship between molding moisture content and dry unit weight of the base course aggregate (compaction curve) was determined using Standard Proctor test, according to ASTM D698. A standard compaction effort of 600 kN-m/m³ was applied to laboratory specimens to determine the characteristics of the base course material. The results of the compaction tests are shown in Figure 3.10. The measured maximum dry unit weight and the corresponding optimum moisture content for the portion of the total aggregate specimen used in performing the compaction test (finer fraction) were 20.7 kN/m³ and 8.6%, respectively. The optimum moisture content and maximum dry unit weight of the base course material were
corrected for oversize particles using ASTM D 4718. For an oversize fraction of 18.1% by weight, the corrected optimum moisture content and maximum dry unit weight of the total base course material were found to be 8.2% and 21.7 kN/m³, respectively.

![Graph showing 100% saturation curves for specific gravities of 2.8, 2.7, and 2.6](image)

**Figure 3.10:** Dry density versus moisture content (compaction curve) for the base course material

*In-situ Density Measurements During Construction:* After placement and compaction of the base course material in the field, nuclear density tests were conducted on the base layer to measure its in-situ dry density and moisture content and thereby predict the adequacy of construction compaction efforts. The tests were performed in accordance with ASTM D6938 using a Troxler nuclear moisture-density gauge Model No. 3411-B. The gauge was operated in the direct transmission mode, where the radioactive particles pass through the full distance from the source (located 152 mm below the road surface) to
the detectors in the gauge base (at the road surface). The number of radioactive particles detected by the detectors is converted into a density reading. The moisture content is measured at the same time using a separate source of radioactive particles. Dry unit weights measured in the field using the nuclear density gauge ranged from 20.4 to 21.5 kN/m³, with an average dry unit weight of 20.9 kN/m³. Field moisture contents for the base course ranged from 3% to 4.5%, with an average moisture content of 3.6%.

Sand-cone tests were performed on the compacted base course in accordance with ASTM D1556. The measured in-situ dry unit weight and moisture content of the compacted base layer ranged from 20.6 to 20.9 kN/m³ and 3 to 6%, respectively. These values are in general agreement with those obtained by the nuclear density gauge.

*California Bearing Ratio*: CBR tests were performed on laboratory-compacted base course material specimens in accordance with ASTM D 1883. Average values of in-situ moisture content and dry density determined from nuclear density and sand-cone testes were used to obtain the CBR value of the base course material. CBR test specimens were compacted at a moisture content equal to the average in-situ value using different compaction efforts to achieve dry densities below and above the desired in-situ dry density. The CBR value corresponding to the average in-situ dry density of the base course was approximately 60%.

### 3.1.4 Properties of Geosynthetics

Four geogrid products were used in this study to evaluate their relative performance: three biaxial geogrids and one triangular aperture geogrid. The three biaxial geogrids are designated herein as G1 or light-duty geogrid, G2 or medium-duty geogrid, and G3 or heavy-duty geogrid, while the triangular aperture geogrid is designated as G4. Additionally, one geotextile was used as a separation layer and herein designated as GT. The average values of physical and mechanical properties of the geosynthetic products used in this study as reported by the manufacturers are listed in Table 3.1 and Table 3.2, respectively. Where applicable, properties are listed in both machine direction (MD) and cross-machine direction (XD) of the geosynthetics.
The biaxial geogrids G1, G2, and G3 were manufactured by Terrafix Geosynthetics Inc. and marketed as TBX1500, TBX 2000, and TBX3000. The three products were made of virgin polypropylene (PP) using the same manufacturing process. The manufacturing method consists of extruding a flat sheet of polypropylene, punching a controlled pattern of holes (square apertures), and stretching the sheet in both directions, orienting the ribs. The only difference between these geogrids is the thicknesses of the original sheets used to manufacture them. Accordingly, the end products of this manufacturing process differ essentially in the level of robustness, unit weight, rib thickness, and mechanical properties.

The triangular aperture geogrid G4 was manufactured by Tensar Corporation and marketed as TriAx TX160. It was also manufactured from a punched polypropylene sheet. However, the sheet was oriented in three substantially equilateral directions so that the resulting product should have equilateral triangular shaped openings. It should be noted that although the biaxial geogrids and the triangular aperture geogrid appeared to have approximately equal aperture sizes (in terms of aperture dimensions reported in Table 3.1), they had quite different aperture areas (approximately 1,521 mm$^2$ for the biaxial geogrids compared to only 693 mm$^2$ for the triangular aperture geogrid). The separator used was a needle-punched light weight non-woven geotextile, commonly used for separation applications. It was manufactured by Terrafix Geosynthetics Inc. and marketed as 270R. It was composed of polypropylene yarns (PPY) with an apparent opening size of 0.212 mm.

As can be seen in Table 3.1, the aperture size of all of the geogrids was around 40 mm. Given that most of the base course aggregates pass the 37.5-mm sieve, it is essential that the geogrid interact with the granular base material through interlocking between the geogrid and the aggregate particles. Interlocking depends on several factors, such as geogrid aperture size relative to aggregate particle size, geogrid aperture shape and geogrid mechanical properties. In order to ensure effective interlocking between geogrids and aggregate particles, geogrids should be selected such that the geogrid aperture to aggregate particle size ratio is close to an optimum value (Brown et al., 2007; Holtz et al., 2008; Palmeira and Góngora, 2016; Sakleshpur et al., 2019; Goud and Umashankar, 2018; Suddeepong et al., 2020). All geogrids used in this study met the aperture size
requirements outlined in Holtz et al. (2008) for stabilization specifications; that is geogrid apertures must be between 12.5 and 75 mm (0.5 and 3 in), larger than the average particle size \(D_{50} = 3\) mm, and smaller than twice the particle size corresponding to 85% finer \(D_{85} = 23\) mm.
### Table 3.1: Physical properties of geosynthetics

<table>
<thead>
<tr>
<th>Designation</th>
<th>Geosynthetic type</th>
<th>Structure</th>
<th>Polymer composition</th>
<th>Aperture shape</th>
<th>Roll width (m)</th>
<th>Aperture dimensions (mm)</th>
<th>Average rib width (mm)</th>
<th>Average rib thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>Biaxial geogrid</td>
<td>PSD</td>
<td>PP</td>
<td>square</td>
<td>3.95</td>
<td>39 x 39</td>
<td>3.0</td>
<td>1.0</td>
</tr>
<tr>
<td>G2</td>
<td>Biaxial geogrid</td>
<td>PSD</td>
<td>PP</td>
<td>square</td>
<td>3.95</td>
<td>39 x 39</td>
<td>3.3</td>
<td>1.3</td>
</tr>
<tr>
<td>G3</td>
<td>Biaxial geogrid</td>
<td>PSD</td>
<td>PP</td>
<td>square</td>
<td>3.95</td>
<td>39 x 39</td>
<td>4.0</td>
<td>2.0</td>
</tr>
<tr>
<td>G4</td>
<td>Triangular aperture geogrid</td>
<td>PSD</td>
<td>PP</td>
<td>triangular</td>
<td>4.0</td>
<td>40&lt;sup&gt;a&lt;/sup&gt;</td>
<td>1.0 x 1.2</td>
<td>1.6 x 1.4</td>
</tr>
<tr>
<td>GT</td>
<td>Geotextile</td>
<td>NW-NP</td>
<td>PPY</td>
<td>-</td>
<td>4.57</td>
<td>0.212&lt;sup&gt;b&lt;/sup&gt;</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

<sup>a</sup> Reported as “rib pitch” in the manufacturer’s specification sheet

<sup>b</sup> Apparent Opening Size (A.O.S), ASTM D4751

PSD = Punched, sheet drawn (integrally-formed geogrids)
NW-NP = Nonwoven, needle-punched

### Table 3.2: Mechanical properties of geosynthetics

<table>
<thead>
<tr>
<th>Designation</th>
<th>Radial stiffness (kN/m @ 0.5% strain)</th>
<th>Aperture stability (kg-cm/deg)</th>
<th>Junction efficiency (%)</th>
<th>Junction strength (kN/m) MD</th>
<th>Flexural stiffness (g-cm) MD</th>
<th>Tensile strength (kN/m)&lt;sup&gt;a&lt;/sup&gt; @ 2% MD</th>
<th>Tensile strength (kN/m)&lt;sup&gt;a&lt;/sup&gt; @ 5% MD</th>
<th>Tensile strength (kN/m)&lt;sup&gt;a&lt;/sup&gt; Ultimate MD</th>
<th>Tensile strength (kN/m)&lt;sup&gt;a&lt;/sup&gt; Ultimate XD</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>209.9</td>
<td>2.4</td>
<td>-</td>
<td>15.5</td>
<td>6.5</td>
<td>16.0</td>
<td>11.5</td>
<td>12.5</td>
<td>16.0</td>
</tr>
<tr>
<td>G2</td>
<td>284.9</td>
<td>3.4</td>
<td>-</td>
<td>19.0</td>
<td>8.0</td>
<td>19.0</td>
<td>14.0</td>
<td>14.0</td>
<td>19.0</td>
</tr>
<tr>
<td>G3</td>
<td>384.9</td>
<td>5.7</td>
<td>93</td>
<td>27.9</td>
<td>12.0</td>
<td>27.9</td>
<td>21.6</td>
<td>22.0</td>
<td>30.0</td>
</tr>
<tr>
<td>G4</td>
<td>300</td>
<td>3.6</td>
<td>93</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>GT</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>445&lt;sup&gt;b&lt;/sup&gt;</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

<sup>a</sup> MD = Machine direction, XD = Cross-machine direction

<sup>Grab tensile strength in Newtons</sup>
3.2 Description of Field Testing

Ten unpaved test sections were constructed at the research site and trafficked to investigate the benefits of geogrid inclusion and collect response data from controlled traffic loads in a full-scale field test. Test variables included geogrid aperture shape, geogrid level of robustness (i.e., tensile modulus), and thickness of the base course layer, \( h \). The test area consisted of two road lanes of five sections that run side by side. Each section was 14 m long and 4 m wide. Two different base course thicknesses were evaluated (one for each lane), 200 and 250 mm. For each base course thickness, four test sections were reinforced with different geogrids while one test section was unreinforced in order to evaluate performance for each base course thickness. The geogrids used were previously described in Section 3.1.4. Table 3.3 summarizes the design details of the test sections. The general layout of the test sections and the support infrastructure are depicted in Figure 3.11. Construction of the test sections and the support infrastructure will be discussed in Chapter 4.

The geogrids were placed at the subgrade-base course interface on top of a non-woven geotextile separator. The optimal location for installing a geogrid in a road system has been studied by many researchers (Abu-Farsakh et al., 2016; Durga Prasad et al., 2016; Mousavi et al., 2017). Placing the geogrid at the subgrade-base course interface is commonly accepted in the current state of practice. Geogrids placed at the interface could provide some degree of separation and lateral restraint of aggregate and they could reduce vertical deformation through decreasing of shear stress at the geogrid-subgrade interface.

Measurements of rut depth and surface deformation were taken to analyze and evaluate the respective performance of each test section. Additionally, the test sections were instrumented for measuring road response to traffic loading so that the stabilizing behavior of the geogrids can be evaluated through analyzing the mechanical response and the field performance of the unpaved system. The instrumentation used in the field testing program are described in the following section.
Table 3.3: Details of test sections

<table>
<thead>
<tr>
<th>Section</th>
<th>Reinforcement type</th>
<th>Base course thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>G4, triangular aperture geogrid</td>
<td>200</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>250</td>
</tr>
<tr>
<td>S3</td>
<td>G1, biaxial geogrid</td>
<td>200</td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td>250</td>
</tr>
<tr>
<td>S5</td>
<td>G3, biaxial geogrid</td>
<td>200</td>
</tr>
<tr>
<td>S6</td>
<td></td>
<td>250</td>
</tr>
<tr>
<td>S7</td>
<td>Control</td>
<td>200</td>
</tr>
<tr>
<td>S8</td>
<td></td>
<td>250</td>
</tr>
<tr>
<td>S9</td>
<td>G2, biaxial geogrid</td>
<td>200</td>
</tr>
<tr>
<td>S10</td>
<td></td>
<td>250</td>
</tr>
</tbody>
</table>

Note: All sections contain non-woven geotextile separator at the subgrade-base course interface

Figure 3.11: Layout of the test sections
3.3 Instrumentation

The test sections were instrumented with different sensors to understand their field performance as well as the response of the unpaved system to traffic and environmental loads. The data collected from the instruments allowed for the description of reinforcement mechanisms. A total of 38 instruments were embedded in the full-scale test sections. The instrumentation plan was designed to measure several variables that are critical to the performance of the unpaved road system. Vertical stresses at the top of the subgrade layer were measured using earth pressure cells to provide an insight into the dynamic vertical stress transferred to the subgrade during traffic loading as well as into any “residual” or “locked-in” vertical stresses that may remain in the soil after the traffic load is removed. Correlation between the dynamic vertical stresses transferred to the subgrade and surface deformations generated at these stress levels can also be investigated. Foil strain gauges were attached to the geogrids in the machine direction (direction of traffic or longitudinal direction) and in the cross-machine direction (direction perpendicular to traffic or transverse direction) to measure dynamic and permanent geogrid strains and provide some insight into the strain distribution in the geogrid under traffic loads. The ambient temperature near the surface of the subgrade (and the geogrid) was also monitored during the field testing using the built-in thermistors in the pressure cells to help separate spurious effects of temperature from real measurements. Motion sensors were installed at the road sides to provide a traffic count. The following sections detail the instrumentation used to measure the response of the unpaved test sections to traffic and environment loads.

3.3.1 Measurement of Subgrade Vertical Stress

3.3.1.1 General Purpose

Field measurement of stresses at various locations in an unpaved system aids in understanding the complex behavior of such a composite system. In addition, it can verify validity of design approaches and can be used to develop better models to describe the response of the system under traffic loads. In particular, determination of the vertical
stress transferred to the subgrade is used to evaluate rutting performance and to examine the role of geogrid reinforcement in reducing the magnitude of vertical stresses transferred to the subgrade (i.e., improvement of stress distribution).

### 3.3.1.2 Description of Earth Pressure Cells Used

The pressure cell used in this study was a Geokon Model 3500-2 (Figure 3.12) hydraulic cell. It consists of two circular stainless steel plates welded together around their periphery and the gap between the plates is filled with hydraulic fluid. The earth pressure applied on the plates is transferred to the fluid. A pressure transducer connected to the fluid converts the fluid pressure to an electrical signal and send it to the data acquisition system. The cell uses a semi-conductor, strain gauge type pressure transducer that is capable of measuring dynamic pressure up to 400 kPa with 0-5 VDC output. This cell features a high relative stiffness with respect to the soil and a high aspect ratio (i.e., ratio of the diameter of the cell to its thickness) with the 230 mm-diameter, 6 mm-thick plates. The cell’s small thickness minimizes stress change from cell inclusion, and the large diameter helps reduce stress concentration at the rigid perimeter. This enables the cell to provide reliable estimates of earth pressure with the least possible impact on the existing state of stress in the soil due to cell inclusion. A Geokon 4 twisted pairs-conductor cable was used as a lead wire for the cells. The cable was shielded and consisted of 22 AWG tinned copper conductors and PVC jacket. All EPCs were equipped with lead wires of sufficient length attached to avoid splicing in the field.

### 3.3.1.3 Layout of Stress Measurement Points

In order to manage the cost of field testing as well as time and budgetary constraints, the pressure cells were installed only in the subgrade layer. The objective of the cells was to monitor the transient and permanent vertical stresses induced by traffic loads at the top of the subgrade by installing them beneath the wheel-path. As the wheel approached and departed from the measurement location, stress could be measured by the cell and recorded in the data acquisition system. A total of eight pressure cells were installed.
Measurements were collected from each section except sections 2 and 10 because of budgetary constraints, at a single measurement point directly under the wheel-path during construction and trafficking.

![Figure 3.12: Earth pressure cell used in field experiment](Image)

### 3.3.2 Measurement of Geogrid Strain

Direct measurements of dynamic and permanent strains under traffic loading were made on the biaxial geogrids using foil strain gauges. Strain gauges are excellent for the measurement of dynamic strain conditions (Hoffmann, 2012). Seventeen active strain gauges were installed in the geogrids. The gauges were attached to the topside of the biaxial geogrid ribs. Additionally, three dummy gauges were attached to 400 mm-long square samples from each biaxial geogrid type for temperature correction. The objectives of the geogrid strain measurement as well as a detailed description of the strain gauges, gauge selection, and the procedure used for attaching these gauges to the geogrids are given in this section.
3.3.2.1 Objectives of Geogrid Strain Measurement

a) Quantifying the Amount of Tension Generated in the Geogrid:

Measurements of both vertical stresses transferred to the subgrade and the tensile strains induced in the geogrid by traffic loads are needed to evaluate the contribution of the geogrid reinforcements to the performance of the unpaved system. The amount of tensile strain accumulated in the geogrid indicates the extent to which the geogrid tensile resistance is mobilized and how much the geogrid is interacting with the aggregate. Therefore, the main purpose of measuring the geogrid strain was to quantify the tension generated in the geogrid reinforcement under traffic loading. To achieve this objective, foil strain gauges were attached to the geogrid to measure dynamic and permanent strains of the geogrid under the wheel-path. The gauges were attached to the topside of the biaxial geogrid rib. Strain gauges were not attached to the triangular aperture geogrid because of the limited width of its ribs.

b) Understanding the Geogrid Strain Behavior in Road-Reinforcement Applications:

i. Strain Anisotropy:

The second objective was to investigate strain anisotropy in the geogrid reinforcement under traffic loads (due to the anisotropic loading condition); in other words, to examine the strain behavior of the geogrid with respect to the direction of traffic (transverse versus longitudinal strain).

In order to evaluate strain anisotropy in the geogrid under actual traffic loading conditions, strain gauges were attached to the geogrids in both the machine direction (direction of traffic or longitudinal direction) and in the cross-machine direction (direction perpendicular to traffic or transverse direction). The majority of strain gauges were oriented to measure strains in the transverse direction as it is generally accepted that geogrid tensile resistance is mobilized in the direction perpendicular to traffic.
ii. Strain Distribution across the Road:

The design of geosynthetic-reinforced unpaved roads is fundamentally based on two principal concepts of reinforcement mechanism: lateral restraint (small displacement approach) and tensioned-membrane effect (large displacement approach).

The large displacement design approach (Giroud et al., 1984; Giroud and Han, 2004) is based on the concept that the geosynthetic is in constant or uniform strain across the width of the pavement (Nieuwenhuis, 1977), while in the small displacement approach (Milligan et al., 1989) it is assumed that the maximum tension in the geosynthetic occurs directly under the center of the loaded area (i.e., under the wheel-path).

Accordingly, another important objective of the measurement of geogrid strain was to validate these concepts of constant versus variable transverse strain distribution. Additional strain gauges were attached to the geogrid in the transverse direction, away from the wheel-path at the measurement locations of the thinner test sections (Sections S3, S5, and S9) to examine the distributions of dynamic and permanent geogrid strains across the pavement width.

iii. Identification of the Governing Reinforcement Mechanism:

The distribution of geogrid strain under traffic loads can aid identifying the governing reinforcement mechanism. Thus, the distribution of tensile strains in the geogrid reinforcements must be examined based on measurements of permanent geogrid strain. Conclusions drawn from those measurements will be used to help explain how reinforcement is able to provide an improvement in road performance. In other words, geogrid strain measurement will help identify which reinforcement mechanism is causing the performance improvement.
3.3.2.2 Selection of Foil Strain Gauges

a) Background and General Guidelines:

Electrical resistance foil strain gauges were installed directly on the geogrids to provide a direct measurement of geogrid strain under traffic loading. The foil strain gauge consists of a strain-sensing element (the grid) mounted on a plastic backing or carrier that serves as an insulator, a bonding surface and a means for handling (Figure 3.13). There are several main parameters to consider when selecting a strain gauge for a particular application: grid alloy, backing material, self-temperature compensation number, gauge size (length and width), gauge configuration or pattern (grid and tab geometry) and gauge resistance. These parameters are selected to best satisfy the installation and operating requirements (e.g. accuracy, stability, elongation, duration of use, magnitude and type of load, ease of installation and operating environment). However, the parameters are not subject to completely independent selection because strain gauges are usually constructed in particular combinations of parameters. Therefore, compromises must be made in the selection process.

Figure 3.13: General outline view of an electrical resistance foil strain gauge (after Vishay Micro-Measurements, 2010a)
The grid *alloy* is selected to accommodate operating conditions, such as temperature range, duration of measurement, and type of loading. Strain gauges can be designed to produce minimum thermal output (temperature-induced apparent strain) using grid alloys that can be supplied in *self-temperature-compensation (STC) numbers* to match a wide range of substrate material expansion coefficients. The STC number is the approximate thermal expansion coefficient in ppm/°F of the substrate material on which the strain gauge is to be mounted.

The stiffness of the *backing material* should be selected to match the stiffness of the substrate onto which it is bonded. The strain gauge is usually designed as a complete system that consists of a particular combination of foil and backing materials, and includes special features, such as encapsulation, preinstalled lead wires, or solder dots. Thus, the alloy and backing combination and the special construction features must be selected from among the available *gauge construction systems*.

The strain gauge *length* (i.e., the active length of the grid) is normally selected to suit the geometry of the bonding specimen and the anticipated strain distribution. The first criterion for a homogeneous field of strain is the space available on the measurement object for bonding the gauge. Short gauge lengths should be selected in strain measurements made at the test specimen’s most strained points in order to minimize error due to stress concentrations and steep strain gradients. Long gauge lengths are preferable for other considerations including ease of handling, better heat dissipation, and inhomogeneity of the substrate material. When the mean strain value is required (such as for an inhomogeneous strain field), a long gauge or grid should be used. Similarly, *grid width* should also be considered when selecting a strain gauge. A narrow grid reduces the averaging error when there is steep strain gradients across the gauge width in the substrate surface, while a wide grid (i.e., greater grid area) improves the heat dissipation and enhances gauge stability (Vishay Micro-Measurements, 2018).

Gauge configuration or *pattern* refers to a particular combination of grid shape, solder tab geometry, and number and orientation of the grids in a multiple-grid gauge. The gauge pattern should be selected such that the size and orientation of the solder tabs are
compatible with the space available for mounting the gauge. The tab geometry should also provide a suitable arrangement for proper and convenient lead wire soldering.

Finally, strain gauges are commonly available in two resistance options: 120 ohms and 350 ohms. A gauge with higher resistance is preferable since it reduces heat dissipation, minimizes lead wire effects (e.g., spurious signals caused by lead wire resistance changes with temperature variations), and improves the signal-to-noise ratio when the gauge circuit includes sources of resistance change (Vishay Micro-Measurements, 2018). Nevertheless, 120-ohm gauges are usually suitable for the majority of applications.

b) Selection Steps and Gauge Description:

The foil strain gauges used in this study were manufactured by Vishay Micro-Measurements. The gauge designation system used by Micro-Measurements comprises five main parts including gauge series (i.e., backing material and grid alloy), STC number, gauge length, gauge pattern (i.e., grid and tab configuration), and gauge resistance. It should be noted that the type (i.e., geogrid or geotextile), polymer composition (i.e., polypropylene, polyester, or polyethylene) and structure (i.e., punched-sheet drawn, heat welded, woven, or non-woven) of the geosynthetic used should be critically considered in the selection process. In this case, the geosynthetic under consideration is an integrally-formed, punched-sheet drawn geogrid made of polypropylene. The selection process of the strain gauge used in this study along with the features of the selected gauge components are discussed below.

For integrally-formed geogrids, the strain varies along the length of the rib such that the highest strain will be near the node and the lowest strain will be near the mid-length of the rib (Warren et al., 2010). Therefore, a longer gauge would improve strain averaging. In addition, a longer and wider gauge (i.e., greater grid area) introduces lower wattage per unit of grid area and thus minimizes heat dissipation caused by the poor heat transfer properties (low thermal conductivity) of the polymer material. However, the selection of the strain gauge size was restricted by the limited dimensions of the geogrid rib; in particular, the rib width (2.5 to 3 mm). Accordingly, the smallest available overall gauge width (i.e., matrix width) of 3 mm was specified. Subsequently, the longest gauge
available in this width that fits the length of the geogrid rib was selected. This gauge has an active gauge length of 5.84 mm (230 mils). The overall length of the gauge is 9.53 mm which covers a reasonable portion of the rib length between the nodes.

A single-grid gauge was employed in this application. The “230DS” pattern was specified since it is the only configuration to provide the preselected gauge size. The 230DS pattern comes with solder tabs located on opposite ends of the gauge. This tab geometry is recommended since it allows for designing a wiring configuration with all wires remain attached to the surface of the geogrid rib while not touching each other (Warren et al., 2010). In all cases, wires and cables should not be allowed to cross over the grid apertures to prevent otherwise inevitable damage to the wires in the field by aggregate particles penetrating into the apertures. Additionally, this pattern features larger tabs (being on opposite ends of the gauge), which is preferable arrangement for wire soldering on the tabs.

The selected gauge had backing constructed from polyimide, designated as “E” backing. It provides superior elongation capabilities that are conformable with the polymer material and it minimizes reinforcement of the geogrid (provides lower gauge stiffness) because of its low stiffness. The grid alloy was constantan or “A” alloy which has an adequately high strain sensitivity. It is characterized by good fatigue life and relatively high elongation capability (Vishay Micro-Measurements, 2018). In addition, it can be processed for self-temperature-compensation to match a wide range of test material expansion coefficients. Accordingly, the EA series (constantan grid alloy on a polyimide backing) was used for the geogrid material in this study. The EA series gauges can sustain strains up to 5%, which is suitable for most field applications.

Some researchers tend to select high-elongation annealed constantan or “P” alloy, used primarily for measurements of large post-yield strains, with the E backing. The EP series gauges can sustain strains up to 20%. They are considered appropriate by some practitioners since the geosynthetic is constructed of plastic that has the ability to elongate excessively (approximately 10% for geogrids and 15–50% for geotextiles). However, this selection is not necessary to say the least since the strains anticipated under traffic loads are typically only as high as 2–4%. In fact, using gauges with strain ranges
that are considerably larger than those anticipated in the field could jeopardize an
otherwise well-planned measurement program. It was noticed from the literature review
that EP series strain gauges were less successful in measuring geogrid strain in pavement
applications despite following strict selection guidelines and rigorous attachment
techniques (Maxwell et al. 2005; Warren and Howard, 2007). Further, it was noticed that
most of the reinforced pavement studies, from which researchers were able to obtain
meaningful geogrid strain data, used strain gauges that had strain ranges of 5% or less.
Those gauges were either constructed with constantan alloy grid and polyimide backing
with 5% strain range (Hufenus et al., 2006; Tang et al., 2013; Tang et al., 2015) or with
other materials with 3% strain range (Helstrom et al., 2006; Henry et al., 2009). Over and
above that, the EP series gauges are more expensive due to their limited availability; and
the “P” alloy shows zero shift under high cyclic strains because of the poor fatigue life,
and the grid tends to fail prematurely with repeated straining (Vishay Micro-
Measurements, 2018).

The 230DS pattern was not available in high grid resistance for the EA series. Therefore,
120-ohm strain gauges were used in this study. It should be noted that the lead wire
resistance could produce a circuit desensitization and thermal drift, particularly in low
resistance gauges. In order to minimize the effects of lead wire resistance on the 120-ohm
strain gauges used, lead cables with low lead wire resistance were used with the gauges
and efforts were made to minimize lead lengths. The lead wire should always be as short
and as thick as possible. Additionally, the gauges were connected independently to the
Wheatstone bridge in a quarter bridge configuration using three-wire connection as will
be discussed later in this chapter.

To account for the effect of temperature fluctuations in the field, dummy strain gauges
identical to the active ones were mounted on small unstressed geogrid specimens placed
in the roadway away from the traffic. The dummy gauges were used to measure the
thermally induced strain on the geogrid materials without the effect of traffic loading.
This thermally induced strain can then be used to correct the measured strains by the
active strain gauges.
From the above steps, the foil strain gauges selected in this study had the following identification number (Figure 3.14):

EA-06-230DS-120

In summary, the selected gauge consisted of constantan foil (in self-temperature-compensation form, STC = 6 ppm/°F) in combination with tough, flexible, polyimide backing. The gauge had a grid length of 5.84 mm. The overall length and width of the gauge matrix were 12.7 mm and 3 mm, respectively. The gauge can reliably measure strains up to 5% which is suitable for unpaved road applications. The lead wire used for the foil strain gauges was a Belden 24 AWG stranded tinned copper 3-conductor shielded cable with PVC outer jacket. All strain gauges were prepared with full field-required lead wire lengths attached so that no cable splicing was performed.

![Foil strain gauge used in this study](image)

**Figure 3.14: Foil strain gauge used in this study**

### 3.3.2.3 Procedure for Attaching Foil Strain Gauges to Geogrids

Concerns regarding preparation of the bonding surface that are specific to plastics must be carefully addressed in advance. The unsupported planar grid product is very weak under out-of-plane loads, which makes it difficult to provide a stable structure during gauge installation. Further, foil strain gauges are very delicate sensors and must be protected from mechanical and moisture damage in a harsh field environment. Previous
procedures available in the literature for installing foil strain gauges on geogrids were reviewed (Cuelho, 1998; Farrag and Morvant, 2003; Warren et al., 2010; Morris, 2013) and combined with techniques and supplies developed in this study to synthesize an attachment procedure that fits the specific needs of this research project. Essential part of this procedure is a modified and refined version of the work of Warren et al. (2010).

a) Layout and Wiring Configuration:

Each single strain gauge was attached to the topside of each targeted biaxial geogrid rib and connected using a three wire quarter bridge circuit as shown in Figure 3.15. The connection was arranged such that one lead wire appears in one bridge arm and the other appears in an adjacent arm. This arrangement offers an intrinsic bridge balance and automatic compensation for the thermal drift due to lead wire resistance, and increases measurement sensitivity (Vishay Micro-Measurements, 2010b).

Two bondable terminals were required for each gauge to bridge the hookup wires from the gauge tabs to the trunk or lead cable. The terminals were useful in minimizing the risk of damage to the gauge by eliminating the trunk cable direct connection to the gauge solder tabs. Micro-Measurements CPF-50C terminals were selected (copper foil laminated on a polyimide film backing). The gauge was attached to one rib and the terminals were placed on the adjacent ribs for the aforementioned space limitations. The trunk cable was positioned on the remaining rib in the gauged geogrid aperture at the opposite side of the gauge. This layout ensured that all wires were accommodated on top of the geogrid ribs and nodes (Warren et al., 2010). Figure 3.16 shows the working space used for performing the attachment of foil strain gauges to the geogrids. The essential installation supplies and accessories used throughout the attachment process are listed in Table 3.4.
Figure 3.15: Three-wire, quarter-bridge circuit (after Vishay Micro-Measurements, 2010b)

Figure 3.16: Working space arrangement for performing strain gauge attachment to geogrids
Table 3.4: Essential installation supplies and accessories used for strain gauge attachment

<table>
<thead>
<tr>
<th>Product</th>
<th>Installation phase</th>
<th>Use</th>
<th>Additional notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>GC-6 Isopropyl Alcohol</td>
<td>Surface preparation</td>
<td>Surface degreasing</td>
<td>Preferable cleaning of plastics</td>
</tr>
<tr>
<td>GSP-1 Gauze pads/sponges</td>
<td></td>
<td>Surface cleaning</td>
<td></td>
</tr>
<tr>
<td>CSP-1 Cotton-tipped applicators</td>
<td></td>
<td>Surface cleaning</td>
<td></td>
</tr>
<tr>
<td>SCP-3 Silicon-carbide paper, 400 grit</td>
<td></td>
<td>Surface abrasion</td>
<td></td>
</tr>
<tr>
<td>MCA Conditioner A</td>
<td></td>
<td>Surface conditioning</td>
<td></td>
</tr>
<tr>
<td>MN5A M-Prep Neutralizer 5A</td>
<td></td>
<td>Surface neutralizing</td>
<td>Removable silicon contamination</td>
</tr>
<tr>
<td>PCT-2M Gauge Installation Tape</td>
<td>Gauge bonding</td>
<td>Gauge handling</td>
<td>Provides non-bondable base</td>
</tr>
<tr>
<td>TFE-1 Teflon film</td>
<td></td>
<td>Gauge bonding</td>
<td>Provides non-bondable base</td>
</tr>
<tr>
<td>brush-cap bottle Catalyst</td>
<td></td>
<td>Gauge bonding</td>
<td>Provides non-bondable base</td>
</tr>
<tr>
<td>M-Bond 200</td>
<td></td>
<td>Adhesive</td>
<td>Controls the reactivity rate of the adhesive</td>
</tr>
<tr>
<td>134-AWP Hookup wire - 34 AWG</td>
<td>Attachment of wires</td>
<td>Gauge hookup wire</td>
<td>Improves bondability to other coatings</td>
</tr>
<tr>
<td>M-Coat B (Nitrile Rubber Coating) Drafting Tape</td>
<td></td>
<td>Prime for Vinyl-insulation protection during soldering</td>
<td></td>
</tr>
<tr>
<td>Rosin Solder with flux core</td>
<td></td>
<td>Wire solder</td>
<td></td>
</tr>
<tr>
<td>RSK-4 (M-LINE) Rosin Solvent</td>
<td></td>
<td>Removal of rosin residues</td>
<td>Prevent corrosion and coating degradation</td>
</tr>
<tr>
<td>M-Coat D (Acrylic coating)</td>
<td>Waterproofing and protection</td>
<td>Base coating</td>
<td>Anchors and insulate the wires</td>
</tr>
<tr>
<td>M-LINE RTV Primer No. 1</td>
<td></td>
<td>Prime for 3145 RTV</td>
<td>Improves long-term protection</td>
</tr>
<tr>
<td>3145 RTV (silicone rubber coating)</td>
<td></td>
<td>Waterproofing</td>
<td></td>
</tr>
<tr>
<td>Aqua Seal</td>
<td></td>
<td>Insulation and protection</td>
<td></td>
</tr>
</tbody>
</table>
b) Preparation of Geogrid:

- In order to provide a flat and stable structure during gauge installation, two small aluminum pipes were attached surrounding the application area and parallel to the gauge axis, which remained attached until the gauges were in their permanent position in the field.

- The surface was degreased with isopropyl alcohol using cotton swabs.

- The application area was lightly abraded using 400-grit silicon-carbide paper to clean and roughen the surface (Figure 3.17). All ribs and nodes within the gauged geogrid aperture and the ribs adjacent to the gauged geogrid aperture were abraded.

- The geogrid surface was slightly burnished to initiate surface oxidation by exposing to Ultraviolet light in an air atmosphere because abrasion is not sufficient to achieve optimum bond to the polypropylene. Additional chemical surface treatment was used subsequent to abrasion to improve bondability by chemically altering the surface (Chu et al., 1996).

- The surface was cleaned by scrubbing with M-Prep M-Prep Conditioner A, and then was dried with a gauze sponge.

- The surface was neutralized immediately by applying M-Prep Neutralizer 5A, and scrubbing with a clean cotton-tipped applicator, and then was dried again.

Figure 3.17: Preparation of the geogrid surface (abrasion)
c) Strain Gauge Bonding:

- The gauge was removed from its envelope, and placed (bonding side down) on a clean glass plate.

- Gauge installation tape (10 to 15 cm long) was prepared for each gauge and terminal to be attached and was placed sticky side down over the gauge.

- The gauge/tape assembly was placed over the surface of its respective rib such that the gauge was aligned in its desired location in the rib. The entire geogrid working area was supported during gauge bonding. A thin plastic board was placed underneath the geogrid to serve as a clean work surface (Figure 3.18).

- One end of the tape was lifted slightly until the gauge was free of the surface and was then folded back over itself and the catalyst was applied to the bonding surface of the gauge.

- A small drop of the adhesive (M-Bond 200) was applied to the rib beyond the actual gauge bonding area, then the tape was brought back at a shallow angle, held tight, and wiped over firmly with a piece of gauze to bring the gauge down to its desired location.

- The gauge installation tape was removed after about two minutes and any excess adhesive was removed using a razor blade.

- The gauge resistance was measured across the gauge between the tabs and was recorded for later reference.
d) Preparation of Wires:

i. Preparation of Hookup Wire:

- The insulation was stripped from a short length of the 34 AWG gauge hookup wire.
- The hookup wire was curved to form strain relief loops (Figure 3.19).

ii. Preparation of Trunk Cable:

- The outer jacket of the 24 AWG trunk cable was stripped (Figure 3.19) and the internal shielding surrounding the internal conductor wires was trimmed, then approximately 40 mm was stripped of the insulation of each conductor wire.
- 8 to 10 mm of the individual strands of the red conductor wire near the end of the insulation were twisted and tinned to create a bundle of wires. All strands were cut but
one to fit the size of the terminal’s copper pad. The two remaining conductor wires were prepared together the same to create another bundle with one extending strand.

- Two single strands were curved to form strain relief loops, and were later used as the jumper wires connected to the terminals.

- The outer jacket of the trunk cable and all exposed insulation of the three conductor wires were coated using M-Coat B and were left to cure for at least 24 hours.

Figure 3.19: Preparation of hookup wires and trunk cables

e) Attachment of Wires:

- The solder tabs and the terminals were cleaned using isopropyl alcohol and the entire gauge grid area was masked with drafting tape, leaving only the tabs exposed (Figure 3.20).
• The solder tabs and terminals were tinned using Rosin Solder with Flux Core (create a small solder point), and then were cleaned.

• One end of each hookup wire was connected to a solder point on the tab, and the other end was attached to the solder point on the terminal.

• The drafting tape was peeled off the gauge grid surface and the rosin residues were removed after soldering using RSK Rosin Solvent. The resistance between the two terminals was then checked to ensure it is the same as taken before.

• The trunk cable was secured to the geogrid rib using zip ties while ensuring there was sufficient strain relief in the cable.

• Each single strand from the trunk cable was connected to the unoccupied solder point on the terminals as described previously.

Figure 3.20: Wiring of a strain gauge on a geogrid
f) Waterproofing and Protection:

- All areas within the gauged geogrid aperture and the ribs adjacent to this aperture were coated using M-Coat D. Similarly, the exposed hookup wires, gauges, terminals, and at least 2 cm of the jacketed trunk cable were coated and the coat was left to dry for 24 hours. The surface was then primed with M-LINE RTV Primer No. 1 (for best long-term protection), and after dried, it was overcoated with RTV Silicon Rubber Coating and left to dry for 24 hours.

- The entire area within the instrumented geogrid aperture (four ribs and four nodes) and 3 cm of the extended jacketed trunk cable were sealed with Aqua Seal for waterproofing and insulating the electrical connections, and then wrapped the Aqua Seal with Teflon film (Figure 3.21).

- Finally, the total resistance was checked, then the geogrid was touched gently to confirm that the gauge is reading.

![Figure 3.21: Waterproofing and protection of a bonded strain gauge](image_url)
3.3.3 Measurement of Temperature

The mechanical properties (in particular, strain-time behavior) of geogrids can be influenced by elevated temperatures (Kaliakin and Dechasakulsom, 2001). The pressure cells used in this study were equipped with a built-in thermistor to obtain static measurements of ambient temperature near the surface of the subgrade and the geogrid layer. The used thermistors were constructed of semiconductor materials that has temperature-dependent resistance properties (Zhang and Hoshino, 2019), and provided accurate measurement owing to large resistance changes with temperature. They give a varying resistance output as ambient temperature changes. This measured resistance is then converted to temperature. The thermistors used feature a temperature range of -80 to +150°C and an accuracy of ±0.5°C.

The thermistors measurements were collected by the data acquisition system, and were used to assess variations in the ambient temperature of the geogrids placed directly on top of the subgrade during trafficking. In addition, the measured temperature could be used to separate spurious effects of temperature from real measurements data collected by other instruments. The temperature readings were obtained periodically throughout the field testing.

3.3.4 Motion Sensors

Motion sensors were installed near each instrument location as shown in Figure 3.22. The sensors provided a traffic count and helped separate the measured road response to traffic loading from the entire set of collected data during post-test processing. The motion sensor detected the vehicle as it approached the instrument location and sent a signal coinciding with the response signals recorded by the instruments.
3.3.5 Instrument Locations and Identification

Instruments were mainly concentrated in one location in each section, mid-length of the section under the left-side wheel-path with the exception of three additional active geogrid foil strain gauges that were located between the two wheel-paths. Dummy strain gauges were mounted on small geogrid specimens and placed in the field away from the loading area, so that the gauges don’t experience traffic induced strain. Details of the type, number, and locations of instruments used in the field experiment are presented in Table 3.5. Figure 3.23 presents a profile view of the instrumented test sections exhibiting the majority of the instrumentation. Test sections with smaller base layer thickness were anticipated to deteriorate first and undergo greater amount of damage, and provide more information about the mechanical response of the unpaved system in a shorter monitoring period. Accordingly, more instrumentation were installed in the 200 mm-base test sections than in the 250 mm-base test sections. Pressure cells were installed near the top of the subgrade layer of each test section except sections 2 and 10.
Each instrument installed in the test sections was assigned a unique identification number to clearly identify its type, location, direction, and assigned data acquisition system connection. This identification number consists of four main parts as follows:

- First part of the identification number describes the type of instrument.
  
  EPC = Earth pressure cell  
  SG = Strain gauge  
  MS = Motion sensor  

- Second part of the identification number designates the number of the test section.
  
  Sections 1 to 10  

- Third part of the identification number designates the layer or material of installation, and optionally direction or type of measurement (for geogrid strain gauges).
  
  S = Subgrade  
  GM = Geogrid, machine direction  
  GX = Geogrid, cross-machine direction  
  GXB = Geogrid, cross-machine direction between the wheel-paths  
  GD = Geogrid, dummy gauge  

- Fourth and final part of the identification number describes the components of data acquisition hardware to which the instrument should be connected (i.e., chassis, module/terminal, and channel).
Table 3.5: Details of instrumentation

<table>
<thead>
<tr>
<th>Section</th>
<th>Geogrid strain gauges&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Under the wheel-path</th>
<th>Between wheels</th>
<th>Dummy gauges</th>
<th>Earth pressure cells</th>
<th>Connected thermistors</th>
<th>Motion sensor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>MD</td>
<td>XD</td>
<td>XD</td>
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<td></td>
<td></td>
</tr>
<tr>
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<td></td>
<td>—</td>
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<td>—</td>
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<td>N/A</td>
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<tr>
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<td>1</td>
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<td>1</td>
</tr>
<tr>
<td>S10</td>
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<td>1</td>
<td>1</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

<sup>a</sup> MD = machine direction, XD = cross-machine direction
Figure 3.23: Schematic diagram for instrumentation of the test sections: (a) 200 mm-base sections, (b) 250 mm-base sections
3.3.6 Data Acquisition System

Several key factors were considered in planning the field experiment and data acquisition to ensure that the project goals and budget requirements are met. These factors included test variables, instrument types, and number of instruments. Considerations regarding the type and number of measurements (such as availability and cost of an instrument, precision and accuracy of the measurement, frequency of the measurements, and the number of measured variables) were evaluated in light of the project budget. The test variables investigated included base thickness, geogrid type and distribution of geogrid strain. The number of instruments was optimized to provide accuracy and ensure repeatability and survivability. Finally, the data acquisition system was configured to collect response data from the 38 sensors embedded in the test sections.

3.3.6.1 Data Acquisition Hardware

Data acquisition (DAQ) system manufactured by National Instruments was set up for data collection from the field test (Figure 3.24). The data acquisition system consisted of two chassis. The main chassis was an 8-slot PXI-1042 that housed the controller, PXI-8820 RT, and two DAQ cards, PXI-6052E and PXI-6070E. The RT controller (National Instruments), which was high-performance PXI system controller that integrated standard I/O features in a single unit, provided reliable operation during measurement. Combining an NI PXI-8820 RT embedded controller with the NI PXI-1042 chassis resulted in a fully PC-compatible computer in a compact rugged package. Embedded LabVIEW Real-Time applications continued to run even if the host PC was interrupted or rebooted. The PXI-6052E and PXI-6070E DAQ cards were high-performance multifunction analog, digital, and timing I/O devices for PXI computers. The PXI-6052E DAQ card was able to sample data at rates up to 333 kHz while the PXI-6070E DAQ card could stream data at a 1.25 MHz rate.

The second chassis was a 12-slot SCXI-1001. This chassis housed the analog input modules, SCXI-1520, which interfaced to strain-gauge bridges. The SCXI chassis supplies a low-noise environment for signal conditioning, supplying power and control
circuitry for the modules. The SCXI-1520 provides a programmable DC voltage excitation between 0 and 10 V and two filtering stages with an overall response of a four-pole Butterworth filter.

The foil strain gauges were connected to the SCXI-1520 modules via universal strain terminal blocks, SCXI-1314, following the proper bridge configuration. An SCXI-1349 shielded cable adapter was used to connect the SCXI-1520 modules to the PXI-6052E DAQ card in the main chassis. The earth pressure cells and attached thermistors were connected to the PXI-6070E DAQ card through unshielded 68-pin screw terminal, NI TBX-68. The instrument circuitry for the pressure cells was completed in the terminal box by supplying an excitation voltage (12 volt) to the cells using an external power supply. Before transportation to the site for installation, all instruments were checked for proper functioning and output signals from the instruments were acquired by connecting to the DAQ system.

Figure 3.24: Data acquisition hardware

3.3.6.2 Data Acquisition Software

LabVIEW™ 14 was used to communicate with the data acquisition hardware and to control data collection from all sensors embedded in the test sections. Customized data acquisition code was written in LabVIEW to monitor and record the instruments response to traffic loading. A user interface, or front panel, was built with controls and indicators.
Then a code was incorporated using the virtual instruments (VIs) and structures in LabVIEW to control the front panel objects. The VI included a block diagram that contains this code.

Previous field studies have used different mechanisms for triggering data acquisition. Brandon et al. (1996) used piezoelectric sensors to initiate continuous data collection during the entire duration of each vehicle pass for all sections. Collecting data continuously from all sections will produce a very large amount of data that requires a huge post-processing effort. Howard and Warren (2005) developed a customized program to trigger and collect data independently in each section during a short acquisition period using asphalt strain gauge as a trigger. Despite being promising, this concept requires extensive programming, imposes unnecessary delays on the project, and has its own hidden/invisible costs. For instance, such triggering mechanism must be tested in a preliminary pilot field test before executing in a full scale testing. In addition, it was difficult, in this study, to find a sensor that can be reliably used to trigger data acquisition without adding significantly to the project cost.

In this study, an intermediate approach was used to minimize data post-processing effort. Data acquisition was triggered manually and data was sampled continuously at a sampling rate of 200 Hz in order to obtain enough data to describe the response. However, the data of interest are only those collected while the truck is passing over the sensors. Therefore, signals generated by the motion sensors as the truck passed over the instrument locations were used to identify these response signals. Specifically, a Matlab code was written to extract these measured response data from the rest of collected data in order to minimize data post-processing effort. In order to further facilitate post-processing, the data collected from the various instruments were stored in different files by LabVIEW. The file size was limited to 20 passes per file.
3.4 Summary

The research site for the field testing program was described and the materials used were discussed. The site investigation program conducted to characterize the subgrade soil at the research site was detailed. In-situ and laboratory tests performed to determine the engineering properties of the subgrade soil and the base course material were discussed. The geosynthetics used in this study and their physical and mechanical properties were outlined. Description of the field testing program and configuration of the full-scale unpaved test sections was provided.

The instrumentation used to measure the response of the unpaved test sections during field trafficking along with the data acquisition system used to collect response data were also detailed. The objectives of the instrumentation were highlighted and important instrument selection guidelines were discussed in detail. Specific concerns and challenges related to field applications, performance and survivability of some of the instrumentation used in the field experimental program were highlighted and solutions were proposed. A detailed procedure for installing and protecting foil strain gauges on geogrids for actual field conditions and pavement applications was presented. The procedure was developed from a combination of previous techniques, case studies, and other resources identified in the literature and from consulting the manufacturer application instructions and technical documents, considering the specific material and operating requirements.
Chapter 4

4 CONSTRUCTION, TRAFFICKING AND DATA COLLECTION

4.1 Construction of Test Sections

Prior to commencing the construction work, the test site was prepared and supplied with the necessary tools and equipment. A mobile container was located near the road area to be used as an office space, storage, and shelter for data acquisition. A power source was supplied next to the container to provide instruments and DAQ system with power.

Ten instrumented unpaved test sections were constructed in the test site. Each test section was 14 m in length and 4 m in width. The test area consisted of two road lanes that run side by side, each included five test sections. The two lanes were constructed with different base course thicknesses, 200 and 250 mm.

The construction of the test sections took place in four main phases: excavation of subgrade (completed in May), instrumentation and placement of geosynthetics (completed in July), placement of the base layer (completed in August), and trafficking (performed in August-September). The construction of the field test sections is described in more detail in the following sections.

4.1.1 Excavation and Preparation of Subgrade

The construction was initiated by excavating a section measuring approximately 9 m by 75 m in the subgrade to remove existing fill material and expose the native subgrade soil. Based on preliminary information of the test site indicating soft subgrade, the construction plan was initially developed with an expected excavation depth of 300 mm. However, a deeper fill layer of approximately 1 m thick was encountered during excavation resulting in a change of excavation depth. As a result of this change in excavation depth, the level of the road surface relative to the surrounding area has significantly changed. Key aspects of the original construction plan such as drainage...
control, surveying works, locating of instruments, cable protection, and aggregate placement were consequently impacted.

In order to construct two road lanes with different base layer thicknesses, the subgrade was cut with a level difference of 50 mm between the two lanes (Figure 4.1). The subgrade surface was prepared with a one-side cross slope of approximately 4% gradient to facilitate water runoff. All debris and large pieces were removed from the subgrade surface to ensure a smooth surface for geosynthetic placement. The edges of the excavated area were cut in a slope of approximately 45° using a backhoe loader to avoid the collapse of the excavation sides. Ramps were constructed at the entrance and exit of the test road to ensure a smooth transition between the test road and the turning areas at the road ends so that the test vehicle can travel the test sections with constant speed.

![Figure 4.1: Excavation of the subgrade](image)

4.1.2 Instrumentation of the Subgrade

After preparing the subgrade surface, its elevation was measured at predetermined grid points. Specifically, instrument positions were precisely located on the wheel-path using
Topcon total station (Figure 4.2). The coordinates of the grid points and instrument locations were saved in the total station so that these locations can be easily recovered during subsequent construction and testing phases. Eight pressure cells were installed in the subgrade such that the top of the cell is 25 mm below subgrade surface. The cell locations were first determined and marked on the subgrade surface using spray paint. The subgrade was then excavated using hand tools for placement of the cells. The cells were bedded on compacted fine sand. After aligning and positioning the cells in their final locations, they were protected from damage by sharp aggregate pieces in the base course by placing selected fine material (mix of sand and natural subgrade soil) on top of the cells. The fill material was then compacted with extreme caution using hand tools (Figure 4.3).

Cables of the pressure cells were housed in 19 mm standard Polyethylene (PE) tubes using a fishing tape (Figure 4.4) to provide protection against damage by angular particles. An expanding, insulating foam sealant was used to seal the ends of the tubes against moisture intrusion. The PE tubes were then accommodated in shallow trenches manually excavated in the subgrade from the cell location to the road side (Figure 4.5). The PE tubes were embedded inside the trenches in a layer of compacted selected fine material to provide further protection against damage by construction equipment and traffic loads. The parts of the cables extending outside the PE tubes were temporarily secured in plastic bags at the sides of the test road to protect them from rain. After backfilling the trenches, the cell and trench locations were covered immediately with tarps awaiting placement of geosynthetics.
Figure 4.2: Locating instrument positions on the wheel-path

Figure 4.3: Installation of earth pressure cell
Figure 4.4: Feeding of a pressure cell cable into a polyethylene tube

Figure 4.5: Placement of a pressure cell cable inside a trench
4.1.3 Placement of Geosynthetics

After the subgrade instrumentation was installed, the subgrade surface was once again cleaned of all remains from previous activities and was levelled prior to the installation of geosynthetics. The geogrids and geotextiles were cut to the proper size of each test section and instrumented in the laboratory before being transported to the test site. A geotextile separator was then placed directly over the prepared subgrade surface (Figure 4.6). No tensioning was performed on the geotextiles so that they are not anticipated to provide a reinforcing effect (Holtz et al., 2008).

Biaxial geogrids were installed in the test sections such that the geogrid machine direction was parallel to the vehicle wheel-path (aligns with the longitudinal direction), while the cross-machine direction of the geogrid was perpendicular to the vehicle wheel-path (aligns with the transverse direction). The instrumented geogrid rolls were placed at the center of their respective test sections such that the strain gauges were approximately positioned in their predetermined locations (Figure 4.7). The geogrids were then carefully unrolled from both ends in the direction of traffic. Geogrids were next adjusted such that the strain gauges were positioned at their final locations using the previously surveyed coordinates. The geogrids were then pulled tight and pinned in place at their edges. All geosynthetics were placed such that they overlap one another by approximately 60 cm in the direction of traffic.
Figure 4.6: Geotextiles placed over the prepared subgrade surface

Figure 4.7: Positioning of an instrumented geogrid in the approximate strain gauge location in the test section
4.1.4 Field Protection of Foil Strain Gauges

As discussed in Chapter 3, foil strain gauges were attached to the biaxial geogrids in the laboratory. Prior to transporting the geogrid to the field, the gauged areas were wrapped with a sealing bituminous material in the laboratory to waterproof and insulate the gauges and their electrical connections and to reduce the risk of mechanical damage in the field. In addition, small aluminum pipes were attached to the geogrid at each side of the installed gauge to help maintain a flat and strain free area around the strain gauge. The instrumented geogrids were then rolled up from both ends towards the installed gauge and carefully transported to the site and stored in a covered place awaiting installation. While in storage, the resistance of the attached strain gauges were regularly checked to monitor attachment quality and integrity and detect any possible defects.

After placing the geogrids on the subgrade following the procedure described in the previous section, the foil strain gauges attached to the geogrids were temporarily protected from the rain by lifting the geogrid at the gauge location using small wood pieces and covering the entire area around the gauge with tarp (Figure 4.8). In order to reduce the risk of accidental damage to the strain gauges during construction, activities of personnel and equipment at the site were directed away from the instrument locations by temporarily pinning traffic or construction cones near these locations.

Prior to the placement of base course material, the strain gauges temporary protection was removed. The strain gauges were covered with a layer of sand to provide permanent protection from damage during compaction and trafficking (Figure 4.9). In addition, the compacted fine material on top of the pressure cells were used as cushions underneath the strain gauges to reduce the risk of damage to the gauges by soil clumps or sharp pieces that might existed in the subgrade.

All cables attached to the strain gauges and extended across the top surface of the geosynthetics were housed inside PE tubes extended to the roadside. The ends of the PE tubes were made water-tight using expanding, insulating foam sealant. The cables and PE tubes were fastened to the geogrid using plastic zip ties. The protective sand layer was continued along the exposed length of the strain gauge cables outside the PE tubes to protect the cables from puncturing by the base course aggregate.
Figure 4.8: Temporary protection of geogrid strain gauges

Figure 4.9: Permanent protection of geogrid strain gauges
4.1.5 Protection and Identification of Instrumentation Cabling

A protective cable conduit network was constructed near the test road in order to house all instrument cables and protect them from moisture intrusion and physical damage by construction and testing activities (Figure 4.10). All instrument cables exited the test sections at one side of the road, where the DAQ system was located, in order to minimize lead lengths. Secondary 75 mm ABS pipes received the instrument cables (housed in PE tubes) at their exit points from each test section. These pipes extended perpendicular to the road direction from the cable exit locations to a “main” collecting 75 mm ABS pipe running parallel to the road direction. A 75 mm flexible PVC pipe was connected to the main pipe at the middle of the roadside to allow cables to be secured inside the container hosting the data acquisition system.

All instruments were connected with lead cables of sufficient length so that no cable splicing was performed in the field. In order to facilitate the process of feeding these long cables into the ABS pipe, the cables were fed through individual pipe sections of shorter lengths one at a time before these pipe sections were glued together through proper fittings. The gaps between the PE tubes exiting each test section at their entrances to the receiving ABS pipes were carefully filled with expanding foam sealant. This created a waterproof seal around the PE tubes to prevent moisture intrusion into the protective cable conduit system. The pipe network was provided with access points for possible cable repair works (Figure 4.10). Cleanout T-fittings with sealed caps were used for this purpose. The pipe network was laid on the ground surface and no excavation works were necessary since the testing was planned for a relatively short period of time. Unburied access points would ensure easy and quick access to the cables and facilitate repair works. The pipes were zip tied to metal stakes fixed in the ground along the pipe line to secure them in position.
All cables were labeled at multiple locations for identification and repair purposes. The labeling scheme used included instrument type, instrument location (layer and test section), chassis and module names, and channel number. For each cable, the main identification labels were located near the sensor, at the exit point of the cable from the test section, and at each access point of the cable protection conduit, and near the end of the cable at the data acquisition system. Additional identification tags were attached to each cable at regular intervals along the cable to facilitate identification if cable splicing is needed. The length of the cable connected to each instrument was also printed in the back of the identification tag. A color-coding system was followed in the labeling to facilitate visual identification of the instrument cables in case repair was required.
4.1.6 Placement and Compaction of the Base Course Layer

All Instruments were connected to the DAQ system so that the instrument performance could be monitored during base course placement and compaction. The base course material was gently placed on the geosynthetics using a Kubota SVL75 compact track loader. No aggregate was tipped directly over the instrumented areas. A working platform of aggregate was first constructed, ahead of the equipment, between the two lanes and away from the instrumented areas (Figure 4.11). The aggregate was then spread laterally across the test road and over the instrumented areas with as much care as possible using the bucket of the loader (Figure 4.12). A small dual smooth-drum vibratory roller (Wacker Neuson RD12) was used to compact the base course in order to minimize the risk of damage to the instruments (Figure 4.13).

The elevation information of the grid points established earlier in the subgrade was used to achieve the required final elevation of the base layer during placement and compaction of base course material. A laser level was utilized to ensure the required thickness is being added to the subgrade level at each point of the surveying grid. A cross slope matching the slope in the subgrade was established on the base layer surface. Figure 4.14 shows the test road at the end of construction.

Prior to trafficking, the base course density and moisture content were measured to ensure that the uniformity and the desired amount of compaction was achieved. Nuclear density tests were conducted on the base course. The measured average in-situ dry unit weight and average in-situ moisture content for the base course layer were 3.6% and 20.9 kN/m$^3$, respectively. Variations for each test section from the average dry unit weight are summarized in Table 4.1. The results show a reasonable consistency of dry density throughout the base layer.

Additionally, sand-cone tests were conducted on the compacted base course to determine its in-situ unit weigh and moisture content. The measured dry unit weight ranged from 20.6 to 20.9 kN/m$^3$ and the moisture content ranged from 3 to 6%. These results are in general agreement with those obtained by the nuclear density gauge.
### Table 4.1: Results of nuclear density tests on the compacted base layer

<table>
<thead>
<tr>
<th>Test section</th>
<th>Dry unit weight (kN/m³)</th>
<th>Moisture content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>20.4</td>
<td>3.4</td>
</tr>
<tr>
<td>S2</td>
<td>21.3</td>
<td>4</td>
</tr>
<tr>
<td>S3</td>
<td>20.8</td>
<td>3.8</td>
</tr>
<tr>
<td>S4</td>
<td>21.4</td>
<td>3.4</td>
</tr>
<tr>
<td>S5</td>
<td>21.5</td>
<td>3</td>
</tr>
<tr>
<td>S6</td>
<td>20.8</td>
<td>4.1</td>
</tr>
<tr>
<td>S7</td>
<td>21.2</td>
<td>3.4</td>
</tr>
<tr>
<td>S8</td>
<td>20.9</td>
<td>3</td>
</tr>
<tr>
<td>S9</td>
<td>20.8</td>
<td>4.5</td>
</tr>
<tr>
<td>S10</td>
<td>20.4</td>
<td>3</td>
</tr>
</tbody>
</table>

Figure 4.11: Placement of base course material (construction of working platform)
Figure 4.12: Lateral spreading of aggregate over the instrumented areas

Figure 4.13: Compaction of the base layer
4.1.7 Drainage Control

Controlling of water drainage into and from the test road was important to ensure same test conditions for all test sections. The original plan for drainage control included two measures: to prepare the subgrade and the base course surfaces with a crown-slope towards the sides of the road sections; and to install a corrugated drain tile surrounded by a bed of stone along both sides of the road at a suitable depth to facilitate water flows from the base course and subgrade surface into the tile. The drain tile then drains into a solid pipe leading to a discharge area. However, the unexpected deep excavation of the subgrade forced changes to be made to the drainage control plan.

Alternatively, drainage was controlled by establishing a one-side cross slope (towards the southern side of the road) of approximately 4% gradient in both the subgrade and the base course surfaces to facilitate water runoff. A shallow ditch was excavated along the southern side of the test road to collect water drained from the road surface. Water drains from the ditch into two large sump holes excavated at the low areas near the roadside.
Collected water inside the sump holes is eventually pumped to a remote discharge area using two submersible sump pumps installed inside the sump holes.

**4.2 Field Trafficking**

Trafficking was performed on the test sections using a dual-wheel, 80 kN single-axle dump truck (Figure 4.15). All tires were inflated to a pressure of approximately 586 kPa. Before commencing any trafficking, the truck was loaded with concrete blocks to achieve the required standard load on the rear axle. The concrete blocks were positioned on the truck such that to ensure equal loads on the axle wheels. A calibrated scale was then used to weigh the truck, axles, and wheels. The rear axle load was 80 kN while the front axle load was 33 kN. Longitudinal lines were painted on the surface of the two road lanes using a road marker spry to guide the vehicle along predetermined wheel-paths passing over the instrument locations (Figure 4.14). The test sections in the two lanes were trafficked as a two-way road. The vehicle traveled across the test sections along the wheel-path in the same direction in a loop configuration at a speed of approximately 15 km/h. During trafficking, the vehicle was regularly filled with gas and weighed and the tire pressure was regularly checked to ensure that the traffic load remained constant throughout testing.

Trafficking was originally scheduled to commence in the spring when the shear strength of the subgrade surface is at its lowest value and it was planned to continue until an average “allowable” rut of 100 mm (a condition of serviceability failure) is developed in each of the test sections or until 2,000 traffic passes (which represents the design life of the road) is reached. However, the construction of the field test sections was delayed and, accordingly, trafficking did not commence until the end of the summer. The months of August and September, when trafficking was accomplished, were hot and dry and the subgrade surface was much stiffer than originally anticipated. Consequently, complete collapse or serviceability failure condition were not observed in the test sections after 1,600 passes of trafficking (due to the stiffer than expected subgrade conditions) except
for one area in section 1 which was attributed to a disturbance in the subgrade soil during the preparation of the subgrade surface.

Postponing of trafficking to the spring was not an option since the site was available for only a limited time. Therefore, it was decided to continue the field test by subjecting the test sections to additional passes of a full load axle in an attempt to increase the rate of deformation or generate complete collapse in the test sections. It was judged by the truck operator that the maximum load that the truck can carry safely is approximately 100 kN. Hence, two additional concrete blocks, approximately 1 ton each, were placed on the truck above the rear axle (Figure 4.16). The truck was then reweighed and the rear axle weighed 10,200 kg (approximately 100 kN axle load) while the front axle load was kept at approximately 33 kN. An additional 900 passes were then applied to the test sections with the 100 kN axle load before the unreinforced section reached serviceability failure, so the road was ultimately trafficked by a total of 2,500 passes.

During trafficking, it was difficult to maintain a clearly painted wheel-path on the surface of the dirt road which was essential to ensure accepted level of controlled traffic is achieved. Accordingly, wheel-paths were repeatedly repainted throughout the entire trafficking period (Figure 4.17). On the other hand, it was necessary to repair failed test sections (Section 1) during testing so that trafficking can continue on unfailed sections. Additional base course material was placed in the excessively rutted areas of the failed section and the entire area was flattened and compacted using a Bobcat compact track loader.
Figure 4.15: Vehicle used for trafficking

Figure 4.16: Adding of concrete blocks to increase axle load
4.3 Data Collection

Up to 2,500 traffic passes have been applied to the unpaved test sections. The test sections were instrumented with pressure cells, geogrid strain gauges, thermistors, and motion sensors so as to measure the response of the road to the traffic loading and monitor environmental changes during testing. Along with the instrument responses, measurements of base layer thickness, rut depth and surface deformation were performed in the field throughout trafficking.

The monitoring program was controlled by the DAQ system housed in the site office. A mobile high-speed internet connection was established at the site to communicate with the data acquisition controller through its static IP address. A research team member was always in attendance during testing activities to ensure full monitoring of testing and trafficking activities, immediate detection of possible errors and making necessary repairs, and observation of signs of distress in the test road surface.
4.3.1 Mechanical Response Data

All load monitoring instruments were installed under the left-side, outer wheel-path in the middle of each test section. The instruments were monitored using data acquisition system manufactured by National Instruments (Figure 4.18). The components of this data acquisition system were previously described in Chapter 3. All instrument cables were fitted with plug sockets to ensure proper and speedy connection to the DAQ system. Earth pressure cells and attached thermistors were connected to the PXI-6070E DAQ card through unshielded 68-Pin Screw Terminal, NI TBX-68. Attached geogrid strain gauges were connected to the SCXI-1520 modules via universal strain terminal blocks, SCXI-1314, following the proper bridge configuration. LabVIEW 14 was used to program and manage data collection by the DAQ system.

During the compaction of the base course layer, instrument responses to the construction activities were carefully monitored by the research team to detect any undesirable effects on the instruments by construction loads. Also, instrument readings were collected and recorded by the DAQ system to help characterize the response of the various unpaved test sections during construction. A baseline reading was recorded for all sensors prior to commencing any trafficking activities.

Data acquisition was triggered manually during traffic testing and data were sampled continuously at a sampling rate of 200 Hz to obtain enough data to describe the measured response by the instrument to the applied traffic load. Heath Zenith motion sensors were installed at the two sides of the road track near each instrument location (Figure 4.19). The motion sensor detected the vehicle as it approached the instrument location and generated a signal coinciding with the response signals recorded by the instruments. Accordingly, they were used to record the count of the number of vehicle passes as well as to identify response signals in order to minimize post-testing data processing effort.

Figure 4.20 shows the LabVIEW front panel during field trafficking. As demonstrated in the figure, the front panel allows monitoring of data collection during filed testing so that observations can be made and errors can be immediately detected. The responses of the geogrid strain gauges (top) and earth pressure cell (bottom) to the moving load of the test vehicle can be clearly seen in the display.
Figure 4.18: Data acquisition system at the test site

Figure 4.19: Motion sensor installed near an instrument location
4.3.2 Environmental or Static Data

When practically possible, environmental instruments were installed along the centerline of the lane, between the two wheel-paths. The static data (i.e., thermistor and dummy strain gauge measurements), were collected at large intervals since environmental readings are unlikely to change in a short period of time. As previously discussed, thermistors attached to the earth pressure cells were connected to the PXI-6070E DAQ card. The earth pressure cells were connected to the terminal block in differential or floating mode to minimize noise infiltration. Due to restrictions on the number of input channels available in the PXI-6070E DAQ card, it was only possible to read the eight pressure cells installed in the test sections using differential voltage measurement. In other words, no thermistor readings can be collected while connecting all pressure cells differentially. The main benefit of a differential measurement is noise rejection. Thus, to collect thermistor readings, ground referenced “single-ended” voltage measurement were used since this type of configuration allowed for the DAQ system to support more channels. However, the single-ended measurement was very susceptible to noise.
Accordingly, two of the earth pressure cells were connected in a single-ended mode when thermistor readings were collected at two different locations of the test sections.

4.3.3 Performance Evaluation Data

4.3.3.1 Measurement of Base Layer Thickness

Prior to the placement of geosynthetics and base course aggregate, the subgrade surface elevation was measured at a grid points established along the two wheel-paths. The coordinates of the grid points were surveyed and recorded using a total station so that each point could be easily relocated at all subsequent construction phases. The elevation of the subgrade was then compared to the elevation of the finished base layer surface after compaction to determine the actual initial base layer thickness.

In order to determine the final thickness of aggregate at the end of trafficking, the elevation of the base layer surface was measured after trafficking at the same grid points established earlier in the subgrade. Holes were then excavated in the base layer at the surveyed locations exposing the subgrade surface, and the elevation of the subgrade surface was measured at these locations. The final thickness of the base layer at each grid point was then calculated by comparing the results of the two measurements. The average measured base layer thicknesses of the test section are summarized Table 4.2.
Table 4.2: Summary of measured base layer thicknesses of the test sections

<table>
<thead>
<tr>
<th>Section</th>
<th>Geogrid type</th>
<th>Base course thickness (mm)</th>
<th>Design</th>
<th>Measured (average)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>G4, triangular aperture geogrid</td>
<td></td>
<td>200</td>
<td>208</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td></td>
<td>250</td>
<td>250</td>
</tr>
<tr>
<td>S3</td>
<td>G1, biaxial geogrid</td>
<td></td>
<td>200</td>
<td>203</td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td></td>
<td>250</td>
<td>235</td>
</tr>
<tr>
<td>S5</td>
<td>G3, biaxial geogrid</td>
<td></td>
<td>200</td>
<td>198</td>
</tr>
<tr>
<td>S6</td>
<td></td>
<td></td>
<td>250</td>
<td>180</td>
</tr>
<tr>
<td>S7</td>
<td>Control</td>
<td></td>
<td>200</td>
<td>208</td>
</tr>
<tr>
<td>S8</td>
<td></td>
<td></td>
<td>250</td>
<td>248</td>
</tr>
<tr>
<td>S9</td>
<td>G2, biaxial geogrid</td>
<td></td>
<td>200</td>
<td>205</td>
</tr>
<tr>
<td>S10</td>
<td></td>
<td></td>
<td>250</td>
<td>238</td>
</tr>
</tbody>
</table>

Note: All sections contain non-woven geotextile separator at the subgrade-base course interface.

4.3.3.2 Measurement of Rut and Surface Deformation

The respective performance of each test section would be evaluated based on rut measurements. Therefore, rut development under the wheels was monitored at selected traffic intervals throughout testing to record rutting as a function of number of passes. Measurements were made more frequently during earlier traffic passes and declined over time to capture the response to traffic loading at small deformations.

A straightedge was placed across the rut, transverse to the direction of traffic, and resting upon the road surface at two contact areas. A measuring tape was then placed between the two contact areas perpendicular to the straightedge and was used to measure the depth from the bottom surface of the straightedge to the road surface (Figure 4.21). Rut was measured in each section at five stations, 3 m-spaced in the road direction; three stations were on the left (outer) wheel-path and two stations were on the right (inner) wheel-path. The average of the five measurements within each section was recorded and considered as the rut depth for that section.
In addition, surface deformation under the wheels was measured at selected traffic intervals using a digital level. Measurements were taken at recoverable grid points established originally at the subgrade surface. A benchmark was established near the road and level differences between the grid points and the benchmark were recorded. The surface deformation at each point was calculated by comparing current readings at the specific traffic level to a baseline reading taken before trafficking. At each measurement station, the average of two surface deformation measurements, made at the center of the inner and outer wheel paths of the dual wheels, was recorded.

Figure 4.21: Measurement of rut depth

4.3.4 Damage Evaluation and Observational Data

In addition to the measurements of surface rutting, the test sections were regularly monitored during trafficking looking for signs of distress to the unpaved structure such as cracking or failure. The condition of the unpaved road inside and outside the wheel-path was visually assessed and photographs were regularly taken. As time permitted, forensic test pits were excavated in the test road to evaluate the condition of the base and subgrade
layers and to assess damage to the geosynthetics from trafficking. Additionally, earth pressure cells were carefully retrieved from the test sections and their integrity after trafficking was assessed. Instrumented geogrid areas were extracted from the test sections to visually inspect and assess the impact of traffic loading on the foil strain gauges and evaluate the attachment and protection methods used in this study. In addition, general survivability and functionality of the instruments during construction and trafficking activities was monitored and evaluated.

4.4 Noise Reduction and Data Processing

Data acquisition is a very critical part of any field testing program. Data acquisition uses sensors of different types that are suited to detect certain physical parameters and convert them to electrical signals. In most cases, the sensor output analog signal may need amplification, filtering, converting, and isolation to make it suitable for processing.

In addition, data initially collected must be processed or organized for further analysis. Analysis of data collected by any acquisition program can only be as good as the input data. Therefore, an effective and careful data-acquisition is necessary in order to provide high quality data.

4.4.1 Noise Reduction and Signal Filtering

Analog signals collected by the sensors are usually accompanied with electrical noise that is superimposed by the presence of electric and/or magnetic fields. Noise is any undesirable electrical signal, which distort or interfere with an original signal (signal of interest). Noise is present in all measurement systems to a certain degree. It can be generated from within the measuring device or system (internal noise) or from an outside source (external noise). Internal noise comes from internal sources such as semiconductors, resistors, and capacitors, while external noise comes from external sources such as AC power lines, motors, generators, vibrators, transformers, soldering irons, welders, radio transmitters. If not controlled, electrical noise can result in distortion
of the measured signal and misinterpretation of the results or, in extreme situations, can completely obscure the signal of interest.

Noise reduction, or noise filtering of the original signal, is a very essential step in signal processing. There are a number of techniques that can be used to control the noise level and reduce the noise picked up by a sensor:

- Avoiding ground loops.
- Cable shielding (conductive shield).
- Implementing twisted pair wiring.
- The use of shorter sensor leads.
- Amplification of the signals at the sensor.
- The use of software-based and hardware-based filters.

Sensors are typically designed to make the measured signal much larger than the noise that accompanies it. A parameter defined as the signal-to-noise ratio (SNR) is commonly used to compare the level of a desired signal to the level of background noise. High signal-to-noise ratio is particularly critical for strain gauges, which inherently have low signal levels that are close to the typical noise signals in electrical measurements.

In addition to the noise that may be introduced from the DAQ system and other external sources into the instruments, there was a concern in this study of noise interference because of using low resistance strain gauges. Accordingly, several measures were taken to control and reduce noise levels in the strain gauges and maximize measurement accuracy.

The selected strain gauges incorporate a feature of removing the excitation from the Wheatstone bridge so that the presence and magnitude of noise can be effectively evaluated. This provided a very useful means for assessing the effectiveness of shielding and grounding, and for making adjustments to control the noise.

The lead cables are commonly the main source of noise pickup in strain gages. They act as antennas and can pick up a variety of electrostatic and magnetic noise. Therefore, the
first provision in reducing noise was to use as short as possible lead cables between sensors and the DAQ system and keeping them away from electrical machinery.

As an additional means of noise control, the cable used with the strain gauges had its conductors twisted and the cable is shielded with conductive foil. Twisted conductors can help reduce electromagnetic coupling between excitation and signal pairs and reduce severe electromagnetic radiation and pickup. Foil shields give 100% cable coverage. The shield was connected to the source of common-mode voltage to minimize the voltage between signal conductors and the shield, which can provide very effective noise reduction.

The differential input terminals of the data acquisition system were used to reject ground-induced interference in the strain gauges. The differential signal connection reduced noise pickup and increased common-mode noise rejection. It also allowed input signals to float within the common-mode limits of the amplifier. A ground connection is usually made at either the amplifier input or the sensor, but not at both. When both devices are grounded, a ground loop current can flow between them. Ground loops are a major cause of noise and using differential inputs will help reduce this noise.

It is noteworthy that the strain gauges used in this study were 120-ohm gauges. There was no 350-ohm gauges available with the size that fits the geogrid rib bonding area and with the pattern that suits the arrangement adopted for gauge wiring. The higher-resistance gauge is preferred because it reduces lead wire effects such as circuit desensitization due to lead wire resistance. In order to increase measurement sensitivity and account for the signal lose due to using a low-resistance gauge, the strain gauges were hooked up in three-wire connection. Configuring the strain gauge input as a three-wire circuit reduced the loss in sensitivity due to lead wires and compensated for the effects of lead wire temperature changes on bridge balance.

Among all the measures taken, the utilization of low pass filter capabilities of the signal conditioners incorporated in the data acquisition hardware was the most important technique utilized to reduce the noise in the strain gauges. Filtering is a class of signal processing, through which unwanted components (frequencies or frequency bands) are removed from the original signal. SCXI-1520 modules were used with the strain gauges
to reduce noise associated with instrumentation and long lead wire lengths and improve the quality of strain measurement. The SCXI-1520 provided DC excitation voltage for the gauge and it had a special calibration feature that enabled LabVIEW to ground the module amplifier inputs so that the amplifier offset could be read. The SCXI-1520 provided two filtering stages and the amplifier had a frequency cutoff, which could be controlled through LabVIEW. Various low pass filter settings were tested using LabVIEW for greater suppression. A 5 Hz setting was eventually chosen in the low pass filter used for the strain gauge signals. It was observed that reducing the filter cutoff frequency below 5 Hz does not have any further effect. As a result of all the mitigation measures implemented, the generated signal from the strain gauges was relatively free of noise. The signal-to-noise ratio was fairly high and a clear response was produced from the gauges.

Similar precautions were taken to ensure clean and viable data are collected by the earth pressure cells. The pressure cells used in the field testing had a multi-conductor cable that is made of individual stranded conductors. The individual conductors are twisted into pairs and shielded inside a conductive Mylar-type shield, which helps minimize common-mode noise voltages. Shielding provides protection against picking up electromagnetic radiation from nearby electrical equipment, power lines, etc. Because of the mutual inductance between the shield and the inner conductors in the shielded, twisted-pair cable, the shield acts as a common-mode filter for signals above the shield cutoff frequency. It was ensured that the shield drain wire was connected to ground. The drain wire is connected electrically to Mylar-type shields to provide a simple means of connecting all the shields to a common ground.

In order to reduce noise infiltration, the pressure cells were connected to the terminal block using differential input mode instead of using single-ended input mode to minimize noise pickup and increase common-mode noise rejection. Additionally, the pressure cell cables were kept as short as possible to help reduce noise pickup. Lead cables were kept away from sources of interference and, as previously discussed, they were shielded inside a pipe conduit.
Despite the above discussed mitigations, noise infiltration was much more significant in the pressure cell signals as compared to the strain gauges. In order to effectively assess the presence and magnitude of noise, a frequency analysis was performed using fast Fourier transform algorithm in Matlab. It was used to process out noise in the baseline signal, show its nature, reveal the source of noise, and identify its frequency components.

Figure 4.22 shows the amplitude of baseline signal as a function of frequency, which is a common metric used in signal processing. It can be seen that there are some very specific frequencies that are contributing to the majority of the noise. The spikes in amplitude spectrum correspond to the signal's frequency components of 20 Hz and 60 Hz. It seems that the primary source of noise in the pressure cells was 60 Hz power line noise. The 20 Hz frequency generally represents the lower limit of audibility or low-frequency sound. Although a 20 Hz frequency signal could be generated by machinery such as diesel engines, wind turbines and specially designed mechanical transducers, it is believed that the 20 Hz signal appeared in the frequency domain of the baseline signal because of the insufficiency of the sampling rate to capture the entire 60 Hz power line signal.

Figure 4.22: Typical noise collected by earth pressure cells (Fourier transform of baseline signal)
It was necessary to use software-based filtering to eliminate the noise in the pressure cells signal. Using Matlab algorithm, a low-pass filter was applied to the signals to eliminate noise. It is a filter that passed signals with a frequency lower than a chosen cutoff frequency and attenuates signals with frequencies higher than the cutoff frequency. Various frequencies were investigated as a cutoff for the low pass filter in Matlab. A cutoff frequency of 5 Hz appeared to be the lowest frequency to effectively remove the noise and was accordingly selected and used to filter the signals of the pressure cells. Figure 4.23 displays a typical filtered signal for an earth pressure cell as the test vehicle travelled over the instrument location. The unfiltered signal for the same cell and the same vehicle pass is also shown in the figure for comparison purposes. It can be seen that the noise was successfully filtered. The signal-to-noise ratio in the figure is fairly high so that the response signals produced by the vehicle wheels are clearly distinct.

Figure 4.23: Typical response for an earth pressure before and after filtering
4.4.2 Separation of Traffic Response Data

As a result of the continuous acquisition of data employed in this study, a significant amount of data was collected by the sensors that had to be processed and reduced before it can be analyzed and interpreted. Data processing is the manipulation of data items to produce meaningful information for analysis. The collected data was managed by the LabVIEW program. The data was saved in separate files, each contains 15 to 20 traffic passes. Data from different sensors (i.e., strain gauges, pressure cells, motion sensors, and thermistors) were saved in separate files.

In order to expedite data processing, a Matlab algorithm was developed to process and format the mechanical response data generated by the LabVIEW program. The Matlab algorithm was implemented to call all data files, one at a time, and precisely identify for each sensor data points that correspond to each pass of traffic loading “event”. As mentioned beforehand, motion sensors were installed near every sensor location to approximately identify the points that corresponded to dynamic loading. The motion sensor sends a signal when the vehicle approaches the sensor location. These signals were recorded in the data files generated by LabVIEW. In order to detect events, a peak threshold for each sensor type was defined in the Matlab algorithm. The algorithm locates the signals recorded by the motion sensors and use these locations to search for events within each structural sensor. Once an event is found, sufficient points to describe instrument response to traffic pass (i.e., event window) was then saved in a Microsoft Excel file. Detailed information for the measured instrument response during each event (i.e., response to a traffic pass), including maximum or peak values (corresponded to maximum dynamic responses to front and rear axle loads) as well as baseline data points (corresponded to unloaded state/permanent strain), were saved in a separate file. The basic features of the algorithm are graphically represented as a flowchart in Figure 4.24.

In addition to the data files generated from LabVIEW, input parameters used in the algorithm include sampling rate, peak threshold (to detect event or response signal), event window (how many samples to be saved around each event), and event lead (how many samples before event to start event window). The processed data were categorized by sensor and pass number to facilitate the analysis and interpretation of the results.
Figure 4.24: Flowchart explains the Matlab algorithm used for data processing
4.5 Post-Testing Evaluation

4.5.1 Assessment of Vehicle Wander

Vehicle wandering is commonly observed in roadways. The influence of vehicle wander on the position and width of wheel-path is an important consideration in conducting field trafficking trials. However, wandering is rarely addressed in designed field traffic tests. While it is unlikely to be completely prevented in real traffic conditions, vehicle wander should be assessed and effective measures should be taken to reduce it and minimize its influence on the measured response of the test sections.

Since the early stages of the field works, design wheel-paths were established on the surface of the subgrade using surveying methods. Positions of field instrumentation were precisely located on the predetermined wheel-paths. The coordinates of the instrument locations and wheel-paths were saved and used at all subsequent construction and testing phases to ensure recovery of these locations on the road surface. In addition, the instrumented wheel-paths were clearly marked on the surface of the two road lanes using a road marker spry to guide the driver of the test vehicle along these predetermined wheel-paths passing over the instrument locations.

A simple procedure was used to give rough estimates of vehicle wander during field trafficking. Surveying measurements were performed after trafficking to relocate the centerline of the instrumented wheel-paths on the road surface. The actual width and position of the trafficked wheel-path (i.e., width of the tires footprint) at the end of testing was then measured at multiple locations of the test sections. Accordingly, wander of the test vehicle wheels from the instrumented wheel-path was determined.

The measurements showed that the average width of the area trafficked by the vehicle tires was approximately 680 mm extending approximately 340 mm on each side of the centerline of the instrumented wheel-path. Given that the test vehicle had an overall dual-wheels width of 560 mm, vehicle wander on either side of the wheel-path could be estimated. It appears that vehicle wander during field trafficking has been within a reasonable range.
It was noted that the exits of the test road lanes experienced greater vehicle wander that was characterized by vehicle shift towards one side of the lane more than the other. Within a distance of approximately 1.5 to 2 m from the road exits, the wheels shifted towards the lift- or the right-side edge of the road lane depending on the direction of the curved path the vehicle followed at each exit. There was not enough space at the road ends for the test vehicle to make a U-turn, so the vehicle traveled in a curved path once exiting the test road to access the specified space for turning around. A horizontal road curve (geometric characteristics of the road) is a significant factor in generating vehicle shift (Luo and Wang, 2013). The curved path along with the ramp at the lane exit compelled the driver to deviate the vehicle towards one side of the lane as he exits the test area. Nevertheless, the areas most affected by vehicle wander (i.e., 1.5 to 2 m from the road exits) had insignificant or no effect on the measured response of the test sections because no test measurements have been taken at these areas. The 14m-long test sections provided sufficient length to avoid collecting performance and mechanical response measurements near the ends of the test sections.

4.5.2 Assessment of Unpaved Road Distress

Visual assessment has been performed on the test sections at various times since construction and photographs were taken looking for signs of distress other than the measured surface rutting. Failure was observed in part of Section 1 almost from the onset of trafficking (within the first 40 passes). The surface rutting at this location rapidly exceeded the allowable value as shown in Figure 4.25. The author recollected an event that may have caused the failure in Section 1. The subgrade soil was disturbed at the same location of the failure during the preparation of the subgrade surface. The rutted area was filled in with base course material so that trafficking could continue on the unfailed test sections.

Visual inspection of the test road indicated that closely spaced transverse cracks appeared in the surface of the aggregate layer in the wheel-path (Figure 4.26). In some cases, however, these transverse cracks were connected through thin longitudinal cracks. This interlaced cracking pattern resembles alligator cracking or fatigue cracks that normally
appears in flexible pavements and that is caused by load-related deterioration resulting from a weakened base or subgrade layer. Limited amount of longitudinal cracks of larger lengths were also observed right outside the wheel-path near the edge between the two lanes (Figure 4.27). It was possible to see a movement in the base layer surface in the areas around these localized cracks when the vehicle travels over.

Figure 4.25: Failure in part of Section 1 at an earlier stage of trafficking
Figure 4.26: Cracks in the unpaved road (in the wheel-path)

Figure 4.27: Longitudinal crack outside the wheel-path
4.5.3 Forensic Investigation

Unfortunately, the time available for conducting post-test investigation was very limited due to out of control circumstances that required handing the test site over to the owner. However, limited excavation work and observational activities were conducted within the available time frame. Forensic test pits were excavated in the wheel-paths and were restricted in areas that had experienced excessive surface rutting in addition to the instrument locations.

4.5.3.1 Separation and Interlocking

No evidence of migration of subgrade soil particles to the base course layer was noted in the test pits. This indicates that all benefits of the geosynthetics were due to reinforcement. Post-trafficking excavations of the geogrid-reinforced test sections revealed signs of interlocking between the geogrid and aggregate particles. Large individual aggregate particles from the base layer were seen to strike through the apertures of the geogrid. It was noted that aggregate particles did not strike through the geogrid apertures near the instrumented ribs. The protective sand layer prevented the aggregate from penetrating the instrumented apertures, which is exactly the purpose of placing this layer on top of the strain gauges. Although this might have slight local effect on the interaction between the geogrid and the aggregate, the benefit of protecting the foil strain gauges are much more significant than any potential underestimation of geogrid strain. In addition, although interlocking is generated in a small scale within the individual apertures, its effect extends in the large or global scale. Therefore, the instrumented geogrid ribs would be influenced by the global effect of interlocking even though no aggregate particles have penetrated the instrumented apertures.

4.5.3.2 Damage Assessment of Geogrids

Forensic investigation was conducted on the test site to evaluate damage to the geogrids from trafficking. The majority of the damage in the geogrids was in the junctions and the geogrid ribs in the cross-machine direction aligned with the transverse direction (i.e.
perpendicular to the direction of traffic). The geogrid ruptured along a line perpendicular to the traffic direction and generally through the middle of a junction. Cracks propagated along a line of geogrid nodes or junctions and through the middle of the transverse (cross-machine) ribs connected to the nodes, splitting the ribs in half lengthwise (Figure 4.28). The geogrid ribs in the transverse direction were generally cracked along their length but not broken across their width.

Rupture was also observed in few cases through the edge of the junction (Figure 4.29). The rupture line propagated through the connections between the nodes and the longitudinal ribs (i.e., ribs in the machine direction) with relatively intact transverse ribs. In rare occasions, a secondary crack line was seen to propagate from the damaged node through one of the longitudinal ribs connected to the node in addition to the primary crack extending through the two adjacent transverse ribs (Figure 4.30).

It was also noted that longitudinal geogrid ribs near the damaged nodes were generally deformed or buckled. It appears that the longitudinal ribs buckled under compression load and were bearing on the nodes. The nodes were ultimately failed under the bearing from the longitudinal ribs in a plane perpendicular to the direction of the applied compression load. It should be noted that local rupture occurred in the geogrid at strain levels lower than the ultimate strain measured in the laboratory. As will be demonstrated by strain measurements in Chapter 5, recorded geogrid strains were generally less than 1%. Little (1993) observed that local rupture can occur at any weak points in the geogrid material in the field at a lower strain level. As concluded from forensic observations, geogrid rupture is believed to be occurring under compression loading, which geogrid materials cannot sustain. This could explain why the rupture occurred in the geogrid while the measured tensile strains are very low compared with the ultimate values measured in laboratory tests.
Figure 4.28: Rupture through the middle of geogrid nodes and splitting of transverse (cross-machine) ribs along their length

Figure 4.29: Rupture through the edge of the geogrid nodes
4.5.4 Field Survivability of Instruments

The instrument survivability is a significant reason for concern in any instrumented full-scale field testing, especially in instrumented field test sections subjected to traffic loading. Detailed procedures for field installation and protection were implemented for each instrument in this study to ensure it survives the construction and traffic loads. Most of the reported sensor damage in the previous field tests took place during construction. Therefore, in addition to following strict installation guidelines, numerous precautions were taken during construction and modifications were made to maximize instrument survivability. Instruments were carefully monitored during the compaction of the base layer. Changes in instrument readings during construction were recorded and analyzed. At the end of field testing, instrument survivability was assessed to evaluate instrument installation techniques and assess subsequent instrument field performance. The instrument survivability was evaluated based on signal presence, signal stability, magnitude and shape of response to dynamic loads, and presence of anomalies or outliers in the instrument response.
4.5.4.1 Field Survivability of Foil Strain Gauges

Field installation of foil strain gauges presents several challenges due to their delicate nature and the harsh environmental conditions in the field and the involved risk of mechanical damage. A wide variety of techniques have been used in previous research studies to protect strain gauges in field testing environment (Brandon et al., 1996; Perkins and Lapeyre, 1996; Helstrom et al., 2006; Warren et al. 2010; Morris, 2013). Despite the efforts made to improve survivability of strain gauges under field conditions, very low gauge survivability rates have been reported. Brandon et al. (1996) reported foil strain gauge survivability rates between 6 and 28% from their field testing. Foil strain gauge survivability rates of 32 and 35% were achieved during field tests conducted by Morris (2013) and Helstrom et al. (2006), respectively. A good foil strain gauge post-construction survivability rate of 81% was reported by Warren and Howard (2007) from their full-scale field testing; however, none of the gauges recorded any meaningful data during testing.

In this study, available protective techniques were carefully reviewed in an attempt to develop a comprehensive and more efficient approach to install and protect foil strain gauges in a full-scale roadway traffic testing (Chapter 3). The goal was to achieve high gauge field survivability while obtaining good quality measurements that can help describe the geogrid response. Table 4.3 summarizes the results of the survivability investigation for each strain gauge installed in the geogrids. A strain gauge was considered survived a construction or trafficking phase and denoted “functional” if it provided a stable signal during the entire time period of that phase. A strain gauge would be considered “unfunctional” if it failed to provide a viable signal. Loss of signal can result from mechanical damage from aggregate particles, electrical malfunction due to water intrusion or damage to the wiring, or gauge detachment. If the shape and magnitude of the response signal conformed to the normal, expected behavior, the data recorded by the gauge was considered “reliable”.

The data in Table 4.3 showed that 100% of the attached foil strain gauges survived the construction and field trafficking and provided a viable signal. 82% of the gauges
provided good quality and reliable data throughout the entire construction and testing period.

### Table 4.3: Summary of foil strain gauge survivability

<table>
<thead>
<tr>
<th>Gauge number</th>
<th>Test section</th>
<th>End of construction</th>
<th>End of trafficking Phase 1</th>
<th>End of trafficking Phase 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>M3</td>
<td>S3</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
<td>X3-1</td>
<td>S3</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
<td>X3-2</td>
<td>S3</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
<td>XB3</td>
<td>S3</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
<td>M4</td>
<td>S4</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
<td>X4</td>
<td>S4</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
<td>M5</td>
<td>S5</td>
<td>F</td>
<td>F-RR</td>
<td>F-UR (1993 passes)</td>
</tr>
<tr>
<td>X5-1</td>
<td>S5</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
<td>X5-2</td>
<td>S5</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
<td>XB5</td>
<td>S5</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
<td>M6</td>
<td>S6</td>
<td>F</td>
<td>F-RR</td>
<td>F-UR (2005 passes)</td>
</tr>
<tr>
<td>X6</td>
<td>S6</td>
<td>F</td>
<td>F-UR (297 passes)</td>
<td>F-UR</td>
</tr>
<tr>
<td>M9</td>
<td>S9</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
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<tr>
<td>X9</td>
<td>S9</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
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<td>S9</td>
<td>F</td>
<td>F-RR</td>
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<td>F-RR</td>
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<tr>
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<td>S10</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
</tbody>
</table>

F = Functional or provide viable signal; UF = Unfunctional (no signal)
RR = Reliable Readings; UR = Unreliable Readings

Readings from the foil strain gauge installed in the cross-machine direction of the geogrid in Section 6 were not considered reliable after approximately 300 passes of field trafficking. Although the gauge continued to provide viable signal until the end of testing, reliability of the readings was questionable. Stable signal appeared to be received from the strain gauge during unloaded state; however, abnormally high signals were recorded
under the applied wheel loads (i.e., exceeding the strain limit of the gauge). It was noted that localized rutting started to develop near the instrument location in Section 6. Therefore, it is believed that the unreasonable response signal was induced due to a soft-spot existed in the subgrade. Two other strain gauges showed similar but less drastic response towards the end of testing. This response was associated with heavy precipitation, which possibly led to a reduction of the subgrade shear strength. It is noteworthy that the transient response of these gauges to the applied loads appeared to be typical in shape; the magnitude of the response was unreasonable though. Accordingly, the affected gauges were considered functional but their readings were deemed unreliable for analyzing the mechanical response of geogrids.

In order to assess the integrity of the foil strain gauges after trafficking, all gauges were carefully extracted from the test sections during post-test excavations. A geogrid area of approximately foot by foot was cut around each strain gauge installed in the field and the instrumented geogrids were carefully transported to the laboratory for further inspection. The gauges remained functional even after the extraction process and provided vital signal when they were tested in the laboratory. This demonstrates the durability and integrity of the gauges and the efficiency of the attachment and protection techniques used in this study. The gauges, especially those that showed unreasonable response, were dissected and visually inspected for signs of mechanical and/or moisture damage. Visual inspection indicated that the gauges were in good condition. No signs of mechanical damage, damage in gauge wiring, or water intrusion has been found. This supports the hypothesis that the high spikes that were recorded under the dynamic loads were due to weak subgrade conditions near the gauge location rather than due to gauge detachment, water intrusion, or wiring issues. Accordingly, the foil strain gauges demonstrated good reliability and durability under harsh environmental and construction conditions, and traffic loads and provided an excellent means of measuring geogrid response in the field. These results indicate the attachment and protection techniques described in this study resulted in high instrument survivability rate in the full-scale field testing.
4.5.4.2 Field Survivability of Earth Pressure Cells

The field performance of the earth pressure cells is summarized in Table 4.4. The data shows that the installation and protection procedures used in this study resulted in field survivability rate of 100%. All earth pressure cells installed in the subgrade operated properly during the construction and trafficking phases. The cells provided viable signal and ideal response to traffic loads and they were generally seen to work reasonably well and give comparable results. The incorporated thermistors survived the construction and trafficking as well and provided sensible readings of the temperature of the unpaved structure.

It is worth noting that readings from the pressure cells in Sections 1 and 6 were slightly higher than expected due to reasons not related to the field performance of the cells since both cells produced viable and meaningful signal throughout the test. An early failure took place at Section 1 near the cell location. It appeared that the cell readings were influenced by the large rutting and the subsequent repair works that were performed in the area. On the other hand, the thickness of the aggregate layer above the cell in Section 6 didn’t meet the requirement of the design thickness, which resulted in recording greater stresses. The measured stresses from cells 1 and 6 were reasonable for the given conditions of these test sections but were not suitable for use in comparing the performance of the test sections with other test sections due to the difference in testing conditions.

The pressure cells were retrieved from the test sections at the end of testing for examination. It was found that the measures taken to protect the cells and the cables were successful in protecting the cells and cables against mechanical and moisture damage. Readings from the earth pressure cells under no load conditions were recorded in the laboratory and compared with zero readings measured before the field testing. It was clear that the impact of trafficking on the cells was insignificant. The excellent survivability rate achieved in this study demonstrate that the earth pressure cells selected to measure the stress response, and the installation and protection practices used resulted in reliable data.
Table 4.4: Status of the earth pressure cells during field testing

<table>
<thead>
<tr>
<th>Gauge number</th>
<th>Test section</th>
<th>End of construction</th>
<th>End of trafficking Phase 1</th>
<th>End of trafficking Phase 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>EPC1</td>
<td>S1</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
<td>EPC3</td>
<td>S3</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
<td>EPC4</td>
<td>S4</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
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<td>EPC5</td>
<td>S5</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
<td>EPC6</td>
<td>S6</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
<td>EPC7</td>
<td>S7</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
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<td>EPC8</td>
<td>S8</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
<tr>
<td>EPC9</td>
<td>S9</td>
<td>F</td>
<td>F-RR</td>
<td>F-RR</td>
</tr>
</tbody>
</table>

F = Functional or provide viable signal; UF = Unfunctional (no signal)
RR = Reliable Readings; UR = Unreliable Readings

4.6 Summary

A detailed description of the full-scale field testing program conducted to study the behavior of geogrid-reinforced unpaved road test sections under traffic loading was presented. The various stages of the construction of the instrumented field test sections were thoroughly discussed and methods used to install and protect the instrumentation in the field were detailed. The traffic loading scheme of the test sections was described and the procedures used to collect, manage, and process the various types of field data were discussed.

Examination of instrument survivability provided information regarding the effectiveness of installation and protection methods of the instrumentation. The installation and protection practices used in this study resulted in achieving excellent instrument survivability rates. 100% of the foil strain gauges and earth pressure cells survived the construction and field trafficking and provided a viable signal. Out of the 17 active strain gauges installed, 82% provided good quality and reliable data throughout the entire construction and testing period. The results of the survivability investigation showed that the foil strain gauges can provide an excellent means of measuring geogrid response to
traffic loads. Although the installation of foil strain gauges in harsh field conditions requires great skill and experience and involves laborious and elaborate procedure, it was demonstrated that proper design, attachment, and protection can result in extremely stable and reliable measurements.

Post-testing excavations at the test site revealed signs of interlocking between the geogrid and aggregate particles. The forensic investigation to assess damage to the geogrids from trafficking indicated that the geogrid ruptured along a line perpendicular to the traffic direction and through the middle (or less frequently through the edge) of a node. The transverse (cross-machine) ribs connected to the damaged nodes, were split along their length. It was also observed that the longitudinal geogrid ribs experienced some mode of buckling most likely under compression load.
Chapter 5

5 RESULTS OF FIELD EXPERIMENT

This chapter presents the results of the field traffic testing on the unpaved road test sections. Measurements of rut depth and surface deformation at selected traffic levels are presented and discussed. In addition, measurements of the mechanical response of the unpaved structure to the applied traffic loads recorded by instruments installed in the road layers are presented, including vertical stresses on top of the subgrade and strains in the geogrids. Instrumentation measurements collected during construction are discussed as well.

5.1 Performance of Test Sections

5.1.1 Surface Rutting under Traffic Loading

The performance of the unpaved test sections under traffic loading is described in this section based on rut depth and surface deformation measurements. Surface deformation, usually referred to as “elevation rut”, was determined by comparing measurements at selected traffic passes to a baseline measurement before trafficking, while rut depth, usually referred to as “apparent rut”, included both the surface deformation and the upheaval adjacent to the wheel-path. A rut depth of 100 mm was used to define the failure of the unpaved test sections in this field experiment. The rut depth and surface deformation for the thin-base test sections are presented in Figure 5.1 and Figure 5.2, respectively, as a function of cumulative traffic passes. Section 1 is not shown on these figures because of an early failure of the section within the first 40 passes. The premature failure that occurred in Section 1 was attributed to a slight disturbance to the subgrade soil in part of the section that took place during the preparation and grading of the subgrade. Due to this difference in subgrade condition, Section 1 cannot be compared with the other sections and was consequently excluded from the subsequent analysis.
It is observed that, during the first 100 passes of trafficking, all 200 mm-base sections experienced a rapid increase in both rut depth and surface deformation. This initial response seems to be less dependent on reinforcement type and is likely caused by further densification of the base layer in response to trafficking. Similar observations were reported by Fannin and Sigurdsson (1996) and Tang et al. (2015). With further trafficking, the response was characterized by a slight increase in rutting and surface
deformation up to the end of the first phase of loading (approximately 1,600 passes). In the second phase of traffic loading, all test sections exhibited rapid increase in rut depth and surface deformation as shown in Figure 5.1 and Figure 5.2, as a result of the larger applied traffic loads. This increase in rutting implies further deterioration of the base layer under repeated traffic loading.

It should be noted that response curves with steeper slopes indicate poorer performance, while response curves having more gentle slopes indicate better performance. The unreinforced section, Sections 7, showed greater rut and surface deformation than the reinforced sections under the same number of traffic passes. The response of Section 7 indicates even more rapid deterioration compared with the reinforced sections in the second phase of traffic loading. Failure, defined as 100 mm of rut depth, occurred in the unreinforced test section at 2,500 truck passes. None of the reinforced sections reached the 100-mm failure criterion after 2,500 traffic passes. These results demonstrate the significant benefits for geogrids in improving the performance of unpaved roads and indicate that geogrid reinforcing action was mobilized.

Section 5 that was reinforced with the heavy-duty geogrid (G3) showed less amounts of rut and surface deformation compared to Sections 3 and 9 that were reinforced with light-duty (G1) and medium-duty (G2) biaxial geogrids. This response is more noticeable in the surface deformation measurements (Figure 5.2). The heavy-duty geogrid also showed a slower rate of increase in surface rutting than the other biaxial geogrids and the unreinforced section, specifically in the second phase of trafficking. On the other hand, Sections 3 and 9 showed similar performance in terms of both rut depth and surface deformation. This is attributed to the comparable stiffness values of geogrids G1 and G2. In addition, should the subgrade soil was weaker, benefits of the two geogrids would have been more distinguishable. Section 9, however, exhibited slightly improved performance at the end of the testing compared with Section 3. The rut and surface deformation results suggest that all the reinforced thin layers performed better than the unreinforced layer. Rut depth at 2,500 traffic passes was reduced by 25% (G1), 26% (G2), and 32% (G3) relative to the unreinforced section. On the other hand, surface deformation at 2,500 traffic passes was reduced by 21% (G1), 24% (G2), and 31% (G3) relative to the unreinforced section.
Variations of rut depth and surface deformation with the number of passes for the thick-base test sections are presented in Figure 5.3 and Figure 5.4, respectively. It should be noted that Section 6, reinforced with geogrid G3, was constructed with an average base layer thickness that does not meet the thickness requirement for the thick-base sections (Table 4.2). Data from Section 6 was consequently deemed unreliable and was excluded from the discussion. Overall, the thick-base sections showed lower rut depths and surface deformations compared with the corresponding thin-base sections. The magnitude and rate of increase of surface rutting were less for the thick-base sections compared with their thin-base counterparts, leading to a less reduction in surface deformation due to geogrid inclusion. Geogrid reinforcements resulted in only marginal performance improvement with the thick-base sections at larger ruts under the highest traffic levels performed. Furthermore, the results from the thick-base sections were inconclusive regarding the respective performance of the various geogrids tested. This similar response of the different geogrids is likely due to the role of the thicker base in minimizing the reinforcing effect of the geogrids. Less stress is transferred to the subgrade as the thickness of the base increases, and the influence of geogrid becomes less significant. This is consistent with observations by Fannin and Sigurdsson (1996) and Hufenus et al. (2006).

![Average rut depth versus number of passes (250 mm-base sections)](image)

**Figure 5.3: Average rut depth versus number of passes (250 mm-base sections)**
Overall, the rut depths observed in all test sections were shallower than expected. Based on a preliminary estimate of subgrade strength at the spring, test sections were expected to reach a serviceability failure (rut of 100 mm) by achieving 2,000 passes (design life). The comparatively stronger subgrade at the time of trafficking during the summer, as opposed to the softer crust in the spring on which the design was originally based contributed to the shallower ruts. Furthermore, it seems that the impact of installing a separator under the geogrids was underrated. The authors believe that the separation of the base course material from the subgrade has significantly reduced the deterioration of the base layer and has consequently reduced rutting and increased the service life of the road. In addition, placement of the geogrid immediately over the geotextile separator might have reduced interlocking between geogrids and aggregate particularly at the relatively low number of passes achieved. This could have reduced the role of geogrids and therefore caused different sections to behave somewhat similarly at the lower levels of loading. Similar observation was reported by Christopher and Schwarz (2010). With hindsight, the use of the separator should be reconsidered. In order to maintain optimal interlocking between aggregate and geogrids, the authors recommend placing the geogrids directly on the subgrade without a separator. Geogrids with an adequate size can provide some degree of separation by preventing penetration of aggregate particles into

**Figure 5.4: Average surface deformation versus number of passes (250 mm-base sections)**
the subgrade (Maxwell et al., 2005; Holtz et al., 2008) and/or reducing migration of subgrade fines into the aggregate layer through holding it together (Giroud, 2009). However, Al-Qadi et al. (1994) observed mixing of the subgrade and aggregate in geogrid-reinforced sections with a silty sand subgrade. Alternatively, if mixing of aggregate and subgrade fines is a big concern, a thin layer of base course material can be placed between the geogrids and the geotextile separator so that aggregate strike-through can occur as suggested by Hufenus et al. (2006).

5.1.2 Permanent Deformation in the Base Layer

The permanent deformation of the base layer at the end of trafficking for each test section was estimated from surveying measurements on the deformed surfaces of the base and subgrade layers. The base layer surface elevation was measured after trafficking at pre-established grid points along the wheel-path. Holes were then carefully excavated in the base layer at the surveyed points exposing the deformed subgrade surface, and the subgrade surface elevation was measured at the same points along the wheel-path. The two sets of measurements were then compared to estimate the average thickness of the base layer for each test section at the end of trafficking. For each test section, the permanent deformation of the base layer at the end of trafficking was calculated by subtracting the thickness of the layer at the end of trafficking from the as-constructed base layer thickness. Table 5.1 summarizes the permanent deformation of the base layer at the end of trafficking for each test section. Overall, the test sections reinforced with geogrids showed lower permanent deformation in the base layer as compared with the control sections, except for section 6 due to its significantly smaller as-constructed base layer thickness.
<table>
<thead>
<tr>
<th>Test Section</th>
<th>Test lane</th>
<th>Base layer thickness (mm)</th>
<th>Deformation of the base layer (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>As-constructed</td>
<td>End of testing</td>
</tr>
<tr>
<td>S1</td>
<td></td>
<td>208.0</td>
<td>-</td>
</tr>
<tr>
<td>S3</td>
<td>North lane</td>
<td>202.8</td>
<td>178.0</td>
</tr>
<tr>
<td>S5</td>
<td>North lane</td>
<td>198.0</td>
<td>175.5</td>
</tr>
<tr>
<td>S7</td>
<td></td>
<td>207.9</td>
<td>179.4</td>
</tr>
<tr>
<td>S9</td>
<td></td>
<td>205.0</td>
<td>181.0</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>250.0</td>
<td>228.8</td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td>235.4</td>
<td>214.3</td>
</tr>
<tr>
<td>S6</td>
<td>South lane</td>
<td>180.3</td>
<td>155.8</td>
</tr>
<tr>
<td>S8</td>
<td></td>
<td>248.2</td>
<td>226.1</td>
</tr>
<tr>
<td>S10</td>
<td></td>
<td>238.0</td>
<td>217.1</td>
</tr>
</tbody>
</table>

### Table 5.1: Average permanent deformation of the base layer at the end of trafficking

#### 5.1.3 Contribution of Individual Layers to Surface Rutting

There is a lack of information from full-scale field testing studies in the literature regarding the relative contributions of the base and subgrade layers to the total permanent deformation of geosynthetic-reinforced roads. In this study, the subgrade layer was not instrumented to measure the permanent deformation during trafficking. In absence of measurements of subgrade deformation, the deformation of the subgrade at the end of trafficking was estimated by subtracting the base layer deformation determined in the previous section from the total surface deformation reported in Section 5.1.1. Based on the measured and estimated permanent deformations of the base and subgrade layers, their relative contributions to the total permanent deformation were estimated and listed in Table 5.2 as percentages of the total permanent deformation. In general, the data in Table 5.2 shows that the base layer contributes more to the total permanent deformation of the unpaved test sections in both the North and South lanes, as compared with the subgrade layer. On average, the contribution to the total surface rutting from the
individual layers of the unpaved road was 66% from the base layer and 34% from the subgrade.

It can also be seen that the base of the reinforced test sections displayed greater contribution to the total permanent deformation compared to the base of the corresponding unreinforced sections. This response is more pronounced in the test sections with thinner base layer (i.e., North lane). It should be noted that although the geogrids led to more contribution by the base layer to the total permanent deformation, they reduced the amount of permanent deformation in the base layer as previously indicated in Table 5.1. In contrast to the base layer, the subgrade of the reinforced sections showed less contribution to total permanent deformation as compared with the subgrade of the corresponding unreinforced sections. The reduction in subgrade contribution to total permanent deformation due to the presence of geogrids was more notable in the thinner sections (i.e., North lane).

**Table 5.2: Layer contribution to total permanent deformation (surface rutting)**

<table>
<thead>
<tr>
<th>Test Section</th>
<th>Test lane</th>
<th>Percentage of base deformation (%)</th>
<th>Percentage of subgrade deformation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>S3</td>
<td>North lane</td>
<td>62.6</td>
<td>37.4</td>
</tr>
<tr>
<td>S5</td>
<td>North lane</td>
<td>65.2</td>
<td>34.8</td>
</tr>
<tr>
<td>S7</td>
<td></td>
<td>57.0</td>
<td>43.0</td>
</tr>
<tr>
<td>S9</td>
<td></td>
<td>63.2</td>
<td>36.8</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>73.1</td>
<td>26.9</td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td>71.5</td>
<td>28.5</td>
</tr>
<tr>
<td>S6</td>
<td>South lane</td>
<td>64.1</td>
<td>35.9</td>
</tr>
<tr>
<td>S8</td>
<td></td>
<td>67.6</td>
<td>32.4</td>
</tr>
<tr>
<td>S10</td>
<td></td>
<td>70.8</td>
<td>29.2</td>
</tr>
</tbody>
</table>

From another perspective, the data presented in Table 5.2 indicates that the increase in base layer thickness has similar effect on the permanent deformation of the individual
layers of the unpaved road as that of adding a geogrid. By comparing the test sections with different base course thicknesses and same reinforcement type, it can be seen that the contribution from the base layer to the total permanent deformation increased, whereas the contribution from the subgrade decreased with the increase of base course thickness. Section 6 is an exception due to its smaller as-constructed base layer thickness as previously explained.

5.2 Temperature

The road temperature was monitored throughout the field traffic testing using thermistor sensors built into the earth pressure cells. Measurements from the thermistors were collected by the data acquisition system. These temperature measurements were used to assess variations in the ambient temperature of the geogrids placed directly on top of the subgrade. As can be seen in Figure 5.5, both measurement locations showed similar temperature trends. The difference in temperatures between the two locations is insignificant and varies between 0.5 and 1°C. Overall, during the period of trafficking, it was found that the road temperature remained relatively constant at approximately 21.5±2°C. Based on these temperature measurements, the effect of the temperature variation on the response of the polyethylene geogrids is deemed negligible.

Figure 5.5: Temperature measurements during trafficking
5.3 Vertical Stress at the Top of the Subgrade

5.3.1 Dynamic Vertical Stresses during Traffic Loading

Figure 5.6 depicts a typical filtered response of an earth pressure cell installed at the top of the subgrade as the truck travelled over the instrument location. Two peaks are observed under the trafficking pass due to the influence of front and rear axles. Figure 5.7 shows the dynamic subgrade vertical stresses as a function of cumulative truck passes for the test sections with a 200 mm base. The dynamic vertical stress for each pass was defined as the difference between the maximum stress measured under the applied wheel load and the stress recorded at the beginning of the event sampling period under no truck load condition. Data recorded by the pressure cell in Section 1 was believed to be affected by the repairs performed in the section at an early stage of testing. Consequently, vertical stress response of Section 1 cannot be discussed in this section even though the cell itself was functioning for the entire testing period. Rapid decrease in dynamic stress is observed for all sections in Figure 5.7 at the onset of trafficking that is attributed to increase in stiffness of the base course layer overlying the subgrade. Further densification of the base layer (rearrangement of granular aggregates into a denser and tighter layer) can result in an increase of its stiffness. The dynamic stress increased afterward indicating a possible degradation of the base layer, before it eventually stabilized and remained constant with cumulative traffic passes until the end of the first phase of testing. The influence of increasing the axle load in the second phase of testing (at about 1600 passes) on dynamic stress is evident in the results as shown in Figure 5.7. It is noted, however, that the increase in dynamic vertical stress at the top of the subgrade when changing traffic loading from 80kN to 100kN was less than what might have been expected. The stresses did not increase linearly with the increase in axle load. The 25% increase in axle load has led to an average increase of 11% in dynamic stress at the top of subgrade. As a result of the heavier traffic loads in the second phase of loading, the vertical stress continued to increase gradually (except for Section 5), a sign of further degradation of the base layer. The subgrade experienced another rapid increase in vertical stress at about 2,000 passes that is directly linked to heavy precipitation, which possibly led to a reduction of the subgrade shear strength.
Figure 5.6: Typical, filtered response of an earth pressure cell to truck loading

Figure 5.7: Subgrade dynamic vertical stress during traffic passes (200 mm-base sections)

The data in Figure 5.7 shows that the dynamic vertical stress transferred to the subgrade was reduced due to the presence of geogrids. It can be seen in Figure 5.7 that lower
dynamic vertical stresses were recorded in the reinforced sections as compared to the unreinforced section. Figure 5.8 shows the dynamic stress response at the top of subgrade for one pass of the truck’s rear axle at the 1,500-pass level. The reduction in dynamic subgrade vertical stress by the inclusion of geogrids is clearly demonstrated in this figure as well. This behavior suggests less degradation of the base layer for the reinforced sections with the repeated truck loading because of geogrid inclusion. Greater degradation of the base layer leads to greater magnitudes of vertical stress transferred from the base to the subgrade. In Figure 5.7, the magnitudes of dynamic stress for the reinforced sections are seen to increase at a lower rate as compared to the sharp increase in stress seen in the unreinforced section. This behavior demonstrates different rates of degradation of the base layer between the unreinforced and reinforced sections which supports the above suggestion. In addition, lower magnitudes and lower rates of increase of dynamic stress for the reinforced sections, as presented in Figure 5.7, may also indicate greater aggregate compaction for these sections during both construction and initial trafficking stage. As the base layer densifies, its stiffness increases, resulting in lower vertical stress transferred to the subgrade. This effect is demonstrated by the lower vertical stresses recorded in the reinforced sections (especially, Section 5) in comparison with the unreinforced section at the beginning of the test (end of compaction). The reduction in subgrade vertical stress due to the inclusion of geogrids is in general agreement with the rutting performance of the test sections discussed in Section 5.1.1.

The results also show that the reduction in subgrade vertical stress became more obvious when a heavier duty geogrid was included. Figure 5.7 shows that Sections 5 (heavy-duty geogrid) is clearly set off from the unreinforced section and the other reinforced sections. On the other hand, the difference between section 3 and 9 (light- and medium-duty geogrids) is almost indistinguishable for most of the testing period as a result of their comparable stiffness values. Difference in magnitudes of the peak subgrade vertical stress among test sections reinforced with different geogrids is also shown in Figure 5.8.
Figure 5.8: Subgrade vertical stress response for one pass of rear axle at the 1,500-pass level (200 mm-base sections)

The dynamic subgrade vertical stresses measured in the 250 mm-base sections are presented in Figure 5.9 as a function of cumulative truck passes. The stresses were generally lower than those recorded in the corresponding 200 mm-base sections. The results of Section 6, reinforced with the heavy-duty geogrid, were not included in the figure because it was constructed with an average base layer thickness that did not meet the thickness requirement for the 250 mm-base sections. Consequently, the limited data presented in Figure 5.9 provides little information on the effect of geogrids on the amount of vertical stress transmitted to the subgrade in the 250 mm-base sections. However, the general trend in Figure 5.9 is similar to that of the 200 mm-base sections.

Lower stress magnitudes were recorded in the 250 mm-base sections as compared to the 200 mm-base sections. With the increase of base layer thickness, the vertical stresses at the top of subgrade were reduced in both the reinforced and unreinforced sections, as expected. Figure 5.9 also shows that, under the low axle load, the dynamic vertical stress in the 250 mm-base sections did not increase by much with the increase of traffic passes (compared to the 200 mm-base sections). However, the increase in vertical stress with cumulative traffic passes was more evident under the higher axle load. This observation indicates more degradation of the base layer under the high axle load. With respect to the
effect of geogrid, the data shows no clear difference between the reinforced and the unreinforced sections under the low axle load. The similar response is likely due to the role of the relatively thicker base in minimizing the reinforcing effect of geogrids. Under the high axle load, nevertheless, the data shows a moderate reduction in dynamic vertical stress by the inclusion of geogrids. This reduction in stress was less significant compared to that for the 200 mm-base sections, however. It appears that the effect of the geogrid decreased by the increase of base layer thickness, perhaps because of the less stress transferred to the subgrade, as indicated above. This result is consistent with observations by Sun et al. (2015) on vertical stress measurements at the base-subgrade interface of reinforced and unreinforced aggregate bases over weak sand subgrade under large-scale plate load tests.

![Subgrade dynamic vertical stress during traffic passes](image)

**Figure 5.9: Subgrade dynamic vertical stress during traffic passes (250 mm-base sections)**

Table 5.3 summarizes the average vertical stresses recorded in all test sections at the end of the field test. The data demonstrates the effectiveness of geogrids in terms of reducing the dynamic vertical stresses transferred to the top of the subgrade. The stress decreased
by 16 to 36% due to the inclusion of geogrids for the 200 mm-base sections, and by only 11% for the 250 mm-base sections (considering only a light-duty geogrid). From another perspective, under the same base thickness (200 mm), the dynamic vertical stress reduced by 33 kPa, 40 kPa, and 75 kPa due to the inclusion of the light-, medium, and heavy-duty geogrid, respectively.

<table>
<thead>
<tr>
<th>Test section</th>
<th>Base thickness (mm)</th>
<th>Subgrade dynamic vertical stress (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3</td>
<td>200</td>
<td>177</td>
</tr>
<tr>
<td>S5</td>
<td>200</td>
<td>135</td>
</tr>
<tr>
<td>S7</td>
<td>200</td>
<td>210</td>
</tr>
<tr>
<td>S9</td>
<td>200</td>
<td>170</td>
</tr>
<tr>
<td>S4</td>
<td>250</td>
<td>155</td>
</tr>
<tr>
<td>S8</td>
<td>250</td>
<td>175</td>
</tr>
</tbody>
</table>

5.3.2 Residual (locked-in) Vertical Stresses

An increase in vertical load applied to soil would generally result in an increase in the vertical and horizontal stresses in the soil. If the vertical load is subsequently removed, part of this stress increase may remain in the soil and this remaining increase in stress is commonly referred to as the “residual” or “locked-in” stress. Locked-in stress is generally more significant in horizontal stresses, but locked-in vertical stresses have also been observed in the existing research. Only locked-in vertical stresses developed at the top of subgrade are evaluated in this study due to the absence of measurements of the horizontal stress and vertical stresses in the base layer. The evaluation is made for locked-in stresses developed due to base layer construction as well as subsequent traffic loading.
5.3.2.1 Residual Vertical Stresses during Construction

Locked-in stress is typical in backfill compaction during earthwork construction. Unloaded state vertical stresses at the top of the subgrade at the end of construction were zeroed with respect to the stress readings recorded in the pressure cells before the construction. The subgrade vertical stresses after construction are summarized in Table 5.4, for each test section. It is noteworthy that the stresses presented represent the ground stresses after construction including the initial “geostatic” or “overburden” stresses. The geostatic vertical stress was expected to be small (around 4 to 5 kPa, depending on the base layer thickness). Stress measurements in Table 5.4 indicate that the vertical stresses after compaction of the base layer were in the range of 4 to 9 kPa. Stress values in excess of the expected geostatic stresses are believed to be the result of locked-in stresses and principal stress rotation from compaction loads during construction. It is noted that lower magnitudes of vertical stress were recorded in the reinforced test sections as compared to corresponding unreinforced sections, regardless of the thickness of the base layer. This result indicates that vertical stresses locked in the subgrade due to aggregate compaction were reduced by the presence of the geogrids. In addition, the reduction in locked-in stresses was more obvious when a heavier duty geogrid was used. These results suggest that the geogrid may offer some stabilization benefits during construction, which may influence the performance of the unpaved road during its service life. This conclusion supports the observations made in Section 5.3.1, based on dynamic vertical stress measurements, in which possible benefits of geogrids during compaction were discussed. As expected, greater magnitudes of vertical stress were recorded in the test sections with a 250 mm as compared to corresponding test sections with a 200 mm, indicating greater locked in stresses developed in the former.
Table 5.4: Vertical stresses after construction

<table>
<thead>
<tr>
<th>Test section</th>
<th>Test lane</th>
<th>Reinforcement type</th>
<th>Unloaded-state vertical stress after construction (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td></td>
<td>G4, triangular aperture geogrid</td>
<td>6.2</td>
</tr>
<tr>
<td>S3</td>
<td>North</td>
<td>G1, biaxial geogrid</td>
<td>5.4</td>
</tr>
<tr>
<td>S5</td>
<td>North</td>
<td>G3, biaxial geogrid</td>
<td>4.4</td>
</tr>
<tr>
<td>S7</td>
<td></td>
<td>Unreinforced</td>
<td>6.6</td>
</tr>
<tr>
<td>S9</td>
<td></td>
<td>G2, biaxial geogrid</td>
<td>4.8</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>G4, triangular aperture geogrid</td>
<td>—</td>
</tr>
<tr>
<td>S4</td>
<td>South</td>
<td>G1, biaxial geogrid</td>
<td>7.0</td>
</tr>
<tr>
<td>S6</td>
<td>South</td>
<td>G3, biaxial geogrid</td>
<td>4.1</td>
</tr>
<tr>
<td>S8</td>
<td></td>
<td>Unreinforced</td>
<td>9.1</td>
</tr>
<tr>
<td>S10</td>
<td></td>
<td>G2, biaxial geogrid</td>
<td>—</td>
</tr>
</tbody>
</table>

5.3.2.2 Residual Vertical Stresses during Traffic Loading

The subgrade vertical stresses measured after each traffic pass (unloaded state) for the test section with a 200 mm base are presented in Figure 5.10. The measurements presented in Figure 5.10 include the residual stresses developed during construction as well. The data shows that the subgrade vertical stresses recorded after trafficking passes were in the range of 8 to 17 kPa. Stresses in excess of the overburden stresses are locked-in stresses from previous compaction loads and trafficking passes. It can be seen that the locked-in stresses developed during trafficking were approximately between 4 and 11 kPa. The reduction in vertical (locked-in) stress by the inclusion of geogrids was more pronounced during trafficking as compared to construction; the reduction became more pronounced when a heavier duty geogrid was used. The reduction in locked-in vertical stresses at the top of the subgrade attributed to the presence of geogrids, as shown in Table 5.4 and Figure 5.10, suggest greater locked-in vertical stresses in the base layer for the reinforced sections compared to the unreinforced section. This behavior was observed in locked-in horizontal stresses developed from compaction and traffic loads reported by White et al. (2011). Greater locked-in vertical stresses in the base layer leads to an
increase in the stiffness of the base layer. This explains the reduction in dynamic vertical stresses transferred to the subgrade, discussed in Section 5.3.1.

Figure 5.10: Vertical stress at the top of subgrade after test vehicle passes (200 mm-base sections)
5.4 Strain Developed in Geogrids

Foil strain gauges were attached to the geogrid ribs to measure strains developed in the geogrids under traffic loads. Strain gauges were installed in the transverse (cross-machine) and longitudinal (machine) directions along the wheel-path. Geogrid strains were monitored during the entire traffic period by continuously collecting data at a sampling rate of 200 Hz. 100% of the gauges were functioning throughout the test. However, strain measurements were affected by a heavy precipitation shortly after 2,000 passes. The impact varied from one gauge to another with readings from some gauges considered completely unreliable (see Table 4.3). Based on measurements from dummy strain gauges installed in geogrid samples adjacent to the active strain gauges, the variation of ambient temperature for strain gauges had negligible effect on the measured strain. Accordingly, the geogrid strain measurements were not corrected for temperature changes.

In order to quantify the strain accumulated in the geogrids due to traffic loading only, the strain measurements were zeroed with respect to the unloaded state reading recorded at the end of construction. This was done so that strain responses to traffic loading for the different types of geogrids utilized can be compared. The strains induced in the geogrid as a result of the traffic loads are discussed in this section in terms of both dynamic and permanent strains. The geogrids used as road reinforcements are exposed to dynamic loading conditions under traffic loading. Dynamic strain measurements are key to identifying those dynamic conditions. The dynamic strain for each pass was calculated by subtracting the unloaded strain from the maximum measured response under the applied wheel load. The dynamic strain in the geogrid can be used to compute the dynamic load induced in the geogrid for each traffic pass. On the other hand, permanent strain measurements allow for comparison of mechanical responses of different reinforcements. Permanent strains will also be used to evaluate mobilization of reinforcement and their engagement with the surrounding material, investigate a potential correlation between reinforcements and surface rutting (performance), and examine the effect of base layer thickness on mobilization of geogrids.
5.4.1 Typical Geogrid Strain Response to Traffic Loading

Geogrid strain response of the unpaved test sections is examined using the data from the foil strain gauges. Two different patterns were distinguished depending on the orientation of the geogrid ribs, on which the strain is being measured, relative to the direction of traffic. Figure 5.11 displays a typical response from a foil strain gauge attached to the cross-machine direction of the geogrid (transverse direction) directly under the wheel-path as the truck passed over the instrument location in a reinforced section. The distinction is clear between the front (left) and the rear (right) axle of the truck. Positive strain values are taken as tension, thereby the transverse geogrid ribs under the wheel-path experienced tensile strains due to traffic loading. It appears that the strain response is mainly recoverable with a tiny yet noticeable, irrecoverable portion (Figure 5.11). A typical response from a foil strain gauge installed in transverse direction away from the wheel-path (between the two wheel-paths) is show in Figure 5.12. In contrast to the response under the wheel-path, the transverse geogrid ribs in the area between the two wheel-paths experienced compressive strains. A small irrecoverable compressive strain can be seen as well.

![Figure 5.11: Typical geogrid strain response to truck load (transverse direction, under the wheel-path)](image-url)
Figure 5.12: Typical geogrid strain response to truck load (transverse direction, between wheel-paths)

Figure 5.13 depicts a typical geogrid strain response recorded in the longitudinal direction (machine direction of the geogrid) to a dump truck travelling across a reinforced test section. The strain response in the longitudinal direction was significantly different and had a relatively complex pattern as compared to the response in the transverse direction. The longitudinal strain response varied in both sign and magnitude as the wheel passed over the instrument location. As can be seen in Figure 5.13, the geogrid initially experiences compressive strains when the wheel approaches the measurement point. As the wheel gets closer to the measurement point, larger tensile strains develop, reaching a maximum value when the wheel is directly over the strain gauge. Then, as the wheel moves away from the measurement point, the state of strain turns back to compression before returning gradually to the steady unloaded condition. This response is an expected result of the moving wheel pattern. Janoo et al. (1999) and Perkins (2002) reported similar longitudinal strain responses at the bottom of the base layer of pavement test sections loaded with a heavy vehicle simulator. A similar response was also seen in the longitudinal strain at the bottom of the asphalt layer in the pavement test sections described by Al-Qadi et al. (2012). Longitudinal geogrid strain data from laboratory-scale pavement test sections reported by Tang et al. (2013) also showed a similar pattern of
response but was not highlighted by the authors. Unlike the strain response in the transvers direction, Figure 5.13 shows a small irrecoverable compressive strain in the longitudinal direction as a result of one pass of the moving truck. Despite their ability to carry high tensile loads, geogrids do not sustain compression and develop large amounts of plastic strain under compression loads.

![Geogrid strain response to truck load (longitudinal direction)](image)

**Figure 5.13: Typical geogrid strain response to truck load (longitudinal direction)**

*Influence of Vehicle Wander:*

The response patterns described above represent the typical responses from foil strain gauges when the moving wheel passes precisely over the strain measurement point (the actual wheel-path coincides with the design wheel-path). Despite the efforts made to control vehicle travel path across the road sections, vehicle wandering is inevitable in such full-scale field traffic conditions. The influence of vehicle wandering was observed in the measured response from few foil strain gauges and for a limited number of isolated traffic passes. This influence was seen more in strain measurements from gauges installed in the transverse direction.
In general, it was observed that dynamic strain response in the transverse direction is more sensitive to vehicle wandering compared with the longitudinal direction. Figure 5.14 depicts representative response patterns that demonstrate the influence of vehicle wander on the measured strain response in the transverse direction. It is noteworthy to point out that these response patterns are very infrequent in the strain measurements. Consequently, the influence of wandering as discussed in this section is limited to a small part of the data where these patterns were observed. Figure 5.14 shows that the influence of vehicle wandering in the transverse direction is more noticeable in the strain response of the dual-wheel rear axle load. This may be attributed to the unique pattern of the loaded area of the dual-wheel and more specifically due to the effect of the unloaded area entrapped between the two tires in the dual-wheel rear axle. As can be seen in Figure 5.14, the measured dynamic strain (peak response) in the transverse direction can be markedly influenced by vehicle wandering resulting in a double peak response under the dual-wheel rear axle load (Figure 5.14a and b). In a case of extreme vehicle wonder, the measured dynamic strain response can be compressive, particularly from the dual-wheel rear axle load (Figure 5.14c and d). In other words, the transient response to a wheel load is represented by a compression peak (valley) instead of an extension peak. Nevertheless, it is believed that the influence of vehicle wandering on the measured permanent strains was insignificant. The irrecoverable strain portion was always tensile (Figure 5.14), regardless of the amount of wandering. Even in cases of greater wandering, when one of the axle loads results in compression peak (compressive dynamic strain) at a measurement point in the geogrid rib, the same point will usually experience extension peak (tensile dynamic strain) under the other axle load. In addition, geogrid areas adjacent to that measurement point (e.g., directly under the wheel) will also experience larger magnitudes of pure tensile dynamic strain.

These tensile dynamic strains are sufficient to produce irrecoverable tensile deformations in the loaded geogrid area (of a certain width) when the vehicle departs from the measurement location (compatibility of strains). Also, accumulation of permanent strains with traffic passes minimizes any possible effect of incidental vehicle wandering. The influence of vehicle wander was less noticeable in geogrid strain measurements in the longitudinal direction. Unlike the transverse direction, the influence of vehicle wander in
the longitudinal direction was seen in the measured strain response to the front axle load but it was limited to underestimating the amount of peak responses. Measured strain response of the front axle is more influenced by wandering than that of the rear axle due to the narrower loading area of the single-wheel front axle compared with the dual-wheel rear axle.

![Graphs showing strain response in the transverse direction](image)

**Figure 5.14:** Influence of vehicle wander on the measured strain response in the transverse direction: (a) and (b) double peak response to the rear axle load; (c) and (d) compression response to the rear axle load
5.4.2 Dynamic Geogrid Strain

5.4.2.1 Dynamic Strains in Transverse Direction

The dynamic geogrid strain was defined for each pass as the peak tensile strain value minus the value when the truck was well away from the measurement point. The dynamic geogrid strains measured during traffic loading in the transverse direction are presented in Figure 5.15 as a function of cumulative traffic passes. Overall, dynamic strain in the reinforcement remained constant or slightly increased with cumulative traffic passes. Constant dynamic strain in the geogrids is expected for a constant dynamic load induced in the geogrids. The increase in axle load at the beginning of the second phase of the test (at approximately 1,600 passes) had minor effect on dynamic strains induced in the geogrid in the transverse direction. The slight increase in dynamic strain with traffic passes is more evident in the light- and medium-duty geogrids installed in the 200 mm-base sections and this increase may be attributed to the degradation of the base layer under repeated traffic loading. The rapid decrease in dynamic strain observed at the onset of trafficking is attributed to the increase in stiffness of the base course layer overlying the geogrids due to further densification of the base layer (rearrangement of granular aggregates into a denser and tighter layer) and to the increase of lateral confinement from the geogrids (leads to a buildup of locked-in horizontal stresses, an increase in the mean stress, and an increase in the stiffness of the granular material). Similar response was observed in the measurements of dynamic vertical stress at the top of the subgrade (Figure 5.7). The highest dynamic strain registered in the transverse direction was around 0.4% at the onset of trafficking, while strain values throughout the rest of the test ranged between 0.1% and 0.2%. This magnitude of induced dynamic strain is only sufficient to mobilize part of the tensile strength of the geogrids.

The dynamic strains were generally lower for the heavy-duty geogrid than for the light-duty and medium-duty geogrids. It is noted that the lower dynamic strains were associated with test sections in which lower dynamic vertical stresses were also recorded in the subgrade. However, it should be noted that special attention must be paid when making such comparisons of response of different geogrids based on strain data. Permanent and dynamic strains do not necessarily provide conclusive information about
the respective engagement of different geogrids with the unpaved structure because of the differences in their elastic moduli. The estimation of tensile forces mobilized in geogrids will rather provide more conclusive information about the respective behavior of geogrids. Figure 5.15 shows also that the dynamic geogrid strain is slightly larger for the 200 mm-base test sections than for the 250 mm-base test sections. Henry et al. (2009) reported similar observations from limited strain gauge data. These observations are generally consistent with the rutting performance of the test sections discussed in Section 5.1.1. This agreement may reflect a possible correlation between dynamic geogrid strains in the transverse direction and rutting performance. However, some researchers reported that dynamic stress and strain measurements rarely provided a good correlation to long-term performance (Perkins, 1999; Sun et al., 2015). These observations, however, were based on model-scale tests. Furthermore, problems with correlating dynamic response to long-term performance are usually encountered when attempting to correlate dynamic “strain” responses, associated with geogrids of different stiffnesses, to rutting performance of the respective test sections, as previously discussed.
Figure 5.15: Dynamic geogrid strain versus number of passes (transverse/cross-machine direction)

(a) 200 mm-base sections

(b) 250 mm-base sections
5.4.2.2 Dynamic Strains in Longitudinal Direction

Figure 5.16 displays the dynamic geogrid strains measured in the longitudinal direction during traffic loading as a function of cumulative traffic passes. The general trend is similar to that of the dynamic geogrid strains measured in the transverse direction. Rapid decrease in dynamic strain is observed at the onset of trafficking because of further densification of the base layer. The dynamic strain then remained constant with cumulative traffic passes during each loading phase (i.e., for a constant axle load). The influence of increasing the axle load on dynamic strains is slightly more evident than that observed in the transverse direction. Nevertheless, it is observed that the dynamic strains did not increase linearly with the increase in axle load, in a similar manner to the subgrade vertical stresses and the dynamic geogrid strains in the transverse direction. The dynamic geogrid strains in the longitudinal direction were generally larger than those obtained in the transverse direction. As can be seen in Figure 5.16, the highest dynamic strain registered was around 0.6% at the onset of testing, while throughout most of the testing period strain values ranged between 0.2% and 0.3%.

No clear correlation was observed between dynamic geogrid strains in the longitudinal direction shown in Figure 5.16 and rutting performance. The results were inconclusive regarding the effect of geogrid stiffness on the magnitude of dynamic strain induced in the geogrid under traffic loading. Moreover, higher dynamic strains were recorded in the 250 mm-base sections than in the 200 mm-base sections. This response does not correspond to the rutting performance observed in the test sections. This is to be expected, however, since the geogrid strain response in the longitudinal direction seems to have little or no influence on the mobilization of the reinforcement as will be demonstrated later by the permanent geogrid strain measurements. In addition, the transient compressive strains that characterize the geogrid strain response in the longitudinal direction (refer to Figure 5.13) could have direct impact on the magnitude of the peak or dynamic tensile strains associated with them. This argument may corroborate the previously discussed causes of discrepancies in correlating dynamic response (in a general sense) to long-term performance that are usually encountered.
Figure 5.16: Dynamic geogrid strain versus number of passes (longitudinal/machine direction)
5.4.3 Permanent Geogrid Strain

5.4.3.1 Permanent Strains in Transverse Direction

The permanent strains measured over the testing period by strain gages attached to geogrid ribs perpendicular to traffic direction (transverse direction) are presented in Figure 5.17 as a function of cumulative traffic passes. The permanent strain is defined as the irrecoverable strain portion after the wheel load moves away from the measurement point. There is a clear general trend of increased permanent strains in the transverse direction with cumulative traffic passes. The data in Figure 5.17 show an immediate accumulation of permanent strain at the onset of trafficking. This demonstrates that the generation of forces in the geogrids did not require a significant amount of rut to be developed in the unpaved structure. Similar responses were observed by many researchers (e.g. Perkins, 1999; Tingle and Jersey, 2005). Overall, permanent geogrid tensile strains between 0.45 and 0.6% were accumulated in the transverse direction at a traffic level of 2,200 passes (the highest traffic level up to which strain gauge data is considered to be sound). Even though these strain magnitudes indicate that tension was partially mobilized in the geogrids, the amounts of tension mobilized in the geogrids were sufficient to manifest the interaction between the geogrid and the surrounding aggregate as will be discussed later. Full mobilization of tensile strength is less likely to happen under the specified test conditions with the level of stresses imposed on the relatively firm subgrade.

Apparently, the responses of the various reinforced sections with respect to permanent geogrid strain are more distinguishable than the geogrid dynamic strain responses discussed in the previous section. It can also be seen from Figure 5.17a that the magnitudes of permanent strain in the geogrids correlate reasonably well with the long-term performance of the test sections reported in Figure 5.2. Larger permanent strains were generally developed in geogrids with lower stiffness. However, it is noteworthy to mention that such correlation does not necessarily exist for strain measurements since the development of permanent strain in the geogrid depends on material properties of the geogrid as well as soil geogrid interaction properties (e.g., for the same magnitude of developed geogrid force, smaller permanent strain will accumulate in geogrids with
greater tensile modulus). Therefore, direct comparison of responses of different geogrids cannot be made based only on strain data because of the differences in their elastic moduli. The tensile forces mobilized in geogrids will rather provide more insight into this matter.

![Graph showing permanent geogrid strain versus number of passes for different geogrids](attachment:geogrid-strain-number-passes.png)

(a) 200 mm-base sections

(b) 250 mm-base sections

Figure 5.17: Permanent geogrid strain versus number of passes (transverse/cross-machine direction)
Figure 5.17 also demonstrates that, for the same geogrid grade, larger magnitudes of permanent geogrid strain were generally recorded in the 200 mm-base sections compared with the corresponding 250 mm-base sections. The measurement of transverse strains in Section 6 was an outlier because the strain gauge installed in that section started to record sharp spikes of dynamic strain under the wheel load after approximately 200 passes of the truck. The larger geogrid strains developed in the 200 mm-base sections can be attributed to the greater magnitudes of vertical stress transferred to the subgrade in these sections (compared with those in the 250 mm-base sections). An average permanent strain difference of over 0.15% was observed after 2,200 passes. In such cases of geogrid products of the same grade installed underneath base layers of different thicknesses, differences in permanent geogrid strain can only be attributed to the difference in base layer thickness. Furthermore, the amount of permanent strain accumulated in geogrids is related to their mobilization and the level of their engagement in a tensile capacity with the surrounding structure. Accordingly, these results suggest that the geogrids were mobilized and engaged more with the unpaved structure in the case of test sections with a 200 mm base than in test sections with a 250 mm base. Regardless of differences in magnitude, geogrid strain measurements from the two base layer thicknesses showed generally similar trends (Figure 5.17). This implies that strain measurements carried out in this study using foil strain gages are fairly reproducible and reliable. Overall, measurements from the duplicate strain gauges attached to adjacent XD geogrid ribs in both sections 3 and 5 demonstrated reasonable repeatability.

As previously mentioned, additional strain gauges were installed in the transverse direction of the thin sections away from the wheel-path (between the two wheel-paths) in order to provide some insight into the geogrid strain distribution in the transverse direction. The permanent strains recorded from these strain gauges are shown in Figure 5.18 as a function of cumulative traffic passes. It can be seen that the geogrids experienced small magnitudes of compressive permanent strain in the transverse direction between the two wheel-paths. This is an expected response based on the typical response from these strain gauges discussed in Section 5.4.1 (Figure 5.12). These results showed that the geogrid was not entirely under tension across the road. In fact, this is to be expected given the loading condition and the anticipated geogrid-aggregate interaction. In
channelized traffic, the aggregate particles at the bottom of the base layer attempt to
spread laterally (in the transverse direction) away from the wheel load as they are being
pushed downward by traffic loads (Perkins, 1999). When geogrids exist at the interface,
they can restrain lateral movement of the aggregate through shear interaction between the
geogrid and the aggregate (i.e., interlocking). Consequently, tensile load is transmitted
from the aggregate to the geogrid and tensile strain accumulates in the geogrid with
traffic passes (in the transverse direction). This tensile strain, however, peaks under the
center of the wheel load but diminishes with distance away from center. Furthermore, as
a result of lateral spreading away from the wheel load, the aggregate particles at the
bottom of the base layer between the two wheels will be laterally compacted as they are
being laterally pushed from both sides by the two wheels. In case of geogrid inclusion,
interlocking between the geogrid and the aggregate will result in the transmission of
compression load from the aggregate to the geogrid. The compression load leads to the
accumulation of compressive strain in the geogrid in the transverse direction with traffic
passes. This compressive strain is clearly demonstrated by measurements from foil strain
gauges installed in transverse ribs between the two wheel-paths Figure 5.18. It is worth
noting that the magnitudes of compressive permanent strain developed in the geogrids
between the two wheel-paths are significantly smaller than the magnitudes of tensile
permanent strain measured under the wheel-path (where greatest lateral movement of the
base aggregate occurs) reported in Figure 5.17a; however, these two values are
proportional to each other, as expected. In both cases, larger permanent strains (tensile or
compressive) are generally developed in geogrids with lower stiffness.
5.4.3.2 Permanent Strains in Longitudinal Direction

Figure 5.19 presents the permanent geogrid strains measured in the longitudinal direction as a function of cumulative traffic passes. Overall, the data indicate that geogrid strains measured in the longitudinal direction were clearly tending towards compression. The relationship between permanent strains in the longitudinal direction and cumulative traffic passes was not as easy to perceive as the one observed in the transverse direction.

For the test sections with a 200-mm base layer (Figure 5.19a), compressive strains were developed in the geogrids throughout the test. The geogrids did not show any tendency for mobilizing tension in the longitudinal direction. Strain decreased rapidly (compression) and immediately upon load application, then the decrease in strain slowed down at different rates for the different geogrids. However, a relationship can be seen between the development of permanent strains in the longitudinal direction and cumulative traffic passes. The geogrid with the higher tensile stiffness (G3) showed greater compressive strains with evidently correlated to the number of traffic passes. Development of compressive permanent strains in the longitudinal direction is anticipated based on the typical strain response of the geogrid to the truck load in the longitudinal direction discussed earlier (Figure 5.13). While geogrids can carry high tensile loads,
they do not sustain compression and develop large amounts of plastic strain under compression loads.

Figure 5.19: Permanent geogrid strain versus number of passes (longitudinal /machine direction)
On the other hand, for the test sections with a 250 mm base layer (Figure 5.19b), longitudinal strains were small and tending towards compression. Compressive strains were initially developed in the geogrids upon load application; then shortly after, a state of tension was induced in the geogrids, leading in the cases of geogrids of lower stiffness to the development of tensile permanent strains of small magnitude for part of the first loading phase. Nevertheless, compression strains were seen to prevail again during the second loading phase.

As previously discussed, the amount of permanent tensile strain accumulated in the geogrid indicates the extent to which the geogrid’s stiffness was mobilized and how much the geogrid was engaged with the surrounding material. In this regard, it appears from the results shown in Figure 5.19 that tension was not mobilized in the longitudinal direction. The results presented in this section along with those discussed in previous section demonstrate that permanent strains in the longitudinal direction were significantly less important than transverse strains for the description of the mechanical response of the different geogrids and for the evaluation of reinforcement mobilization. However, the measured longitudinal geogrid strains certainly offer a valuable and clearer insight about the response of geogrid reinforcements under traffic loading.

### 5.4.4 Geogrid Strains Developed during Construction

Previous research shows that geogrids provide some stabilization benefits during road construction. However, views are conflicting on the mechanism by which geogrids affect the road system during construction. Some studies suggest that significant amounts of tensile strain develop in geogrids during construction and cause improvement by mobilizing tension (Helstrom et al., 2006; Tang et al., 2013), while others argue that tensile strains develop during construction are negligible due to the random nature of the construction process (Perkins et al., 2004). Results from this study suggest a new conception that seems to reconcile the two opposing views.

Strains in the geogrids were measured during the construction process of the base layer to investigate possible reinforcement effects on the construction process. Table 5.5
summarizes strains measured by strain gauges during construction at each test section. These strain values were recorded immediately after construction and zeroed with respect to the strain readings following geogrid installation and immediately before construction. The measurements show that reasonable amounts of strain were developed in the geogrids; the magnitudes and directions of these strains were highly variable, however. The variability of geogrid strain exhibited in Table 5.5 is consistent with the random nature of compaction where aggregate particles do not have predominant direction of movement as they attempt to escape compaction loads (Perkins et al., 2004). Overall, the strain values caused by compaction during construction of the base layer ranged between -1085 and 2520 µε (-0.1 and 0.25%). Negative values in Table 5.5 indicate that the strain in an instrumented rib was compressive.

Table 5.5: Permanent strains developed in the geogrids during construction

<table>
<thead>
<tr>
<th>Test section</th>
<th>Base thickness</th>
<th>Geogrid</th>
<th>Geogrid strain developed during construction (µε)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>MD</td>
</tr>
<tr>
<td>S3</td>
<td>Thin sections</td>
<td>G1</td>
<td>546</td>
</tr>
<tr>
<td>S5</td>
<td>G3</td>
<td>-1085</td>
<td>1102</td>
</tr>
<tr>
<td>S9</td>
<td>G2</td>
<td>-573</td>
<td>1242</td>
</tr>
<tr>
<td>S4</td>
<td>G1</td>
<td>55</td>
<td>2520</td>
</tr>
<tr>
<td>S6</td>
<td>G3</td>
<td>-512</td>
<td>900</td>
</tr>
<tr>
<td>S10</td>
<td>G2</td>
<td>475</td>
<td>867</td>
</tr>
</tbody>
</table>
As was discussed previously (Sections 5.4.1 and 5.4.3), under moving loads, the permanent strains accumulate in biaxial geogrid ribs can be either tensile or compressive depending on the orientation of the rib relative to the direction of traffic as well as the position of the applied wheel load with respect to the rib (or the point on the rib where the strain is being measured). In channelized traffic, the aggregate particles attempt to spread laterally (transverse direction) at the bottom of the base layer as they are being pushed downward by traffic loads (Perkins, 1999). Geogrids can restrain lateral movement of the aggregate through interlocking between the geogrid and the aggregate. As a result, tensile load is transmitted from the aggregate to the geogrid and tensile strain accumulates in the geogrid with traffic passes in the direction perpendicular to traffic (transverse direction). This tensile strain peaks under the center of the wheel load and diminishes with distance away from center. Meanwhile, permanent compressive strains develop in the geogrid in the direction of traffic (longitudinal direction) as demonstrated by measurements from foil strain gauges in this study.

The process of compaction employed in the construction of road bases, however, is different from channelized traffic. The direction of lateral movement of the aggregate particles under compaction loads is just as liable to change as the direction of motion of the compaction equipment itself. Consequently, development of tensile and/or compressive strains in the geogrid reinforcement will be of a random nature, direction-wise, leading to these strains be canceled and/or reversed during compaction. Even when the compaction equipment operates in a predominant direction (i.e., parallel to the road centerline, as was generally followed in this study), geogrid strains develop during compaction (in the transverse direction) may get neutralized or even reversed when the equipment operates in an adjacent location. This process may result in developing strains in the geogrid that do not follow a particular pattern.

Regardless of the ultimate amounts, directions and distribution of geogrid strains at the end of compaction, the lateral restraint provided by the geogrid to the aggregate during each pass of compaction will result in an increased placement density and a buildup of horizontal stresses locked into the aggregate. The increase in horizontal stress means an increase in the mean stress which leads to an increase in the stiffness of the granular material (Perkins, 1999). Consequently, the outcome of the compaction process is an
increased stiffness and placement density of the compacted aggregate due to the lateral restraint provided by the geogrid, yet a state of strain in the geogrid that largely depends on the pattern of compaction.

That being said, the results in Table 5.5 also suggest that following a strict pattern of compaction along a single direction parallel to the road centerline (which is the common practice), will increase the likelihood of developing more tensile strains (mobilize tension) in the geogrid in the transverse direction of the road. Specifically, strain values in Table 5.5 indicate that a general trend existed of developing tensile strains in the transverse direction (XD) and compressive strains in the longitudinal direction (MD) of the road. This trend is consistent with the pattern of compaction followed in this study of operating the compaction equipment predominantly in a direction parallel to the road centerline. This pre-tensioning of geogrid in the transverse direction during construction will lead to an improved performance during trafficking.

In summary, it appears based on results from this study that the reinforcing benefits of geogrids during construction is not linked to the amount of tensile strain developed in the geogrids due to compaction loads, but rather to the restraint of lateral movement of aggregate during compaction which leads to an increase in stiffness and placement density of the base layer. Attempts to experimentally perceive the potential reinforcing benefit of geosynthetics during construction as “global” pre-tensioning of geosynthetics were unsuccessful (Perkins et al., 2004). In fact, development of compressive strains in geogrids during construction was reported even in studies that are in favor of the geosynthetics pre-tensioning effect (Helstrom et al., 2006) but they were attributed to possible difficulties encountered during geogrid installation. However, pre-tensioning the geogrid in the transverse direction during road construction by following a single-direction compaction pattern parallel to the road centerline will benefit the road in the long term.
5.5 Summary

The data collected from the unpaved test sections during the traffic testing were presented and discussed. Performance of the unpaved test sections under traffic loading was described based on rut depth and surface deformation measurements. The base layer in all test sections experienced initial compaction or further densification under traffic loading, which can result in a stiffness increase of the base layer. The results demonstrated the benefits of geogrid reinforcement in reducing surface rutting of unpaved test sections with thin base layers (i.e., 200 mm). The decrease in surface rutting due to geogrid inclusion was most pronounced for the heavy-duty geogrid, followed by the medium and light-duty biaxial geogrids, which suggest that the achievable degree of reinforcement may depend on the stiffness of the geogrid. Data were insufficient to evaluate the performance of the triangular aperture geogrid. Improvement by geogrid reinforcements decreased as the thickness of base course was increased from 200 mm to 250 mm (and the geogrids showed similar performance).

The results also indicated that placement of the geogrids immediately over a geotextile separator resulted in reduced interlocking between the geogrid and aggregate. It is recommended to place the geogrid directly on the subgrade without a separator to maintain optimal interlocking between the geogrids and aggregate. Overall, the base layer contributed more to the surface deformation of the unpaved test sections as compared with the subgrade layer. For the same base layer thickness, the contribution from the base layer to the total permanent deformation increased, whereas the contribution from the subgrade decreased with the presence of geogrid. In general, the latter response was more notable in the thinner sections and it was more pronounced when a heavier duty geogrid was used. For the same reinforcement type, the contribution from the base layer to the total permanent deformation increased, whereas the contribution from the subgrade decreased with the increase of base course thickness.

The vertical dynamic stresses on top of the subgrade decreased rapidly initially at the onset of trafficking for all sections, which is attributed to an increase in the base layer stiffness, followed by an increase in stress indicating a possible degradation of the base layer. The inclusion of geogrids reduced the dynamic vertical stresses transferred to the
top of the subgrade by 16 to 36% for the 200 mm-thick base layers, and by only 11% for the 250 mm-thick base layers. The reduction in subgrade vertical stress became more obvious when a heavier duty geogrid was used.

The increase in subgrade vertical stresses with traffic passes was reduced in both reinforced and unreinforced sections with the increase of base layer thickness, indicating less degradation of base layer. However, the effect of geogrid reinforcement (improvement in stress transfer by the base layer) was decreased by the increase of base layer thickness.

The traffic loading changed from 80 kN to 100 kN, which resulted in some increase in subgrade dynamic vertical stress; an increase of 25% in axle load resulted in an average increases of 11% in dynamic stress at the top of subgrade. The increase in vertical stress with cumulative traffic passes was more evident under the higher axle load, indicating more degradation of the base layer under the high axle load.

Residual or locked-in vertical stresses were recorded at the top of subgrade during both construction and trafficking. These locked-in stresses were reduced by the presence of the geogrids. The reduction was more pronounced during trafficking as compared to construction, and it became more pronounced when a heavier duty geogrid was used. The reduction in locked-in vertical stresses at the top of the subgrade by the presence of the geogrid suggests greater locked-in vertical stresses in the base layer for the reinforced section compared to the unreinforced section, which indicates an increase in the base layer stiffness by the reinforcement.

The geogrid strain data indicated that the mechanical response of geogrid reinforcements measured in a laboratory test may not be sufficient to understand their performance under actual traffic loading in the field. Examination of the typical strain response of the geogrid indicated that the geogrid ribs in the transverse direction experienced significant tensile transient strains directly beneath the wheel load and relatively small compression transient strains away from the load (between the wheel-paths). The measured strain in the longitudinal or traffic direction indicated a relatively complex response in terms of strain changes as the wheel load approaches and moves directly on top of the strain measurement point, and then as the wheel moves away from the measurement point. The
loading setups used in laboratory tests are not capable of replicating these loading conditions.

The results of permanent geogrid strain further demonstrated that the geogrid was not under constant tension across the road. Tension was only mobilized in the transverse direction, beneath the wheel-path, and the geogrid was not in tension between the wheel-paths (i.e., away from the load) and in the longitudinal direction. Permanent strains were accumulated in the geogrid immediately at the onset of the traffic loading and well before any appreciable surface rutting was developed in the unpaved structure. It was also observed that the amount of tension mobilized in the geogrid decreased with the increase in base layer thickness, for the same reinforcement type.

 Permanent geogrid tensile strains accumulated in the transverse direction ranged from 0.45 to 0.6%. A clear relationship was observed between the developed permanent tensile strains in the transverse direction and cumulative traffic passes. In addition, the permanent geogrid strains were correlated to rutting performance. Furthermore, the developed permanent compression strains (in the longitudinal direction and laterally away from the load) were correlated to the cumulative traffic passes in the 200-mm base sections.

Previous research indicated inconsistencies with respect to the dynamic strain response of geogrid reinforced base layers and problems with correlating this response with rutting performance. However, this study suggests that a correlation between dynamic geogrid strains in the transverse direction and long-term rutting performance may exist.
Chapter 6

6 ANALYSIS OF EXPERIMENTAL DATA

This chapter presents the analysis of the data collected from the full-scale field traffic testing discussed in Chapter 5. Different analyses are performed to investigate the behavior of the geogrid-reinforced unpaved structure and evaluate the various effects of geogrid reinforcement on its performance. The performance benefits of using geogrid reinforcements to stabilize layers of unpaved road are quantified and evaluated in terms of both extending the service life of the road and reducing the required aggregate thickness. Furthermore, the degradation of the base layer is investigated through the changes in stress distribution angle and base-to-subgrade modulus ratio with cumulative traffic passes. In addition, the tensile forces mobilized in the geogrid reinforcements are computed and their role in stabilizing the unpaved layers is discussed. Results from the full-scale field experiment are utilized to investigate the reinforcement mechanisms through which the geogrids stabilized the unpaved test sections. Finally, the quantified reinforcement benefits of various forms are used to identify properties of geogrid reinforcement most relevant to their effectiveness in improving the performance of unpaved structures.

6.1 Quantification of Geogrid Benefits Related to Performance

Performance of test road sections is usually evaluated based on their ability to support a given number of traffic passes, which is measured by changes in surface rutting of road segments conducted in full-scale field testing. In Section 5.1.1, the respective performances of the test sections were compared based on rut depth and surface deformation measurements. However, there are several methods that can be used to evaluate the performance benefit of geogrid reinforcement in a more quantitative manner. In road design, two main benefits of geogrid reinforcement are generally considered: extended service life of the road (i.e., additional traffic passes), and reduced aggregate thickness (i.e., reduced aggregate quantities and initial construction cost). The American
Association of State Highway and Transportation Officials (AASHTO) recommends the use of the traffic benefit ratio (TBR) and the base course reduction factor (BCR) to quantify the benefit derived from geogrid-reinforcement (AASHTO, 2009). Accordingly, the TBR and BCR analyses are used to evaluate the benefits of geogrid reinforcement in this study. Only the 200 mm-base test sections are evaluated in this section since the reinforcement benefits were not as evident in the 250 mm-base test sections.

6.1.1 Equivalent Single-Axle Load

The pavement system is usually subjected to traffic loads of various magnitudes and axle configurations, each having a specific damage effect. Before any evaluation of reinforcement benefits can be made, influences of vehicle axles on damage of the test sections must be assessed. The concept of an equivalent single-axle load (ESAL) is commonly used by pavement engineers to convert the loading from various axle configurations and load magnitudes into an equivalent number of reference single-axle loads that cause the same damage. The reference is the single axle, with dual wheels, loaded to 80 kN. The factor that defines the number of passes of an equivalent (80 kN) single-axle load \( N_{ESAL} \) needed to cause pavement damage equivalent to that caused by one pass of a given axle load is called “relative damage factor” or “load equivalency factor” (LEF). LEF can be defined as (AASHTO, 1993):

\[
LEF = \frac{N_{ESAL}}{N}
\]  

where

\( N_{ESAL} \) = number of passes of the equivalent (reference, 80 kN) single-axle load,
\( N \) = number of passes of the given axle load,

In terms of relative damage, LEF is defined as the ratio of the damage caused by any given axle load to the damage caused by the reference axle load, or (AASHTO, 1993):

\[
LEF = \frac{\text{Damage due to any axle load}}{\text{Damage due to 80 kN SAL}}
\]
The LEF values for the axle loads applied in this study were calculated using the fourth power rule that is generally utilized to express relative damage from increased axle loads. For rear axle (dual-wheel single axle) (Scala, 1970):

\[
\text{LEF} = \left( \frac{W}{80} \right)^4 \quad 6-3)
\]

where \( W \) = the load on a given single axle with dual wheels (kN).

A common pitfall of using the LEF expression is employing the same LEF factors for both single-wheel (e.g., in a steering axle) and dual-wheel axle configurations considering only the axle load and not including the number of wheels in the axles. This can lead to erroneous results because different wheel configurations may cause different amounts of damage. In fact, front or steering axles are more damaging on pavements than rear axles because they are fitted with single wheels (Gillespie et al., 1992). It was found that single wheels are about three to four times as destructive as dual wheels for equal load magnitudes (Scala, 1970). Based on deflection data caused by single axle with single tires, Scala (1970) suggested that a reference 80 kN single-axle load on dual wheels is equivalent to 53 kN single-axle load on single wheels. A reference 53 kN single-wheel, single-axle load is considered a conservative measure to estimate damage from steering axles and is used in this study to calculate the load equivalency factors for the steering or front axle.

For steering/front axle (single-wheel single axle) (Scala, 1970):

\[
\text{LEF} = \left( \frac{W}{53} \right)^4 \quad 6-4)
\]

where \( W \) = the load on a given single axle with single wheels (kN).

Table 6.1 summarizes the LEF values for the axle loads applied in the field trafficking calculated using Equations 6-3 and 6-4. Knowing the values of LEF listed in Table 6.1, Equation 6-1 is used to calculate the number of equivalent single-axle load passes (\( N_{\text{ESAL}} \)) achieved during different stages of the field trafficking. Overall, the total number of truck passes of 2,500 performed during field trafficking corresponded to 4,160 ESAL
passes (ESALs). The number of equivalent single-axle load passes are used to replace the number of truck passes for a better assessment of pavement damage (i.e., better assessment of reinforcement benefits). The rut depth for the test sections previously discussed in Section 5.1.1 (Figure 5.1) is presented in Figure 6.1 in an alternative way as a function of cumulative ESAL passes.

Table 6.1: Load equivalency factors for axle loads used in the field testing

<table>
<thead>
<tr>
<th>Axle configuration</th>
<th>Phase I</th>
<th>Phase II</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Load (kN)</td>
<td>LEF</td>
</tr>
<tr>
<td>Rear axle</td>
<td>80</td>
<td>1.00</td>
</tr>
<tr>
<td>Steering axle</td>
<td>33</td>
<td>0.15</td>
</tr>
<tr>
<td>Truck</td>
<td>113</td>
<td>1.15</td>
</tr>
</tbody>
</table>

Figure 6.1: Average rut depth versus number of ESAL passes (200 mm-base sections)
6.1.2 Traffic Benefit Ratio Analysis

Benefit of geogrid reinforcement is commonly expressed in terms of extension of service life of the road. In that case, extension of life can be directly measured by a Traffic Benefit Ratio (TBR). It is defined as the number of ESAL passes sustained by a reinforced section divided by that of an equivalent unreinforced section at a given level of rut. The TBR analysis was conducted based on rut depth results at varying levels of rut depth (25, 45, 55, 65 and 75 mm). The 25 and 45 mm rut depths represent the first loading phase while the 55, 65 and 75 mm represent the second loading phase. The first step was to view the data presented in Figure 6.1 in an alternative way in which the number of ESAL passes, $N_{ESAL}$, needed to reach the prescribed levels of rut depth for each test section is determined and displayed. The results are presented in Table 6.2. Based on the data in Table 6.2, TBR values for the reinforced sections were then computed at the given levels of rut depth. The results of the TBR analysis are summarized in Table 6.3. The additional number of ESAL passes, $N_{ESAL(add)}$, that each reinforced section carried as compared to the unreinforced section (corresponding to the given rut levels) is also presented in Table 6.3.

<table>
<thead>
<tr>
<th>Test section</th>
<th>Reinforcement type</th>
<th>$N_{ESAL}$ at various rut levels</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>25 mm</td>
</tr>
<tr>
<td>S7</td>
<td>Unreinforced</td>
<td>22</td>
</tr>
<tr>
<td>S3</td>
<td>G1</td>
<td>36</td>
</tr>
<tr>
<td>S5</td>
<td>G3</td>
<td>42</td>
</tr>
<tr>
<td>S9</td>
<td>G2</td>
<td>38</td>
</tr>
</tbody>
</table>

It is evident that for an unpaved test section with a 200 mm base thickness, installing the geogrids improved the road system ability to sustain higher loading applications. Overall, the data in Table 6.3 shows that the geogrids helped support around 1.2 to 3.6 times more ESAL passes when evaluated at varying levels of rut depth. The results of TBR analysis indicate that the greatest benefit was achieved by using the heavy-duty geogrid (resulting
in an improvement of up to 3.6 times the traffic sustained by the unreinforced test section), followed by the medium-duty geogrid (TBR of up to 2.4) and the light-duty geogrid (TBR of up to 2.3). The overall benefit ranking of the geogrids did not change with the change of rut depth.

Table 6.3: Summary of Traffic Benefit Ratio

<table>
<thead>
<tr>
<th>Test section</th>
<th>Reinforcement type</th>
<th>Trafficking stage</th>
<th>Rut level (mm)</th>
<th>$N_{ESAL(\text{add})}$</th>
<th>TBR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase I</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S3</td>
<td>G1</td>
<td>25</td>
<td>14</td>
<td>25</td>
<td>1.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>45</td>
<td>681</td>
<td></td>
<td>2.3</td>
</tr>
<tr>
<td>Phase II</td>
<td></td>
<td>55</td>
<td>462</td>
<td></td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>65</td>
<td>963</td>
<td>1.4 (1.38)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>75</td>
<td>1198</td>
<td></td>
<td>1.4</td>
</tr>
<tr>
<td>Phase I</td>
<td></td>
<td>25</td>
<td>20</td>
<td></td>
<td>1.9</td>
</tr>
<tr>
<td>S5</td>
<td>G3</td>
<td>45</td>
<td>1330</td>
<td></td>
<td>3.6</td>
</tr>
<tr>
<td>Phase II</td>
<td></td>
<td>55</td>
<td>808</td>
<td></td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>65</td>
<td>1642</td>
<td></td>
<td>1.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>75</td>
<td>--</td>
<td></td>
<td>&gt;&gt;1.7</td>
</tr>
<tr>
<td>Phase I</td>
<td></td>
<td>25</td>
<td>16</td>
<td></td>
<td>1.7</td>
</tr>
<tr>
<td>S9</td>
<td>G2</td>
<td>45</td>
<td>724</td>
<td></td>
<td>2.4</td>
</tr>
<tr>
<td>Phase II</td>
<td></td>
<td>55</td>
<td>589</td>
<td></td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>65</td>
<td>1099</td>
<td>1.4 (1.44)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>75</td>
<td>1334</td>
<td></td>
<td>1.5</td>
</tr>
</tbody>
</table>

It is noted that, within each loading phase, larger TBR values were obtained at larger rut depths. However, as the trafficking moved from the lighter to the heavier axle load, the benefit of the geogrids was less evident (decrease in TBR values). In general, the change in TBR with increased rut level is inversely proportional to the rate of deterioration of the reinforced test section (relative to the unreinforced section). When a reinforced test section deteriorates in an accelerating rate with traffic loading (i.e., shows an accelerating
rate of increase of surface rutting with cumulative ESAL passes), the TBR decreases with increased rut level, and vice versa. Similar TBR trends (smaller TBR values at greater rut level) reported by Austin and Coleman (1993) and Qian et al. (2013) support this observation. However, the drop in TBR values corresponding to rut levels reached in the second (heavier) loading phase is likely attributed to underestimating the number of ESAL passes in the second phase by Equations 6-3 and 6-4. It is believed that the fourth power rule underestimated the load equivalency factor for the heavier axle load as well as for the steering axle load. A corresponding drop in $N_{ESAL(add)}$ when transitioning from the lighter to the heavier axle load may support this conclusion.

### 6.1.3 Base Course Reduction Analysis

An alternative way to evaluate the relative performance benefit of geogrid reinforcement is to consider the reduction in aggregate quantities and initial construction cost by performing a base course reduction (BCR) analysis. The BCR factor is defined as a percentage savings of the unreinforced base thickness. It essentially quantifies the amount of thickness reduction that takes place in a geogrid reinforced base layer with respect to the thickness of a hypothetical unreinforced layer that performs equally, at a specified level of surface rutting. In order to accomplish this, unreinforced test sections would need to be constructed with base thicknesses so as to match the measured performance (TBR) of each of the reinforced test sections at the given rut depth.

A simple analysis based on the AASHTO 1993 Pavement Design Procedure (AASHTO, 1993) and the TBR values obtained in the previous section was used to evaluate the equivalent unreinforced sections and estimate the reduction in base course thickness due to the inclusion of geogrid reinforcement. The AASHTO 1993 procedure is based on performance prediction (rather than traffic prediction) in which the number of equivalent single-axle load (ESAL) applications that causes a tolerated reduction in serviceability index, $\Delta PSI$, is determined. This allowable number of ESAL applications during the design period is designated $W_{18}$, referring to the 18 kip (80-kN) equivalent single-axle load, and can be estimated by (AASHTO, 1993):
\[
\log W_{18} = Z_R S_0 + 9.36 \log(SN + 1) - 0.2 + \frac{\log \left[\frac{(PSI - p_t)}{(4.2 - 1.5)}\right]}{0.4 + \frac{1094}{(SN + 1)^{0.19}}} + 2.32 \log M_r - 8.07
\]

6-5)

where

\(Z_R\) = normal deviate for a given level of reliability, \(R\) (Table B.1 and Table B.2),

\(S_0\) = overall standard deviation (0.35 to 0.45),

\(SN\) = structural number of pavement (for unpaved road: structural number of base layer),

\(PSI\) = initial serviceability index (function of pavement type and construction quality);

  typical value from the AASHO Road Test was 4.2 for flexible pavements (Huang, 2004),

\(p_t\) = terminal serviceability index (the lowest index that will be tolerated before requiring rehabilitation or resurfacing),

\(M_r\) = resilient modulus of the subgrade soil which is calculated from the equation suggested by Georgia Department of Transportation for cohesionless soil (Webb and Campbell, 1986):

\[
M_r (\text{MPa}) = 21.48 \times [CBR]^{0.478}
\]

6-6)

The structural number is considered to determine the overall structural capacity of any roadway pavement. For unpaved test sections, the structural number is calculated from the following equation (AASHTO, 1993):

\[
SN = a_2 D_2 m_2
\]

6-7)

where

\(a_2\) = layer structural coefficient for the base course (Figure B.1),

\(D_2\) = base layers thicknesses,

\(m_2\) = drainage coefficient of base course (Table B.3).
The first step in the BCR analysis is to determine $W_{18}$ for the unreinforced section (S7). The design parameters used in the calculation of $W_{18}$ for the unreinforced section are summarized in Table 6.4. The actual base layer thickness of the unreinforced test section was used to calculate the structural number of the test section (Equation 6-7). Using the value of $SN$ and Equation 6-5, $W_{18}$ for the unreinforced section was determined.

In order to express the benefit of geogrid inclusion in terms of an equivalent unreinforced section, the contribution of TBR (at a specified rut depth) is factored into the design ESAL applications as follows (Webster, 1993):

$$W_{18(\text{reinforced})} = TBR \times W_{18(\text{unreinforced})}$$  \hspace{1cm} (6-8)

### Table 6.4: Input parameters for AASHTO 1993 Pavement Design Procedure

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Z_R$</td>
<td>-0.253</td>
</tr>
<tr>
<td>$S_0$</td>
<td>0.35</td>
</tr>
<tr>
<td>$P_S I$</td>
<td>4.2</td>
</tr>
<tr>
<td>$p_t$</td>
<td>1.5</td>
</tr>
<tr>
<td>$M_r$ (MPa)</td>
<td>32</td>
</tr>
<tr>
<td>$a_2$</td>
<td>0.13</td>
</tr>
<tr>
<td>$m_2$</td>
<td>1.2</td>
</tr>
</tbody>
</table>

The number of ESAL applications for the reinforced section, $W_{18(\text{reinforced})}$, can be used to back-calculate the required structural number ($SN$) and then, the required base layer thickness ($D_2$) of an equivalent unreinforced section using Equations 6-5 and 6-7. This equivalent unreinforced section matches the measured performance (TBR) of the reinforced test section at the given rut depth (sustains the same ESAL applications, $W_{18(\text{reinforced})}$). The reduction in base course thickness due to the inclusion of geogrid can then be calculated by subtracting the base thickness of the reinforced section from that of the equivalent unreinforced section.
The results of the BCR analysis are summarized in Table 6.5. $W_{19}$ of 3,114 ESAL passes was obtained for the unreinforced section (S7). This means that the unreinforced test section could sustain 3,114 80-kN equivalent single-axle load applications before any rehabilitation is required, based on terminal serviceability of $p_t = 1.5$. Factoring in the TBR values at varying levels of rut, the calculated number of ESAL applications for the geogrid reinforced sections (i.e., equivalent unreinforced sections) were between 3,737 and 11,200 ESALs. The results show that the structural number of the equivalent unreinforced sections (i.e., reinforced sections) increased on average by 10% compared to the unreinforced section (S7). For each of the reinforced test sections, the back-calculated equivalent unreinforced base thickness increased by the geogrid presence indicating that a greater unreinforced base thickness would have been required to match the increased structural number as a result of the geogrid inclusion. It is evident from the BCR analysis that the inclusion of geogrid results in a system with lower initial/construction cost that is able to support the same number of ESAL applications (same amount of traffic) with a reduced base layer thickness.

Overall, the data in Table 6.5 shows that the inclusion of geogrids reduced the base layer thickness by 6 to 25% at varying levels of rut depth. The results of the BCR analysis indicate that the greatest reduction in base thickness is achieved by using the heavy-duty geogrid (average of 16.1%), followed by the medium-duty geogrid (average of 9.9%), and the light-duty geogrid (average of 9.8%). In other words, the base layer thickness can be reduced on average by 39, 23, and 22 mm by reinforcing the control test section (S7) with heavy-, medium-, and light-duty geogrid, respectively. It can be seen that the light- and medium-duty geogrids resulted in similar aggregate savings. In addition, back-calculating the structural number of the equivalent unreinforced sections from their TBR factored ESAL applications, showed an average increase in the structural capacity of the unpaved test sections of 8, 10, and 14% as a result of the inclusion of the light-, medium-, and heavy-duty geogrid, respectively.
Table 6.5: Summary of base course reduction analysis for the 200 mm-base test sections

<table>
<thead>
<tr>
<th>Test Section</th>
<th>Reinforcement Type</th>
<th>Actual base thickness (mm)</th>
<th>Measured performance</th>
<th>Equivalent unreinforced section</th>
<th>Reduction in Base thickness (mm)</th>
<th>BCR (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Trafficking stage</td>
<td>Rut level (mm)</td>
<td>TBR</td>
<td>W₁₈</td>
</tr>
<tr>
<td>S7</td>
<td>Unreinforced</td>
<td>208</td>
<td>—</td>
<td>1</td>
<td>3114</td>
<td>1.28</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Phase I</td>
<td>25</td>
<td>4982</td>
<td>1.39</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>45</td>
<td>7162</td>
<td>1.49</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Phase II</td>
<td>55</td>
<td>3737</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>65</td>
<td>4297</td>
<td>1.36</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>75</td>
<td>4359</td>
<td>1.36</td>
</tr>
<tr>
<td>S3</td>
<td>G1</td>
<td>203</td>
<td>—</td>
<td>1.6</td>
<td>45</td>
<td>2.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Phase I</td>
<td>55</td>
<td>3737</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>65</td>
<td>4297</td>
<td>1.36</td>
</tr>
<tr>
<td>S5</td>
<td>G3</td>
<td>198</td>
<td>—</td>
<td>1.4 (1.38)</td>
<td>75</td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Phase II</td>
<td>55</td>
<td>4359</td>
<td>1.36</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>65</td>
<td>5294</td>
<td>1.41</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>75</td>
<td>4671</td>
<td>1.38</td>
</tr>
<tr>
<td>S9</td>
<td>G2</td>
<td>205</td>
<td>—</td>
<td>1</td>
<td>3114</td>
<td>1.28</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Phase I</td>
<td>25</td>
<td>5294</td>
<td>1.41</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>45</td>
<td>7473</td>
<td>1.50</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Phase II</td>
<td>55</td>
<td>4048</td>
<td>1.34</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>65</td>
<td>4484</td>
<td>1.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>75</td>
<td>4671</td>
<td>1.38</td>
</tr>
</tbody>
</table>
6.1.4 Comparison with Giroud and Han (2004) Design Method

The objective of this section is to compare actual performance of the unpaved test sections measured in this study to that predicted by Giroud and Han (2004) design procedure. The design method proposed by Giroud and Han (2004) is widely used to determine the base course thickness required to support traffic on unpaved roads. The method accounts for parameters such as: interlock between geogrid and aggregate, aperture stability modulus of the geogrid, subgrade strength, base layer stiffness, traffic volume, wheel loads, tire pressure, rut depth, and influence of the presence of a geosynthetic on the failure mode of the unpaved road. The results of the full-scale field experiment were used to compare the performance of individual test sections with the predicted performance using the Giroud and Han (2004) design method. Specifically, the actual loading parameters, material properties of the test sections, and observed performance data were used as design inputs. According to Giroud and Han method, the required base course thickness for unreinforced and/or reinforced roads is determined using the following iterative equation (Giroud and Han, 2004):

\[
h = \frac{0.868 + (0.661 - 1.006J^2)\left(\frac{T}{h}\right)^{1.5} \log N}{1 + 0.204 [R_E - 1]} \times \left[ \sqrt{\frac{P}{\pi r^2}} \left(\frac{s}{75}\right) \left[1 - 0.9 e^{-\left(\frac{r}{h}\right)^2}\right] (30N_CBR_{sg}) \right] - 1 \]  \tag{6-9}

where

- \( h \) = required base thickness (m),
- \( J \) = geogrid aperture stability modulus (N-m/deg),
- \( N \) = number of passes,
- \( P \) = wheel load (kN),
- \( r \) = radius of the equivalent tire contact area (m),
- \( s \) = allowable rut depth (mm),
$R_E = \text{limited modulus ratio of base course to subgrade soil,}$

$$R_E = \min (3.48 \, CBR_{bc}^{0.3} / CBR_{bc}, \, 5),$$

$CBR_{bc} = \text{base course California bearing ratio}$

$CBR_{sg} = \text{subgrade California bearing ratio}$

$N_c = \text{bearing capacity factor (3.14 for unreinforced, 5.14 for geotextile-reinforced, 5.71 for geogrid-reinforced unpaved roads).}$

Equation 6-9 was used to predict the required base thickness. The predicted base thicknesses were then compared to the actual base thicknesses as summarized in Table 6.6 and depicted in Figure 6.2. The considered reinforcements have aperture stability moduli as reported by the manufacturers listed in Table 6.6. The required base thicknesses of the test sections were calculated for their respective number of traffic passes observed at a rut depth of 65 mm (rounded value of highest rut depth reached in the test sections).

Overall, the results indicate that the Giroud and Han (2004) method underpredicts the thickness of base layer required to carry the applied traffic by 17 to 49%. In other words, the method overpredicts the performance (i.e., number of traffic passes to develop a 65-mm rut). It is shown in Figure 6.2 that the underprediction of the base thickness is greater for the geogrid-reinforced sections compared with the control section. Furthermore, the underprediction is most pronounced in the case of the heavy-duty geogrid, followed by the medium-duty geogrid, then the light-duty geogrid. This means worst predictions were given for geogrids with a larger aperture stability modulus, the only parameter used to differentiate between various geogrids in the method. These observations are consistent with similar observations reported and discussed by Cuelho and Perkins (2009) based on field data from 12 test sections, including 10 different geosynthetic products. These observations suggest that the aperture stability modulus might not be sufficient on its own to represent the response of the geogrid material, and other geogrid properties should be considered beside the aperture modulus in the design method. More investigations into the adequacy of the aperture stability modulus to represent the behavior of the geogrid material in road applications are needed. The initial compaction of the base layer.
observed during trafficking (discussed in Section 5.1.1) may also have contributed to some extent to this overprediction of performance at small ruts and led to additional underprediction of the required base thickness.

Table 6.6: Summary of comparison of field results with Giroud and Han (2004) method

<table>
<thead>
<tr>
<th>Test section</th>
<th>Reinforcement type</th>
<th>$J^1$ (N-m/deg)</th>
<th>$N_{ESAL}$ (at 65-mm rut)</th>
<th>Actual base thickness (mm)</th>
<th>Predicted base thickness (mm)</th>
<th>Percent error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3</td>
<td>G1</td>
<td>0.235</td>
<td>3481</td>
<td>203</td>
<td>138</td>
<td>32</td>
</tr>
<tr>
<td>S5</td>
<td>G3</td>
<td>0.559</td>
<td>4160</td>
<td>198</td>
<td>100 $^2$</td>
<td>49</td>
</tr>
<tr>
<td>S7</td>
<td>Control</td>
<td>0</td>
<td>2518</td>
<td>208</td>
<td>172</td>
<td>17</td>
</tr>
<tr>
<td>S9</td>
<td>G2</td>
<td>0.333</td>
<td>3617</td>
<td>205</td>
<td>127</td>
<td>38</td>
</tr>
</tbody>
</table>

$^1$ Aperture stability modulus of the geogrids as reported by the manufacturers (Table 3.2)

$^2$ Minimum thickness of 100 mm was used as recommended by Giroud and Han (2004) to prevent disturbance of the subgrade soil due to trafficking.

Figure 6.2: Comparison of actual and predicted base thickness using Giroud and Han method
6.2 Base Layer Degradation Back-Analysis

Performance benefits of geogrid reinforcement were discussed in the previous section in terms of extending the service life of the road and reducing the required aggregate thickness. In this section, the geogrid reinforcement benefit in improving the mechanical properties of the unpaved structure under traffic loading is discussed. Specifically, the benefit of geogrid reinforcement with respect to slowing the progressive deterioration or degradation of the base layer due to traffic loading is considered. The degradation of base layer was demonstrated in the field test results as an increase in maximum or dynamic vertical stress transferred to the subgrade with cumulative traffic passes (Section 5.3.1). The results of the full-scale field testing are analyzed (in this section) to describe the performance with degradation of the base layer. Based on measurements of vertical stress on top of the subgrade, back-calculation analyses are conducted to evaluate the degradation of base layer through the change in stress distribution angle and base-to-subgrade modulus ratio with cumulative traffic passes.

6.2.1 Stress Distribution Angle

The integrity of the base layer is essential for good performance of an unpaved structure. The main role of the base layer is to distribute the applied traffic loads to a wider area of the subgrade so the subgrade stress remains below the level that causes excessive subgrade deformation (Giroud, 2009). The effectiveness of the base layer to distribute applied loads is related to its thickness and its mechanical properties (stiffness). Accordingly, the degradation of base layer can be evaluated using the change in stress distribution angle with cumulative traffic passes. The stress distribution angle plays an essential role in pavement design. It describes the ability of the base layer to distribute vertical stresses induced on the unpaved road surface by applied traffic loads. The stress distribution angle is influenced by the properties of the base course material and the subgrade soil, the number of traffic passes, and the geogrid properties (Giroud and Han, 2004).
For roads subjected to traffic loading, the stress distribution angle can be estimated using one of the following methods:

- based on geometrical measurement of the stress distribution angle using the deflected shapes of the road surface and the base-subgrade interface (Sigurdsson, 1993),
- based on the area under the surface (e.g., at the interface) within which the majority of the applied traffic load is distributed (Lawton, 1996),
- based on uniform distribution of the maximum vertical stress measured under the center of the loaded area (Leng and Gabr, 2002; Giroud and Han, 2004).

The last two methods require that the vertical stress at a particular point below the surface (e.g., at the base-subgrade interface) be known. The method based on the uniformly distributed maximum vertical stress is used in this research. This method is simple, conservative, and by far the most commonly used. In addition, it is suitable for use based on the vertical stress measurements collected from the field testing. The method is based on the following assumptions:

- the applied wheel load is uniformly distributed over the wheel contact area,
- the actual wheel contact area is replaced by an equivalent circular area,
- the maximum stress on the subgrade (i.e., under the center of the wheel contact area) is uniformly distributed according to the approximate method with a stress distribution angle, \( \alpha \),
- the base layer thickness remains relatively constant.

Accordingly, the stress distribution angle, \( \alpha \) is given by the following equation (Giroud and Han, 2004):

\[
\tan \alpha = \frac{r}{h} \left[ \frac{p}{\pi r^2 \sigma} - 1 \right]
\]  \hspace{1cm} 6-10

where

\( h \) = base layer thickness (m),
\[ P = \text{wheel load (kN)},\]

\[ \sigma = \text{vertical stress at the base-subgrade interface (i.e., maximum vertical stress measured on the subgrade.) (kPa)}, \]

\[ r = \text{radius of the equivalent tire contact area (m); } r = \sqrt{\frac{P}{\pi p}}, \text{ } p \text{ is the tire contact pressure in kPa}. \]

From the measured dynamic/maximum vertical stress at the top of subgrade for each traffic pass, and knowing the applied wheel load, the thickness of the base layer, and the radius of the equivalent wheel contact area, the stress distribution angle can be back-calculated for each traffic pass during the field test. The back-calculated stress distribution factor, \( \tan \alpha \), for the 200 mm- and 250 mm-base test sections are shown in Figure 6.3 as a function of cumulative traffic passes. The results are reported up to a traffic level of approximately 2,000 passes to avoid possible effects of the heavy precipitation around this level of traffic on the calculations. Overall, the results of the back-analysis show that the base layer was initially compacted (as discussed in Section 5.3.1), which resulted in an increase in stress distribution angle. This initial improvement was followed by a decrease in stress distribution angle with increased traffic passes indicating degradation of the base layer.

For the same base layer thickness, it is observed that the geogrids improved the base layer ability to distribute vertical stress by reducing the degradation of \( \alpha \) with traffic passes (i.e., increased distribution angle) compared with the unreinforced section. This decreased the rate of increase in the maximum/dynamic vertical stress with traffic passes for the reinforced sections as discussed in Section 5.3.1. In addition, the heavier duty geogrid resulted in increased stress distribution angle (transferred vertical stress to a wider subgrade area) as compared with the lighter duty geogrids. After 2,000 traffic passes, the distribution angle factor for the 200 mm-base test section increased by 28, 35, and 84% due to the inclusion of light-, medium-, and heavy-duty geogrid, respectively. The distribution angle factor for the 250 mm-base test section increased by 14% due to the inclusion of the light-duty geogrid. It is also noted that the inclusion of geogrid reinforcements (the heavy-duty geogrid in particular) resulted in higher initial stress
distribution angle (Figure 6.3). This result suggests that the geogrid may offer some stabilization benefits during construction.

![Graph](image)

(a) 200 mm-base sections

![Graph](image)

(b) 250 mm-base sections

Figure 6.3: Back-calculated stress distribution angle factor ($\tan \alpha$) vs number of passes
For the same reinforcement condition, the decrease in $\alpha$ with traffic passes was less pronounced in the 250 mm-base sections as compared with the 200 mm-base sections. As the thickness of the base layer increases, its degradation with repeated traffic loading decreases. In addition, the improvement in stress distribution angle by the geogrid (G1) was less evident in the 250 mm-base section (14\% increase in $\tan\alpha$) than in the 200 mm-base section (28\% increase in $\tan\alpha$) and the improvement was only clearly distinguished under the heavier axle load. Figure 6.3 also shows that the base layer with greater thicknesses had slightly lower initial stress distribution angles. Accordingly, it is concluded that the change in stress distribution by the base layer with traffic loading or the degradation of the base layer is affected by the grade/properties of the geogrid reinforcement as well as the thickness of the base layer itself.

6.2.2 Modulus Ratio

The deterioration of the unpaved structure results mainly from the degradation of base layer (Grioud et. al., 1985). Therefore, the benefit from the geogrid reinforcement in reducing the degradation (or increasing the modulus) of the base course is discussed in this section. Based on theory of elasticity, the vertical stress transferred to the top of the subgrade depends on the thickness of base layer and the base-to-subgrade modulus ratio (Burmister, 1958). Accordingly, assuming that the base thickness remains relatively constant, the degradation of unpaved structures under repeated traffic loads is evaluated using the change in base-to-subgrade modulus ratio with cumulative traffic passes. However, the unpaved road system consists of two layers, a base and subgrade, with a higher modulus for the base layer compared to the subgrade. In order to accurately determine the effect of base layer modulus on unpaved road performance, the elastic theory for two-layer systems must be used, which involves complex calculations. For simplicity, the concept known as the method of equivalent thickness (MET) or Odemark’s method (Ullidtz, 1987) is used instead to describe pavement responses (i.e., stresses and strains) within such an elastic layered system. The MET is an approximate method to transform a multi-layer pavement system with different moduli into an equivalent homogenous system (one-layer elastic half-space with one elastic modulus) on
which Boussinesq’s solution may be applied. The method has been widely used for pavement response analyses and it is based on the following assumptions:

- the base layer thickness remains relatively constant,
- the behavior of base and subgrade layers can be simplified as linear elastic.

In addition, the use of Boussinesq’s solution involves making the following basic assumptions:

- the applied wheel load is uniformly distributed over the tire contact area,
- the actual wheel contact area is replaced by an equivalent circular area,
- the maximum stress on the subgrade (i.e., under the center of the tire contact area) is uniformly distributed according to the approximate method,
- the base layer thickness remains relatively constant during traffic loading.

According to Odemark, the stiffness of a layer is proportional to the following term (Ullidtz, 1987):

\[
\text{Stiffness} \propto \frac{h^3 E}{1 - v^2}
\]  

(6-11)

where \( h \) = thickness of the layer, \( E \) = elastic modulus, and \( v \) = Poisson’s ratio.

Based on Odemark’s method, the base layer in an unpaved road can be replaced by an equivalent layer that has the same stiffness as the original layer but with an equivalent thickness, \( h_e \), and the same modulus and Poisson’s ratio as the subgrade (Ullidtz, 1987):

\[
\frac{h^3 E_{bc}}{1 - v_{bc}^2} = \frac{h_e^3 E_{sg}}{1 - v_{sg}^2}
\]  

(6-12)

By rearranging Equation 6-12, the equivalent thickness of the base layer, \( h_e \), can be determined (Ullidtz, 1987):

\[
h_e = h \left[ \frac{E_{bc}(1 - v_{sg}^2)}{E_{sg}(1 - v_{bc}^2)} \right]^{1/3}
\]  

(6-13)

where

\( h \) = thickness of base course (m),
\( E_{bc} \) = elastic modulus of base course (MPa),
\( E_{sg} \) = elastic modulus of subgrade (MPa),
\( \nu_{bc} \) = Poisson’s ratio of base course, and
\( \nu_{sg} \) = Poisson’s ratio of subgrade.

For this equivalent homogenous system, stresses and strains at the interface (of originally two-layer pavement system) are calculated by treating the two-layer system as one layer with modulus \( E_{sg} \). The vertical stress under the center of the wheel loaded area at the base-subgrade interface can be expressed using Boussinesq’s solution as (Ullidtz, 1987):

\[
\sigma = p \left[ 1 - \frac{h_e^3}{(r^2 + h_e^2)^{1.5}} \right]
\]

where
\( \sigma \) = vertical stress at the base-subgrade interface (i.e., maximum vertical stress measured on the subgrade) (kPa),
\( p \) = tire contact pressure or tire inflation pressure (kPa),
\( h_e \) = equivalent thickness of the base layer (m),
\( r \) = radius of the equivalent tire contact area (m); \( r = \sqrt{P/\pi p} \).

The Odemark’s or MET method is valid under the following conditions (Ullidtz, 1987): the ratio of the moduli of the base to subgrade layer materials is greater than 2 (i.e. \( E_{bc}/E_{sg} > 2 \)); and the transformed or equivalent thickness of the base layer be larger than the radius of loaded area (i.e., \( h_e > r \)). However, Dalla Valle and Thom (2018) proposed a relaxation for the base-to-subgrade modulus ratio, i.e., \( E_{bc}/E_{sg} \geq 1 \), instead of 2.

The performance of unpaved roads is mainly evaluated based on the stiffness/modulus of the layer materials under repeated traffic loads (e.g. resilient modulus). The subgrade modulus was estimated using Equation 6-6, an empirical relationship with the CBR value suggested by Georgia Department of Transportation for cohesionless soil (Webb and Campbell, 1986). Poisson’s ratios of 0.35 and 0.31 were used in this analysis for the base course and subgrade soil, respectively. With the measured dynamic/maximum vertical stress on the subgrade for each traffic pass, the tire inflation pressure, and the radius of
the equivalent wheel contact area, the equivalent thickness of the base course can be back-calculated using Equation 6-14. The modulus ratio can then be back-calculated using Equation 6-13, and knowing the modulus of the subgrade, the modulus of the base course material can then be estimated for each pass of the field traffic. The results of the modulus ratio back-analysis for the 200 mm-base test sections and the 250 mm-base test sections are presented in Figure 6.4.

The general trend is similar to the one observed in the stress distribution angle discussed in the previous section. It can be seen that, after initial increase due to initial compaction, the modulus ratio decreased with cumulative traffic passes in all test sections indicating degradation of the base layer. The results show that the grade/properties of the geogrid reinforcement and the thickness of the base layer influenced the change in modulus ratio with increased traffic passes. It is shown that towards the end of the test the modulus ratio of the reinforced 200-mm bases increased from the unreinforced case by 61, 76, and 235% due to the inclusion of light-, medium-, and heavy-duty geogrid, respectively. For the 250-mm bases, the modulus ratio increased by 36% as a result of including the light-duty geogrid.

These results show that the geogrids reduced the degradation of the base layer due to repeated traffic loads by providing increased modulus ratio (i.e., slower rate of degradation in modulus ratio) compared with corresponding unreinforced sections. Moreover, the rate of degradation in modulus ratio was generally lower when a heavier duty of geogrid was used. Similar to the stress distribution angle, it is seen that the geogrids showed impact on the initial modulus ratio (Figure 6.4a), which suggests a stabilization benefits provided by the geogrids during construction. These results are consistent with those obtained from measurements of locked-in vertical stress and geogrid strain during construction.

Based on results of laboratory cyclic plate load tests by Gabr (2001), Giroud and Han (2004) assumed in their design procedure that the initial stress distribution angle, has the same value for unreinforced and reinforced cases, the geogrid has no effective before some strain is developed in the base layer with repeated traffic loading in order to mobilize the reinforcement. It should be noted, however, compaction of base course
material in a laboratory test tank cannot replicate the process of compaction under real construction conditions that results in an increased placement density and a buildup of locked-in horizontal stresses in the base layer, as previously indicated by results from this study (Sections 5.3.2.1 and 5.4.4).

Figure 6.4: Back-calculated elastic modulus ratio vs number of passes
It is clear from Figure 6.4 that the degradation rates of modulus ratio were generally higher for the 200-mm bases compared with the 250-mm bases with the same reinforcement condition. In addition, the increase in modulus ratio by the light-duty geogrid was less pronounced for the 250-mm base (improvement was reduced almost by half). The results show also that the base layer with greater thicknesses had lower initial modulus ratio.

It should also be noted that Giroud and Han (2004) limited the maximum base-to-subgrade modulus ratio to 5.0 in their design method based on data reported by Heukelom and Klomp (1962), which was based on unreinforced unpaved test sections. The range of modulus ratio reported were between 1.0 and 5.0. Giroud and Han (2004) recommended further investigations to evaluate if modulus ratio limit greater than 5.0 may be used for the design of geogrid-reinforced unpaved roads. The results of this full-scale field study demonstrate that the inclusion of stiff geogrids allows for improved compaction and results in modulus ratios greater than 5.0 (modulus ratios of up to 10 were calculated in this study). Accordingly, it is recommended that the modulus ratio limit of 5.0 be increased for the geogrid-reinforced case in the Giroud and Han design method.

**General Observations:**

It is noted that the modulus ratio (and stress distribution angle) increased with the increase in axle load in the second phase of trafficking which is attributed to an increase in the modulus of the base course material. Granular materials generally exhibit an increase in elastic and resilient moduli with increasing applied load or mean stress (Uzan, 1985; Perkins, 1999). This is consistence with the observation discussed in Section 5.3.1 that the dynamic stresses on top of the subgrade did not increase linearly with the increase in axle load; an increase of 25% in axle load resulted in an average increases of 11% in dynamic stress at the top of subgrade (only 40% of the increase in axle load). The effect of the heavier axle load was more evident in the modulus ratio than in the stress distribution angle, which is attributed to an increase in aggregate modulus. The slight
improvement noticed in stress distribution angle with the increase of axle load (Figure 6.3) is in fact attributed to the increase in modulus of the base layer discussed above.

It is also noted that the increase in modulus ratio (and stress distribution angle) due to the application of heavier axle load was more evident as the thickness of the base layer increased. In addition, the increase in modulus ratio with the heavier load was more noticeable in the reinforced sections, and became more pronounced when a heavier duty geogrid was used. This may be related to the lateral confinement provided by the geogrid reinforcement to the aggregate at the bottom of the base layer under the greater axle load. The lateral restraint by the geogrid leads to increased horizontal and hence mean stress in the reinforced base compared with the unreinforced base. In other words, the mean stress at the bottom of the base layer increases under the heavier load due to the presence of the geogrid, which leads to an additional increase in the stiffness of the granular material (Perkins, 1999) compared with the unreinforced section. This conclusion is supported by the slight increase in the measured dynamic geogrid strain (i.e., increase in dynamic geogrid force) under the heavier axle load discussed in Section 5.4.2, which indicate an increase of lateral restraint to the aggregate by the geogrids.

It is of interest to note that the previous analyses are conservative with respect to estimating the stress distribution angle and the modulus ratio. The measured maximum or dynamic vertical stress on the subgrade was conservatively taken as the uniformly distributed stress over the subgrade. This will result in lower values of modulus ratio and stress distribution angle. In addition, the assumption of constant base course thickness may overestimate the degradation (underestimate \( \alpha \) and \( E_{bc}/E_{sg} \)) if the actual base thickness decreases under repeated traffic loading. Nevertheless, the influence of these assumptions is insignificant on the evaluation of geogrid benefit in improving the mechanical properties or degradation of the base layer (which involves comparing degradation of test sections) in the sense that all test sections are affected by this in a similar way.
6.2.3 Influence of Various Parameters on Stress Distribution

Unpaved roads deteriorate under repeated traffic loads. Consequently, the stress distribution angle decreases with increasing number of traffic passes. The influence of traffic was demonstrated through the results presented in Figure 6.3 discussed in Section 6.2.1. In addition, the change in stress distribution on the subgrade under traffic loading depends on the thickness of base layer and the base-to-subgrade modulus ratio, as well as the properties of the geogrid (Giroud and Han, 2004).

When traffic loads are applied to the unpaved road, shear or horizontal tensile stresses will be induced at the bottom of the base layer. As a result, the aggregate is susceptible to spread laterally and the modulus of the granular material decreases at the bottom of the base layer. The deterioration of the base layer with repeated traffic loads results in a redistribution of vertical stress over a narrower area of the subgrade and an increase of vertical stress on the subgrade. The base layer must have sufficient stiffness to support the load without failing due to the shear stresses induced by traffic loads. In addition, the thickness of the base layer must be sufficient to distribute the vertical stress over a wider area of the subgrade such that the vertical stress is reduced to a magnitude where plastic deformations of the subgrade soil is minimized. Figure 6.5 shows the degradation of stress distribution angle during traffic loading for each test section as a function of base-to-subgrade modulus ratio. The stress distribution angle is presented in Figure 6.5 in terms of a stress distribution tangent ratio (i.e., ratio of stress distribution angle after $N$ traffic passes, $\tan\alpha_N$, to the initial stress distribution angle, $\tan\alpha_1$).

It can be clearly seen that the stress distribution angle decreases as the base-to-subgrade modulus ratio decreases. It should be noted that curves with steeper slopes indicate greater rates of degradation in stress distribution angle, while curves with shallower slopes indicate lower rates of degradation in stress distribution angle. The data in Figure 6.5 indicates that the distribution angle decreased faster in the case of unreinforced sections compared to the reinforced sections. This can also be surmised from the final values of distribution angle ratio being significantly lower for the unreinforced sections than for the reinforced sections. Accordingly, it is apparent that the geogrid inclusion influenced the stress distribution and resulted in reducing the rate of degradation in stress.
distribution angle under traffic loading. It is also noted that the improvement in stress distribution was more pronounced when a heavier duty geogrid was used.

Figure 6.5: Relationship between stress distribution and base-to-subgrade modulus ratio
It should be noted that plots for the test sections originate from different locations at the top right side of Figure 6.5 due to the differences in their initial distribution angle values (tan\(\alpha_1\)). Sections with greater initial distribution angles are plotted closer to the right side of the figure. For the same value of stress distribution tangent ratio in Figure 6.5 (for any traffic pass \(N\)), test sections on the right side have greater modulus ratio and greater distribution angle as a result of their greater tan\(\alpha_1\). In other words, the position of the test section plot is governed by its initial distribution angle. Greater initial distribution angle results from an inclusion of geogrid and/or a decrease in base layer thickness. This is clearly demonstrated by the data in Figure 6.5 with the geogrid-reinforced test sections plotted further in the right side compared with the unreinforced section (for the same base thickness) while the test sections with larger base thickness (Figure 6.5b) plotted in the left side compared with sections with smaller base thickness (for the same reinforcement conditions). In general, the stiffer the geogrid the greater the initial distribution angle, for the same base thickness.

Results from this study have indicated that the geogrid reduced the degradation of the base layer due to repeated traffic loads by increasing the modulus ratio and consequently improving stress distribution. Without geogrid, the modulus typically decreases at the bottom of the base layer because of lateral spreading and sinking of aggregate, and lack of sufficient support from underlying subgrade soil. However, geogrid reinforcement resists the shear stresses induced at the bottom of the base layer, restricts lateral spreading of aggregate resulting in an increased base modulus, which improves the stress distribution on the subgrade. In addition, the presence of geogrid benefits the base layer during construction and results in a base layer with higher initial modulus.

The results of the modulus ratio back-analysis from this study (Section 6.2.2) were used to develop a relationship between the stress distribution tangent ratio and the parameters discussed above using regression analysis. The following regression equation was derived in this study:

\[
\frac{\tan \alpha_N}{\tan \alpha_1} = a R_{E,N}^{0.5}
\]

\[6-15\)
where

\[ \alpha_N = \text{stress distribution angle after } N \text{ traffic passes}, \]

\[ \alpha_1 = \text{initial stress distribution angle}, \]

\[ R_{E,N} = \text{modulus ratio of base course to subgrade soil after } N \text{ traffic passes}; \]

\[ R_{E,N} = \frac{E_{bc,N}}{E_{sg}}, \]

\[ E_{bc,N} = \text{elastic modulus of base course after } N \text{ traffic passes (MPa)}, \]

\[ E_{sg} = \text{elastic modulus of subgrade (MPa)}, \]

\[ a = \text{function describes the effect of the initial conditions (i.e., the effect of geogrid and base layer thickness)}; \]

\[ a = c \left[ \frac{r}{h} \right]^{-0.792} R_{E,1}^{-0.245} \]

where

\[ c = \text{constant equal to 0.45626}, \]

\[ r = \text{radius of the equivalent tire contact area (m)}, \]

\[ h = \text{thickness of base course (m)}, \]

\[ R_{E,1} = \text{initial modulus ratio of base course to subgrade soil}; \]

\[ R_{E,1} = \frac{E_{bc,1}}{E_{sg}}, \]

\[ E_{bc,1} = \text{initial elastic modulus of base course (MPa)}, \]

\[ E_{sg} = \text{elastic modulus of subgrade (MPa)}. \]

As was discussed earlier, the effect of the geogrid on the stress distribution angle lies in increasing the initial value of the base layer modulus during compaction or construction of the base (\( R_{E,1} \)) and in reducing the rate of degradation in the base modulus with traffic passes (\( R_{E,N} \)). Figure 6.6 displays plots of stress distribution ratio determined from vertical stress measurements versus those predicted using Equation 6-15 for each of the test sections.
Figure 6.6: Estimated degradation of stress distribution angle using Equation 6-15
6.3 Forces in Geogrids

Forces developed in geogrids due to traffic loading are expected to be much lower than their tensile strength. Nonetheless, the estimation of these forces is essential to quantifying the effectiveness of geogrids in reinforcing roadways. The magnitude of these forces produced in the geogrids indicates the amount of traffic-generated horizontal loads confined and distributed by the geogrids, hence provides insights into the respective engagement of different geogrids with the unpaved structure. The tensile forces are determined from the measured permanent strains along with the geogrid tensile moduli at small strains (2%). Only strain data in the transverse direction are used to calculate geogrid forces since tension was not mobilized in the longitudinal direction.

6.3.1 Dynamic Forces Induced in Geogrids

The dynamic geogrid strains measured during traffic loading in the transverse direction discussed in Section 5.4.2.1 can be used to calculate dynamic forces induced in the geogrids. Tensile forces were computed by multiplying the final values of dynamic geogrid strain shown in Figure 5.15 by the geogrid elastic modulus at small strain (2%), which was determined from tensile strength data at small strain in accordance with ASTM D6637. The results are summarized in Table 6.7, which shows that dynamic forces between 1.12 and 1.68 kN/m were induced in the geogrids at the first traffic pass. The dynamic forces decreased to 0.5 and 0.72 kN/m due primarily to the effect of initial compaction previously described. These forces are attributed to the transmission of dynamic tensile loads from the aggregate to the geogrids through the development of shear-interaction between the aggregate and the geogrids. Part of these dynamic forces remains in the geogrids after the wheel load moves away as indicated by the accumulation of permanent geogrid strains in the transverse direction (Section 5.4.3.1). It is noted that larger dynamic tensile forces were induced in the stiffer geogrids. This indicates that shear-interaction between the aggregate and the geogrid was greater for the stiffer geogrids than it was for the less stiff geogrids. In this sense, the dynamic geogrid forces appear to correlate reasonably well to long-term rutting performance of the test
sections. Furthermore, for the same geogrid reinforcement, the induced dynamic force decreased as the thickness of the base layer increased, suggesting less engagement between the geogrid and the surrounding aggregate. This is also in general agreement with the rutting performance of the test sections.

### Table 6.7: Dynamic force induced in the geogrids (transverse direction)

<table>
<thead>
<tr>
<th>Test section</th>
<th>Geogrid</th>
<th>Elastic modulus (kN/m)</th>
<th>Dynamic strain (%)</th>
<th>Dynamic force (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3</td>
<td>G1</td>
<td>350</td>
<td>0.33</td>
<td>0.180</td>
</tr>
<tr>
<td>S4</td>
<td>G1</td>
<td>350</td>
<td>0.32</td>
<td>0.133</td>
</tr>
<tr>
<td>S5</td>
<td>G3</td>
<td>600</td>
<td>0.28</td>
<td>0.120</td>
</tr>
<tr>
<td>S9</td>
<td>G2</td>
<td>400</td>
<td>0.29</td>
<td>0.165</td>
</tr>
<tr>
<td>S10</td>
<td>G2</td>
<td>400</td>
<td>0.28</td>
<td>0.125</td>
</tr>
</tbody>
</table>

#### 6.3.2 Tensile Forces Mobilized in Geogrids

While the dynamic tensile force induced in the geogrid may provide some insight into the transmission of dynamic tensile load from the aggregate to the geogrid (interaction between the geogrid and aggregate), it is the force mobilized in the reinforcement that provides the permanent restraint to lateral movement of aggregate. Accumulation of tensile force in the geogrid with repeated traffic passes means that the geogrid continuously resists lateral movement of the base aggregate (i.e., providing lateral confinement). This lateral restraint provided by the geogrid to the aggregate leads to a buildup of horizontal (confining) stress locked into the aggregate. As a result, the mean stress at the bottom of the base layer increases which causes the stiffness of the granular material to increase as previously discussed. It should be noted that tension was only mobilized in geogrid reinforcement in the transverse direction, directly under the wheelpath (where greatest lateral movement of the base aggregate is expected to occur).
The geogrid tensile forces developed in the transverse direction directly under the wheel-path are presented in Figure 6.7. The data shows that the forces mobilized in the geogrids even with small amount of rut. An immediate accumulation of tensile forces is clearly recorded at the onset of trafficking. Overall, magnitudes of tensile force between 1.5 to 3 kN/m were developed in the geogrids, which indicates an engagement of geogrids in resisting shear stresses at the bottom of the base layer. The magnitude of geogrid forces in the reinforced sections generally follows the same order of response seen in the test sections with regards to the subgrade vertical stress (Section 5.3.1). For the same base layer thickness, the heavy-duty geogrid mobilized greater tensile force as compared with the light- and medium-duty geogrids. Despite the lower strains measured in the heavy-duty geogrid relative to the other two geogrids, its comparatively higher tensile modulus corresponded to greater tensile force. This is also consistent with the rutting performance observed in the test sections. The difference between tensile forces mobilized in the light- and medium-duty geogrids was indistinguishable for a significant part of the test due to their comparable stiffnesses (Table 6.7). Similar response was observed in the dynamic vertical stresses transferred to the subgrade (Figure 5.7), with a better distinction between the two geogrids demonstrated in the mobilized geogrid forces data. It is also noted that, for the same geogrid reinforcement, a lower magnitude of tensile force was mobilized in the geogrid for the thicker base layer. This indicates that transmission of tensile loads or, in other words, the interaction between the geogrid and the aggregate decreases with the increase in base thickness. This is also consistent with the rutting performance of the test sections.
Figure 6.7: Geogrid reinforcement force in transverse direction

Figure 6.8 shows the forces mobilized in the geogrid installed in Section 9, along with corresponding surface deformations for comparison purposes. As can be seen, the tensile force developed in the geogrid correlates reasonably well with the performance of the reinforced test section throughout the test. This strong correlation further demonstrates the engagement of the geogrids with the unpaved structure at different levels of strain.
Examination of the geogrid force data presented in this section along with surface deformation and vertical stress measurements from corresponding test sections discussed in previous parts of this study, reveals that the heavy-duty geogrid (has highest tensile stiffness) showed overall greater mobilized tensile force in the geogrid, less vertical stress transferred to the subgrade, and lower surface deformation. In addition, the geogrid that mobilized greater tensile force also resulted in less contribution from the subgrade to total permanent deformation (Table 5.2) compared with the geogrid of lower mobilized force. These results indicate that the geogrid’s contribution to the enhancement of performance of the unpaved structure is strongly correlated to the tensile forces developed in the geogrid. Furthermore, this conclusion suggests that the effectiveness of geogrids in improving the performance of unpaved structures is strongly related to their tensile stiffness.

![Geogrid force and surface deformation versus number of passes](Figure 6.8)

(Figure 6.8: Geogrid force and surface deformation versus number of passes (Section 9))
6.4 Reinforcement Mechanisms

In order to identify the mechanisms through which the geogrid improves the performance of the unpaved system, it is necessary to understand how the geogrid engages with the surrounding material. The amount of permanent tensile strain accumulated in the geogrid indicates the extent to which the geogrid tensile resistance is mobilized and how much the geogrid is interacting with the aggregate. In addition, having a thorough knowledge of strain distribution in the geogrid under traffic loads is a key element to identify the governing reinforcement effect. Thus, the strain distribution within the geogrid (used as a reinforcement for roadways subjected to traffic loads) must be well characterized. The distribution of tensile strains in the geogrid was previously examined based on measurements of permanent strain (Section 5.4.3). Conclusions drawn from that discussion are utilized here to help identify which reinforcement mechanism is causing the observed performance improvement. It was concluded in Section 5.4.3 that permanent geogrid strain measurements from this study had the following features:

- Strain accumulated immediately upon the first traffic pass and well before any appreciable surface rutting was developed in the unpaved system.
- Geogrid was not under constant tension across the road.
- Significant tensile permanent geogrid strain was developed only in the transverse direction directly beneath the wheel-path. This tensile strain, peaked under the center of the wheel load but diminished with distance away from center.
- The geogrid experienced small magnitudes of “compressive” dynamic and permanent strains in the transverse direction away from the wheel-path (between the wheels).
- The geogrid did not show any tendency for mobilizing tension in the longitudinal direction (i.e., tension was not mobilized in the longitudinal direction).

According to the tensioned membrane effect, the tensile capacity of the geogrid plays no role until the unpaved road experiences large amounts of surface rutting (Giroud, 2009). Therefore, tension is assumed to develop in the geogrid as a result of excessive rutting and associated large deformation of base and subgrade layers (concave shape forms) rather than due to shear interaction between the geogrid and the aggregate. The concave
shape suggests that tension develops laterally in the geogrid almost across the entire width of the road or at the very least to a great extent between the two wheel-paths. Similarly, the mechanism of subgrade vertical confinement or “inverted tensioned membrane” is based on the assumption of developing tension in the geogrid between the wheels to provide vertical confinement to the subgrade soil in the area away from the wheel loads. However, the distribution of geogrid strain described above collides with this assumption. In addition, the immediate accumulation of strain at small rut depths suggests different mechanism of reinforcement that doesn’t rely on the development of large amounts of rut. Since both the tensioned membrane effect and the subgrade vertical confinement rely on the assumption of tensioned geogrid across the width of the road, it can be concluded based on the geogrid strain distribution that the improved performance was not due to any of these two mechanisms.

It is very important to note that, based on the process through which tension is mobilized in geogrid under traffic loads previously discussed in this study (Section 5.4.3), it is unlikely to mobilize tension in geogrid across most of the road width under tolerated rut levels (i.e., within the tolerated reduction in serviceability). For such tension mobilization to occur in the reinforcements (i.e., for the reinforcing mechanism to come into effect), excessive and impermissible amounts of rut must develop so that rehabilitation is already required. This argument raises serious doubts regarding the overall validity and soundness of the mechanisms of tensioned membrane and subgrade vertical confinement even for unpaved roads.

The mechanism of shear-resisting interface, on the other hand, suggests that development of tension laterally in the geogrid is localized under the wheel-path where lateral movement of aggregate under traffic loads is expected to occur. The geogrid strain data discussed above demonstrates a strain distribution that supports the development of such interaction under the wheel-path in the lateral/transverse direction. Thus, improved performance of the unpaved system in this study is likely due to the mechanism of shear-resisting interface.

In essence, the interface shear resistance develops through shear interaction between base aggregate and the geogrid. According to Giroud (2009), shear interaction mechanisms are
different for various types of geogrids. For extruded geogrids in particular, interaction is provided primarily by “interlocking” with aggregate particles. Therefore, the term “interlocking” is occasionally used in this section to refer to this mechanism. Perkins (1999) discussed four distinct but interdependent reinforcing actions under the reinforcement mechanism of shear-resisting interface. Results from the full-scale field experiment conducted in this study are discussed below to further support the existence of the reinforcement mechanism of interface shear resistance and the reinforcing actions it provides. These reinforcing actions are discussed in the order of importance and occurrence.

1. Lateral Restraint of Base Aggregate:

*Description of reinforcing action (Cause):*

As a result of shear or horizontal tensile stresses generated by traffic loads at the bottom of the base layer, local failure will occur in zones where horizontal compressive stresses are smaller than the induced tensile stresses.

The aggregate would spread laterally (i.e., in transverse direction) and the modulus of the granular material would decrease at the bottom of the base to reduce or relieve the tensile stresses and stress would be redistributed.

With geogrid reinforcement, shear interaction develops between the aggregate and the geogrid; the aggregate transmits part of the induced shear stress from the bottom of the base layer to a lateral tensile load in the geogrid. As the geogrid is much stiffer in tension than the aggregate, it restrains lateral movement of the aggregate through the interlocking (shear interaction) between the geogrid and the aggregate and minimizes the reduction of the granular material modulus under traffic loads.

*Support or evidence from the results:*

- Dynamic tensile forces of up to 1.68 kN/m were induced in the geogrid in the transverse direction, only under the wheel-path where greatest lateral movement of aggregate was expected to occur (Table 6.7), indicating shear interaction between the
aggregate and the geogrid under the wheel-path and exhibiting transmission of lateral loads.

- Tensile forces were mobilized in the geogrid, only under the wheel-path in the transverse direction, which indicates resistance or restraint to lateral movement of aggregate along the wheel-path provided by the geogrid (Figure 6.7).
- Tension was not mobilized in the longitudinal direction. This indicates that shear interaction (the shear-resisting interface) is limited to the transverse direction.

**Result (Effect):**

- The lateral restraint of aggregate results in far less lateral deformation developing in the base layer.
- Lower lateral deformation in the base results in less vertical deformation of the base layer and less surface deformation.

**Support or evidence from the results:**

- Significant permanent tensile geogrid strain developed in the transverse/lateral direction under the wheel-path with repeated traffic passes (Figure 5.17). Accumulation of tensile strain in the geogrid indicates that the geogrid was responding to and resist lateral movement of aggregate and suggests that less lateral deformation developed in the base layer.
- The permanent tensile strain peaked under the center of the wheel load and diminished with distance away from center (Figure 5.17 and Figure 5.18), which further demonstrates that the development of tensile strain in the geogrid was due to resisting lateral movement of aggregate under the wheel-path.
- Vertical deformation of the base layer decreased due to geogrid inclusion (Table 5.1).
- Surface deformation/rutting of the unpaved road decreased due to geogrid inclusion (Figure 5.1 to Figure 5.4).
2. Increase in Modulus of the Base Layer:

*Description of reinforcing action (Cause)*:

The lateral restraint provided by the geogrid to the aggregate results in an increase in lateral stress within the base (i.e., buildup of locked-in horizontal or confining stress). The increase in lateral or confining stress leads to an increase in the mean stress at the bottom of the base layer, which in turn causes the stiffness of the granular base course material to increase compared to the unreinforced case.

*Support or evidence from the results*:

- As previously discussed, tensile geogrid forces developed under the wheel-path in the transverse direction (Figure 6.7) indicate lateral confinement to the aggregate. This lateral confinement results in an increase in lateral stress at the bottom of the base layer.
- In addition, locked-in vertical stresses measured at the top of the subgrade decreased by the presence of geogrids (Figure 5.10), which may suggest increased locked-in vertical stresses in the base layer (based on observations by White et al. (2011)). Increased locked-in vertical stresses in the base due to the geogrid presence suggest increased locked-in horizontal stresses as well (higher horizontal stresses are usually locked into the aggregate than vertical stresses).
- The geogrids reduced the degradation of the base layer modulus ratio due to repeated traffic loads (i.e., increased the modulus of the base) compared with the unreinforced section (Figure 6.3a and Figure 6.4a).

*Result (Effect)*:

- The increased modulus of the base results in less vertical deformation in the base.

*Support or evidence from the results*:

- Inclusion of geogrids resulted in reduced vertical deformation of the base layer compared with the unreinforced section (Table 5.1).
3. Improvement of Stress Distribution by the Base layer:

_description of reinforcing action (Cause):

The increase in the modulus of the base layer due to presence of geogrid results in an improved distribution of vertical stress by the base on the underlying weaker subgrade layer. Therefore, applied vertical stress on the surface would be distributed over a wider area of the subgrade (i.e., distribution angle increases).

Support or evidence from the results:

- The results of the back-calculation analysis showed that the degradation of stress distribution angle was reduced by the inclusion of geogrid. In other words, the stress distribution angle increased due to the presence of the geogrid (Figure 6.3b and Figure 6.4b).

Result (Effect):

- Less dynamic vertical stress is transferred to the subgrade directly under the center of the loaded area (i.e., maximum vertical stresses under the centerline of the wheel-path decreases).
- This results in less surface deformation/rutting of the subgrade.

Support or evidence from the results:

- Dynamic vertical stress transferred to the subgrade (directly under the wheel-path) was reduced by the inclusion of geogrids (Figure 5.7).
- The subgrade of the reinforced sections showed less vertical deformation as compared with the subgrade of corresponding unreinforced section (Table 5.2).
4. Reduction of shear stress in the subgrade soil:

Description of reinforcing action (Cause):

As the base transmits shear load to a lateral tensile load in the geogrid through shear interaction (interlocking), the shear stress transmitted from the base layer to the subgrade decreases.

Support or evidence from the results:

- Due to lack of proper instruments to capture the existence of this reinforcing action, there is no direct data to support reduction of subgrade shear stress; however, it can be inferred from indirect observations. Transmission of lateral tensile load from the aggregate to the geogrid at the bottom of the base layer was demonstrated by inducing dynamic tensile forces in the geogrid (Table 6.7), as well as by accumulation of tensile force in the geogrid (Figure 6.7). Transmission of lateral load indicates reduction of shear stress at the bottom of the base layer which resulted in less shear stress transferred from the base to the subgrade.

Result (Effect):

- Reduction of shear stress in the subgrade decreases the shear strain near the top of subgrade and result in reduced horizontal/lateral deformation in the subgrade, which leads to less vertical deformation in the subgrade.

Support or evidence from the results:

- The geogrid inclusion resulted in reduced magnitudes of subgrade vertical deformation (Table 5.2).
6.5 Key Properties of Geogrid Reinforcement

The effectiveness of geogrid reinforcement in stabilizing the unpaved road depends on their physical and mechanical properties. Identification of key geogrid properties is critical for effective design to improve pavement performance. This can be accomplished by correlating these properties with the response of reinforced pavement measured in a laboratory or field experiment. Previous research indicated that effectiveness of geogrids was primarily related to their tensile stiffness (Miura et al., 1990; Dawson and Little, 1997; Leng and Gabr, 2005; Perkins and Ismeik, 1997; Hufenus et al., 2006; Tang et al., 2008; Cuelho and Perkins, 2009; Palmeira and Gongora, 2016) and junction strength (Dawson and Little, 1997; Perkins and Ismeik, 1997; Tang et al., 2008; Cuelho et al., 2014; Palmeira and Gongora, 2016). However, Austin and Coleman (1993) concluded that performance of geogrid-reinforced roads seems to be independent of the tensile strength. Based on results from field tests, Webster (1993) reported that geogrid aperture stability modulus showed good correlation with the traffic improvement factor.

The aperture stability modulus is becoming the main material property currently used in design. Giroud and Han (2004) considered the aperture stability as the only geogrid property in their design method for unpaved roads. However, other studies indicated poor or no correlation between geogrid aperture stability modulus and performance (Morris, 2013; Palmeira and Gongora, 2016). Simac et al. (2006) criticized using the aperture stability modulus as the only geogrid mechanical property in Giroud and Han design method and recommended utilizing tensile strengths at 2 to 5 % strain for design. Similar recommendations of considering geogrid properties other than aperture stability modulus have been made by Cuelho and Perkins (2009). These inconsistencies regarding which geogrid properties are most related to performance indicate that there is still a need to investigate the relationship between geogrid properties and performance in instrumented full-scale field tests.

This section aims to evaluate the key mechanical properties of the biaxial geogrids used in this study from correlations with the measured field responses. A linear regression analysis was carried out to evaluate relationships between various mechanical properties of geogrid reinforcement and improvements in measured performance of the test sections.
Through the regression analysis, the geogrid mechanical properties that show stronger correlations/relationships with the improvement in measured performance of the test sections were identified. In addition to its mechanical properties, physical properties of the geogrid such as aperture size to aggregate particle diameter ratio, thickness and the aperture area available for bearing are important for the performance improvement of unpaved roads (Palmeira and Gongora 2016). Evaluation of the geogrid’s physical properties is beyond the scope of this analysis. The mechanical properties evaluated in the analysis include:

- Ultimate tensile strength; referred to as $T$
- Tensile strength at 5% strain in the machine direction; referred to as $T5\% (M)$
- Tensile strength at 5% strain in the cross-machine direction; referred to as $T5\% (X)$
- Tensile strength at 2% strain in the machine direction; referred to as $T2\% (M)$
- Tensile strength at 2% strain in the cross-machine direction; referred to as $T2\% (X)$
- Junction strength in the machine direction; referred to as $JS(M)$
- Junction strength in the cross-machine direction; referred to as $JS(X)$
- Flexural rigidity in the machine direction; referred to as $FR(M)$
- Flexural rigidity in the cross-machine direction; referred to as $FR(X)$
- Aperture stability modulus; referred to as $J$
- Radial stiffness at low strain; referred to as $RS$

Direction dependent properties are evaluated in both machine and cross-machine directions in order to investigate whether the improvement in the measured performance is related to the geogrid properties in the machine (aligned with longitudinal direction of the road) or cross-machine direction (aligned with the transverse direction of the road). Sixteen measures of improvement are used in this analysis to identify which geogrid properties were most relevant to the performance of the test sections. These improvement measures include:

- TBR at 25, 45, 55, and 65 mm of rut
- BCR at 25, 45, 55, and 65 mm of rut
- Reduction in final rut depth
- Reduction in final surface deformation
• Reduction in subgrade vertical stress
• Reduction in stress distribution angle
• Reduction in modulus ratio
• Reduction in permanent deformation of the base layer
• Reduction in subgrade contribution to total permanent deformation
• Mobilization of geogrid forces

These measures are based on four independent sets and one dependent set of field measurements performed in this study. The linear regression was used because there were not enough data points to describe a more advanced relationship. In any case, linear regression is simple and adequate for comparing individual correlations within the same type of response and for comparing trends in correlations from multiple types of response. The intercept Y in the linear regression was forced to zero except in the TBR analysis it was set to 1.0 in order to represent the reference case of no improvement in performance of the unreinforced section. The coefficient of determination, $R^2$, was used to assess how well a fitted linear relationship based on a particular property predicts the measured field response or performance. In general, the higher the $R^2$, the better the regression line fits the data. $R^2$ normally ranges from 0 to 1. However, $R^2$ can be negative when the best-fit regression line, given the chosen regression and its variable (in this case the geogrid property), fits the data worse than a horizontal line (i.e., does not follow the trend of the data). The results of the regression analysis is summarized in Table 6.8.

Table 6.8 shows that the correlations with improvement in TBR and BCR were poor for all geogrid properties at the low rut level of 25 mm, and improved at the higher rut levels. It is not surprising to obtain poor correlations between geogrid properties and improvement in final rut depth since the rut depth or “apparent rut” includes the upheaval adjacent to the wheel-path which is highly variable and might be affected by vehicle wander; therefore, can’t be correlated with geogrid properties.
Table 6.8: Summary of R² results of linear regression analysis to identify geogrid properties most related to performance

<table>
<thead>
<tr>
<th>Measure of improvement</th>
<th>Geogrid properties</th>
<th>T</th>
<th>T5%(M)</th>
<th>T5%(X)</th>
<th>T2%(M)</th>
<th>T2%(X)</th>
<th>JS(M)</th>
<th>JS(X)</th>
<th>FR(M)</th>
<th>FR(X)</th>
<th>J</th>
<th>RS</th>
</tr>
</thead>
<tbody>
<tr>
<td>TBR @ 65mm</td>
<td></td>
<td>0.973</td>
<td>0.976</td>
<td>0.99</td>
<td>0.983</td>
<td>0.997</td>
<td>0.989</td>
<td>0.993</td>
<td>-1.437</td>
<td>-2.216</td>
<td>0.746</td>
<td>0.934</td>
</tr>
<tr>
<td>TBR @ 55mm</td>
<td></td>
<td>0.929</td>
<td>0.948</td>
<td>0.932</td>
<td>0.97</td>
<td>0.965</td>
<td>0.983</td>
<td>0.981</td>
<td>-1.766</td>
<td>-2.495</td>
<td>0.714</td>
<td>0.985</td>
</tr>
<tr>
<td>TBR @ 45mm</td>
<td></td>
<td>0.962</td>
<td>0.949</td>
<td>0.953</td>
<td>0.93</td>
<td>0.922</td>
<td>0.914</td>
<td>0.91</td>
<td>0.141</td>
<td>-0.283</td>
<td>0.926</td>
<td>0.283</td>
</tr>
<tr>
<td>TBR @ 25mm</td>
<td></td>
<td>0.437</td>
<td>0.474</td>
<td>0.593</td>
<td>0.56</td>
<td>0.726</td>
<td>0.648</td>
<td>0.72</td>
<td>-6.876</td>
<td>-8.829</td>
<td>-0.526</td>
<td>0.624</td>
</tr>
<tr>
<td>BDR @ 65mm</td>
<td></td>
<td>0.944</td>
<td>0.928</td>
<td>0.968</td>
<td>0.916</td>
<td>0.945</td>
<td>0.909</td>
<td>0.922</td>
<td>-0.668</td>
<td>-1.356</td>
<td>0.767</td>
<td>0.796</td>
</tr>
<tr>
<td>BCR @ 55mm</td>
<td></td>
<td>0.984</td>
<td>0.974</td>
<td>0.982</td>
<td>0.96</td>
<td>0.958</td>
<td>0.948</td>
<td>0.948</td>
<td>-0.209</td>
<td>-0.725</td>
<td>0.903</td>
<td>0.869</td>
</tr>
<tr>
<td>BCR @ 45mm</td>
<td></td>
<td>0.826</td>
<td>0.817</td>
<td>0.909</td>
<td>0.83</td>
<td>0.926</td>
<td>0.853</td>
<td>0.894</td>
<td>-2.729</td>
<td>-3.934</td>
<td>0.348</td>
<td>0.726</td>
</tr>
<tr>
<td>BCR @ 25mm</td>
<td></td>
<td>0.721</td>
<td>0.721</td>
<td>0.835</td>
<td>0.754</td>
<td>0.882</td>
<td>0.796</td>
<td>0.851</td>
<td>-4.021</td>
<td>-5.505</td>
<td>0.072</td>
<td>0.678</td>
</tr>
<tr>
<td>Final rut depth</td>
<td></td>
<td>-0.368</td>
<td>-0.303</td>
<td>-0.046</td>
<td>-0.136</td>
<td>0.212</td>
<td>0.04</td>
<td>0.19</td>
<td>-12.63</td>
<td>-15.8</td>
<td>-2.141</td>
<td>0.02</td>
</tr>
<tr>
<td>Final surface deformation</td>
<td></td>
<td>0.557</td>
<td>0.596</td>
<td>0.672</td>
<td>0.674</td>
<td>0.791</td>
<td>0.747</td>
<td>0.798</td>
<td>-5.837</td>
<td>-7.502</td>
<td>-0.231</td>
<td>0.754</td>
</tr>
<tr>
<td>Subgrade vertical stress</td>
<td></td>
<td>0.938</td>
<td>0.929</td>
<td>0.911</td>
<td>0.91</td>
<td>0.879</td>
<td>0.889</td>
<td>0.877</td>
<td>0.513</td>
<td>0.244</td>
<td>0.97</td>
<td>0.834</td>
</tr>
<tr>
<td>Distribution angle</td>
<td></td>
<td>0.87</td>
<td>0.859</td>
<td>0.838</td>
<td>0.837</td>
<td>0.802</td>
<td>0.813</td>
<td>0.799</td>
<td>0.786</td>
<td>0.61</td>
<td>0.948</td>
<td>0.759</td>
</tr>
<tr>
<td>Modulus ratio</td>
<td></td>
<td>0.699</td>
<td>0.686</td>
<td>0.667</td>
<td>0.663</td>
<td>0.629</td>
<td>0.639</td>
<td>0.625</td>
<td>0.989</td>
<td>0.926</td>
<td>0.805</td>
<td>0.589</td>
</tr>
<tr>
<td>Base deformation</td>
<td></td>
<td>0.838</td>
<td>0.864</td>
<td>0.874</td>
<td>0.904</td>
<td>0.935</td>
<td>0.936</td>
<td>0.95</td>
<td>-3.06</td>
<td>-4.09</td>
<td>0.444</td>
<td>0.946</td>
</tr>
<tr>
<td>Subgrade deformation</td>
<td></td>
<td>0.598</td>
<td>0.625</td>
<td>0.724</td>
<td>0.69</td>
<td>0.823</td>
<td>0.757</td>
<td>0.815</td>
<td>-5.544</td>
<td>-7.238</td>
<td>-0.193</td>
<td>0.715</td>
</tr>
<tr>
<td>Mobilized geogrid force</td>
<td></td>
<td>0.308</td>
<td>0.34</td>
<td>0.507</td>
<td>0.433</td>
<td>0.649</td>
<td>0.533</td>
<td>0.626</td>
<td>-7.789</td>
<td>-10.01</td>
<td>-0.822</td>
<td>0.469</td>
</tr>
</tbody>
</table>
It is noted that the tensile strength at 2% and the junction strength generally provided the best correlations with the measured improvements, especially with those improvements that are based on measurements of rut, surface deformation, and geogrid strain. The radial stiffness, tensile strength at 5%, and ultimate tensile strength showed also good correlations with performance. The aperture stability showed generally inconsistent relationship with improvements, while flexural rigidity showed the poorest correlations among all properties.

For better assessment of the results, exploratory data analysis (EDA) is utilized. Specifically, a box-and-whisker plot is used to graphically depict and analyze $R^2$ values in Table 6.8 and summarize their main characteristics. Plotting data as boxplots allows for a simple visual examination of how a geogrid property is related to the overall performance by providing a tool to compare trends in correlations ($R^2$ values) from multiple types of response. $R^2$ values of zero in Table 6.8 indicate that the regression line is worse than using the mean or a horizontal line fit (i.e., poorest correlations); therefore, negative $R^2$ values were replaced with zeros in the construction of the box plots. The graphical representation of the $R^2$ data is shown in Figure 6.9.

![Figure 6.9: Box-plots of $R^2$ values for fitted linear correlations between geogrid properties and improvement in measured performance (using all improvement measures)](image-url)
Each box extends from the first quartile (25th percentile) to the third quartile (75th percentile) of the $R^2$ data; horizontal lines inside the boxes denote median values; the “×” symbols denote mean values; the whiskers (vertical lines extending from the boxes) indicate variability outside the upper and lower quartiles and denote the lowest and highest $R^2$ values for each property excluding any outliers; outliers are plotted as individual points or dots.

Considering the direction dependent properties, Figure 6.9 indicates that the measured performance was better correlated to the properties in the cross-machine direction than those in the machine direction. For example, the tensile strength at 2% strain in the cross-machine direction has higher average and more importantly higher median of $R^2$ values than that in the machine direction. The cross-machine direction has less overall variation in $R^2$ (i.e., smaller range) and also less spread around the median (smaller interquartile range, IQR). More consistent $R^2$ values indicate reliable correlation with performance. This means that the tensile strength at 2% has more consistent correlation with performance in the cross-machine direction compared with the machine direction. Moreover, it is consistent in the upper levels of $R^2$ (more consistent and stronger correlation). Finally, the machine direction has by far the smallest $R^2$ (poorest relation) among the two direction. Comparisons between tensile strengths at various strains by following the same approach used for comparing direction properties reveal that the tensile strength at 2% strain has the strongest correlation with performance, followed by the tensile strength at 5% strain, while the ultimate tensile strength showed the least consistent and poorest correlation among the three.

Among all mechanical properties evaluated in this analysis, the tensile strength at 2% in the cross-machine direction and the junction strength in the cross-machine direction exhibited the strongest correlations with the measured performance of the test section. Based on the box plots, they showed the highest medians and maximum $R^2$ values and the least overall variations in $R^2$ and the least variations around the median (most consistent correlations). It is also noted that nearly 75% of the $R^2$ values in the two properties are higher than 0.8, which clearly indicates strong correlation with the measured performance.
A second EDA analysis was conducted considering only one improvement measure from each measurement set performed in this study. $R^2$ values from duplicate measures of improvement (i.e., derived from the same set of measurements) were removed from the data. The purpose of the second analysis was to reduce any possible representation bias in the data by resampling in such a way that all members of the intended population (i.e., the improvement measures) have an equal sampling representation. One improvement measure was selected from each of the four groups of field measurement. Whenever possible, measures were selected to give direct reference on the improvement of the measured parameter or response. TBR at 65 mm rut depth was chosen to represent the improvement in rut development since final rut depths was shown to have poor correlation with all of the geogrid properties as previously discussed. The improvement measures included in the second exploratory data analysis were as follows:

- TBR at 65 mm of rut
- Reduction in final surface deformation
- Reduction in subgrade vertical stress
- Mobilization of geogrid forces

The results are depicted in Figure 6.10. By excluding duplicate measures of improvement from the Box plot, the results show generally less variability in $R^2$ or more consistent correlation with performance; the trend of the results, however, doesn’t change. The analysis revealed that the response improvements were primarily related to the tensile strength at 2% and junction strength in the cross-machine direction. It can be seen that the tensile strength at 2% in the cross-machine direction exhibited more consistent correlations so that all of its $R^2$ values were nearly higher than the lowest 25% $R^2$ values from other tensile properties. It is also noted that the radial stiffness showed less spread of $R^2$ around the median (more consistent correlations) in contrary to the ultimate tensile strength, for example, which showed more spread in data. It is believed that the good correlations with performance provided by the radial stiffness are attributed the low strain level at which this radial stiffness is measured (0.5%), which is more consistent with the range of strains measured in the field experiment. This suggests that a tensile stiffness
measured at smaller strain level same as those experienced by the geogrid in real traffic conditions would have produced even stronger correlations with performance.

![Box-plots of $R^2$ values for fitted linear correlations between geogrid properties and improvement in measured performance (using main improvement measures only)](image)

**Figure 6.10:** Box-plots of $R^2$ values for fitted linear correlations between geogrid properties and improvement in measured performance (using main improvement measures only)

### 6.6 Summary

Results from the full-scale field traffic testing were analyzed to investigate the behavior of the geogrid-reinforced unpaved test sections and to evaluate the influence of geogrid reinforcement on its performance. A traffic benefit ratio analysis was used to quantify the performance benefit of the geogrids in terms of “extending the service life” of the reinforced test sections over unreinforced sections at various levels of rut depth. It was found that the number of ESAL passes was increased as much as 3.6 times by using geogrids. Greatest TBRs were observed when using the stiffer, heavy-duty geogrid followed by the medium- and light-duty geogrids.

The performance benefit of the geogrids was also evaluated in terms of "savings in base course material" by conducting a base course reduction analysis. The results indicated that the greatest reduction in base thickness was observed in the heavy-duty geogrid
The light- and medium-duty geogrids resulted in similar aggregate savings (average of 10%).

The widely used design procedure of Giroud and Han (2004) was employed to predict the performance of unreinforced and reinforced field sections tested in this study. The results demonstrated that the method significantly underestimated the thickness of base layer required to support the traffic loads applied during the test. Furthermore, it was observed that the predictions of the method are worse for geogrids with larger values of aperture stability.

Based on measurements of subgrade dynamic vertical stress, back-calculation analyses were conducted to evaluate degradation of the base layer through the change in stress distribution angle and base-to-subgrade modulus ratio with cumulative traffic passes. It was concluded that the degradation of the base layer was affected by the grade or properties of the geogrid reinforcement as well as the thickness of the base layer itself. The results showed that the degradation of modulus ratio and stress distribution angle of the geogrid-strengthened unpaved test sections decreased. Also, increasing the base layer thickness reduced the degradation in modulus ratio. Heavier duty geogrids provided more improvement than did the lighter duty geogrids. In addition, the results of the back analysis suggested that the geogrids offered some stabilization benefits during construction that resulted in higher post-construction values of stress distribution angle and modulus ratio. It was also demonstrated that the inclusion of stiff geogrids allows for improved compaction and results in modulus ratios greater than 5.0. Accordingly, it is recommended that modulus ratio limit greater than 5.0 should be used in the design procedure of Giroud and Han (2004) for the geogrid-reinforced case. The change in stress distribution on the subgrade under traffic loading depends on the traffic, the thickness of base layer, the base-to-subgrade modulus ratio, and the properties of the geogrid. The relationship between the change in stress distribution angle and these parameters was described using regression analysis.

The geogrid strains measured during traffic loading in the transverse direction along with the geogrid tensile moduli at small strains (2%) were used to compute geogrid tensile forces. Tensile forces of 1.5 to 3 kN/m were mobilized in the geogrids indicating that
significant part of the shear stresses generated at the bottom of the base by traffic loads was transmitted to the geogrids. The reinforcement forces correlated reasonably well with long-term rutting performance of the test sections as well as to the reduction in vertical stress transferred to the subgrade. The results suggested that effectiveness of geogrids as reinforcements is likely related to their tensile stiffness.

The reinforcement mechanisms through which the geogrids stabilized the unpaved test sections were examined and discussed based on the results of the field testing. It was concluded that improved performance was not due to a tensioned membrane effect or vertical confinement of subgrade. Rather, it was due to the combined mechanism of shear-resisting interface commonly known as “lateral restraint” or “lateral confinement”.

The distribution of measured geogrid tensile strains from the field trafficking didn’t support the assumptions embedded in the mechanisms of tensioned membrane and subgrade vertical confinement that tension develops in the geogrid almost across the entire width of the road and that it is contingent upon the development of large amounts of rut. Moreover, the mobilization of tension in geogrid under traffic loads as demonstrated by geogrid strain measurements from this study challenges the overall validity and soundness of the two mechanisms even for unpaved roads. On the other hand, the geogrid strain distribution was fully consistent with the concept of shear-resisting interface that is based on the mobilization of tension only under the wheel-path.

The observation of low geogrid strains away from the wheel-path validates the lateral movement of base aggregate away from the loaded area. In addition, the results from this study further supported the existence of the four reinforcing actions included in the mechanism of shear-resisting interface.

Several linear regression analyses were conducted using mechanical properties of geogrid to identify which properties were most related to performance of the unpaved test sections in this study. The results indicated that the improvement in performance was related to the cross-machine direction of the geogrid (aligned with the transverse direction of the road). It was concluded that performance of unpaved test sections in this study was primarily related to the tensile strength at 2% and junction strength in the cross-machine direction.
Chapter 7

7 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary

A full-scale field study was conducted to quantify the effectiveness of geogrids in unpaved roads, evaluate their reinforcing mechanism, and provide an understanding of which geogrid properties are most directly related to performance improvement. To achieve these objectives, ten full-scale unpaved road test sections were constructed and instrumented to collect response data from controlled traffic loads. Test variables included geogrid aperture shape, geogrid level of robustness (i.e., tensile modulus), and thickness of the base course layer. Five of the test sections were constructed with a 200-mm nominal base course thickness and the other five were constructed with a 250-mm nominal base thickness. For each base course thickness, four test sections were reinforced with different geogrids while one test section was unreinforced in order to evaluate their performance. Four geogrid products were used in this study: three biaxial geogrids and one triangular aperture geogrid. The subgrade soil was a silty, clayey sand with a CBR of approximately 2.3% and the base course material used was well-graded gravel with silt and sand. The geogrids were placed at the subgrade-base course interface on top of a non-woven geotextile separator.

The test road was trafficked using a dual-wheel, 80 kN single-axle dump truck. Measurements of rut depth and surface deformation were taken to analyze and evaluate the respective performance of each test section. Additionally, the test sections were instrumented for measuring road response under traffic loading. Dynamic and permanent geogrid strains were measured using foil strain gauges attached to the geogrid ribs. Earth pressure cells were installed at the top of the subgrade layer in order to measure dynamic and permanent vertical stresses transferred to the subgrade soil.
7.2 Conclusions

Based on the results of this study, the following conclusions are drawn that reinforce the current understanding of geogrid reinforcement behavior in unpaved roads and reconcile contradicting observations reported in the literature.

7.2.1 Rutting Performance

- The geogrids improved the rutting performance of the unpaved test sections with thin base layers (i.e., 200 mm-thick bases).
- The decrease in surface rutting due to geogrid inclusion was more pronounced when a heavier duty geogrid was used.
- Performance improvement by geogrid reinforcements decreased as the thickness of base course was increased from 200 mm to 250 mm.
- The base layer contributed more to the surface deformation of the unpaved test sections as compared with the subgrade layer.
- For the same base layer thickness, the contribution from the base layer to the total permanent deformation increased, whereas the contribution from the subgrade decreased with the presence of geogrid. This response was more notable in the thinner sections and it was more pronounced when a heavier duty geogrid was used.
- For the same reinforcement type, the contribution from the base layer to the total permanent deformation increased, whereas the contribution from the subgrade decreased with the increase of base course thickness.

7.2.2 Vertical Stress Transferred to the Subgrade

- The inclusion of geogrids reduced the dynamic vertical stresses transferred to the top of the subgrade.
- The reduction in subgrade vertical stress became more obvious when a heavier duty geogrid was used.
- The effect of geogrid reinforcement (improvement in stress transfer by the base layer) was decreased by the increase of base layer thickness.
• Residual or locked-in vertical stresses were recorded at the top of subgrade during both construction and trafficking.

• The geogrids reduced the locked-in stresses. This reduction was more pronounced during trafficking as compared to construction, and it became more pronounced when a heavier duty geogrid was used.

• The reduction in locked-in vertical stresses at the top of the subgrade by the geogrid suggests greater locked-in vertical stresses in the base layer for the reinforced section compared to the unreinforced section, which indicates an increase in the base layer stiffness by the reinforcement.

7.2.3 Strain Developed in Geogrids

• Permanent strains were accumulated in the geogrid immediately at the onset of the traffic loading and well before any appreciable surface rutting was developed in the unpaved structure.

• The amount of tension mobilized in the geogrid decreased with the increase in base layer thickness, for the same reinforcement type.

• Permanent geogrid tensile strains accumulated in the transverse direction ranged from 0.45 to 0.6%.

• The permanent geogrid strains were correlated to rutting performance.

• A relationship was also observed between the development of permanent compression strains (in the longitudinal direction and laterally away from the load) and cumulative traffic passes in the 200-mm base sections.

• The geogrid strain data from this study suggest that a correlation between dynamic geogrid strains in the transverse direction and long-term rutting performance may exist.

7.2.4 Quantification of Geogrid Benefits

• TBR values of up to 3.6 were obtained using the geogrids.

• Greatest TBRs were observed in the stiffer, heavy-duty geogrid followed by the medium- and light-duty geogrids.
• The results of the base course reduction (BCR) analysis indicated that the inclusion of geogrids reduced the base layer thickness by 6 to 25% at varying levels of rut depth. The greatest reduction in base thickness was observed in the heavy-duty geogrid (average of 16%). The light- and medium-duty geogrids resulted in similar aggregate savings (average of 10%).

7.2.5 Degradation of Modulus Ratio and Stress Distribution Angle

• The analysis of measured data indicated that the degradation of the base layer was affected by the grade or properties of the geogrid reinforcement as well as the thickness of the base layer itself.
• The degradation of modulus ratio and stress distribution angle of the geogrid-stabilized unpaved test sections decreased by the presence of geogrid as well as by increasing the base layer thickness. Heavier duty geogrids provided more improvement than did the lighter duty geogrids.
• The results suggested that the geogrids offered some stabilization benefits during construction that resulted in higher post-construction values of stress distribution angle and modulus ratio.
• The inclusion of stiff geogrids allowed for improved compaction and resulted in modulus ratios greater than 5.0. Accordingly, it is recommended that modulus ratio limit greater than 5.0 should be used in the design procedure of Giroud and Han (2004) for the geogrid-reinforced case.
• The results showed that the change in stress distribution on the subgrade under traffic loading depends on the traffic, the thickness of base layer, the base-to-subgrade modulus ratio, and the properties of the geogrid. The relationship between the change in stress distribution angle and these parameters was described using regression analysis.

7.2.6 Forces in Geogrids

• Tensile forces of 1.5 to 3 kN/m were mobilized in the geogrids indicating that significant part of the shear stresses generated at the bottom of the base by traffic loads was transmitted to the geogrids.
The reinforcement forces correlated reasonably well to long-term rutting performance of the test sections as well as to the reduction in vertical stress transferred to the subgrade.

The results suggested that effectiveness of geogrids as reinforcements is likely related to their tensile stiffness.

7.2.7 Reinforcement Mechanisms

The distribution of geogrid strains evaluated from the field trafficking didn’t support the assumptions of tensioned membrane and subgrade vertical confinement mechanisms, i.e., tension develops in the geogrid almost across the entire width of the road.

The geogrid strain distribution was fully consistent with the concept of shear-resisting interface or “lateral restraint” that is based on the mobilization of tension only under the wheel-path. The results further supported the existence of the four reinforcing actions included in the mechanism of shear-resisting interface.

7.2.8 Other Conclusions

Linear regression analysis indicated that the improvement in performance was related to the cross-machine direction of the geogrid (aligned with the road transverse direction). Performance of unpaved test sections in this study was primarily related to the tensile strength at 2% and junction strength in the cross-machine direction.

The results of this study indicated that the design method of Giroud and Han (2004) significantly underestimated the thickness of base layer required to support traffic loads applied during the test. Furthermore, the method gave worse predictions for geogrids with larger values of aperture stability.
7.3 Research Findings

The following findings are exclusively derived from the results of this study:

- Examination of the typical strain response of the geogrid under traffic loads indicated that the geogrid ribs in the transverse direction experienced pure tensile strain directly beneath the wheel load and pure compressive strain of relatively smaller magnitudes away from the wheel load (between the wheel-paths).

- Geogrid strain measurements in the longitudinal or traffic direction showed a relatively complex typical response where the state of strain changes from when the wheel load is approaching (compression) to when it is directly on top of the strain measurement point (tension) and then when the wheel moves away from the measurement point (compression). Loading setups used in most laboratory tests are not capable of replicating these loading conditions.

- Examination of the permanent geogrid strain behavior with respect to the direction of traffic indicated that the strain response under the actual traffic loading conditions was an anisotropic one.

- The permanent geogrid strain data demonstrated that the geogrid was not under constant tension across the road. Tension was only mobilized in the transverse direction, beneath the wheel-path, and the geogrid was under compression between the wheel-paths (i.e., away from the load) and in the longitudinal direction.

- The distribution of geogrid strains under traffic loads as demonstrated by strain measurements from this study challenges the overall validity and soundness of the tensioned membrane effect, the widely accepted reinforcing mechanism for unpaved roads.

- Results from this study indicated that the reinforcing benefits of geogrids during construction is not linked to the amount of tensile strain developed in the geogrids due to compaction loads, but rather to the restraint of lateral movement of aggregate during compaction which leads to an increase in stiffness and placement density of the base layer.
7.4 Study Limitations

The results presented in this study are based on full-scale field testing performed under the following testing conditions:

- Base course thickness ranging between 200 and 250 mm.
- Subgrade soil with CBR ranging between 2 and 3%.
- The geogrid materials used in this study.
- Geogrids placed at the base-subgrade interface.

Further research is needed to verify the generality of the findings obtained from this study under broader test conditions.

7.5 Recommendations for Future Research

Based on the results and the conclusions from this study, the following points are recommended for future research:

- Research is needed to investigate interlocking between geogrids and aggregate and the factors that affect it (such as grid aperture size and other physical and mechanical properties) using a similar full-scale field testing setup.
- Similar research is needed for other types of geogrid reinforcements of different structure and polymer composition.
- Further field studies to investigate other placement locations of geogrids with thicker base layers may be beneficial to gain additional knowledge about the behavior of geogrid reinforced unpaved roads.
- The results of this study raised serious doubts regarding the overall validity and soundness of the mechanisms of tensioned membrane and subgrade vertical confinement even for unpaved roads. A field study that further evaluates the performance of geogrids at increased rut depths should be conducted to validate whether or not a strain distribution of constant tension is possible to exist in the geogrid under traffic loads.
REFERENCES


Webster, S.L. (1993). *Geogrid reinforced base courses for flexible pavements for light aircraft: test section construction, behavior under traffic, laboratory tests, and design*


# APPENDIX A

## BOREHOLE LOGS AND SOIL GRADATION CURVES

### Table A.1: Log of boring BH1

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description</th>
<th>Legend</th>
<th>Sample No.</th>
<th>SPT-N (Blows/ft)</th>
<th>w (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
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Remarks:
- SS: Split spoon
- SH: Shelby Tube Sample
### Table A.2: Log of boring BH2

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**Remarks:**

SS: Split spoon

SH: Shelby Tube Sample
Table A.3: Log of boring BH3

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Remarks:
- SS: Split spoon
- SH: Shelby Tube Sample
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Remarks:
- SS: Split spoon
- SH: Shelby Tube Sample

Borehole No.: BH4
Type of Drilling: Solid-Stem Flight Auger
Equipment: CME 45 Truck Mount
Logged By: Abdalla El Tawati

Job No.: UWO Road Project
Location: 1915 Crumlin Sideroad, London, ON
Job Name: UWO Road Project
Type of Drilling: Solid-Stem Flight Auger
Equipment: CME 45 Truck Mount
Logged By: Abdalla El Tawati
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Remarks:
- SS: Split spoon
- SH: Shelby Tube Sample

Table A.5: Log of boring BH5

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<tr>
<th>Job No.:</th>
<th>UWO Road Project</th>
</tr>
</thead>
<tbody>
<tr>
<td>Job Name:</td>
<td></td>
</tr>
<tr>
<td>Location:</td>
<td>1915 Crumlin Sideroad, London, ON</td>
</tr>
<tr>
<td>Borehole No.:</td>
<td>BH5</td>
</tr>
<tr>
<td>Type of Drilling:</td>
<td>Solid-Stem Flight Auger</td>
</tr>
<tr>
<td>Equipment:</td>
<td>CME 45 Truck Mount</td>
</tr>
<tr>
<td>Logged By:</td>
<td>Abdalla El Tawati</td>
</tr>
</tbody>
</table>
# Table A.6: Log of boring BH6

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description</th>
<th>Legend</th>
<th>Sample No.</th>
<th>SPT-N (Blows/ft)</th>
<th>w (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>SILTY, CLAYEY SAND with some gravel, light brown, loose to medium-dense, moist.</td>
<td>SH1</td>
<td></td>
<td>11.6</td>
<td>18.2</td>
<td>13.6</td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>SILTY, CLAYEY SAND with some gravel, grey, loose to medium-dense, wet.</td>
<td>SH2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td></td>
<td>SS1</td>
<td></td>
<td>32</td>
<td>12.3</td>
<td>20.1</td>
<td>14.2</td>
</tr>
<tr>
<td>2.5</td>
<td></td>
<td>SS2</td>
<td></td>
<td>45</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td>Boring terminated</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Remarks:**
- SS: Split spoon
- SH: Shelby Tube Sample

**Job No.:**
- BH6

**Job Name:**
- UWO Road Project

**Location:**
- 1915 Crumlin Sideroad, London, ON

**Borehole No.:**
- BH6

**Type of Drilling:**
- Solid-Stem Flight Auger

**Equipment:**
- CME 45 Truck Mount

**Logged By:**
- Abdalla El Tawati
## Table A.7: Log of boring BH7

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description</th>
<th>Legend</th>
<th>Sample No.</th>
<th>SPT-N (Blows/ft)</th>
<th>w (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td><strong>SILTY, CLAYEY SAND</strong> with some gravel, light brown, loose to medium-dense, moist.</td>
<td>SS1</td>
<td>11</td>
<td>10.9</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td><strong>SILTY, CLAYEY SAND</strong> with some gravel, grey, loose to medium-dense, wet.</td>
<td>SS2</td>
<td>12</td>
<td>10.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td></td>
<td>SS3</td>
<td>32</td>
<td>14.5</td>
<td>21.3</td>
<td>16.5</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td></td>
<td>SS4</td>
<td>33</td>
<td>12.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td></td>
<td>SS5</td>
<td>38</td>
<td>10.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Remarks:**
- SS: Split spoon
- SH: Shelby Tube Sample

**Notes:**
- **SS1:** Sample No. 11
- **SS2:** Sample No. 12
- **SS3:** Sample No. 32
- **SS4:** Sample No. 33
- **SS5:** Sample No. 38

**Location:**
1915 Crumlin Sideroad, London, ON

**Job No.:**
UWO Road Project

**Type of Drilling:**
Solid-Stem Flight Auger

**Equipment:**
CME 45 Truck Mount

**Logged By:**
Abdalla El Tawati
Table A.8: Log of boring BH8

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description</th>
<th>Legend</th>
<th>Sample No.</th>
<th>SPT-N (Blows/ft)</th>
<th>w (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>SILTY, CLAYEY SAND with some gravel, light brown, loose to medium-dense, moist.</td>
<td>SH1</td>
<td></td>
<td>9.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>SILTY, CLAYEY SAND with some gravel, grey, loose to medium-dense, wet.</td>
<td>SH2</td>
<td></td>
<td>11.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td>Boring terminated</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Remarks:
- SS: Split spoon
- SH: Shelby Tube Sample
Table A.9: Log of boring BH9

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description</th>
<th>Legend</th>
<th>Sample No.</th>
<th>SPT-N (Blows/ft)</th>
<th>w (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td><strong>SILTY, CLAYEY SAND</strong> with some gravel, light brown, loose to medium-dense, moist.</td>
<td></td>
<td>SH1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td><strong>SILTY, CLAYEY SAND</strong> with some gravel, grey, loose to medium-dense, wet.</td>
<td></td>
<td>SH2</td>
<td></td>
<td>11.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td></td>
<td></td>
<td>SS1</td>
<td>23</td>
<td>9.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td></td>
<td></td>
<td>SS2</td>
<td>27</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td>Boring terminated</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Remarks:

SS: Split spoon
SH: Shelby Tube Sample
### Table A.10: Log of boring BH10

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description</th>
<th>Legend</th>
<th>Sample No.</th>
<th>SPT-N (Blows/ft)</th>
<th>w (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>SILTY, CLAYEY SAND with some gravel, light brown, loose to medium-dense, moist.</td>
<td>SS1</td>
<td>14</td>
<td>8.8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>SILTY, CLAYEY SAND with some gravel, grey, loose to medium-dense, wet.</td>
<td>SS2</td>
<td>19</td>
<td>10.7</td>
<td>18.3</td>
<td>14.6</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td></td>
<td>SS3</td>
<td>50</td>
<td>9.0</td>
<td>17.2</td>
<td>12.4</td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td>Boring terminated</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Remarks:
- SS: Split spoon
- SH: Shelby Tube Sample
Figure A.1: Particle size distribution for the subgrade soil (sample No. BH2-SS3)

Figure A.2: Particle size distribution for the subgrade soil (sample No. BH3-SS2)
Figure A.3: Particle size distribution for the subgrade soil (sample No. BH5-SS1)

Figure A.4: Particle size distribution for the subgrade soil (sample No. BH6-SS1)
Figure A.5: Particle size distribution for the subgrade soil (sample No. BH7-SS1)

Figure A.6: Particle size distribution for the subgrade soil (sample No. BH7-SS2)
Figure A.7: Particle size distribution for the subgrade soil (sample No. BH8-SH1)

Figure A.8: Particle size distribution for the subgrade soil (sample No. BH10-SS2)
Figure A.9: Particle size distribution for the subgrade soil (sample No. BH10-SH3)
APPENDIX B

DESIGN DATA FOR AASHTO 1993 PAVEMENT DESIGN PROCEDURE

Table B.1: Suggested levels of reliability for various functional classifications (after AASHTO, 1993)

<table>
<thead>
<tr>
<th>Functional classification</th>
<th>Recommended level of reliability</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Urban</td>
</tr>
<tr>
<td>Interstate and other freeways</td>
<td>85─99.9</td>
</tr>
<tr>
<td>Principal arterials</td>
<td>80─99</td>
</tr>
<tr>
<td>Collectors</td>
<td>80─95</td>
</tr>
<tr>
<td>Local</td>
<td>50─80</td>
</tr>
</tbody>
</table>

Table B.2: Standard normal deviates ($Z_R$) for various levels of reliability (after AASHTO, 1993)

<table>
<thead>
<tr>
<th>Reliability (%)</th>
<th>Standard normal deviate ($Z_R$)</th>
<th>Reliability (%)</th>
<th>Standard normal deviate ($Z_R$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>0.000</td>
<td>93</td>
<td>-1.476</td>
</tr>
<tr>
<td>60</td>
<td>-0.253</td>
<td>94</td>
<td>-1.555</td>
</tr>
<tr>
<td>70</td>
<td>-0.524</td>
<td>95</td>
<td>-1.645</td>
</tr>
<tr>
<td>75</td>
<td>-0.674</td>
<td>96</td>
<td>-1.751</td>
</tr>
<tr>
<td>80</td>
<td>-0.841</td>
<td>97</td>
<td>-1.881</td>
</tr>
<tr>
<td>85</td>
<td>-1.037</td>
<td>98</td>
<td>-2.054</td>
</tr>
<tr>
<td>90</td>
<td>-1.282</td>
<td>99</td>
<td>-2.327</td>
</tr>
<tr>
<td>91</td>
<td>-1.340</td>
<td>99.9</td>
<td>-3.090</td>
</tr>
<tr>
<td>92</td>
<td>-1.405</td>
<td>99.99</td>
<td>-3.750</td>
</tr>
</tbody>
</table>
Table B.3: Recommended drainage coefficients for untreated bases and subbases in flexible pavements (after AASHTO, 1993)

<table>
<thead>
<tr>
<th>Quality of drainage</th>
<th>Percentage of time pavement structure is exposed to moisture levels approaching saturation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Less than 1%</td>
</tr>
<tr>
<td>Excellent</td>
<td>1.40—1.35</td>
</tr>
<tr>
<td>Good</td>
<td>1.35—1.25</td>
</tr>
<tr>
<td>Fair</td>
<td>1.25—1.15</td>
</tr>
<tr>
<td>Poor</td>
<td>1.15—1.05</td>
</tr>
<tr>
<td>Very poor</td>
<td>1.05—0.95</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rating</th>
<th>Water removed within</th>
<th>2 hours</th>
<th>1 day</th>
<th>1 week</th>
<th>1 month</th>
<th>Never drain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Good</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fair</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Poor</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Very poor</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure B.1: Variation in granular base layer coefficient ($a_2$) with various base strength parameters (After Van Til et al., 1972)
CURRICULUM VITAE

Name: Abdalla El Tawati

Post-secondary Education and Degrees:

Bachelor of Science
Civil Engineering Department
University of Benghazi
Benghazi, Libya

Master of Science
Civil, Environmental and Architectural Engineering Department
University of Colorado at Boulder
Boulder, Colorado, USA
2008 – 2010

Doctor of Philosophy
Civil and Environmental Engineering Department
The University of Western Ontario
London, Ontario, Canada
2012 – 2021

Honours and Awards:


Milos Novak Memorial Award, University of Western Ontario
2015

William E. and Ruther Lardner Graduate Award, University of Western Ontario: 2019

Related Work Experience

Teaching Assistant
The University of Western Ontario
2012 – 2017

Consultant Engineer
The Research and Consulting Center, University of Benghazi
2004 – 2007

Teaching Assistant
University of Benghazi
2002 – 2007
Publications:


