REINFORCEMENT OF PAVEMENT SUBGRADE USING GRANULAR FILL AND A GEOSYNTHETIC LAYER

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Dedication

To my mum

Dr. Jolden Alice Acabat Oriokot

For her endless love and support.

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The research described in this thesis was conducted at the University of Cape Town during the period January 2013 and September 2014.

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Abstract

Engineers are often faced with construction of pavement structures over soft and compressible subgrade. Such conditions render the structures unable to withstand required design loads and thus are susceptible to high settlements associated with excessive distress leading to pavement damage. The use of imported quality fill to improve the load-bearing capacity of the subgrade has limited benefits, which leads to the necessity of an alternative construction approach to attain the necessary strength of the soil structure. The use of geosynthetics in soil offers a better alternative to improvement of the soil's stability.

This research was conducted to determine the degree of improvement of the load-bearing capacity and reduction in settlement due to geosynthetic reinforcement of a soft clay overlain by granular material. Compression tests on the unreinforced clay subgrade and geosynthetic-reinforced two-layered soil composite were conducted at bench-scale using a Zwick Universal Compression and Tension machine at the University of Cape Town Geotechnical Engineering Laboratory. The geosynthetic products used in this study included geogrids and geotextiles that provided reinforcement to the soil structure. The reinforcement layer was placed either within the layer of granular material or at the interface of the soil, and a range of fill thicknesses and depths of placement were tested to determine their influence on the improvement of strength.

The results obtained indicated that on inclusion of geosynthetic reinforcement there was an improvement in the load-bearing capacity that ranged from 35% - 160%. In addition there was a reduction in settlement that ranged from 35% - 60%, which was determined using the Percentage Reduction in Settlement (PRS) equation.

Geogrids performed better than geotextiles when included in the two-layered soils. This could be attributed to geogrids being stiffer than geotextiles and the added ability to interlock the soil particles in their apertures forming a stronger composite structure.

There were further benefits observed as a result of geosynthetic-reinforcement that included: reduction in the required fill thickness, which was determined using the Base-Course Reduction Ratio (BRR) and gave a range of 25% - 67%. This would result in a reduction in the material use and increase the ease of the construction process.

It is anticipated that the use of these composites could lead to an extension in the design life of the pavement structure and consequently reduced necessity for maintenance work.

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Abbreviations

AASHTO	–	American Association of State Highway and Transportation Officials
ASTM	–	American Society of Testing and Materials
BCR	–	Bearing Capacity Ratio
BRR	–	Base-Course Reduction Ratio
BS	–	British Standard
CBR	–	California Bearing Ratio
DPW	–	Department of Public Works
EGG	–	Extruded Geogrid
EKG	–	Electrokinetic Geosynthetics
ESAL	–	Equivalent Single Axle Load
GC	–	Geocomposite
GCL	–	Geosynthetic Clay Liners
GG	–	Geogrid
GF	–	Geofoam
GL	–	Geocell
GM	–	Geomembrane
GN	–	Geonet
GT	–	Geotextile
HDPE	–	High Density Polyethylene
kN	–	kilo Newton
LL	–	Liquid Limit
L.O.I	–	Loss on Ignition
LVDT	–	Linear Vertical Displacement Transducer
MC	–	Moisture Content
MDD	–	Maximum Dry Density
m	–	metre
mm	–	millimetre
NGT	–	Non-woven Geotextile
OMC	–	Optimum Moisture Content
PA	–	Polyamide
PE	–	Polyethylene
PET	–	Polyester

PI	–	Plastic Index
PL	–	Plastic Limit
PP	–	Polypropylene
PRS	–	Percentage Reduction in Footing Settlement
PVC	–	Polyvinylchloride
RT	–	Repeatability Test
SAPEM	–	South African Pavement Engineering Manual
UR	–	Unreinforced
USCS	–	Unified Soil Classification System
WGG	–	Woven Geogrid
WGT	–	Woven Geotextile

CHAPTER 1

1. Introduction

1.1. Background to Study

Construction of pavement structures often occurs over soft and compressible subgrades that have low load-bearing capacities and are susceptible to large settlements. This results in damage to the structures erected on them (Ornek et al. 2012). A further problem is that sites may have had the in situ soil tampered with either by previous excavations or dumping of weak material on them, leaving unstable sections on the site. These factors often render the strength of the subgrade unfeasible for construction. As such these subgrades need engineering intervention before they can support applied loads safely.

The problem of encountering soft subgrades consisting clays and silts; the need to build infrastructure like pavement structures over these sites; and the additional mandated environmental regulations that prevent construction on alternative sites, have provided the impetus for the development of a number of ground improvement techniques during the past 25 years (Geosynthetics Materials Association, 2011).

The accumulation of the load as the construction process progresses including the continuous movement of construction machinery on site; and also the active loading during the design life of the pavement structures sometimes leads to migration of the fine grain particles into the granular fill, and penetration of the large granular particles into the soft subgrade (Love et al. 1987). The effect of this tends to lead to deterioration in the structure of the soil layers, resulting in deformations and ultimately leading to failure of the structure. In addition, the limited availability of quality fill to stabilize the soft subgrade is also a problem faced in the construction process. This can cause scheduling delays as a result of having to transport the required material from distant quarries to the construction site, or the use of alternative ground improvement methods which are relatively expensive, affecting the feasibility of projects (Moayed and Nazari 2011).

When dealing with difficult sites for construction purposes, the conventional practice was limited to either replacing the unsuitable soils, or bypassing them with costly deep foundations. The solution involving addition of granular material, which acts as the fill layer, distributes the loads laterally decreasing the stresses on the soft subgrade. This allows support of greater loads, and leads to minimal failure occurrences such as settlements and pavement distress (Erickson and Drescher, 2001). The granular fill takes up most of the applied loads which reduces the load transferred to the in situ subgrade. This is attributed to granular fills having higher strength properties than the in situ clay found on the sites hence improving the load-bearing capacity of the pavement structure.

Innovative ground improvement approaches are now used to solve these unique soil-related problems, and often are considered to be the most economical means to improve an undesirable site condition. Construction with geosynthetics is one of the approaches that have been incorporated in the design of pavement structures. Its aim is to stabilize the soils, making them more suitable for engineering applications (Maxwell et al. 2005). The use of geosynthetics in geotechnical construction projects has gained tremendous popularity over the past 30 years, and their use in large-scale civil construction projects has made the resulting structures safer (Erickson and Drescher, 2001). The inclusion of geosynthetics in soil layers has shown the benefits of enhancing the load-bearing capacity of the soil, and reducing the settlement undergone by the structures (Kazimierowicz-Frankowska 2007).

1.2. Justification of Study

It has become clear that construction sites with marginal soils which are highly susceptible to settlements and pavement distress require engineering intervention before construction progresses. The conventional practice of overlaying these soils with granular fill leads to an improvement in the load-bearing capacity of the soil structure. However, the increase in traffic loads and the use of heavy machinery during the construction process has led to the necessity for thicker fill layers. There is also the destabilization of the soil structure due to mixing of soil particles as the machinery moves about the site during the construction process. This occurs through pumping of fines into the granular fill, and penetration of the granular material into the soft marginal soils.

In this study, the inclusion of a geosynthetic layer in the multi-layered soil is addressed to quantify its potential benefits to pavement subgrades. Geosynthetic reinforcement is used in pavement structures to aid in support of traffic loads, where loads may be due to vehicular traffic over the life of the pavement, or equipment loads on the unpaved subbase and subgrade course during construction.

Geosynthetics incorporated at the interface offers separation of the different soil layers, thus reducing the potential mixing of the particles that would result in a reduction in the strength. Geotextile products are primarily used in separation of multi-soil layers; however they also provide adequate reinforcement to the soil. On the other hand geogrids are primarily used for reinforcement, but the interlocking granular particles in the apertures of the geogrid form a composite structure that also provides separation of soil layers. Therefore both geotextiles and geogrids are beneficial in reinforcement and separation of multi-layered soils. In addition, areas with high traffic loads are susceptible to rutting, and the incorporation of a geosynthetic layer in the pavement structure has the benefit of reducing the rut depths. This is all dependent on the depth of placement of the reinforcement layer and whether it can provide the additional support before excessive ruts formed in the structure.

According to Bouazza and Heerten (2013), when conventional methods of construction were compared with those that use geosynthetics, it was found that the use of geosynthetics results in a considerable reduction in construction costs, construction time, as well as a reduction in the volumes and masses that need to be excavated and imported. This has economic and environmental benefits such as the carbon reduction of the earth. Using geosynthetic reinforcement on the subbase and subgrade layers may allow the use of fill material of reduced strength, as the geosynthetic layer will provide the additional strength necessary.

1.3. Research Objectives

The main objectives of this research were to determine the degree of improvement of the load-bearing capacity of clay soil using combined reinforcement of granular fill and geosynthetics; and determine the degree of reduction in settlement. The objectives involved determining the optimum thickness of fill, determining the optimum depth of placement of the geosynthetic layer, determining the effect of footing size, and identifying which of the geosynthetic products would provide the desirable benefits.

1.4. Research Limitations

The research conducted herein concerned the reinforcement of pavement subgrade using both granular fill and a geosynthetic layer, with the determination of the benefits as a result of their inclusion in the pavement subgrade. However, the tests conducted were limited to static loads, and no cyclic loading and rolling wheel tests included in the research. In addition, a limited number of geosynthetics were investigated from a single supplier.

1.5. Thesis Outline

Chapter 1 introduces the topic to be investigated in this study with the appropriate background, the current problems being faced and the importance of this research.

Existing theory and research applicable to the current study is reviewed in three chapters. Chapter 2 describes the literature associated with pavement structures. A review of bearing capacity is presented, and the problematic soils that are encountered on sites in South Africa are identified. Chapter 3 looks at geosynthetics as the selected reinforcement material in multi-layered soils with the theory involved in the reinforcement, applications of the materials, the different types available and soil-geosynthetic interaction. Chapter 4 introduces the literature review on geosynthetic reinforcement related to the current research with identification of the gaps in these studies.

Chapter 5 describes the methodology followed in assessing the improvement in bearing strength of the geosynthetic reinforced soil composite. The research materials used in the study are also described with their material properties obtained from classification tests.

Chapter 6 presents the data collected from the plate-load tests conducted and these results are discussed in detail in Chapter 7.

Chapter 8 looks at the practical applications of geosynthetics, with modifications of the equations for bearing capacity due to inclusion of geosynthetics.

Finally, Chapter 9 summarizes and highlights the most important conclusions derived. Recommendations to further this study are presented at the end of the chapter.

CHAPTER 2

2. Review on Pavement Structure, Bearing Capacity and Problematic Soils

2.1. Introduction

This review begins with coverage of pavement structures used in the South African road industry. The load-bearing capacity of soil, both homogenous and multi-layered, is then covered, followed by an in-depth description of the different problematic soils and conditions encountered on civil engineering project sites. Focus on soft compressible soils, and identification of the various methods of improvement of these soils is reported. Finally the concept of soil reinforcement, as well as the different methods and benefits of soil reinforcement are discussed.

2.2. Pavement Structure

Pavement structures are the general definition for civil engineering infrastructure that include highways, rail and road embankments and even parking structures. There are two types of pavement structures depending on the materials used: flexible pavements and rigid pavements as shown in Figures 1 and 2 respectively. There are further subcategories including; permanent paved roads, temporary paved roads, permanent unpaved roads, and temporary unpaved roads. All these have similarities in that they are founded on the natural ground on the site, which is classified as the in situ subgrade (SAPEM, 2013a).

Figure 1: Flexible pavement (South African Pavement Engineering Manual (SAPEM), 2013a):

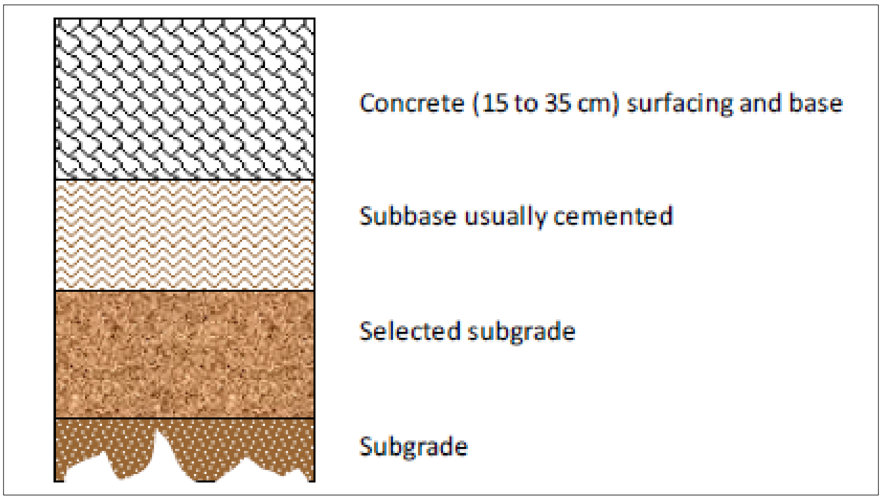

Figure 2: Rigid pavement structure (South African Pavement Engineering Manual (SAPEM), 2013a)

The pavement structure is a combination of layers of materials and the subgrade on which it is founded. The purposes of the various layers in the pavement are described below (South African Pavement Engineering Manual, SAPEM, 2013a, Section 4: Page 10).

- **Surfacing**: This is a functional wearing course that provides waterproofing, skid resistance, noise-damping, durability against the elements, visibility and drainage. For surfaced roads, the upper layer is bound, consisting of spray seals, asphalt or concrete.
- **Base**: This is a load spreading layer that is the most important structural component of the pavement. The layer must provide the required support for the surfacing and distribute the high tyre pressures and wheel loads uniformly over the underlying layers and subgrade. The base comprises bound material, e.g., asphalt, concrete or stabilised, or can be unbound material, e.g., crushed stone or gravel base.
- **Subbase**: This layer provides support for the base as well as a platform upon which to construct a structural base layer of high integrity. It also protects the underlying selected subgrade layer by further spreading the load.
- **Selected subgrade**: These layers are primarily capping for the subgrade to provide a workable platform to construct the imported base and subbase layers. At the same time, these layers provide depth of cover over the subgrade to reduce the stresses in the subgrade to acceptable levels.
- **Subgrade**: This is the existing material upon which the pavement is constructed. It can be modified with stabilisers to reduce plasticity, ripped and recompacted to achieve uniform support, or undercut and replaced, depending on its quality.

2.2.1. Materials Used

The typical materials used in construction of pavement structures are dependent on the material available on the site and whether it has the strength requirements to carry the design loads. In the event that the material found on site is not adequate for the project, then quality fill is imported to improve the conditions on the site. Figure 3 shows typical materials used in pavement structures and the sourcing of materials from quarries.

Figure 3: Sourcing material for pavement structure showing typical materials used (top left); the crushing of materials sourced from quarries (top right) and typical examples of quarries (bottom left and right) (SAPEM, 2013c).

2.2.2. Stress Distribution through Pavements

The selection of material for the different layers should have adequate strength to resist the exerted traffic loads, otherwise there would be failure of the structure. Figures 4 and 5 show the distribution of loads through typical flexible and rigid pavement layouts respectively, and the required strengths for each of the layers.

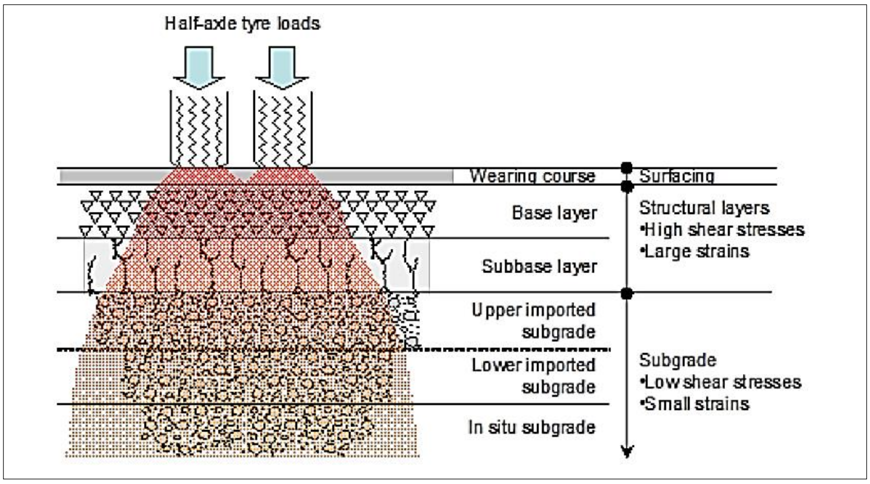

Figure 4: Stress distribution in a typical South African flexible pavement (SAPEM, 2013d)

From Figure 4 it can be seen that there is an apparent lateral spread of the load through the subsequent layers. The degree of the spread is dependent on the angle of friction of the material in the layer. The top most layers, which are the structural layers, have a higher spread as they have higher angles of friction, while the lower layers, which are the subgrade, have a lower spread as they have lower angles of friction.

The structural layers undergo high shear stresses and large strains. As such they should have high strength properties to provide resistance. The subgrades undergo low shear stresses and strains. However, they should still possess the strength to support the accumulative loads during the construction phase and the design life of the pavement.

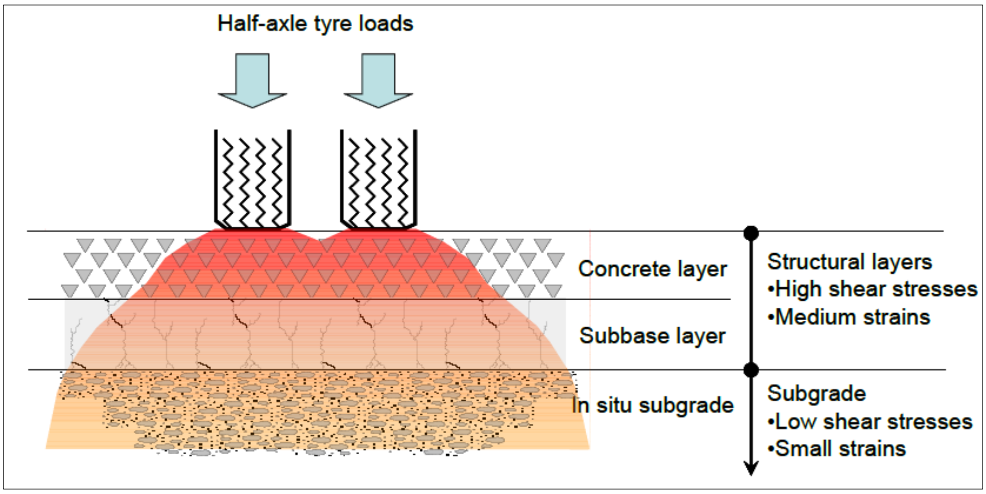

Figure 5: Stress distribution in a typical South African rigid pavement (SAPEM, 2013d).

From Figure 5 the same phenomenon of load spread through the layers is demonstrated. The structural layers consisting of concrete undergo high shear stresses, medium strains, and there is a higher degree of spread attributed to the properties of concrete material. The degree of spread reduces once the load is distributed to the subgrade layer that undergoes low shear stresses and small strains.

2.2.3. Pavement Structure Failure

Yoder and Witczak (1975) defined two types of pavement distress. The first is a structural failure, in which a collapse of the entire structure, or a breakdown of one or more of the pavement components, renders the pavement incapable of sustaining the loads imposed on its surface. The second is a functional failure. This occurs when the pavement, due to unevenness, is unable to carry out its intended function without causing discomfort to drivers or passengers or imposing high stresses to vehicles. Figure 6 shows the distresses that could occur due to construction over soft subgrade. The potential result of pavement distress could be damage of the structure as shown in Figure 7. According to SAPEM (2013a), excessive loads, excessive repetition of loads, and high tire pressures can cause either structural or functional failures given;

Structural failure (Surfacing distress):

- Surface cracking (Figure 6(a))
- Bleeding
- Permeability
- Aggregate loss/ravelling
- Surface failure
- Surface texture

Functional failure (Traffic associated distress):

- Rutting
- Deformation
- Pumping (Figure 6(b))
- Potholes (Figure 6(a))
- Patching (Figure 6(b))

Figure 6: (a) Surface cracking and potholing and (b) Pumping of stabilized base and patching (SAPEM, 2013a)

Figure 7: Stability failure of a pavement structure (SAPEM, 2013b).

The distresses presented in the figures are the focus of this study, and are the ones that the research seeks to mitigate through the inclusion of geosynthetics in the pavement structure.

2.2.4. Mitigation of Pavement Distress

There are different civil engineering methods to deal with pavement distress that range from maintenance to preventative actions before construction commences, which minimize the development of distress on the structures during its design life. The application of geosynthetics to the pavement structures is discussed below and further detail is presented in Chapter 3.

2.2.4.1. Subgrade Reinforcement

Subgrade reinforcement involves having the geosynthetic layer at the interface of the base/subbase and the in-situ subgrade. The geosynthetic layer acts as both reinforcement and separation as it reduces the dispersion of the base/subbase material and also the mixing of the different soil particles which would lead to pavement destabilization. This is necessary as it provides subgrade restraint for construction of the road over soft subgrade conditions. Figure 8 shows an illustration of subgrade reinforcement due to geosynthetic inclusion (Berg et al., 2000).

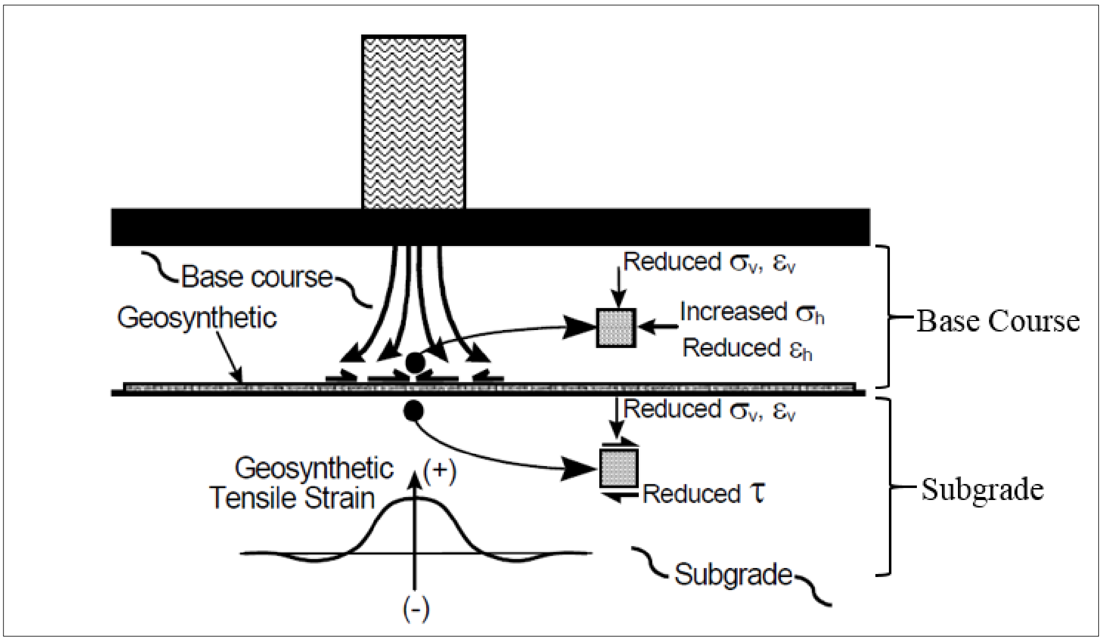

Figure 8: Subgrade reinforcement due to geosynthetic inclusion at the interface of the soils (Perkins et al., 1998; Berg et al., 2000).

With the inclusion of the reinforcement layer at the interface of the soils there is distribution of the applied load along the geosynthetic, with the particles in the base course restrained laterally. As a result there is a reduction in the vertical stress and strain; and a subsequent increase in the lateral restraint and reduced lateral strain of the particles in the base course layer. There is also the added effect of reduced vertical stress, vertical strain, and shear on the particles in the subgrade layer. All this occurs as the geosynthetic layer takes up the tensile stress and strain.

2.2.4.2. Subbase Reinforcement

Subbase reinforcement involves having the geosynthetic layer placed within the base/subbase course as shown in Figure 9. This mobilizes the reinforcement property of the geosynthetic layer earlier which reduces the transfer of applied load to the soft subgrade below. There is also the increased dispersion of the load applied, thus reducing the distress on the subgrade. Paved permanent roadway design methods that incorporate base/subbase reinforcement generally allow the user to evaluate a reduction in base course thickness, or an increase in design life of a section containing a layer of geosynthetic reinforcement (Berg et al., 2000).

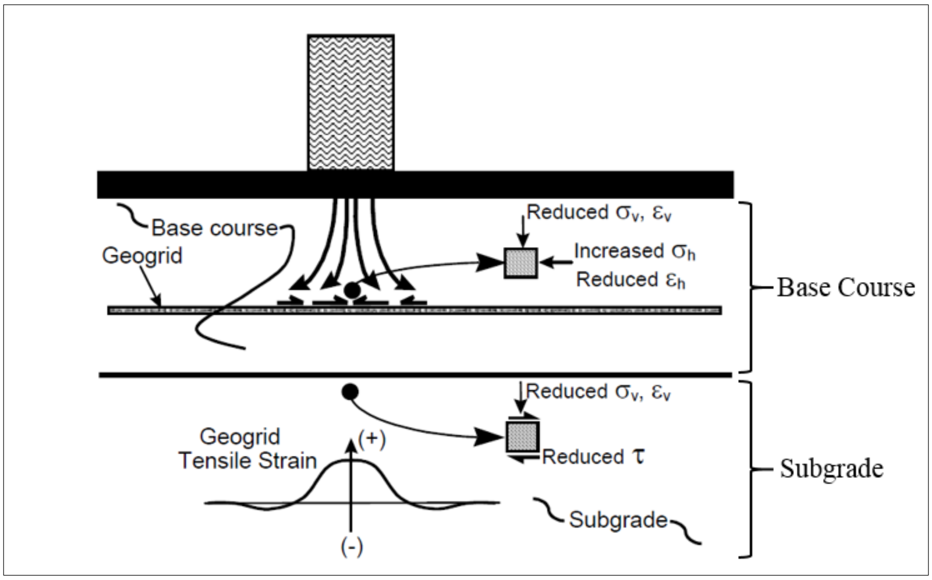

Figure 9: Base/Subbase reinforcement due to geosynthetic inclusion within the base course (Perkins et al., 1998; Berg et al., 2000).

The inclusion of the geosynthetic layer within the base course leads to distribution of the load within the base course layer, such that the transferred load to the subgrade is even less than if the reinforcement was at the interface. Similar to the subgrade reinforcement the inclusion of the geosynthetic has the effect of an increase in the restraining horizontal stress, with reduced horizontal strain, and a reduced vertical stress and vertical strain for the particles in the base course layer. There is also the added effect of reduced vertical stress and vertical strain on the particles in the subgrade layer, with reduced shear. However, in this configuration there is expected less stress and strain subjected onto the subgrade.

2.3. Bearing Capacity

The bearing capacity of a soil is its ability to withstand the exerted loads on it without failure occurring. Bearing capacity is an indication of the strength of a soil, and is used in the design phase to determine the feasibility for construction to proceed on a specific site. In the event that the ultimate bearing capacity of a soil is exceeded, failure would occur that could lead to damage of structures erected on the soil. The shear failure of the soil causes heaving of the soil next to the pavement, which could lead to damages to adjacent structures.

2.3.1. Modes of bearing capacity failure

There are three different modes of bearing capacity failure; general shear failure, local shear failure and punching shear failure. The different modes of bearing capacity failure are dependent on the type of soil and Figure 10 shows the different types that could occur.

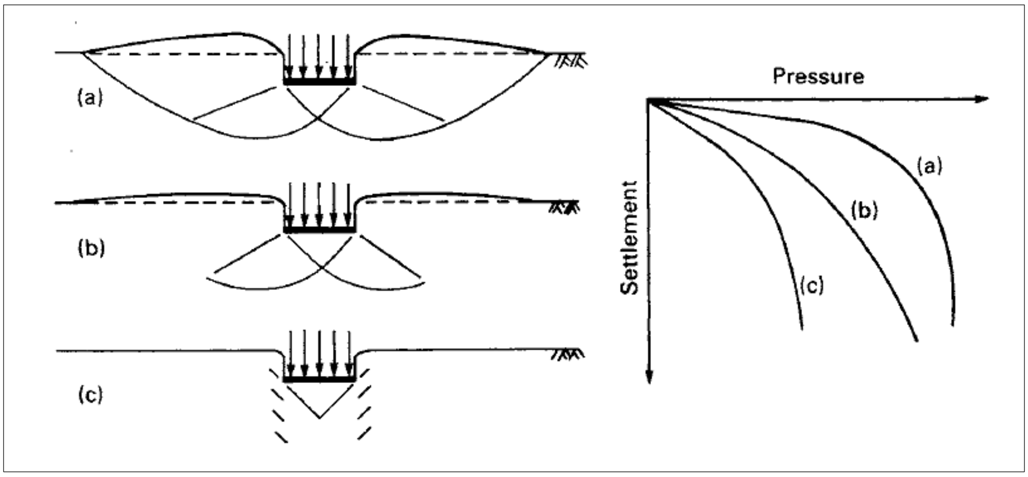

Figure 10: Modes of failure: (a) General Shear, (b) Local Shear and (c) Punching Shear (Craig, 2004).

2.3.1.1. General Shear Failure

There is a development of continuous failure surfaces between the edges of the footing and the ground surface (Craig, 2004). As the pressure is increased the value of the state of plastic equilibrium is reached initially in the soil around the edges of the footing and then gradually spreads downwards and outwards, as shown in Figure 10 (a). According to Das (2007) the state of plastic equilibrium is fully developed throughout the soil above the failure surfaces that leads to heaving of the ground surface on both sides of the footing, although the final slip movement would occur only on one side, accompanied by tilting of the footing. This mode of failure is typical of soils of low compressibility

2.3.1.2. Local Shear Failure

There is significant compression of the soil under the footing and only partial development of the state of plastic equilibrium. The characteristics of this failure is that slight heaving occurs as the failure surfaces do not reach the ground surface; tilting of the foundation would not be expected; and it is associated with soils of high compressibility (Das, 2007). This is shown in Figure 10 (b).

2.3.1.3. Punching Shear Failure

Punching shear failure occurs when there is relatively high compression of the soil under the footing, accompanied by shearing in the vertical direction around the edges of the footing. The characteristics of this failure are; no heaving of the ground surface away from the edges and no tilting of the footing; with relatively large settlements and in addition the ultimate bearing capacity is not well defined, as shown in Figure 10 (c). Punching shear failure will occur in a soil of low compressibility if the foundation is located at considerable depth (Whitlow, 1995).

2.3.2. Bearing Capacity of Layered Soils: Stronger Soil underlain by Weaker Soil.

In practice, layered soil profiles are encountered on sites in which the failure surface may extend through two or more soil layers. The development of the failure surface is dependent on the depth, H, between the base of the structure and the underlying layer of soil. If H is relatively small, then punching shear failure shall occur in the stronger soil with general shear failure in the weaker soil below. However, if H is relatively large, then general shear failure shall occur in the stronger soil only with no failure surface within the weaker soil. (Das, 2007). Figure 11 illustrates this occurrence.

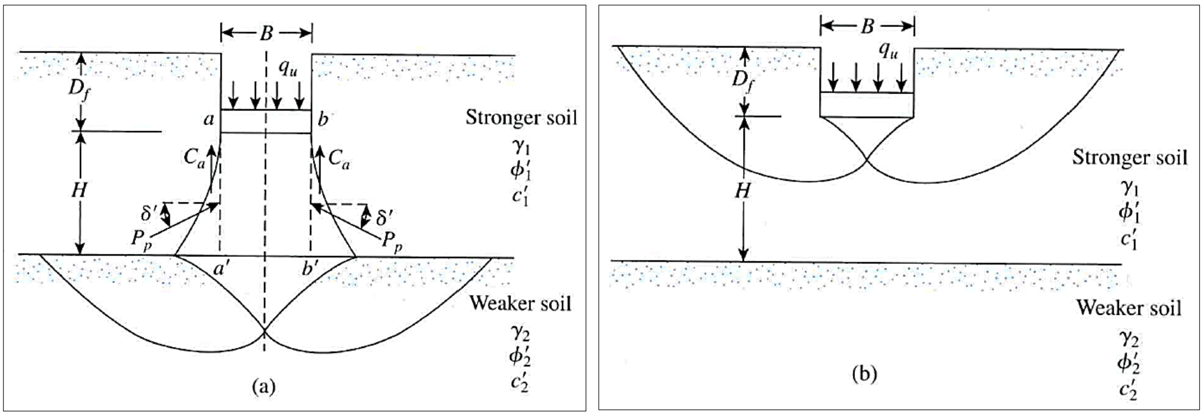

Figure 11: Development of failure surface in a two-layered soil showing (a) punching failure and (b) general shear failure (Das, 2007).

2.3.3. Zone of Influence

One phenomenon that is important in the distribution of loads below the pavement structure, is the zone of influence that is dependent on the width of the structure. In homogenous soils the range of the zone influence is dependent on the shape of the base as shown in Figure 12. In strip footings the zone ranges up to 3B while in circular footings the range is up to 1.5B. Beyond these depths there are negligible stresses transferred to the soil.

In determination of the zone of influence for a multi-layered soil, the Westergaard distribution was referred to, as was the case in the study conducted by Collin (2007). As shown in the Figure 13, the zone of influence ranges up to twice the footing width. In multi-layered soils the zone of influence determines which of the soils provides the load-bearing capabilities to the exerted loads, and to what degree. The zone of influence is important as it determines the maximum thickness that should be applied in the experimental testing, so as to limit wastage of material by exceeding the zone of influence.

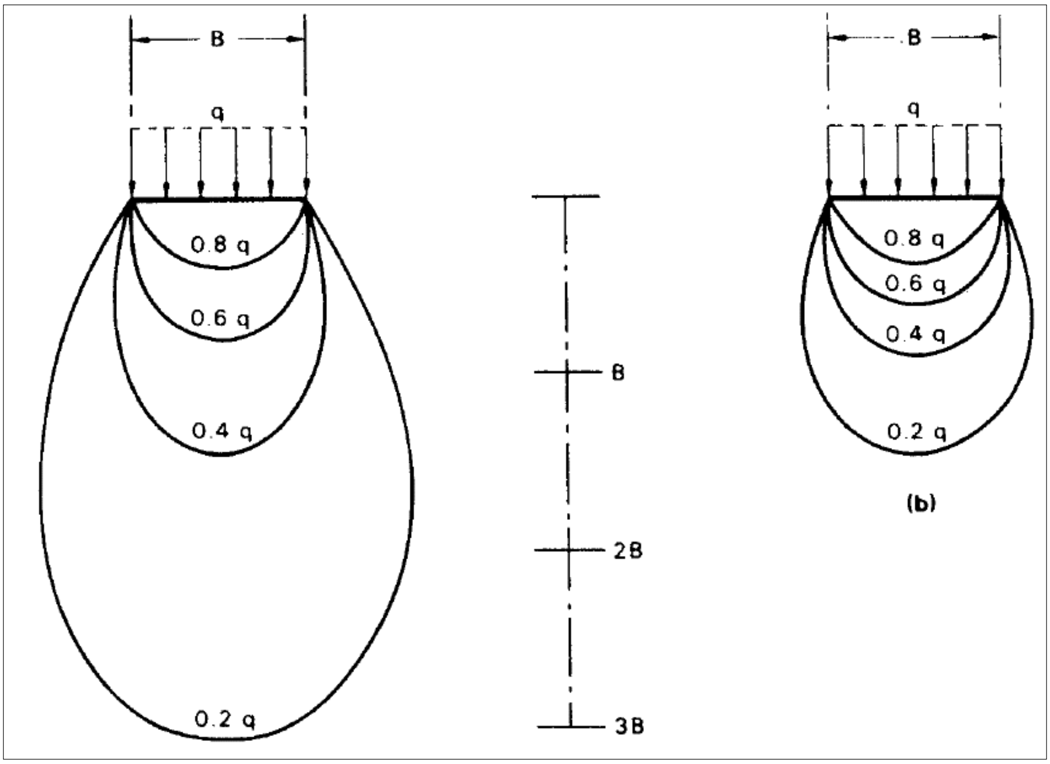

Figure 12: Zone of influence for strip footings (left) and circular footings (right) (Craig, 2004).

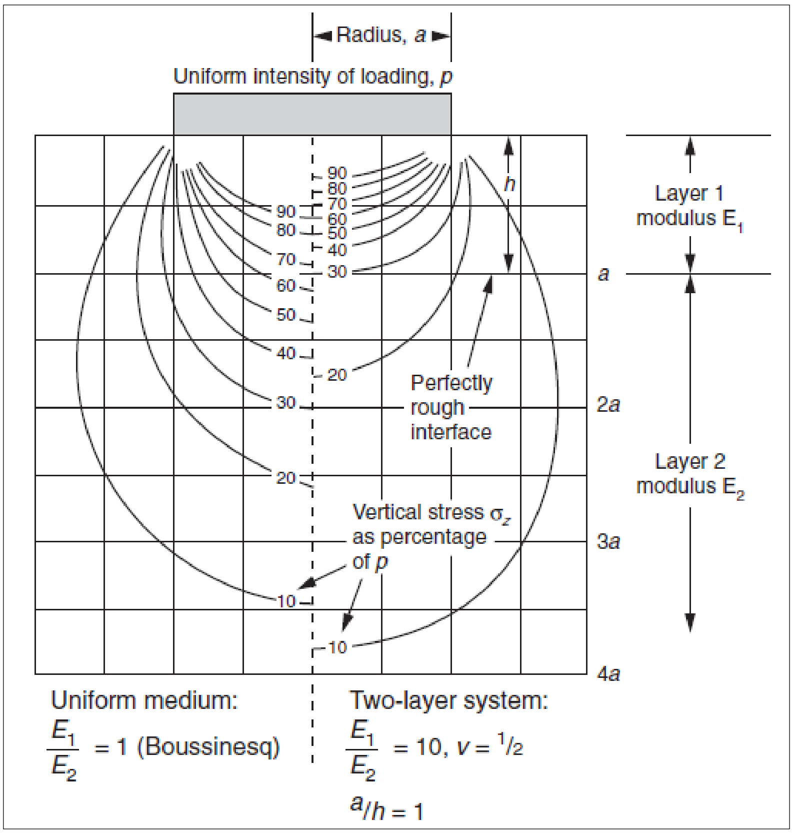

Figure 13: The Westergaard stress distribution for a two-layer system (Munfakh et al. 2001).

2.3.4. Equations of Bearing Capacity

The bearing capacity of reinforced soil is dependent on a number of variables that include the soil parameters and in the case of inclusion of a geosynthetic; the tensile strength of the reinforcing layer that is mainly under tension. The Terzaghi equation gives a simple estimate of the ultimate bearing capacity of the soil, as shown in the equation:

$$q_u = cN_c + qN_q + \frac{1}{2}\gamma' B N_\gamma \tag{1}$$

Where N_c, N_q and N_γ are the bearing capacity factors.

This equation is however applicable to single layered soils, and does not take into consideration the reinforcing layer or critical features like shape, inclination, depth and base factors. To cover these factors, the Hansen equation is usually considered in the calculations of the ultimate bearing capacity of soils, as shown in the equation:

$$q_u = cN_c s_c d_c i_c b_c + qN_q s_q d_q i_q b_q + \frac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma \tag{2}$$

Where s, d, i, b are the shape, depth, inclination and base factors respectively.

To verify the load-bearing capacity of the geosynthetic reinforced two-layered soil composite, different analysis approaches were considered that determined the strength of the composite and included:

- Hansen's Method for two layered soils,
- The Projected Area Method, and
- Chen's Method.

2.3.4.1. Hansen's Method for two layered soils

This method suggests determining the average values of: cohesion ($\bar{c}$), angle of internal friction ($\bar{\varphi}$), and unit weight of the soils ($\bar{\gamma}$), and the equivalent significant depth, z_{max} for the layered soils. With these strength parameters, the ultimate bearing capacity can then be determined by using the bearing capacity factors: N_c, N_q, N_γ, which are coefficients in Hansen's method for homogenous soils (Fan-fan and Shu-wang, 2003). The equation used in the calculations is:

$$q_u = \bar{c}N_c s_c + \bar{q}N_q s_q + \frac{1}{2}\bar{\gamma} B N_\gamma s_\gamma \tag{3}$$

Where;

$$\bar{\gamma} = \sum_{i=1}^{n} \gamma_i\, h_i / z_{max} \tag{4}$$

$$\bar{c} = \sum_{i=1}^{n} c_i\, h_i / z_{max}$$

(5)

$$\bar{\varphi} = \sum_{i=1}^{n} \varphi_i\, h_i / z_{max}$$

(6)

$$z_{max} = \lambda B = z_{top\ soil} + z_{bottom\ soil}$$

(7)

λ is coefficient of depth; γ_i, c_i, φ_i, h_i are unit weight, cohesion, angle of internal friction and thickness of every layer.

$$s_c = 1 + \frac{N_q}{N_c}\frac{B}{L}$$

(8)

$$s_q = 1 + (\frac{B}{L}) sin\varphi$$

(9)

$$s_\gamma = 1 - 0.4\frac{B}{L}$$

(10)

2.3.4.2. *Projected Area Method*

The projected area method for layered soils assumes a foundation with increased width and length on the interface as shown in Figure 14. The increased length and width of the foundation are based on the projected angle, α, which is as a result of the load being laterally distributed (Fan-fan and Shu-wang, 2003). The ultimate bearing capacity, q_u, is given by the equation:

$$q_u = \frac{q_b (B + 2Htan\alpha)^2}{B^2}$$

(11)

Where;

q_b is the bearing capacity of the footing on the lower soft layer.

B is the width of the footing

H is the thickness of the top layer

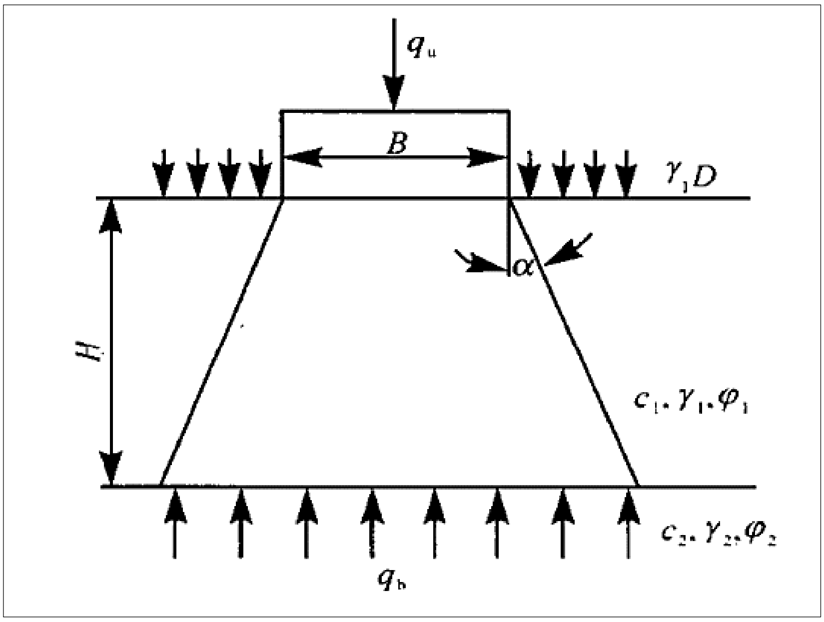

Figure 14: Illustration of the essential parameters in calculation of bearing capacity using the projected area method.

2.3.4.3. Chen's Method

Chen's method of determination of the bearing capacity of a geosynthetic reinforced soil takes into consideration the tensile strength of the geosynthetic as shown in the equation (Chen, 2008):

$$q_u = cN_c s_c + qN_q s_q + \frac{1}{2}\gamma B N_\gamma s_\gamma + \Delta q_T \tag{12}$$

According to Chen's equation, the component of the ultimate bearing capacity provided by the tensile strength of the geosynthetic is determined using the following equation;

$$\Delta q_T = \sum_{i=1}^{n} \frac{4T_i(u + (i-1)h)}{B^2} \tag{13}$$

Where;

T_i - is the tensile strength of the geosynthetic layer in kN/m.

u - is the depth to the first geosynthetic layer

i - is the number of layers

h - is the spacing between each of the other geosynthetic layers

B - is the width of the footing.

The above equation for Δq_T is applicable for 2 or more geosynthetic reinforcement layers in the soil. However, given only 1 layer of geosynthetic reinforcement was included in the soil structure, the equation for Δq_T was;

$$\Delta q_T = \frac{4T_i D}{B^2} \tag{14}$$

Where D is the depth of placement of the reinforcement layer

2.4. Problematic Soils

Many soils can prove problematic in geotechnical engineering because they undergo extreme changes such as expansion, collapse, dispersion, excessive settlement, and have a distinct lake of strength; resulting in severe damages to structures erected on them. The conditions and types of problematic soils are dependent on various factors which makes it possible to group these soils. Each problem soil has characteristics that make them unique and these are determined by various factors that include the nature of the parent rock, the origin of the soil, the climate, vegetation and the topography (Braatvedt et al. 2008).

2.4.1. Types of Problematic Soils

According to Braatvedt et al. (2008) the following problem soils have been identified in the Southern African region:

- Collapsible soils
- Expansive soils
- Dispersive soils
- Dolomites
- Soft clays
- Liquefiable soils

Each of these soils are however specific to certain regions and formations, and are only relevant for specific geotechnical engineering applications.

2.4.1.1. Collapsible soils

According to Jefferson and Rogers (2012), collapsible soils present significant geotechnical and structural challenges, and can be either found naturally or formed through human activity. They are unsaturated soils that undergo large volume changes upon saturation (Das, 2011). They have high void ratios and low unit weights; which would imply that on loading the soil structure would be destabilized as the voids are filled by the particles, and the low unit weight would imply it would not be able to resist high loads exerted. Figure 15 shows the distribution of potentially collapsible soils in Southern Africa as presented by the Department of Public Works (DPW, 2007).

Problems associated with construction on collapsible soils are not only confined to buildings with shallow foundation structures, but also to roads, airfields and railways, as well as earth dams and reservoirs (Braatvedt et al. 2008).

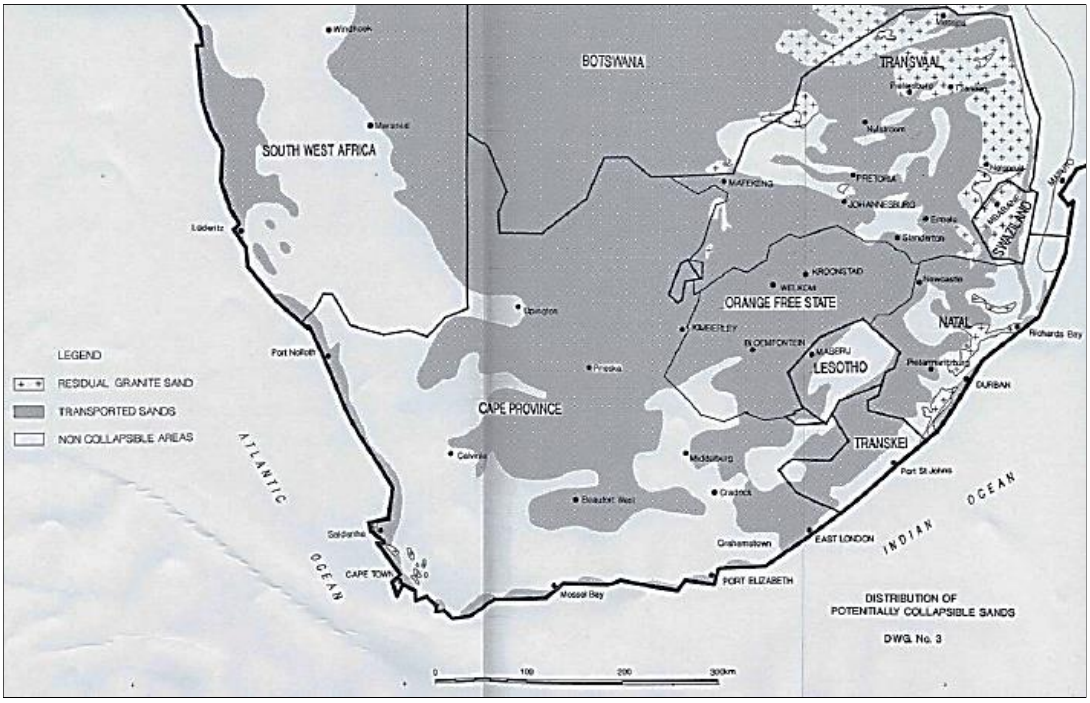

Figure 15: Distribution of potentially collapsible soils in Southern Africa (DPW, 2007).

According to Braatvedt et al. (2008) for collapse settlement to occur, several conditions need to be satisfied;

- The soil must have a collapsible fabric. Soils of low in-situ dry density which are silty or sandy commonly exhibit a collapsible fabric.
- An initial condition of partial saturation must be present. This condition is applicable to the upper horizons of the soil profile in most areas of Southern Africa.
- An increase in moisture content must occur so that a loss of shear strength of bridging colloidal materials can be effected.
- The imposed pressure exerted on the soil fabric by the structure must exceed the overburden pressure.

Figure 16 shows the mechanism of collapse settlement in these collapsible soils.

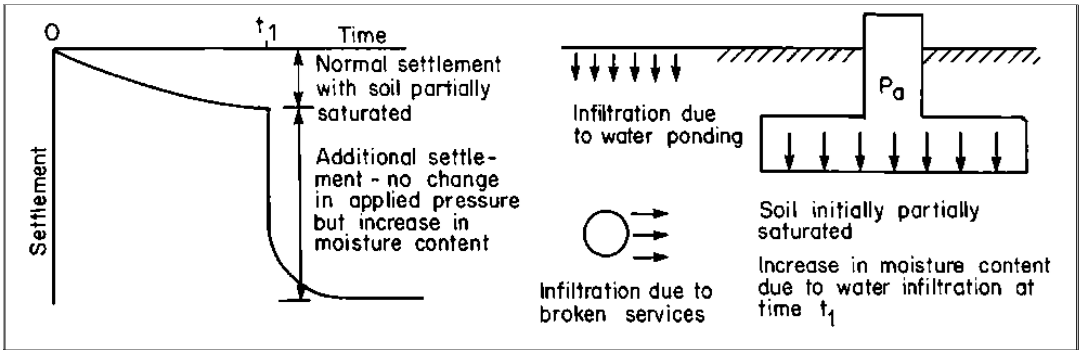

Figure 16: Mechanism of collapse settlement (Braatvedt et al. 2008).

The collapse potential (C_p) is calculated using Equation 15;

$$C_p = \Delta\varepsilon = \frac{e_1 - e_2}{1 + e_0} \tag{15}$$

Where:

$\Delta\varepsilon$ = vertical strain

e_0 = natural void ratio of the soil

e_1 = void ratio before flooding of the soil

e_2 = void ratio after flooding of the soil

Table 1 shows the relation of the collapse potential of a soil to the severity of the problem faced.

Table 1: Relation of collapse potential to the severity of foundation problems (Das, 2010).

Collapse potential, C_p (%)	**Severity of problem**
0 – 1	No problem
1 – 5	Moderate trouble
5 – 10	Trouble
10 – 20	Severe trouble
>20	Very severe trouble

2.4.1.2. Expansive soils

These soils mainly contain clay minerals, such as smectite, and thus tend to be cohesive and plastic. With the existence of the double layer, the clay minerals have a high affinity for water and therefore there is potential swelling in the wet season and shrinking in the dry season (Kalumba, 2013).

Expansive soils are affected by the rapid infiltration and dissipation of water into the soil structure as the water table rises and falls during changes in the climatic conditions. They are associated with low shear strength; semi- to impervious soil; poor compaction and workability; unstable slopes; and uneven bedrock surface. These are the most common of the problem soils in Southern Africa and are widely distributed throughout the region (Braatvedt et al. 2008) as shown in Figure 17. This is attributed to the formation of expansive soils as either residual soils from parent rocks of the igneous and argillaceous formations found in Bushveld Igneous Complex, Karoo Sequence and the Beaufort Group, or categorized as transported soils deposited by streams in the form of alluvium, lacustrine, gulleywash and fine colluvium (Williams et al., 1985). Tables 2 and 3 show the different characteristics and types of residual and transported soils respectively.

Table 2: Residual soils (Department of Public Works (DPW), 2007).

Parent Rock Type	Examples of Rock Types	Type of Material Formed	Associated Engineering Impact
Acid Igneous Rocks	Vein quartz, Pegmatite, Rhyolite, Aplite, Granite	Clayey sand or sandy clay (often mica-rich); clayey gravel; corestone; gravel, cobbles and boulders	Collapsible grain structure; dispersive soil; sand “boils”, high permeability; high erodibility; good compaction and workability
Basic Igneous Rocks	Basalt, Dolerite, Andesite, Diorite, Norite, Pyroxenite	Clay (turf); silty clay changing to sandy clay with depth; corestones; gravel, cobbles and boulders	Expansive clay; low shear strength semi- to impervious soil; poor compaction and workability; unstable slopes; uneven bedrock surface.
Calcareous Rocks	Calcrete, Limestone, Marble, Dolomite	Wad; silty or sandy clays; clayey or sandy gravel; angular gravel, cobbles and boulders; large floaters of dolomite	Cavities, sinkholes and dolines; hard rock bands with interbedded loose or soft layers; highly erodible; highly porous; fair to good compaction and workability; troughs and pinnacles; extremely uneven bedrock surface.
Argillaceous (clayey) Sedimentary Rocks	Claystone, Mudstone, Siltstone, Shale, Coal	Clay, silt, silty clay	Expansive clay; low shear strength; high settlement; slaking on exposure; semi- or impervious soil; dispersive soil; poor compaction or workability; unstable slopes.
Arenaceous (sandy) Sedimentary Rocks	Sandstone, Conglomerate, Tillite, Chert	Clayey sand or gravel; cobbles, boulders or rubble	Expansive clay from tillite; pervious to semi-impervious soil; high erodibility; good to Excellent compaction and workability.

Table 3: Transported Soils (Department of Public Works (DPW), 2007).

Transported Soil Type	Transportation Agent	Source Rock	Soil Type	Problems to Anticipate
Talus (coarse colluvium)	Gravity	Any rock outcropping directly above talus deposit	Unsorted angular gravel and boulders within sandy soil matrix	Slope instability
Hillwash (fine colluvium)	Sheetwash	Acid crystalline, Basic crystalline, Arenaceous sedimentary, Argillaceous sedimentary	Clayey sand, Clay Sand, Clay or silt	Collapsible grain structure Heave High compressibility
Alluvium or gulley wash	Streams or gulleys	Dependent on catchment	Gravel, sand, silt or clay	All possible problems, including dispersivity and erosion
Lacustrine deposit	Stream depositing in pan, lake or subterranean pool in cavernous rock	Usually mixed source	Sand, Silt, Clay	Heave or high compressibility
Estuarine deposit	Rivers and tides	Mixed	Sand, Silt, Clay	Quicksand

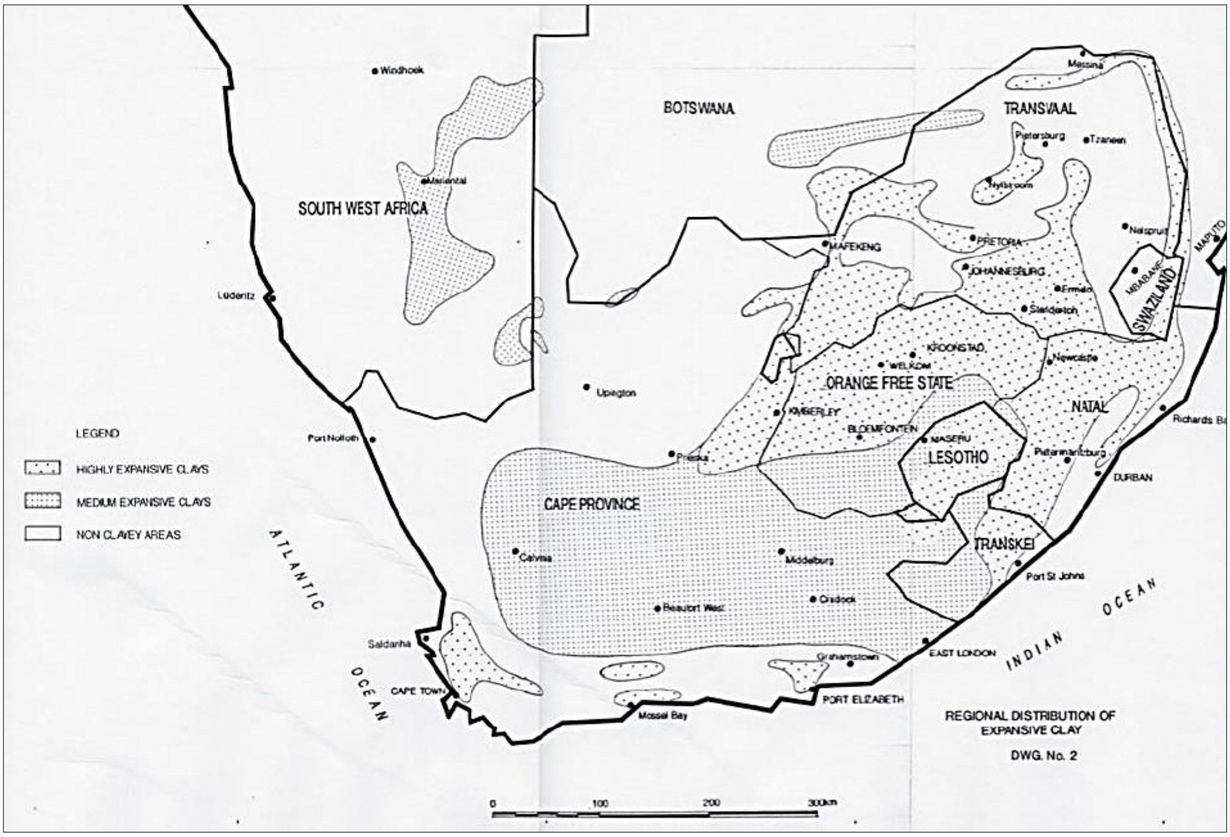

Figure 17: Distribution of potentially expansive soils in Southern Africa (DPW, 2007).

Structures constructed on these soils are subjected to large uplifting forces caused by swelling, which would induce heaving and cracking throughout the structure. When faced with construction on a potentially expansive soil, it is acceptable to follow standard practices if there is low swell potential of the soil. However, if the soil possesses potential to swell marginally or highly, then precaution is necessary. The following methods could be applied to avoid the effects of the soil (Das, 2011);

- Replacing the expansive soil with more suitable soil
- Changing the nature of the soil through compaction; prewetting; installation of moisture barriers; or chemical stabilization.
- Strengthening the structure to withstand the heaving of the soil; structures that are flexible enough to withstand differential heave; or constructing deep foundations passed the expansive soil.

2.4.1.3. Dispersive soils

These soils are susceptible to distresses under pavement layers, as they disperse with the increasing presence of water disrupting the structure of the soil layers. According to Braatvedt et al. (2008) the problem of these soils affect embankments, dams and slopes as there is a tendency of influx of water around these structures. The distribution of these soils in South Africa is shown in Figure 18.

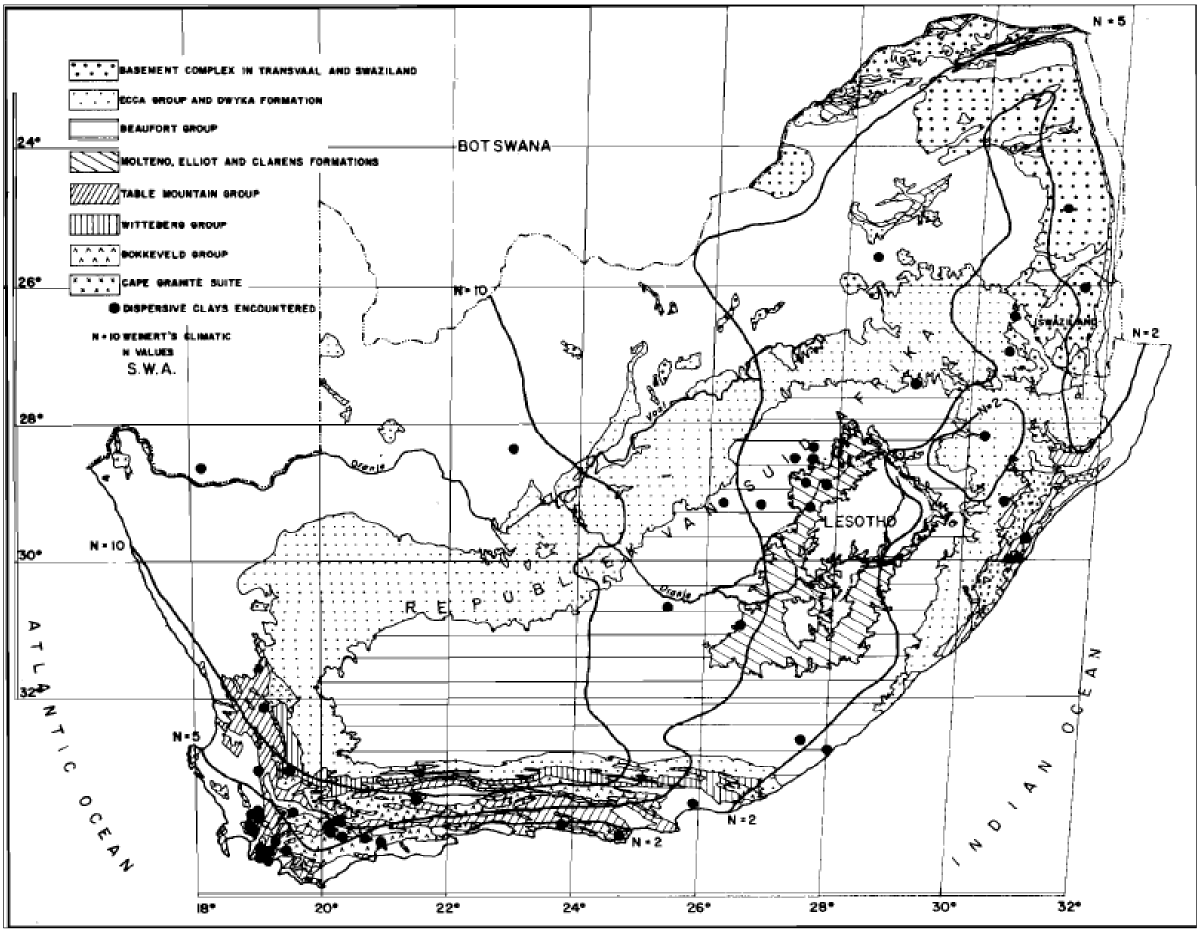

Figure 18: Distribution of dispersive soils in South Africa (Elges, 1985).

According to Elges (1985) the formation of pipes or desiccation cracks within the internal structure of the soil mass eventually becomes severe enough that stability of the soil is compromised and collapses, which is a common occurrence in earth clay dams and natural clay slopes. Particular care should be taken when designing earth dams, drainage channels and lateral support where the soil mass within the structure is dispersive as these soils deflocculate when permeated by relatively pure water and then become susceptible to erosion and piping.

The tendency of dispersion in a soil depends on variables such as the mineralogy and chemistry of the clay and on the dissolved salts in the soils water and the eroding water. When there is a high exchangeable sodium percentage in the clay-water system, there is the potential of piping to occur, which is a common occurrence in soils where the clay fraction is largely composed of smectite and also illites. The phenomenon of dispersion occurs in the clay soils when the repulsive forces between the particles exceed the attractive forces. When the clay soil is in contact with water, the particles progressively detach and are held in suspension, and in the case that the water is flowing they are carried away (Elges, 1985).

2.4.1.4. Dolomitic soils

The nature of the problem associated with dolomitic soils is as a result of changes in the water table and the presence of soluble bedrock (Kalumba, 2013). The dissolution of the dolomitic rock cavities made up of carbonate bedrock results in settlements and punching shear failure. These soils are limited to areas that are underlain by rocks of the Campbell, Witwatersrand and Chuniespoort Groups (Braatvedt et al. 2008) concentrated in the northern regions of South Africa as shown in Figure 19.

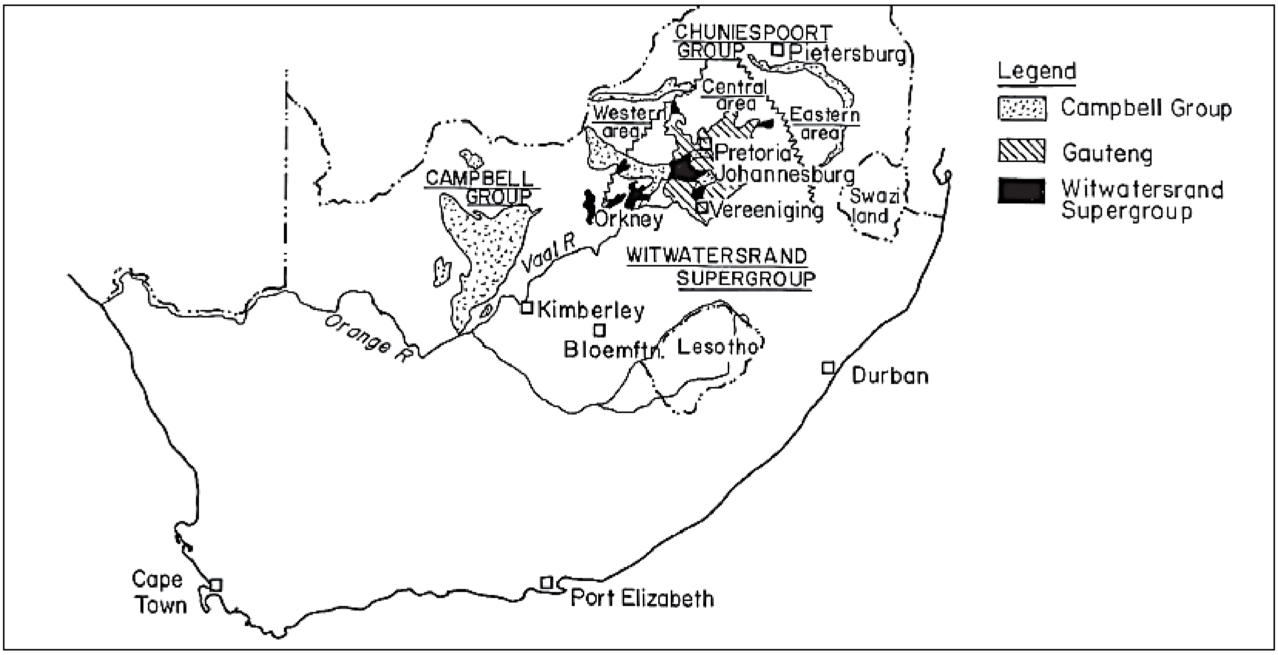

Figure 19: Distribution of dolomitic rocks in South Africa (Braatvedt et al. 2008).

According to Wagener (1985), damages to structures and loss to life have been more severe on dolomitic formations than any other geological formations and problem soils. Areas with mining activity have had inflow of water along the fissures have led to wide-spread damages as a result of surface instabilities in the form of sinkholes. Figure 20 shows the extent of the damage that could occur on sites underlain with dolomitic soils.

Figure 20: Damage to structures due to formation of sinkholes on dolomitic soils (Savage, 2014).

2.4.1.5. Soft Clays

Soft soils exhibit low shear strength, high compressibility, and lead to severe time related settlement problems. In southern Africa these clays are often partially saturated and over-consolidated (Braatvedt et al., 2008). The soft clays are generally limited to the flood plains and estuaries of the Eastern seaboard with the most significant deposits at Durban, Richards Bay and the Natal North and South Coasts, with localised deposits also around Cape Town. (Braatvedt et al. 2008).

Road and railway embankments constructed on these soils have had stability failures characterised by long-term settlements in excess of the predicted values, with rotational failures evident in extreme conditions. These settlements, specifically differential settlements, are the problems associated with construction of embankments on soft clays, and occur over time with observed settlements of 30% of the height of the embankment, with extreme instances of up to 95% (Jones and Davies, 1985). In addition, the low shear strength of the soft clays does not permit the construction of shallow foundations unless a compacted fill is placed over the clay. However, this solution is only suitable for structures that are light and can tolerate differential settlements. The alternative is the use of piles, which would be relatively easy to install through the clays but would still be an expensive option as most of the support would be from the end bearing capacity on the bedrock as there is negative skin friction with the soft clay which may provide challenges in design (Department of Public Works, 2007).

2.4.1.6. Liquefiable soils

Soil liquefaction occurs when loose sands temporarily change from a solid state to having the consistency of a heavy liquid. Soil liquefaction typically occurs in cohesionless sands, silt, and fine-grained gravel deposits, and is a consequence of increasing pore water pressures and corresponding decrease in effective stress induced by loose sands and tendency to decrease in volume when subjected to cyclic undrained loading (Braatvedt et al. 2008). The problems associated with these soils are stability and settlement related. Instability and large settlements for heavy loads such as road embankments present engineering problems to infrastructural developments. Most building structures located on these soils demand a piled foundation solution.

2.4.2. In situ soils of concern in this research

The soils of interest in this study are the soils that are susceptible to high levels of settlement and have relatively low load-bearing capacities as their California Bearing Ratios (CBRs) are below 3%. The soils with these characteristics are widespread throughout South Africa in form of soft clays, collapsible soils, dispersive soils and expansive clays.

Figure 21 shows a combination of distribution of collapsible and expansive soils in Southern Africa that are susceptible to large settlements under loading conditions and generally have low load bearing capacities. The soft clays are found in the Eastern seaboard as discussed in Section 2.4.1.5, and the distribution of dispersive soils covered in Section 2.4.1.3.

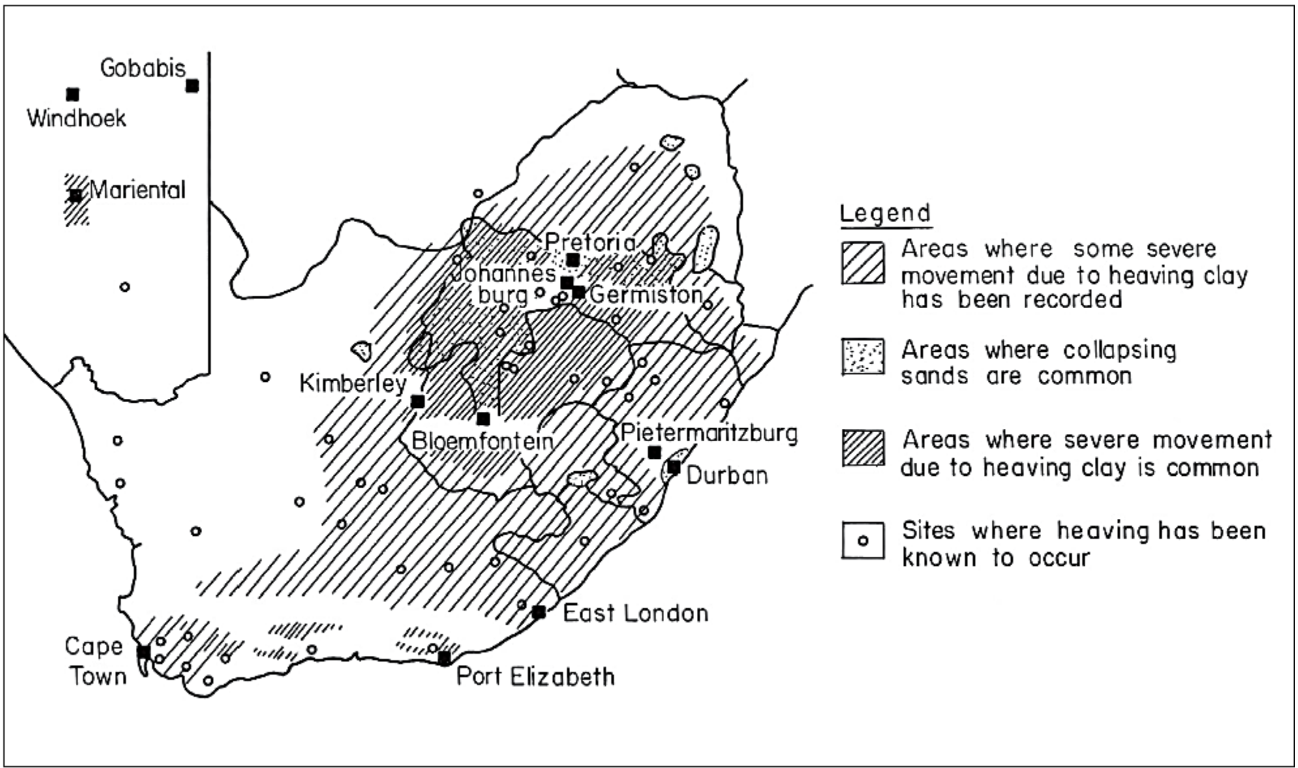

Figure 21: Map of compressible and expandable soils in Southern Africa (Braatvedt et al. 2008).

2.4.3. Consequence of Construction on these soils.

The consequence of construction depends on the type of soil that is faced on the site however much they have the same undesirable properties of low bearing capacities and high susceptibility to settlement. If the soils are primarily collapsible soils, there may not necessarily be an increase in moisture content that leads to collapse but the application of heavy loads in pavement structures like highways, airfields and even railways would be sufficient to cause shear failure and induce collapse. In the case of expansive soils, driving comfort may be severely affected due to movements caused by the underlying expansive clays. There is also the problems at approach contacts at bridges and at drainage structures such as culverts crossing underneath roads that could be affected by differential settlement (Department of Public Works, 2007). Soft clays on the other hand can cause severe problems such as stability failures, construction problems and long-term settlements to road and rail embankments. Most of these problems occur during construction consisting of rotational failures and could also be as extreme as complete displacement of the in-situ soil.

As these soils have low load-bearing capacities and are highly susceptible to settlement, construction on these soils would lead to failure and damage of the structures on them. In pavement structures these failures are associated with rutting of the road surface; surface cracking and the formation of potholes rendering the pavement structure serviceability state low. With these failures occurring on roads there shall be need for maintenance that could be costly and lead to delays to the road users.

2.4.4. Methods to deal with these conditions

Given that these soils are highly compressible and susceptible to settlement, the treatment would have to counter these actions and thus prevent them from occurring under loading. To stabilize these soils either physical or chemical treatment can be applied to improve the strength properties of the soil.

There are a number of techniques to deal with the ground and these are shown in Table 4 grouped into the different approaches that could be used; ground improvement, ground treatment and ground reinforcement.

Table 4: Techniques to deal with problematic ground conditions.

	Ground Treatment	**Ground Improvement**	**Soil Reinforcement**
TECHNIQUES	Soil Cement	Surface Compaction	Stone Columns
	Lime Admixtures	Drainage/Surcharge	Soil Nails
	Fly ash	Electro-Osmosis	Micro-piles
	Dewatering	Compaction Grouting	Jet Grouting
	Heating/Freezing	Blasting	Ground Anchors
	Vitrification	Dynamic Compaction	Geosynthetics
		Pre-Loading	Fibres
			Lime Columns
			Vibro-Concrete Columns
			Mechanically Stabilised Earth
			Biotechnical

The methods presented in the next sections are those suitable for the types of problem soils dealt with, and the methods that could be applied are the ones practiced in the civil engineering field for pavement construction.

2.4.4.1. Ground Treatment

Ground treatment involves mixing some chemicals in the soil like lime or cement to improve the material properties may be attempted to stabilize the soil; but the thickness of the clay, problems with proper mixing, and maintaining the required moisture content can prove this method difficult to implement successfully. The methods applied change the properties of the soil such that they are improved to the desirable level.

The chemical methods involve introducing a reacting agent into the soil that would change the material properties of the soil, thus improving its performance qualities. The use of lime, cement, or fly ash for the stabilization of soft subgrades tends to be labour intensive and sensitive to the construction environment. While these methods may be cost competitive with geosynthetic reinforcement for some applications, consideration must be given to the physicochemical characteristics of the soil to determine the suitability of these techniques. There is also an associated construction delay for mixing and, in some cases, hydration that must be considered with these selections. In addition, construction quality control becomes an issue for these alternatives. (Berg et al., 2000).

2.4.4.2. Ground Improvement

In the construction field it has been discovered that some soils lack the necessary strength properties to support the loads exerted on them by erected structures, and this necessitates the improvement of the soil. Ground improvement can either cause temporary or permanent change, and the choice depends on the specific engineering purpose (Rogers, 2012). Ground improvement involves altering the material properties of the soil to achieve the desired state necessary for construction to occur on the site. The properties that could affect construction are strength and compressibility which could be improved by reducing the void ratio that would reduce settlement.

Pre-loading

Preloading the site is a method applied such that the settlement that will be experienced by the structure is reduced. This method is beneficial in collapsible soils as the air voids are filled by the soil particles prior to applying the structural loads. In collapsible soils the collapse potential of the soil and the depth of occurrence is dependent on, which would qualify in situ densification by impact rolling as the most sufficient method to resolve the problem; and in addition sufficient water must be applied during impact rolling (Department of Public Works, 2007). The limitation to this method is the time it takes to achieve the necessary settlement that would prevent further problems to structures erected on the soils. The addition of vertical and horizontal drains could accelerate the process to a certain degree.

2.4.4.3. Soil Reinforcement

Soil reinforcement is the process of improving the strength characteristic of a soil so that it is able to support or carry more loads, and prevent the occurrence of geo-failure and structural damage. The methods of soil reinforcement are numerous and are mainly dependent on the type of soil to be dealt with, and the scale of project. Reinforcing the soil involves adding structural elements that has better properties than the soil, such that the composite structure has a greater performance. Using one of the many available engineering methods would strengthen the soil. One of the methods will be further discussed in Chapter 3.

According to Kalumba (2013) the reasons for soil improvement include but are not limited to:

- Improving the soft soils that are susceptible to settlement, thus reducing the settlement structures are subjected to improve the load-bearing capacity of the soil and the overall stability of structures.
- Addressing liquefiable soils on the sites, at risk of failure due to seismic activity, mainly in regions on or near fault lines.
- Improving slope stability by increasing the factor of safety, thus preventing development of failure surfaces and landslides occurring. This is applicable to high road and rail embankments.
- Assisting in retaining unstable soils, which is applicable to high embankments and sites with deep excavations that need lateral earth supports.
- Improving the workability and usability of fill materials, which would be the case for importation of fill material to the site either to replace the in-situ fill or to add material to that on site to bring it back to its original level, usually when the soil on site has been recompacted.
- Improving the soil shear strength by enhancing the structure of the soil, thus increasing the bearing capacity of foundations.

2.5. Soil Replacement

2.5.1. Introduction

Construction sites are usually faced with problematic soils that need to be excavated and replaced to achieve the required strength for construction to progress. Replacing the soil under the foundation, by excavation and using imported quality fill with more desirable properties is a conventional method applied on construction sites. The excavated soil can also be used as backfill though the compaction of the material would lead to the necessity to import more fill material to bring the ground back to its original level. This is mainly beneficial in soils with a thin layer of expansive or collapsible qualities that would be beneficial to excavate as opposed to application of any of the other techniques (Braatvedt et al., 2008).

2.5.2. Soil Selection

The choice of soil type for the fill are usually granular materials because they have greater strength and stiffness characteristics, which is mainly dependent on the material density. Their greater stiffness and resistance than loose soils is attributed to the greater interlocking of the grains that allows for greater resistance to stresses (Lopes, 2012). When incorporated with other reinforcing elements, granular soils act to form a composite structure as the soil interacts with the elements, increasing the shear resistance of the soil.

2.5.3. Benefits and Limitations of Soil Replacement

The advantages of this method is that the imported fill can achieve an increase in bearing capacity; and it is a relatively simple and easy method to undertake that is quicker than alternatives. Aggregate is also a natural resource that often requires some level of conservation. However, in areas where good quality base and subbase materials are plentiful and relatively inexpensive, and where over-excavation is not required, there may be little (initially apparent) cost benefit in using additional reinforcement.

The limitation of this method is that the thickness of fill required to achieve adequate strength increase is large, and failure could occur through water ingress during construction (Nelson and Miller, 1992). In certain regions, the use of granular fill is costly due to the distance of the quarries to the project sites; and there are also prohibitions by environmental constraints to exploitation of granular fill (Palmeira, 2013). There is also the issue of time consumption through all the processes needed to carry out this method, such as replacing the unsuitable material (Geosynthetics Materials Association, 2011).

The resilient modulus of unreinforced base and subbase materials tend to be negatively impacted over time by a loss of aggregate to the subgrade and an increase in moisture (Berg et al., 2000). This is through pumping of the fine materials into the granular fill, and the subsequent penetration of the granular particles into the soft subgrade.

2.5.4. Inclusion of Geosynthetics

Given the limitations of the soil replacement method, there is a need to incorporate different methods and materials to further improve the quality of the soils on sites. The shift from the conventional methods of construction and need to improve the strength properties of the soil, has led to geosynthetic materials being included in pavement structures. The benefits of the application of geosynthetics in pavement structures is discussed further in Chapter 3.

CHAPTER 3

3. Geosynthetics

3.1. Introduction

A geosynthetic is a planar product manufactured from a synthetic or polymeric material used in contact with soil, rock, earth, or other geotechnical-related material as an integral part of a civil engineering project, structure, or system (SANS ISO 10318:2013).

3.2. Types of Geosynthetics

The types of geosynthetics depend on the function, application and manufacture process, and these include: Geogrids; Geotextiles; Geocomposites; Geosynthetic clay liners (GCLs); Geonets; Geocells; Geomembranes; and Geofoams. Figure 22 shows the different types of geosynthetics that could be manufactured for different applications and functions.

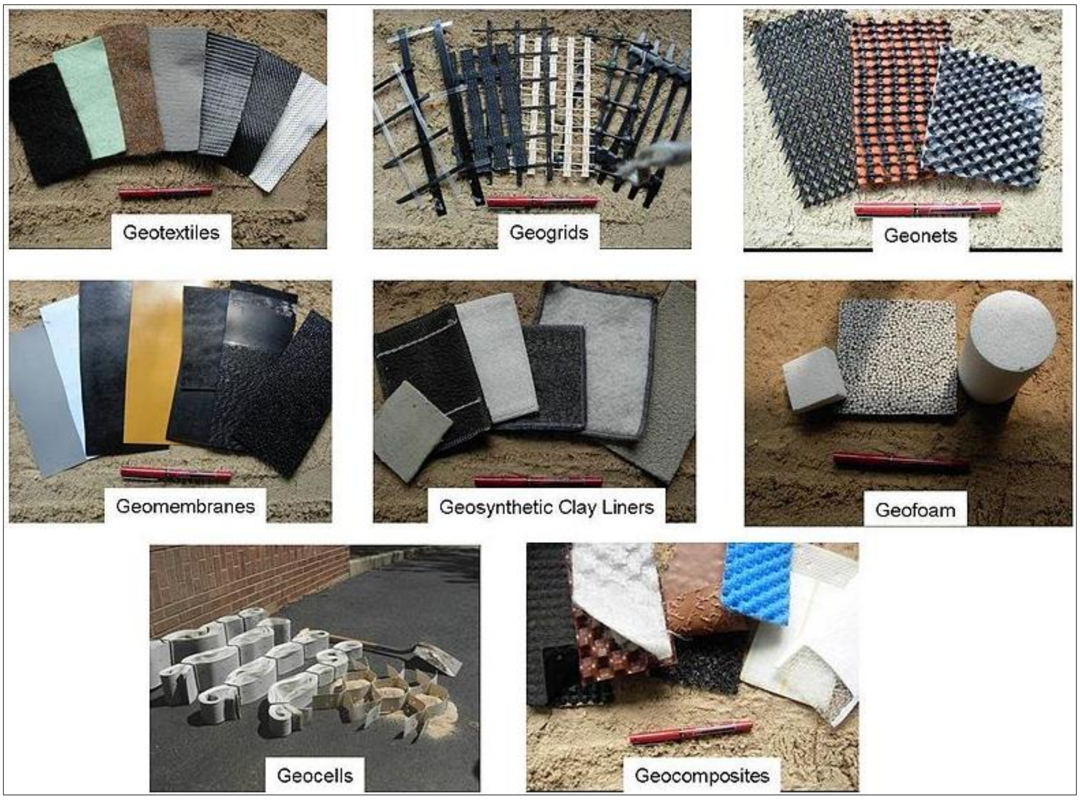

Figure 22: Collage of different types of geosynthetic products (Gorantla Geosynthetics).

3.3. Functions of Geosynthetics

Geosynthetics are generally designed for a particular application by considering the primary function that can be provided. The multiple functions of geosynthetics are dependent on the material they are manufactured from and also on the application intended. The different functions include; separation, reinforcement, filtration, drainage, and containment as shown in Table 5.

Table 5: Identification of the primary functions for each type of geosynthetic product.

Type of Geosynthetic	Separation	Reinforcement	Filtration	Drainage	Containment
Geotextile (GT)	X	X	X	X	
Geogrid (GG)		X			
Geonet (GN)				X	
Geomembrane (GM)					X
Geosynthetic Clay Liner (GCL)					X
Geofoam (GF)	X				
Geocells (GL)	X	X			
Geocomposite (GC)	X	X	X	X	X

Most geosynthetic materials only play a passive role, e.g., geosynthetic barriers stop the passage of liquids; geosynthetic reinforcement provides tensile resistance, but only after an initial strain has occurred; and geo-drains provide a passage for water but do not cause the water to flow (Jones, 2007). This can be changed if the geosynthetics are designed to play an active role, like in the case of electrokinetic geosynthetics (EKGs) that drain the soil of excess pore water, and reinforce the structure thereafter. The functions of geosynthetics related to the research conducted are further discussed.

3.3.1. Filtration

According to Sarsby (2007) the geosynthetic acts as a filter by permitting the flow of liquid and gases but prevents major passage of soil particles, which could cause blockage of the drain or settlement due to loss of ground. Figure 23 shows how a geosynthetic carries out the filtration function.

When low-permeability fills are loaded, excess pore water pressure could be generated that would result in a reduction in the available shear strength of the cohesive fill and also a reduction in the soil–reinforcement bond, requiring more reinforcement to provide an adequate bond length. The dissipation of this excess pore water pressures into geosynthetics would result in consolidation and settlement of the reinforced structure (Jones, 2007).

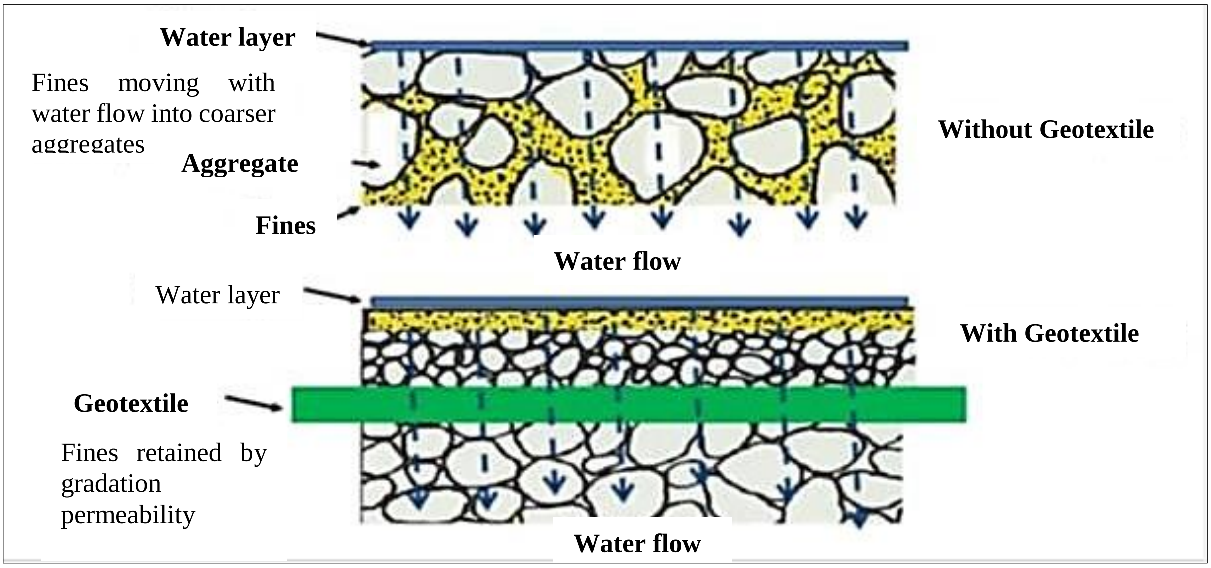

Figure 23: Filtration function of geosynthetics (Textile Innovation Knowledge Platform, 2014)

3.3.2. Separation

In the case of a granular layer over a fine-grained soft soil, the separator must prevent the fine particles of the soft soil from entering the gaps between the particles of the granular material above it, as well as preventing the larger grains of the granular layer from sinking into the soft soil below as shown in Figure 24. The separator must also permit water to pass through to prevent pore water overpressure in the soft soil, and all this must function under loading conditions (Wilmers, 2007).

The separation function is typically used in road construction where the subgrade condition is poor. A geosynthetic will act as a barrier preventing the roadway's base material from being pushed into the weaker subgrade material, resulting in a smaller amount of base material required for the construction of the road (Erickson and Drescher, 2001).

The physical demands for a separator under an unsurfaced access road are more complex. When vehicles pass along the road, the granular layer is pressed down by the wheels and deformed in accordance with the shape and load of the wheels. This deformation widens the gaps between the granular particles, which permits the finer soil particles to penetrate the granular layer. Successive transits by vehicles increase the amount of fine particles in the granular layer and the coarse aggregates start to sink into the fine soil. Eventually, the granular layer collapses and the road becomes impassable (Agrawal, 2011).

The separator must therefore have the following characteristics:

- It must follow the deformation under rolling loads.
- It must have a high elongation, to allow rutting without the layer rupturing.
- It must possess sufficient strength to prevent a local collapse.
- It must be robust enough to withstand mechanical stresses during installation and under traffic.

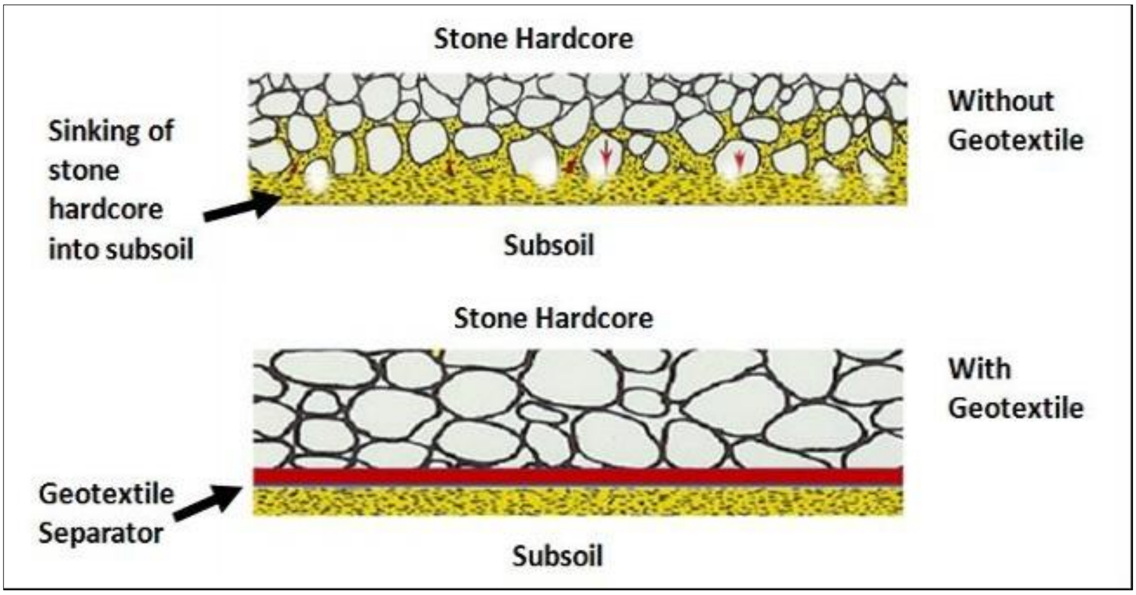

Figure 24: Separation function of geosynthetics (Textile Innovation Knowledge Platform, 2014)

3.3.3. Reinforcement

The bearing capacity relates to the strength of the soil and depends on the thickness of the granular layer and the deformability of the underlying soft soil. The granular layer should be a well-graded granular material and should be strongly compacted, to obtain high friction between the grains and maximum stiffness. When an embankment is constructed over soft compressible ground, the load from the fill promotes foundation failure in the underlying ground without improving its shear strength. The insertion of geosynthetics within the embankment or at its base would provide extra lateral force to prevent the embankment from failing by splitting or rotation. With time, pore water in the foundation will migrate from beneath the embankment and the shear strength of the foundation will increase. The stability of the embankment will thus improve in time as the underlying soft soil consolidates. As the underlying soil strength increases, the stabilizing force which needs to be provided by the geosynthetic diminishes (Sarsby, 2007). Figure 25 shows the reinforcement function of a geosynthetic.

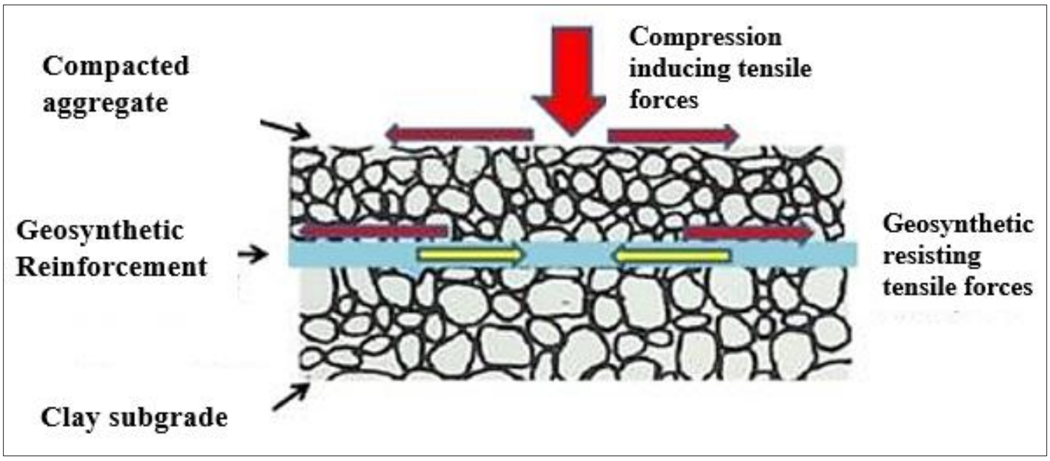

Figure 25: Reinforcement function of geosynthetics (Textile Innovation Knowledge Platform, 2014)

3.4. Properties of Geosynthetics

The properties of geosynthetics could vary from physical, to mechanical, hydraulic, endurance and degradation. The physical properties of concern are the type of structure, mass per unit area, thickness and stiffness. According to Perkins (2007) the structure of the geosynthetic usually dictates the application in which it shall be used; in geotextiles there are woven and non-woven geotextiles, while in geogrids it is according to the process used to manufacture the junctions which include woven, extruded, integral and welded.

The mechanical properties of geosynthetics are associated with the applications where the material is subjected to loading and undergoes deformations. Loading can also be applied in the plane of the geosynthetic resulting in tension of the material. This type of loading is generally associated with the function or operation of the constructed facility and where the mechanical properties of the geosynthetic are typically used in the design of the facility. Mechanical properties pertaining to the shearing resistance between the geosynthetic and the surrounding soil are also important as this resistance is responsible for transferring load from the soil into tensile load in the geosynthetic (Shukla, 2012a). Geosynthetics are mainly beneficial for their tensile properties, as they complement soil that is good in compression but weak in tension.

Endurance properties of geosynthetics are also important in selection, design and use of the material in different geotechnical applications, and the properties that are of concern include installation damage, and abrasion. The deformations and stresses experienced by geosynthetics during installation can be more severe than the actual design stresses for the intended application and arise from the placement and compaction of overlying fill. Damage may occur in the form of holes, tears and ruptures, which influences the mechanical and hydraulic properties of the material. (Perkins, 2007). The abrasion of geosynthetics is defined as the wearing away of any part of a material by rubbing against another surface. Excessive abrasion can lead to a loss of properties, e.g. strength, that are needed for proper functioning.

3.5. Manufacturing Process

Geosynthetics are manufactured from a variety of polymers such as polypropylene, polyester, polyethylene, polyamide and PVC as the common types. These materials are highly resistant to biological and chemical degradation, as compared to natural fibres that can only be used for temporary applications (Sarsby, 2007). Table 6 shows the typical polymers used in manufacturing geosynthetics and their compositions.

Table 6: Commonly used geosynthetic resins and their compositions (Sarsby, 2007).

Type	Resin (%)	Plasticizer (%)	Fillers (%)	Carbon black or pigment	Additives (%)
Polyethylene	95–98	0	0	2–3	0.25–1
Polypropylene (flexible)	85–98	0	0–13	2–4	0.25–2
Poly(vinyl chloride)	50–70	25–35	0–10	2–5	2–5
Poly(ethylene terephthalate)	98–99	0	0	0.5–1	0.5–1
Polyamide	98–99	0	0	0.5–1	0.5–1
Polystyrene	98–99	0	0	0	1–2

In manufacturing geotextiles, elements such as fibres or yarns are combined into planar textile structures. The fibres can be continuous filaments, which are long thin strands of a polymer, or staple fibres, which are short filaments, typically 20 to 100 mm long. The fibres may also be produced by slitting an extruded plastic sheet or film to form thin flat tapes. In both filaments and slit films, the extrusion or drawing process elongates the polymers in the direction of the draw and increases the fibre strength (Sarsby, 2007). A yarn is made of one or more fibres.

Geotextile type is determined by the method used to combine the filaments or tapes into the planar textile structure. The vast majority of geotextiles are either woven or nonwoven. Woven geotextiles are made using any one of these types of yarns; monofilament yarns (made from a single filament), multifilament yarns (made from fine filaments aligned together), spun yarns (made from staple fibres interlaced or twisted together) or fibrillated yarns (made from strands), or of slit films and tapes (Shukla, 2012a). Woven geotextiles are obtained by conventional weaving processes using a mechanical loom as shown in Figure 26.

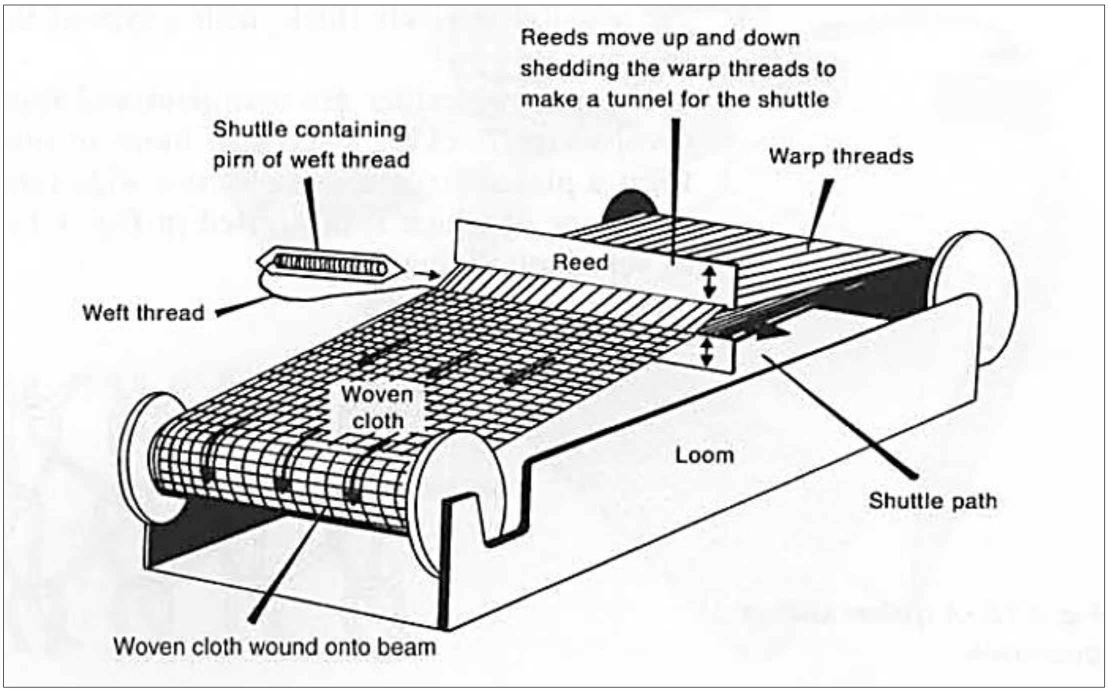

Figure 26: Main components of a weaving loom used in the manufacture of woven geotextiles (Shukla, 2012a).

Although the weaving process is old, non-woven textile manufacture is a modern engineering development in which synthetic polymer fibres or filaments are continuously extruded and spun, blown or otherwise laid onto a moving belt. The mass of filaments or fibres are then either needle punched, in which the filaments are mechanically entangled by a series of small needles, or heat bonded, in which the fibres are welded together by heat and/or pressure at their points of contact in the nonwoven mass (Sarsby, 2007). The final product is then wound into rolls as shown in Figure 27.

Figure 27: Manufacture process of woven geotextile (Alibaba.com, 2014)

Extruded geogrids are manufactured by the method of processing a polymer sheet in two or three stages as shown in Figure 28 where the sheet is first fed into a punching machine which punches holes on a regular pattern; followed by heating and stretching or drawing in the machine direction. Stiff geogrids with integral junctions are manufactured by extruding and orienting sheets of polyolefin. Flexible geogrids are made of polyester yarns joined at the crossover points by knitting or weaving, and coated with a polymer.

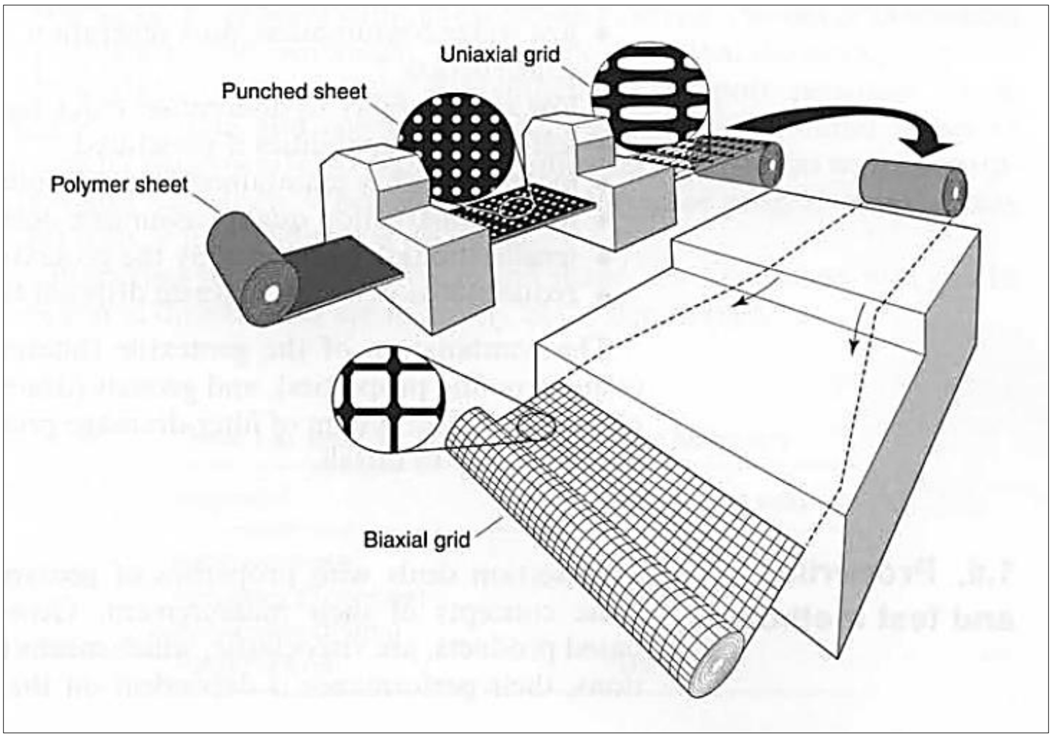

Figure 28: Tensar manufacturing process of geogrids, courtesy of Nelton (Shukla, 2012a).

3.6. Theory of Geosynthetic Reinforcement

In monotonic loading the mechanism for soil improvement can be as simple as separating native soils from fills, or it could be expanded to the mechanisms of soil-reinforcement interaction that include tensile membrane action; lateral restraint; and alteration of failure surfaces (Collin, 2007).

3.6.1. Lateral Restraint or Enhanced Confinement

As the soil structure is subjected to loading, the particles resist the load until the shear strength is reached and exceeded, at which point the particles move laterally as failure occurs. Soft subgrade soils provide little lateral restraint, and so when the aggregate moves laterally, ruts develop on the aggregate surface and also in the subgrade (Zannoni, 2013). If the geosynthetic used is stiff, it acts as a restraint to this lateral movement as it forms a composite structure with the soil particles, thus keeping the soil structure stable, as shown in Figure 29, and forming a stiffer pavement subgrade. According to Erickson and Drescher (2001) if the geosynthetic is placed at a depth of high lateral strain, the shear stress in the soil can be transferred to tensile stress in the geosynthetic.

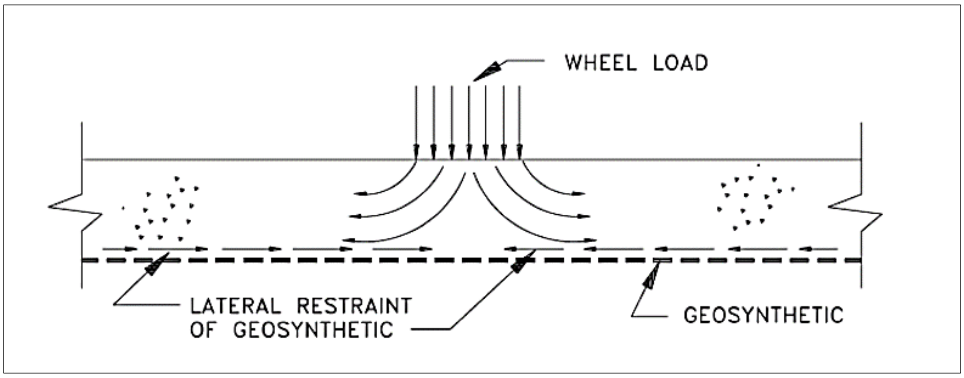

Figure 29: Lateral restraint by geosynthetic (Berg et al. 2000, after Haliburton et al. 1981)

As the geosynthetic layer acts to confine the soil particles, the tensile stresses that develop in the soil are transferred to the geosynthetic. Given that the reinforcement has a greater tensile strength and stiffness than the soil, it can resist the tensile stresses more effectively and restrain the lateral deformation of the soil (Bourdeau and Ashmawy, 2012). The stress that is transferred at the soil-geosynthetic interface results in a confining pressure, within the soil, greater than in the unreinforced soil.

3.6.2. Bearing Capacity Increase

As the geosynthetic layer forms the composite structure with the soil particles, it also reduces the stress exerted on the soil particles below enabling the soil to withstand greater loads before failure occurs. The geosynthetic reinforcement forces the potential bearing capacity failure surface to follow an alternate higher strength path as shown in Figure 30. This tends to increase the bearing capacity of the roadway (Erickson and Drescher, 2001).

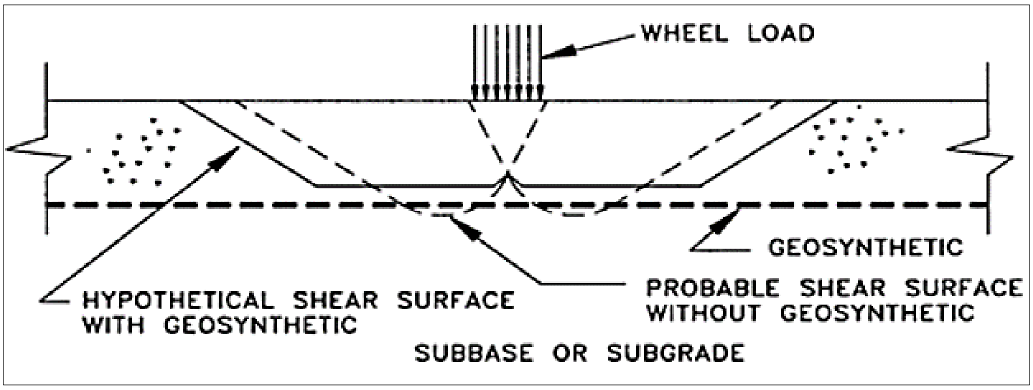

Figure 30: Bearing capacity increase (Berg et al., 2000, after Haliburton et al. 1981)

3.6.3. Tension Membrane Support

Membrane action has two effects, which include providing a direct upward component of force to resist the load and increasing the bearing capacity of the subgrade by a downward loading on its surface to either side of the loaded area (Love et al. 1987). If the geosynthetic has a sufficiently high tensile modulus, tensile stresses will develop in the reinforcement, and the vertical component of this membrane stress will help support the applied wheel loads, as shown in Figure 31. As tensile stress within the geosynthetic cannot be developed without some elongation, wheel path rutting is required to develop membrane-type support (Zannoni, 2013).

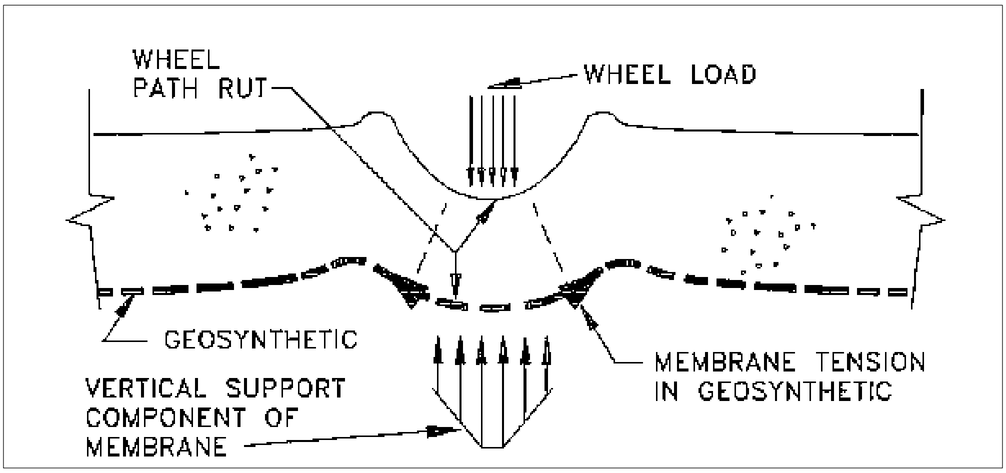

Figure 31: Tensioned membrane support (Berg et al., 2000, after Haliburton et al. 1981)

Under repeated traffic loading, the above mechanisms of reinforcement are still active, but there is the development of two additional mechanisms that are specific to cyclic loading conditions. These include the additional compaction of the aggregate base course and the dynamic interlock of the soil particles.

3.6.4. Additional Compaction of the Base Course

According to Bourdeau and Ashmawy (2012), the additional compaction of the base course is achieved as a result of repeated cyclic loading that leads to an increase in stiffness and the resistance of the granular fill. Tests conducted by other authors (Leflaive, 1985; Nimmesgern and Bush, 1991) showed that due to the additional compaction, there was a significant performance improvement under cyclic loading as compared to monotonic loading. In unpaved roads, the membrane support and lateral restraint mechanisms as provided by the reinforcement layer allow for the additional compaction to occur; and prevents bearing failure during the early cycles of loading.

3.6.5. Dynamic Interlock

This mechanism is specific to geogrid reinforcement, as the aggregate particles become locked into the apertures of the grid. This prevents the elastic part of the reinforcement tensile strain from being fully recovered during the unloading phase; as a result, the geogrid remains stressed and the lateral confinement of the aggregate layer is increased (Bourdeau and Ashmawy, 2012).

3.7. Application of Geosynthetics

The applications of geosynthetics are numerous, however the ones discussed here are those directly related to the research conducted, in terms of how the results and findings could be applied in geotechnical engineering projects.

3.7.1. Unpaved Roads

Unpaved roads are not capped with concrete slabs or covered by an asphaltic wearing course. These designs are found in temporary access roads or tracks, forest roads and haul roads; and generally consist of crushed stone or gravel fill that is laid directly on the subgrade (Bourdeau and Ashmawy, 2012). The main application of geosynthetics in unpaved roads is as a separation layer. In most cases these roads are used by heavy vehicles for a limited period, with the frequency of usage varying from high to none at all. Given that these roads have to be dismantled once they are no longer needed it is important to minimize the quantity of material used, therefore the thickness of the bearing layer is kept as thin as possible.

Access roads are typically narrow and vehicles use the same path in both directions, so rutting is inevitable as shown in Figure 32. Deformation is thus permitted, but not to the level of collapse, because otherwise maintenance demands for the road become too high. Under these conditions, consolidation of the subsoil over time occurs more easily and helps to enhance the bearing capacity. To optimize the thickness of the layer for the given traffic load, field tests are needed because an estimation of the reaction of the subsoil and the influence of the friction characteristics of the fill is not realistically possible. The choice of geotextile separation layer should take into consideration the fill grain size, expected rut depths, and must demonstrate the required filter characteristics (Wilmers, 2007).

Figure 32: Rutting in unpaved road (murderiseverywhere.blogspot.com, 2014)

3.7.2. Paved Roads

Geosynthetics are used in paved roads to provide the necessary reinforcement, separation, drainage and filtration; with the purpose of controlling development of pavement distress features in the form of ruts and cracks (Perkins et al., 2012).

According to Wilmers (2007) the permitted elastic deformation for subgrades under road bases and superstructures is less than 0.5 mm. If the soil does not have adequate strength, geosynthetics cannot guarantee a sufficient strength, because geosynthetics require deformation to occur to develop a reaction force. The vertical deformation of a rut, necessary for an elongation of only 1.0% in a textile on the base of a layer of 300 mm thickness, is more than 50 mm for a wheel rut of 300 mm width.

Geosynthetics as separation layers do not directly improve the strength of a subgrade, but are helpful under a granular layer used to improve the bearing capacity of the subgrade (by excavating soft soil and replacing it with granular soil of higher bearing capacity), because they guarantee that the granular material keeps its properties even when disturbed by construction traffic. This is achieved by preventing the contamination of the granular fill by the migration of fines from the subgrade layer (Perkins et al., 2013). If this is not prevented, then rutting would occur as shown in Figure 33.

Figure 33: Rutting in paved road (sourced from Pavement Interactive)

3.7.3. Railway Tracks

In railway foundation design a separation layer between ballast and the underlying subgrade soil can be efficient because, under a dynamic load, the coarse grains can be pressed or vibrated into the ground, which would lead to destabilization of the soil structure, making maintenance necessary. The high erosion of coarse sharp-edged grains under the dynamic loads of railway circulation destroys even thick geotextiles in a short time. The most common problem in railway track sub-structure is mud pumping of the fine subgrade soils causing ballast (granular fill) fouling and degradation (Tan and Shukla, 2012). Figure 34 shows a fouled ballast from subgrade mud pumping. Therefore, in railroad tracks with frequent circulation, it is better to install a layer of, for example, sandy gravel under the ballast. This layer is filter stable against the ballast under load and is known as the 'protection layer'.

Figure 34: Fouled subgrade from mud pumping (Tan and Shukla, 2012).

A geotextile separation layer should be placed at the interface of the ballast and soft soil, instead of a sand layer, to aid in the prevention of deterioration of the soil structure. In addition, the geotextile would act as a filter allowing water to pass freely from the subgrade to the fill during train passby while preventing pumping of the fines. The geotextile could also act as a drainage layer directing water from precipitation and pumping action to the edge of the track, where it is drained away (Tan and Shukla, 2012). The use of a strong geotextile or geogrid could provide adequate lateral basal confinement that would improve the ballast stability as shown in Figure 35.

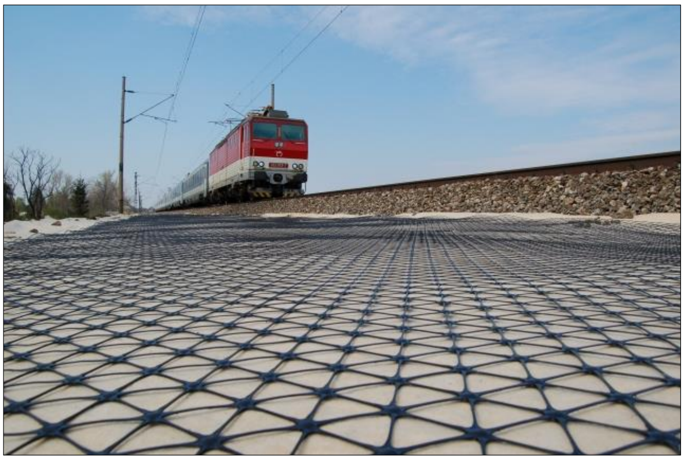

Figure 35: Geosynthetic used in railway ballast reinforcement (Geosynthetica.net)

3.7.4. Parking Lots

Parking lots are stressed under low-velocity circulation and by vehicles that rest in the same place for a long period of time. Parking lots mostly have bound pavements (asphalt or cement concrete) or concrete stone set paving. In all cases, the construction must be stiff enough to hinder local deformations under load, which can be followed by settling. The separation layer between subsoil and first fill aids during construction, because it prevents deformation and mixing of the underlying soil. The bearing capacity is not influenced by the separation layer but only by the bearing capacity of the subsoil and the properties and thickness of the fill (Wilmers, 2007). Figure 36 shows the placement of the geosynthetic between the layers.

Figure 36: Geosynthetics used in parking lot construction (Typar Geosynthetics)

3.8. Geotextiles

A geotextile is a permeable geosynthetic made of textile materials mainly used for separation purposes. Geotextiles have been used since the days of the Pharaohs in road construction and in the construction of the pyramids and as an aid in road construction over soft ground by the Romans (Shukla, 2013a). These geotextiles were however made of natural fibres, fabrics or mixed vegetation that were then mixed with the soil, though recently the products are manufactured using industrial fabrics and must comply with numerous standards.

3.8.1. Types of Geotextiles

There are various types of geotextiles made from polymers such as polyester or polypropylene, but are mainly differentiated by the method of manufacture, which are woven fabrics, non-woven fabrics and knitted fabrics.

3.8.1.1. Woven Fabrics

Woven geotextiles are manufactured using techniques similar to weaving of normal clothing textiles, in which individual filaments are weaved together to create an interlocking structure, having the characteristic appearance of two sets of parallel threads; the warp that runs along the length, and the weft that is perpendicular to it, as shown in Figure 37. These woven geotextiles may comprise slit film tapes, monofilaments, multi-filaments, fibrillated tapes or combinations, by which better performance and cost can be achieved. Higher permeability is obtained with the use of monofilaments and multi-filaments. According to Koerner (2007) woven geotextiles are known to typically fail at elongation strains less than 50%.

Figure 37: Woven geotextile product.

3.8.1.2. Non-woven fabrics

Non-woven geotextiles are manufactured using either short fibres or continuous filaments that are bonded together thermally, chemically, mechanically or by a combination of these techniques (Agrawal, 2011). The type of technique used to manufacture the non-woven fabrics would lead to different thicknesses in the final product; thermally bonded non-wovens typically range from 0.5 – 1 mm, while chemically bonded are usually in the order of 3 mm, and mechanically bonded fabrics range from 2 – 5 mm. According to Koerner (2007) non-woven geotextiles typically fail at elongation strains greater than 50%. Figure 38 (a) shows a non-woven geotextile.

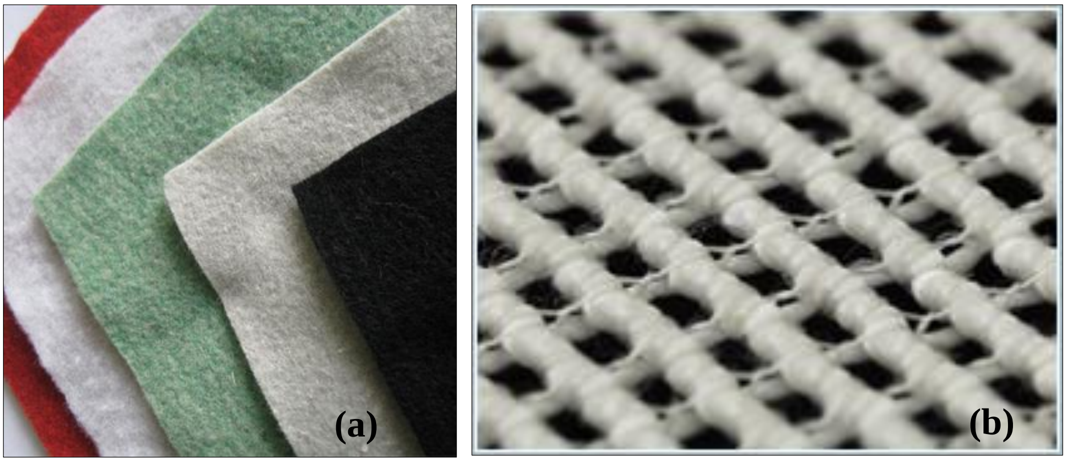

Figure 38: (a) Non-woven geotextile (Fibre Cloths), and (b) Knitted geotextile (Alpe-Adria Textil, 2014)

3.8.1.3. Knitted fabrics

These are manufactured using the knitting process, also adopted from the clothing textiles industry, in which interlocking of loops of yarn is made to form the finished product (Agrawal, 2011). Knitted geotextiles are however used in limited geotechnical applications, and Figure 38 (b) shows a knitted geotextile.

3.8.2. Applications of Geotextiles

The application of geotextiles in civil engineering works are vast, and are mainly dependent on the function required from the geotextile.

Geotextiles are used in road construction mainly for the separation of different layers of soils used in the foundation or embankment structure. There is also the excess pore pressure that builds up under the road bed that needs to be drained away to prevent deterioration of the structure, which can be provided by geotextile membrane if they have a higher degree of permeability than soil.

In railway works, woven and non-woven geotextiles are used to separate layers without impeding the ground water circulation where the ground is unstable. Also having the layers enveloped in the geotextiles prevents the material dispersing laterally as a result of the shocks and vibrations from the running trains.

In river canals and coastal works, woven and non-woven geotextiles are used to prevent erosion, and can also be used as a filter when combined with natural or artificial rock structures.

The use of geotextiles in drainage applications is beneficial, as they prevent the in-situ soil from being washed away clogging the drainage system. They are used in roads and highways; earth dams; reservoirs; behind retaining walls; trenches and in agriculture.

Geotextiles are also used in sports and recreational fields to form a stable base that would allow for support of the working loads, and function as a filter and drainage system, thus preventing waterlogging from occurring.

3.8.3. Benefits of Geotextiles in Reinforcement

Geotextiles are flexible and thus the tensioned membrane action can be accessed when excessive deformation has occurred to the soil, thus preventing further failure. They also act as a drainage path for excess pore pressure, thus limiting build up that would lead to deterioration of the soil structure. Geotextiles also improve the performance of an unpaved road by acting as a separator between the clay and granular fill (Love et al. 1987).

According to Zannoni (2013) geotextiles installed at the interface of the subgrade and base layers have the following benefits:

- Reduce rutting due to construction traffic
- Provide working platform
- Improve subgrade bearing capacity
- Reduce differential settlement when spanning soft zones
- Reduce need for chemical stabilization

Geotextiles can be used in two alternative design approaches; to either reduce the thickness of the aggregate base layer, or to increase the pavement design service life as shown in Figure 39.

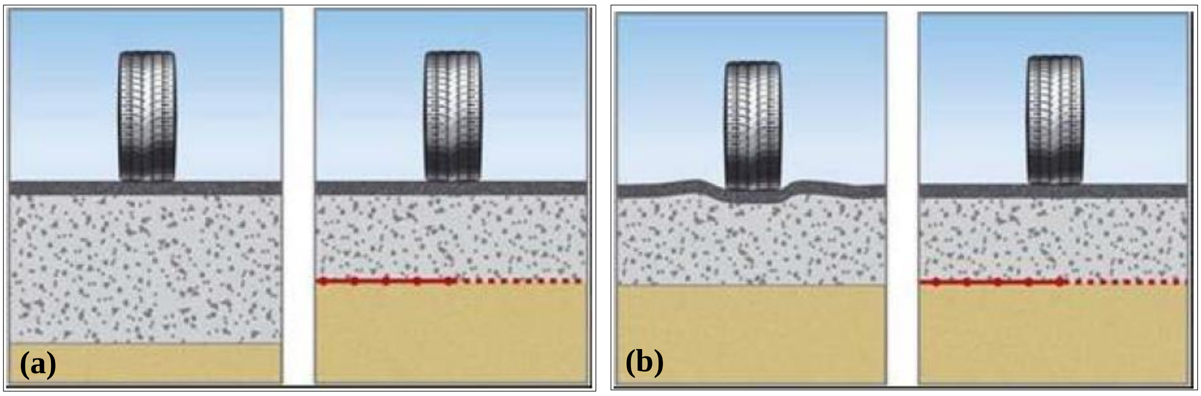

Figure 39: Design approaches illustrating (a) the reduction in the aggregate base layer, and (b) the extension in design service life of the pavement structure (Zannoni, 2013).

3.9. Geogrids

Geogrids are geosynthetic products formed by a regular network of tensile elements with apertures of sufficient size to interlock with surrounding fill material. Geogrids improve the structural integrity of the soil by confining the particles and distributing the loads exerted. Geogrids provide support for the construction of access roads, highways, and structure applications that previously required the use of relatively expensive excavating or piling methods on soft subgrades. Geogrids are also used in base reinforcement applications to reduce aggregate thickness requirements and/or extend roadway performance life. The performance of geogrids in providing reinforcement to soil depends on its rigidity; having a high tensile modulus to take up the tensile strains; and the aperture geometry that accounts for its interlocking with the soil particles (Shukla, 2013a).

3.9.1. Types of Geogrids

The types of geogrids available differ in terms of the manufacturing process and the type of polymer used and include:

- Knitted or woven geogrids
- Punched and drawn geogrids
- Heat or chemically bonded

3.9.1.1. Knitted or Woven geogrids

They are produced using methods similar to the textile industry, as done for the woven and knitted geotextiles. They are manufactured from polyethylene or polyester polymers and coated with bitumen, latex or PVC for added strength. These have high flexibility, and can assess the tensioned membrane effect when considerable strain has undergone in the soil. Figure 40 (a) shows a variety of knitted geogrids and Figure 40 (b) shows a flexible woven geogrid.

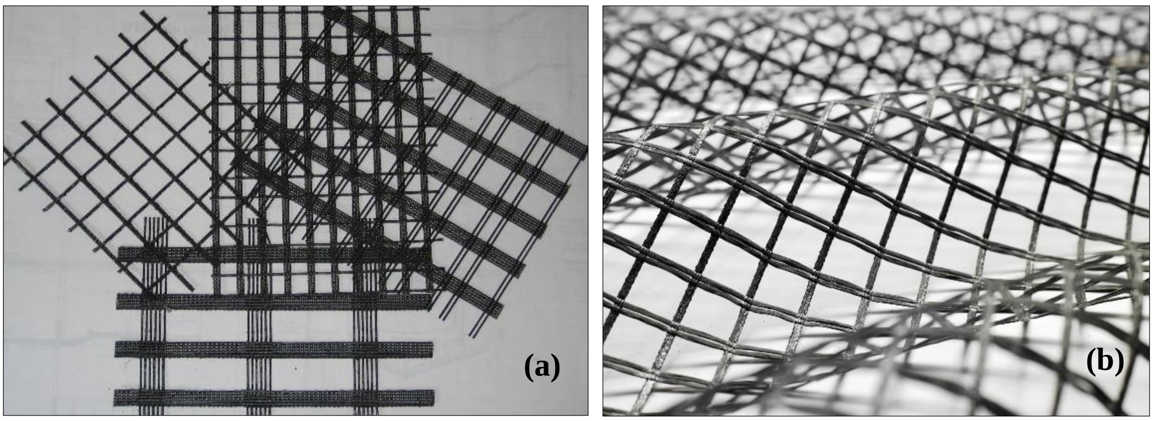

Figure 40: (a) Illustration of a variety of knitted geogrids, and (b) a flexible woven geogrid.

3.9.1.2. Punched and drawn geogrids

These are also known as extruded geogrids and are formed by punching holes in a sheet followed by drawing or stretching to form the desired configuration of geogrid. The polymers used are mainly HDPE and Polypropylene (PP). Figure 41 shows a variety of extruded geogrids that differ in terms of the direction of transfer of the loads exerted on the soil. Uniaxial geogrids transfer the loads in one direction and are mainly used in reinforced concrete retaining block walls. Biaxial geogrids are used for basal reinforcement and transfer the loads in two directions. Triangular geogrids provide a better reinforcement as they transfer the loads in three directions, almost in a radial manner. There also exist Quaxial geogrids that provide similar reinforcement as the triangular geogrids but have increased stiffness.

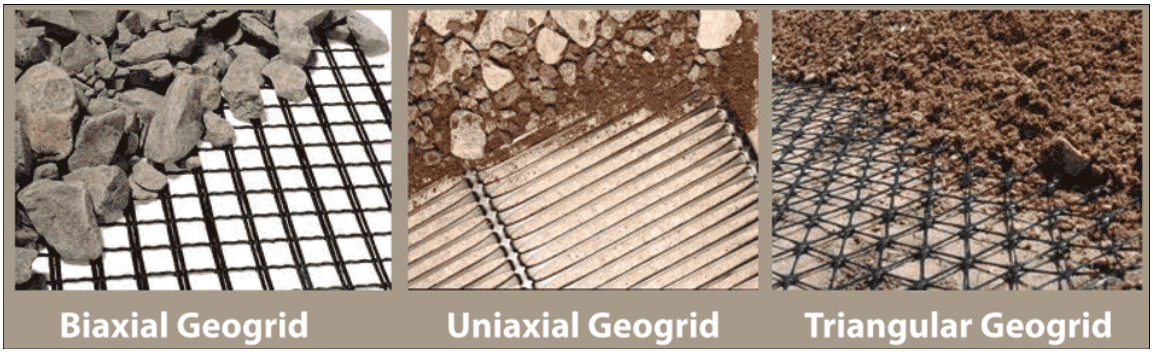

Figure 41: Collage of various types of extruded geogrids (Tensar).

3.9.1.3. Heat or chemically bonded geogrids

These are formed by bonding polymers either chemically or using heat, to form a sheet-like structure with ribs at the point of bonding. They are usually produced for special design cases that need high degrees of stiffness in the geogrid reinforcement.

3.9.2. Applications of Geogrids

Geogrids are mainly used in reinforcement applications that include basal reinforcement and within embankment structures for roads and railways. In all these applications, the geogrid improves the load-bearing capacity of the soil, through different design configurations that include the depth of placement, the number of layers of reinforcement, and the spacing between layers. Geogrids have apertures that assist in interlocking the soil particles, thus forming a composite structure. This forms a separating layer in soils that have different particle sizes; and it also acts to restrain lateral movement of the particles – reducing settlement of pavement structures and also increasing the bearing capacity of the soil. Most geogrids are not that flexible, so the tensioned membrane action is not readily accessed with them. Woven geogrids are used in soils of larger particles as they have larger pore size, while non-woven geogrids are used where clay or silt are formed.

According to Zannoni (2013), when installed at the interface of the base and subbase aggregate layers, geogrids reduce surface deformation by reducing permanent deformation in unbound aggregate and subgrade layers; and reduce fatigue cracking in asphalt concrete layers by reducing dynamic deformation.

3.9.3. Benefits of Geogrids in Reinforcement

According to Moayedi et al. (2009) geogrids are able to improve the performance of subgrade soils through four mechanisms;

- Prevention of local shearing of the subgrade;
  Vehicular loads applied to the roadway surface create a lateral spreading motion of the base aggregate. Tensile lateral strains are created at the bottom of the base as aggregate moves down and out away from the applied load. Lateral movement of the base aggregate allows for vertical strains to develop leading to a permanent rut in the wheel path. Placement of a geosynthetic layer or layers in the base aggregate allows for shear interaction to develop between the base and the geosynthetic as the base aggregate attempts to spread laterally. Tensile load is in effect transmitted from the base aggregate to the geosynthetic layer. Since the geosynthetic is considerably stiffer in tension as compared to the base aggregate, far less lateral tensile strain develops in the system. This first reinforcement mechanism results from less lateral strain being developed in the base, which results in less vertical deformation of the roadway surface. The shear stress developed between the base aggregate and the geosynthetic provides an increase in lateral stress within the bottom portion of the base. This increase in lateral confinement leads to an increase in the mean hydrostatic normal stress in the aggregate. Granular materials generally exhibit an increase in elastic modulus with increasing mean stress, meaning that the base aggregate becomes stiffer when adequate interaction develops between the aggregate and the geosynthetic. (Pietro, 2001)
- Improvement of the load distribution through the base course;
  An increase in modulus due to lateral confinement of the base also results in less vertical strain being developed in the base aggregate. While this mechanism controls the development of rut depth, it might also be expected that an increase in modulus of the base would result in lower dynamic, recoverable vertical deformations of the roadway surface, meaning that fatigue of an asphalt concrete layer in a flexible pavement would be reduced by this mechanism. For layered systems, where a weaker, less stiff subgrade material lies beneath the base aggregate, an increase in modulus of the base also means that this layer will aid in distributing load on the subgrade.
- Reduction or reorientation of shear stresses on the subgrade;
  This reduces vertical stress in the base and in the subgrade beneath the centreline of the wheel. A reduction of vertical stress results in lower vertical strain in these layers. As a result of an improved load distribution, the deflected shape of the roadway surface would have less curvature.
- The tensioned membrane effect;
  Membrane support of the wheel load reduces the vertical stress applied to the subgrade. Confinement of the subgrade increases its resistance to shear failure (i.e. bearing capacity). The reinforcement process is dependent on the rut depth developed, and comes into effect when a substantial amount of settlement (rut depth) has been attained.

3.10. Soil–Geosynthetic Interaction

3.10.1. Introduction

When geosynthetics are used as reinforcement elements in soil, the most important feature in the provision of this reinforcement is the interaction between the soil and the geosynthetic. This is attributed to the necessary transfer of the stress in the soil to the geosynthetic material. The purpose of this is to inhibit the development of tensile strains in the soil, and also to support the tensile stresses that the soil cannot withstand (Lopes, 2012). The tensile stress supported by the geosynthetic improves the mechanical properties of the soil by reducing the shear stress that develops and allows a greater shear resistance. As such the shear strength of reinforced soil relies on the mobilised shear resistance in the soil and the mobilised tensile stress in the reinforcement.

3.10.2. Soil-geosynthetic interaction mechanism

There are many factors that could have an effect on the soil-geosynthetic interaction, such as the material properties of the soil; the construction process; and the mechanical properties of the reinforcement. The mechanisms of interaction that are critical in reinforced systems are:

- Skin friction along the reinforcement,
- Soil-soil friction, and
- Passive thrust on the bearing members of the reinforcement.

The skin friction is the resistance that develops between the soil and the surface of the geosynthetic material as shown in Figure 42(a). In geotextiles this is the only mechanism that is developed, however in geogrids, there is also the development of soil-soil friction as the grains protrude through the apertures of the reinforcement. In addition to that, there is also the passive thrust that the grains exert on the bearing members (ribs and junctions) of the geogrids, as shown in Figure 42(b) (Lopes, 2012).

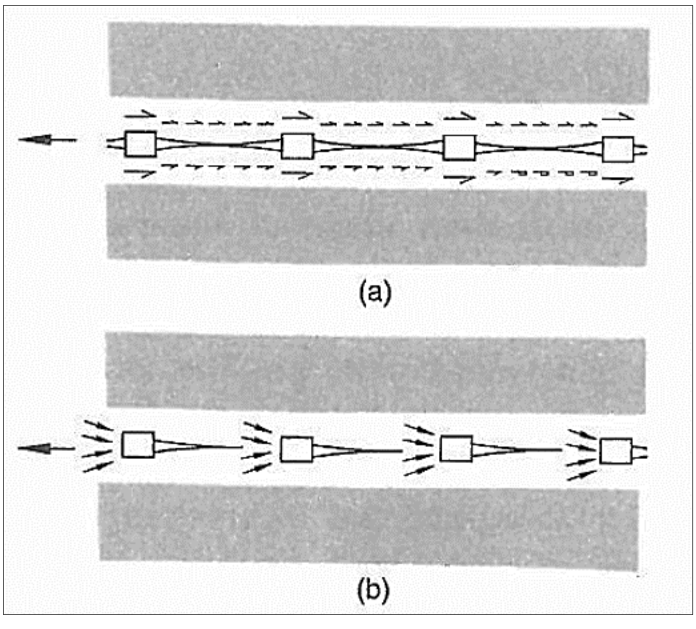

Figure 42: Soil-geogrid interaction mechanisms: (a) shear between soil and plane surfaces and (b) soil bearing on reinforcement surfaces (Lopes, 2012).

3.10.3. Soil-geosynthetic interaction resistance

According to Lopes (2012) reinforced soil stability is directly related to the effectiveness of transfer of the stress subjected on the soil to the reinforcement, and this is dependent on the length of reinforcement available to shear. With geotextiles the only mechanism that contributes to the interaction is skin friction, whereas with geogrids there is the additional interaction through mobilization of passive thrust on the bearing members of the grid; and the soil-soil friction as shown in Figure 42.

The mobilization of strength in soil-reinforcement interfaces is through the sliding of a grain of soil across one side of the reinforcement that is linked on the other side to the other grain of slide, and this is known as direct sliding. In this case when the shear strength of the soil-reinforcement is exceeded failure occurs by direct shear. The soil-reinforcement interface shear strength in direct shear can be defined by Equation 16.

$$T = WL\sigma'_n f tan\varphi' \tag{16}$$

Where 0 < f < 1, f is the soil-reinforcement interface coefficient

φ' is the soil friction angle in terms of effective stresses.

σ'_n is the effective normal stress in the interface.

W and L are the width and length of the reinforcement respectively.

The soil-reinforcement interface coefficient is dependent on the interaction mechanism mobilized and is given by Equation 17.

$$f = f_{ds} = \frac{tan\delta}{tan\varphi'} \tag{17}$$

Where δ is the friction angle at the soil-reinforcement interface.

However, in the case of geogrids the shear strength is the sum of the skin friction mechanism (T_s) and the soil-soil friction mechanism (T_{s-s}), as shown in equations 18 – 20. The passive thrust mobilization on the bearing members of the geogrid is almost negligible in the case of direct sliding, thus not considered.

$$T = T_s + T_{s-s} \tag{18}$$

$$T_s = a_s WL\sigma'_n tan\delta \tag{19}$$

$$T_{s-s} = (1 - a_s)WL\sigma'_n tan\varphi'_n \tag{20}$$

Where a_s is the fraction of the geogrid surface that is solid.

3.10.4. Factors influencing soil-geosynthetic interaction.

3.10.4.1. Soil particle size

The soil particle has considerable influence in the soil-geosynthetic interaction, especially when geogrids are used as the reinforcement. It was determined by Jewell et al. (1984) that the coefficient of direct sliding increases as the particle size increases, and a maximum value is reached when the grain size is similar to that of the geogrid aperture. The minimum value is reached when the particle size is larger than the aperture size such that penetration is inhibited, and interface resistance is only mobilized at the points of contact between the soil and the reinforcement. The recommended ratio for geogrids used as soil reinforcement, according to Jewell et al. (1984) is shown in Equation 21.

$$\frac{minimum\ aperture\ dimension}{average\ soil\ particle\ size} \geq 3 \tag{21}$$

3.10.4.2. Confinement stress

As shown in equation 1, the confinement stress is important in soil-geosynthetic interface resistance as it affects the soil friction angle. The confinement stress is more notable in geotextiles where the strength mobilization in the interface is a three-dimensional phenomenon, in which an increase in the confinement stress can inhibit the dilatancy that tends to occur at the interface in dense soils. This would lead to a greater improvement in the soil-geosynthetic interface strength (Lopes, 2012).

3.10.4.3. Soil density

Soil density affects the interface strength in the same way as the confinement stress. Granular soils that are considerably dense are more resistant and have greater stiffness, which would lead to dilatant behaviour and thus induce higher confinement stresses (Lopes, 2012).

3.10.4.4. Geosynthetic structure

The distance between the bearing members of geogrids is another important parameter to be considered with regard to soil-geogrid interaction. If the distance is lower than an optimum value, then there is interference between the members that makes each member less effective.

3.11. Soil Reinforcement with Geosynthetics

In conclusion, soil reinforcement with geosynthetics relies on the efficiency of the interaction between the soil and the geosynthetic, which is dependent on the properties of the soil and the geosynthetic. In the case of geogrids, the soil particle size is of utmost importance as there are reinforcement mechanisms developed between the two. Stiffer reinforcement materials are more effective as they provide greater tensile support, allowing the soil to have a greater shear resistance. Geotextiles on the other hand only have skin friction mechanism contributing to the soil-geosynthetic interface resistance.

According to the Berg et al. (2000) the following advantages of incorporation of geosynthetics in the construction process can be accessed:

- Space Savings - Sheet-like geosynthetics take up much less space in a landfill than do comparable soil and aggregate layers.
- Material Quality Control - Soil and aggregate are generally heterogeneous materials that may vary significantly across a site or borrow area. Geosynthetics on the other hand are relatively homogeneous because they are manufactured under tightly controlled conditions in a factory. They undergo rigorous quality control to minimize material variation.
- Construction Quality Control - Geosynthetics are manufactured and often factory "prefabricated" into large sheets. This minimizes the required number of field connections, or seams. Both factory and field seams are made and tested by trained technicians. Conversely, soil and aggregate layers are constructed in place and are subject to variations caused by weather, handling and placement.
- Cost Savings - Geosynthetic materials are generally less costly to purchase, transport and install than soils and aggregates.
- Technical Superiority - Geosynthetics have been engineered for optimal performance in the desired application.
- Construction Timing - Geosynthetics can be installed quickly, providing the flexibility to construct during short construction seasons, breaks in inclement weather, or without the need to demobilize and remobilize the earthwork contractor.
- Material Deployment - Layers of geosynthetics are deployed sequentially, but with a minimum of stagger between layers, allowing a single crew to efficiently deploy multiple geosynthetic layers.
- Material Availability - Numerous suppliers of most geosynthetics and ease of shipping insure competitive pricing and ready availability of materials.
- Environmental Sensitivity – Geosynthetic systems reduce the use of natural resources and the environmental damage associated quarrying, trucking, and other material handling activities.

CHAPTER 4

4. Literature Review on Geosynthetic Reinforcement

Since the implementation of geosynthetics in civil construction projects, research has been undertaken to investigate their performance in pavement systems with soft subgrades. Laboratory and field tests have been conducted to determine the configurations that would provide the optimal benefits. From these studies different parameters were varied that included: width of geosynthetic; depth of placement of geosynthetic; type and stiffness of reinforcement; size of footing.

This chapter reviewed work carried out by previous authors with a summary of the discussion presented at the end of this chapter. Conclusions are drawn up from these studies looking into the gaps that could lead to improvements in research conducted in this field.

4.1. Soft Soils

The application of geosynthetics in marginal soils such as clays and silts not only increases the stability of the ground, but also makes sites more accessible during the construction process. Studies conducted by Guido et al. (1985); Mandal and Sah (1992); and Ranadive and Jadhav (2010) showed that the inclusion of geosynthetics in soft soils led to an improvement in the load-bearing capacity and a reduction in the settlements undergone. To attain these improvements, variations in parameters were applied in the marginal soils that included: length of the geosynthetic layer; number of layers; spacing between the layers.

Mawer (2013) investigated marginal soils, specifically soft clay soils from the Cape Town area that pose a problem on sites such as susceptibility to high settlements. The study aimed to address the problem by imbedding geotextiles within the clay bed. Mawer conducted compression tests on clay following variations in the aforementioned parameters; and in addition, the moisture content of the clay was varied to determine the effect it had on the reinforcement benefits. This was seen as necessary as the conditions in the soil change with infiltration of water and that would have an effect on the geosynthetic performance.

The tests were conducted in a similar experimental set-up followed by Hartley (2010); Buratovich (2011); and Oriokot (2012). The compressive load was applied to the geosynthetic-reinforced clay on a footing of dimension 140 mm by 150 mm. The soil that was used in this study was a typical soft clay namely Durbanville clay which had an angle of internal friction of 32°, and an optimum moisture content and MDD of 22% and 14.7 kN/m^3 respectively. The reinforcement material used was BIDIM A7 geotextile described as a nonwoven, continuous filament, needle-punched polyester geotextile. The dimensions of the geotextile used in the tensile tests was 280 mm by 50 mm, and had a tensile strength of 1.3 kN at a strain of 167 mm with a Young’s Modulus of 31.2 GPa.

The results obtained were compared and also analysed using the bearing capacity ratio (BCR) in which the unreinforced clay and geosynthetic-reinforced clay were compared. Figures 43 (a) and (b) show graphs of bearing pressure against settlement comparing the effect of spacing when using 2 geotextile layers and 3 geotextile layers respectively.

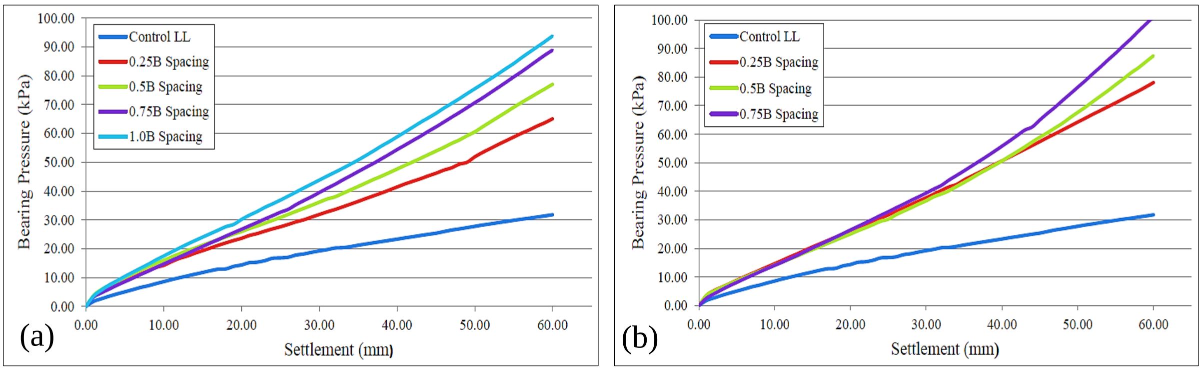

Figure 43: Bearing pressure against settlement showing the effect of spacing when using (a) 2 geotextiles and (b) 3 geotextiles (Mawer 2013).

Figure 44 shows the effect of moisture content on the bearing capacity and the BCR for: (a) the unreinforced set-up and (b) the geosynthetic-reinforced set-up, which shows a decrease and an increase in the bearing capacity respectively. This is attributed to the reduced friction between the particles as the moisture content is increased, leading to a reduction in the strength as shown in the unreinforced results. Whereas in the reinforced set-up, the geotextile provides the additional support to counteract the negative effects of increasing moisture content, thus improving the load-bearing capacity of the soil.

In the study by Ramaswamy and Purushothaman (1992) similar findings of reduced improvement of load-bearing capacity with increase in moisture content were observed.

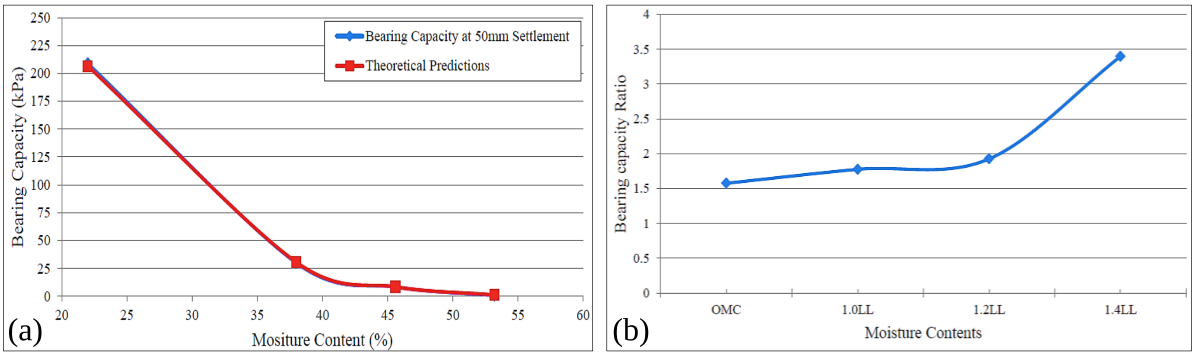

Figure 44: (a) Bearing capacity against moisture content for the unreinforced set-up, and (b) bearing capacity ratio vs moisture contents for the reinforced set-up (Mawer 2013).

4.2. Granular Soils

The structural layers of pavement structures comprise of granular soils such as sands and gravels. These layers consist of selected material and are the most important as they spread the applied loads such that the subgrades are not overstressed (SAPEM, 2013d). These soils have greater load-bearing capacities than soft soils, and their inclusion in pavements allows for higher loads to be supported and reduced settlement experienced. The application of geosynthetics in these soils could lead to further reinforcement benefits as discovered through research conducted by a number of authors.

Studies conducted by Adams and Collins (1997); Patra et al. (2005); Basudhar et al. (2007); El Sawwaf and Nazir (2010) showed improvements to geosynthetic-reinforced sand composites.

In the studies conducted by Hartley (2010) and Buratovich (2011), triangular geogrids were included in sand. Hartley added a single layer of geogrid to the sand at varying depths corresponding to the footing width. This was conducted to determine the optimum depth of placement of the geogrid layer in the sand. Buratovich varied the number of layers of geogrids included in the soil, and the spacing between the layers to obtain the optimum configuration. The study involved a similar experimental set-up as Hartley (2010) that represented a strip foundation. The material used was Klipheuwel sand sourced locally, and the reinforcement material was a triangular geogrid sourced from a South African supplier. The sand had a friction angle of 44.8% and was classified as well graded, GW, using the USCS. The materials were prepared in a specially manufactured loading box, with the parameters varied including: the number of reinforcing layers (N), the spacing between the layers (h), the depth of the first layer (u), and the footing sizes. The widths of the geogrid layers (b) was kept constant throughout all the tests.

The results presented in Figure 45 shows variations in the geogrid spacing in which an increase in the spacing between the layers led to a subsequent increase in the applied load. This was however observed only up to an optimum of 0.5B, thereafter a reduction in the applied load with increased spacing was noted. Figure 46 shows that as the number of layers was increased there was an observed increase in the applied load, with the optimum configuration involving 3 layers.

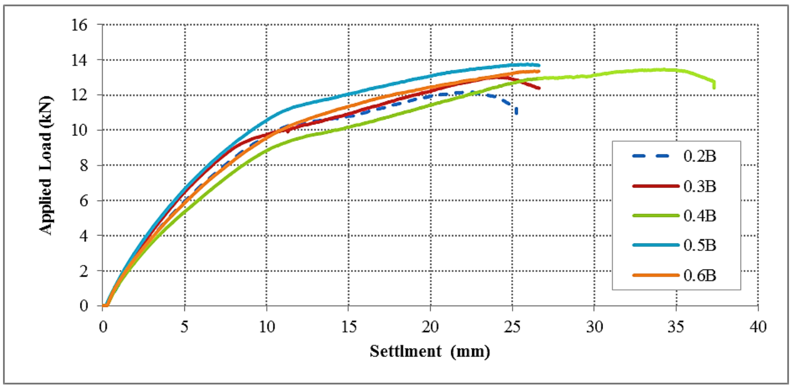

Figure 45: Applied Load (kN) versus Settlement (mm) for Test Series 2; B=100mm, u=0.25B, b=1.5B, N=2 and 0.2 ≤ h ≤ 0.6 (Buratovich 2011).

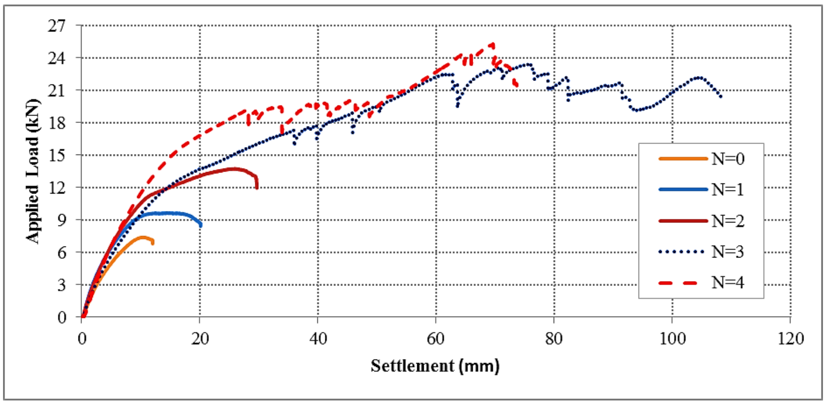

Figure 46: Applied Load (kN) versus Settlement (mm) for Test Series 3; B=100mm, u=0.25B, b=1.5B, h=0.5B and 0 ≤ N ≤ 4 (Buratovich 2011).

Oriokot (2012) also conducted bearing capacity tests on coarse-grained soils, though the material tested was crushed greywacke aggregate. This material is used in the base layer of pavement structures as it is considered to have greater strength and thus higher load-bearing capacity with better benefits in reinforcement. A single layer of triangular geogrid was included with varying in depths and widths of the reinforcement to determine the optimum depth and width.

4.3. Multi-layered Soils

The founding level of pavement structures begins on the natural subgrade and involves addition of soil layers of superior material properties. Given that subgrade soils usually do not have the required strength, the addition of a fill layer is necessary to form a stable structure capable of withstanding exerted loads through the construction phase and design life of the pavement.

Geosynthetics have been included in these multi-layered structures to further improve the performance of pavements. Research has been conducted to verify the reinforcement benefits taking into consideration different design configurations.

Studies conducted by Yetimoglu et al. (1994); Dash et al. (2003); El Sawwaf (2007); Kazimierowicz-Frankowska (2007) have shown that the inclusion of geosynthetics in a two-layered system has led to reinforcement benefits.

Love et al. (1987) also conducted tests with the geosynthetic layer placed at the interface of the soils. The experiments were carried out in a rectangular box of dimensions 1000 mm by 300 mm by 600 mm that represented the length, width and depth respectively. In the tests, one footing size of 75 mm was used, and the fill thicknesses and the strength of subgrades were varied. The thicknesses were 50 mm, 75 mm and 100 mm, while the subgrade soils strengths were 6 kPa, 9 kPa and 15 kPa, all with a CBR of less than 3%. The fill material was a mixture of Leighton Buzzard sand and gravel, and the subgrade soil was kaolin clay. The kaolin was consolidated from a slurry and then allowed to swell to produce fully saturated overconsolidated clay subgrades of different shear strength profiles. The reinforcing layer was placed on the surface of the clay to cover the whole area, and then overlain by the fill material.

From Figure 47 it was observed that the inclusion of the geogrid into the two-layered soil for each of the subgrade strength soils led to an improvement in the load per unit area. However, there is only a greater difference after 5 mm footing penetration. This shows that the geosynthetic material only provides adequate support after considerable deformation has occurred to the soil structure, from beyond 5 – 10 mm penetration, at which the load-bearing capacity is improved by 25 – 100 % when the unreinforced soil and geosynthetic reinforced composite results are compared at the ultimate penetration of 50 mm. The results indicate an increase in the bearing capacity as the subgrade strength increased and also as the thickness of the fill increased.

At the interface the geosynthetic acts to provide both reinforcement benefits and separation of the soils. This prevents progressive destabilization of the soil structure through pumping of the fines into the fill and penetration of the aggregate particles into the soft subgrade.

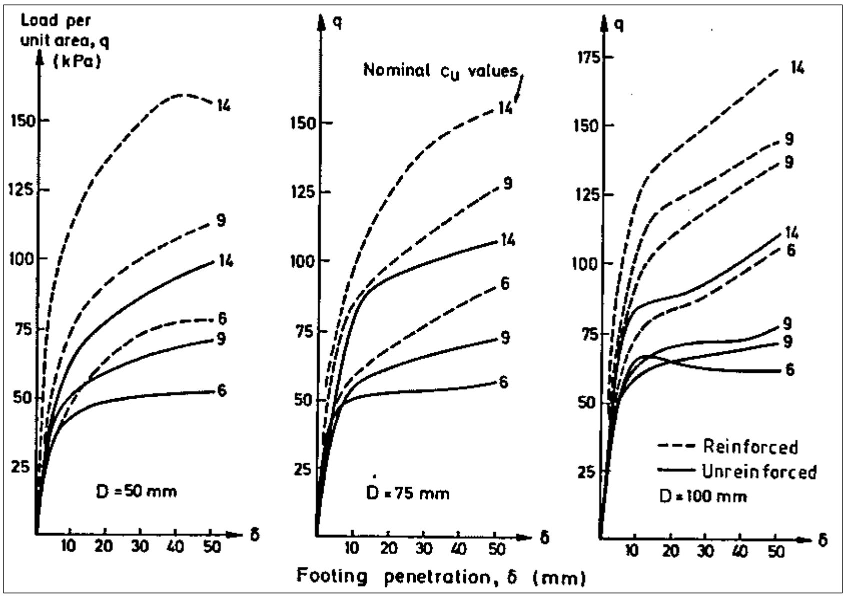

Figure 47: Graph of load per unit area versus footing penetration (Love et al. 1987).

Moayed and Nazari (2011) tested a clay overlain by sand to form a two-layered soil structure. The study was conducted to determine the effect of geosynthetic reinforcement on the improvement in the performance of a two layered soil, and also on the reduction in the thickness of subbase required due to inclusion of different types of geosynthetics in a multi-layered soil.

The underlying subgrade layer was a clay soil classified as CL in the Unified Soil Classification System (USCS), with an optimum moisture content and maximum dry density (MDD) of 16% and 19.20 kN/m^3 respectively. The material was prepared at 90% MDD and at optimum moisture content to give a CBR value of 11%. Overlying the clay soil was the subgrade course aggregate which was a sand classified as SW in the USCS, and had an MDD of 21.4 kN/m^3 at a water content of 9%. The properties of the geosynthetics of interest were the tensile strengths that ranged from 7 – 11 kN/m and the aperture size of the geogrid that was 10 mm by 10 mm. The diameter of the geosynthetics tested was 152.4 mm.

The bearing ratio tests were conducted at unsoaked conditions in accordance with ASTM D1883-05. The bearing ratio mould was a rigid metallic cylinder with an inside diameter of 152 mm and a height of 178 mm. The load was applied on the soils at a uniform rate of 1.2 mm/min. In all the tests, thickness of the compacted cohesive soil is maintained as 116 mm and thickness of the overlaying compacted sand varied as 40 mm, 55 mm, and 70 mm.

Results obtained from the tests conducted indicated an increase in the load-bearing capacity of the two-layered soil, however there was an observed reduction in the effect of the geosynthetic inclusion as the thickness of the subbase increased further. This was backed by the results obtained from the tests in which the thicknesses of the imported fill was varied; at which there was a considerable increase in the effect on bearing ratio for the 40 mm and 55 mm thicknesses, however, a reduced benefit for the 70 mm layer. On comparison of the results obtained, it was observed that the fill thickness of the fill layer could be reduced whilst keeping the load-bearing capacity constant as a result of inclusion of a geosynthetic layer at the interface of the soils. A reduction in fill thickness of 43% could be attained.

The study conducted by Ornek et al. (2012) was a large scale field test and involved natural clay overlain by granular fill material, with a geogrid at the sand-clay interface. The characteristics of the clay were determined from both field and laboratory tests. It was discovered from the boreholes drilled that the water table was at 2.20 m thus giving a degree of saturation of the clay layer, where the tests were conducted, of about 80%. The values of the undrained shear strengths were determined by unconfined compression tests in the range of 60 – 80 kN/m^2. The soil layers were classified as lightly overconsolidated soil (OCR=1–2.65) from odometer tests. From visual observation and the Unified Soil Classification System (USCS) the soil was described as silty clay with high plasticity (CH) that changed to silty clay with intrusion of sand (CL) with depth. The granular material had an internal friction angle of 43° and cohesive strength of 15 kN/m^2, and was prepared at optimum moisture content and maximum dry density of 7% and 21.7 kN/m^3.

In the case of the granular reinforced clay, as the thickness of the granular fill was increased there was a subsequent increase in the load-bearing capacity of the two-layered soil. The test results shown in Figure 48 indicate that the use of granular fill layers over natural clay soil has a considerable effect on the bearing capacity and the settlement characteristics. The max benefit was obtained at H/D = 1.00, where H is the fill thickness, and D is the width of the footing. The BCR values for D = 0.90 m obtained are 1.21, 1.35 and 1.44 for H = 0.33D; H = 0.67D and H = 1.00D, respectively.

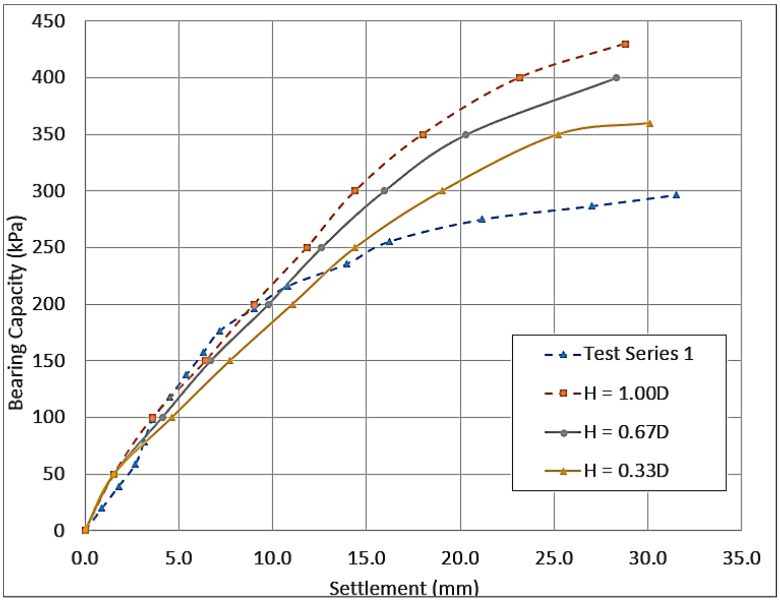

Figure 48: Bearing capacity versus settlement for granular reinforced clay using a 0.90 m footing diameter – Series 2 (Ornek et al. 2012).

From the studies analysed, the geosynthetic layer was placed at the interface of the soils. This is beneficial when the separation function of the geosynthetic in addition to the reinforcement benefits is intended to be mobilized. However, an alternative configuration that involves placement of the geosynthetic layer within the fill layer can be investigated to determine whether pertinent reinforcement benefits can still be obtained. In the case of geogrids, better anchorage of the geogrid could occur; better confinement of the soil particles; and a reduction in the stresses and strains transferred to the soft subgrade.

4.4. Width of geosynthetic

The effect of geosynthetic width is of importance when the lateral restraint mechanism of soil particles is considered. If the width is not adequate, then the particles would disperse outward with transfer of applied loads directly to underlying soil without mobilizing the full reinforcement benefit in the geosynthetic. As such the geosynthetic width should allow for full mobilization of reinforcement.

The study conducted by Hartley (2010) involved two footings of width 50 mm and 100 mm, and the widths of the geogrid tested were 1.5B; 2B; 3B; 4B; and 5B, where B is the width of the footing. Figures 49 (a) and 50 (a) show the comparison of the results of the stress applied on the sand against the displacement for different lengths of the reinforcement layer for the two footings, whereas Figures 49 (b) and 50 (b) show the comparison of the BCR against the displacement for the same results.

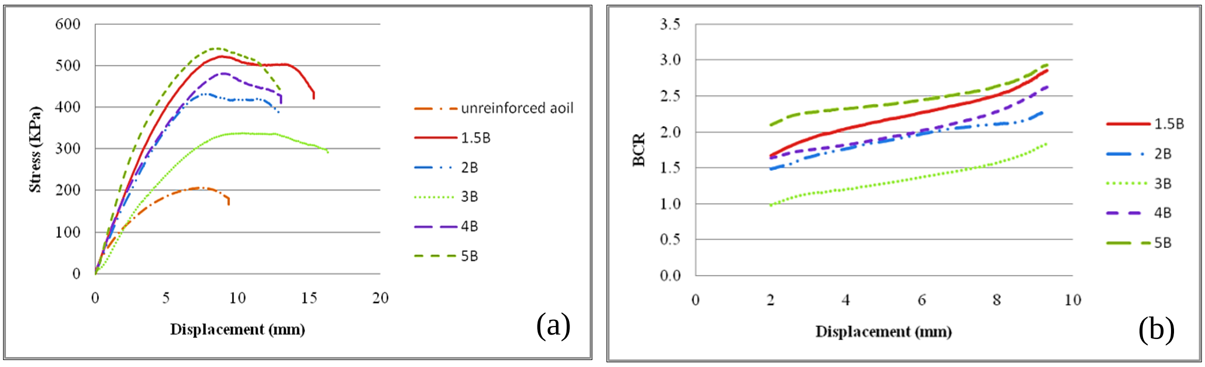

Figure 49: (a) Stress versus displacement, and (b) BCR versus displacement; for the 50 mm footing using different widths of geogrid at a depth of 0.5B (Hartley 2010).

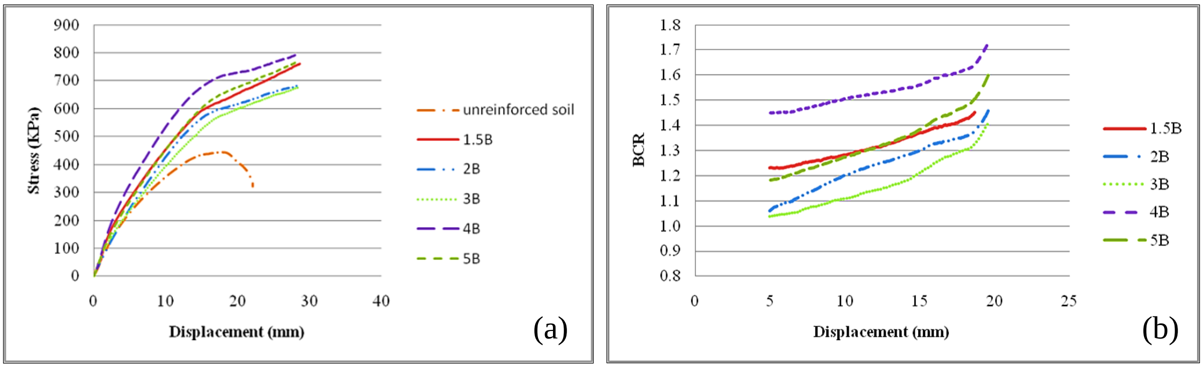

Figure 50: (a) Stress versus displacement, and (b) BCR versus displacement; for the 100 mm footing using different widths of geogrid at a depth of 0.5B (Hartley 2010).

From Figures 49 and 50 it was observed that as the width of the geogrid was increased, there was also a subsequent improvement in the stresses applied and the BCR. The improvement is however only up to the widths of 5B and 4B for the 50 mm and 100 mm footings respectively; after which further increases in the width of the geogrid provides no further increase in the BCR. The optimum geosynthetic widths to footing width ratio were 5B and 4B for the 50 mm and 100 mm footings respectively.

In the study by Oriokot (2012), the optimum width of the geogrid was determined by keeping the depth of placement constant while varying the width of the reinforcement. The widths of the geogrid were of the range 75 mm – 250 mm and 150 mm – 500 mm for the 140 mm and 200 mm footings respectively. The crushed greywacke aggregate used in the study was sourced from the Contermanskloof quarry in Cape Town and consisted of angular to sub angular particles with mechanical properties of particle density 1588 kg/m^3, and a particle size ranging from 0.075 – 9.5 mm, with a mean grain size of 7.1 mm. The triangular geogrid used was specially selected because its grids had a more stable structure and provided nearly uniform tensile resistance in all directions, and therefore more efficient in improving the performance of reinforced bases.

Figures 51 (a) and 51 (b) show the geogrid layer placed at varying depths in the crushed aggregate that ranged from 0 – 125 mm. The loading was applied by a Zwick Universal Compression and Tension machine, shown in Figure 51 (c), at a rate of 1.2 mm/s that led to response of the composite replicating undrained conditions. The loading box used was of dimensions 950 mm by 450 mm by 140 mm.

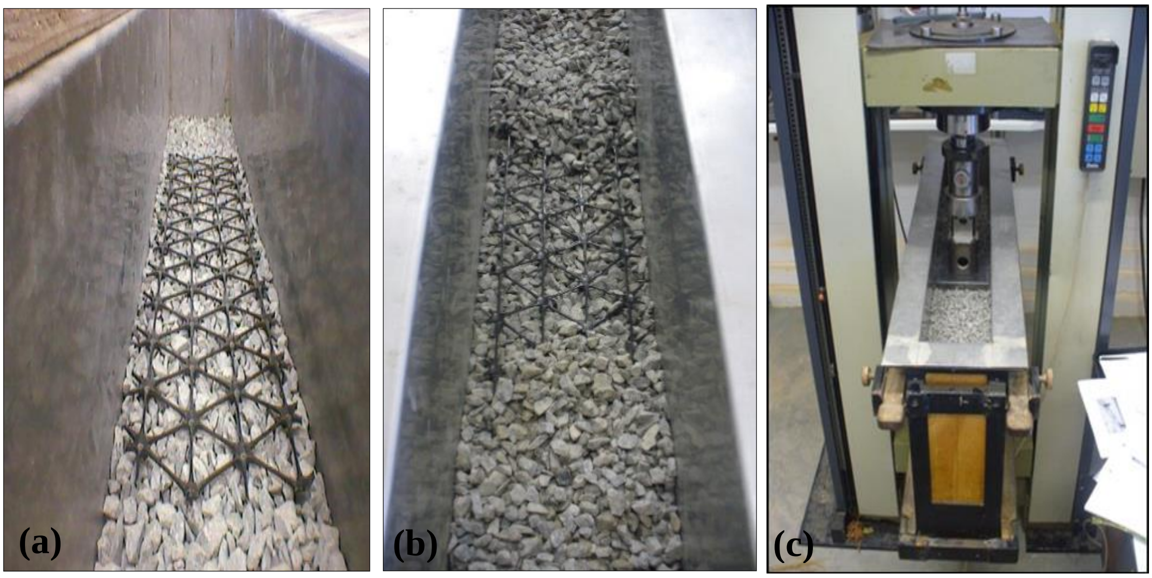

Figure 51: Experimental set-up showing geogrid placed within the aggregate layer (a) and (b); and with loading box and Zwick machine (c) (Oriokot, 2012).

In Figure 52 (a) it was observed that when the width of the geogrid increased there was an increase in the bearing capacity, as shown; but there was a width that was reached where there was not a subsequent increase in the bearing capacity, as shown in Figure 52 (b). From the graph, it was observed that the optimum widths of the geogrid were 200 mm and 150 mm at depths of 25 mm and 125 mm respectively. This corresponds to a range of 0.2B – 0.9B.

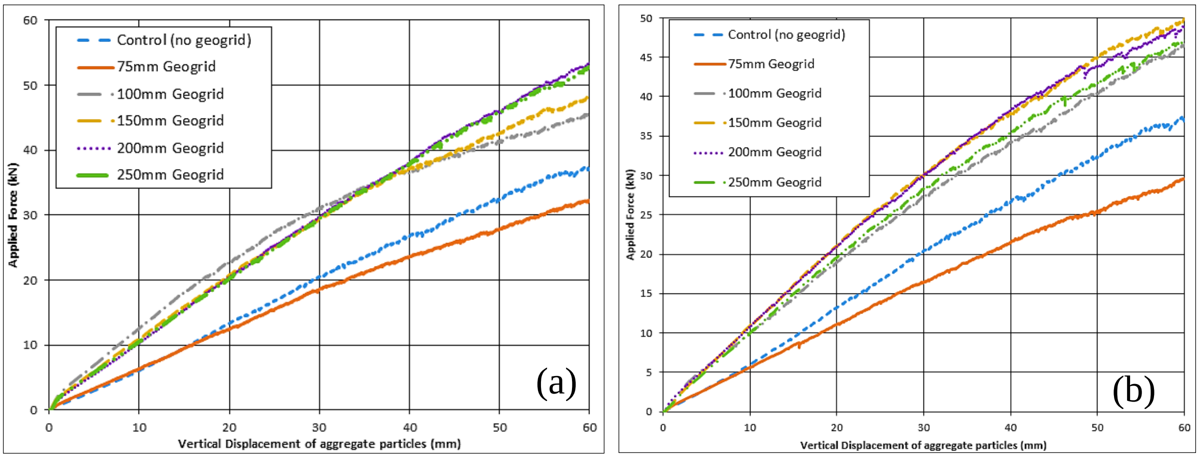

Figure 52: Applied force against vertical displacement for various widths of geogrid at a (a) 25mm depth for the 140mm footing, and (b) 125mm depth for the 140mm footing (Oriokot, 2012).

Mawer (2013) tested varying geotextile lengths of 1.5B; 2B; 3B; and 5B for a footing of width 150 mm in a clay soil. From the results, it was observed that the optimum of geotextile width was 5B, as shown in Figure 53.

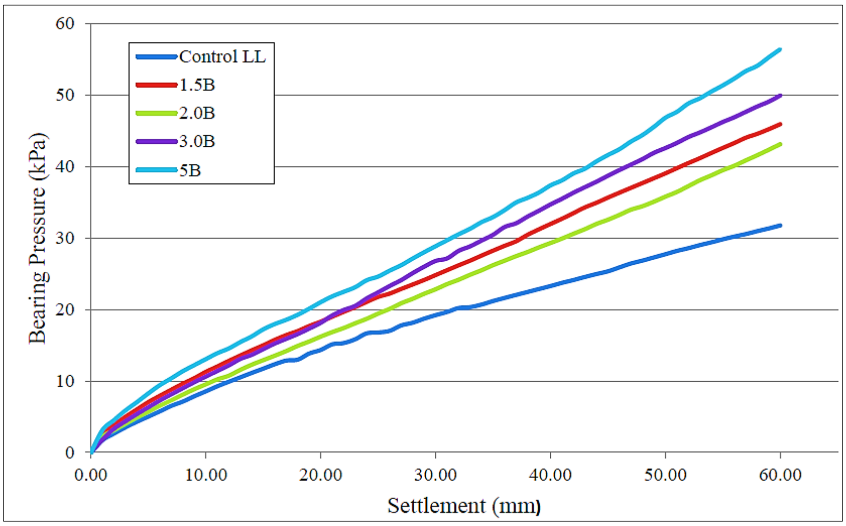

Figure 53: Bearing pressure vs settlement for different widths of geotextiles (Mawer 2013).

4.5. Depth of placement

The position of placement of a layer of geosynthetic in a soil structure is of upmost importance, as it determines how much of the reinforcement is provided by the geosynthetic, and also when it shall be mobilized. One of the mechanisms through which geosynthetics provide reinforcement is the tensioned membrane action. This is achieved only when considerable strain has undergone in the soil, such that the strains in the soils are transferred to the geosynthetic layer. The strain is usually transferred when a large vertical deformation (known as rutting in pavement structures) has occurred. As such the depth of placement is essential in mobilization of the reinforcement provided by the geosynthetic layer.

If the depth is too great, then adequate reinforcement will not be achieved, as observed in the study conducted by Guido et al. (1985) where beyond the depth equivalent to D = 1.0B there was reduced improvement in the load-bearing capacity. However, if the depth is too low, then the geosynthetic will not have the necessary anchorage in the soil and would instead warp and extrude through the ground surface. This corresponds to the findings by Gill et al. (2012), where the optimum depth of placement of the reinforcement is at the interface of the soils. The studies conducted aimed at determining the optimum depths of placement of the geosynthetic layer in the soil.

Hartley (2010) tested two footing sizes of 50 mm and 100 mm, with the corresponding depths of placement of 0B; 0.25B; 0.5B; and 1B, where B is the width of the footing. From the results obtained as shown in Figures 54 and 55, the improvement in the bearing capacity of the sand due to increase in depth of placement of the geogrid were observed. The optimum depths of placement for the 50 mm and 100 mm footings were obtained as 0.5B and 0.25B respectively. This is equivalent to a depth of 25 mm for both footings. Figures 54 (a) and 55 (a) show the comparison of the results of the stress applied on the sand against the displacement for different depths of placement of the reinforcement layer. Figures 54 (b) and 55 (b) show the comparison of the bearing capacity ratio (BCR) against the displacement for the same results.

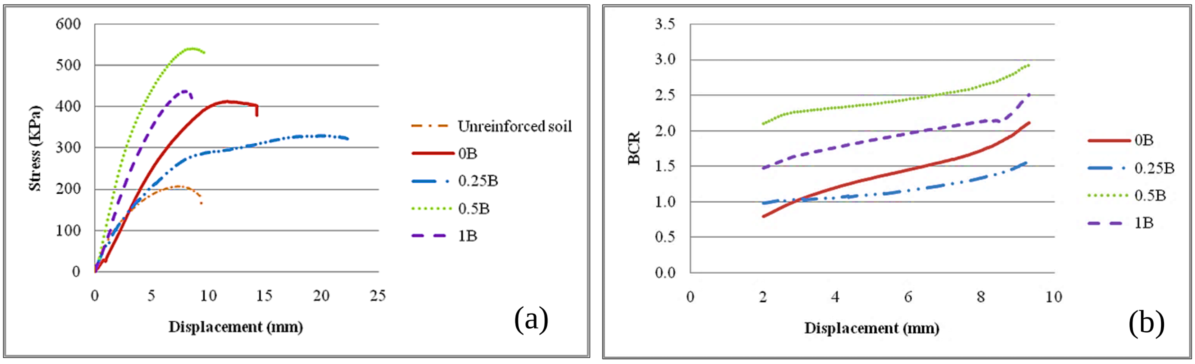

Figure 54: (a) Stress versus displacement, and (b) BCR versus displacement for the 50 mm footing using a 250 mm length of geogrid at different depths (Hartley 2010).

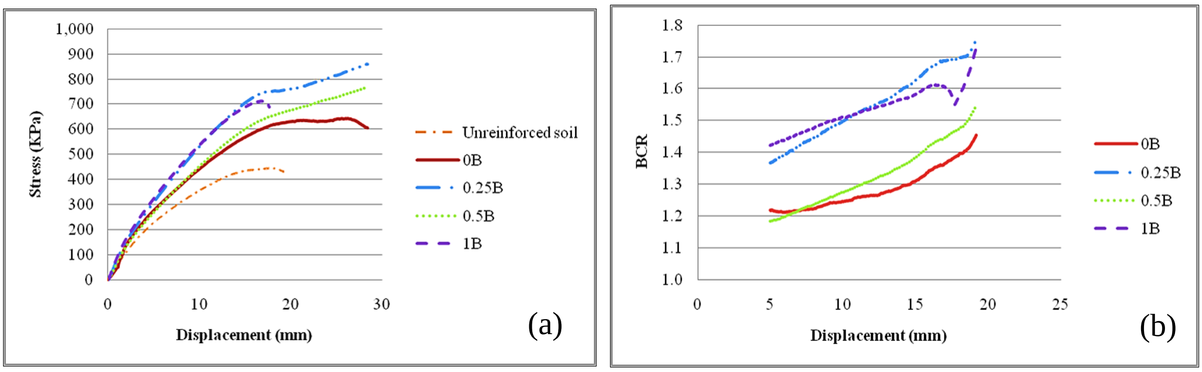

Figure 55: (a)Stress versus displacement, and (b) BCR versus displacement for the 100 mm footing using a 500 mm length of geogrid at different depths (Hartley 2010).

From Figures 54 and 55 it was observed that as the depth of placement increased, there was a subsequent improvement in the stresses and the BCR. However, this occurred up to depths of 0.5B and 0.25B for the 50 mm and 100 mm footings respectively, after which there was a reduction in the improvement as the depth of placement of the geogrid was increased.

In the study by Al-Qadi et al. (2012) the optimum location for installing a geogrid in pavements was determined. This was established by altering the depth of placement of the reinforcement layer in a pavement structure consisting of granular base overlying a soft subgrade. The geogrid was placed at the interface of the two soils with variations in the base layer thickness of 253 mm, 305 mm and 457 mm. Given that the tests were conducted at large scale, the subgrade thickness varied accordingly. From the results, it was observed that there was an increase in the performance with increasing thickness and depth of placement.

An increase in the base layer thickness by 50% led to a reduction in the stresses in the subgrade. This reduction improves the pavement performance and increases the design service life. Inclusion of geogrid-reinforcement at the interface of the base layer and the subgrade led to a further reduction in stresses in the subgrade, which indicated the effectiveness of geogrid-reinforcement. This research study showed that the inclusion of geogrid in the granular base layer reduced the deformation in both transverse and longitudinal directions.

Research conducted by Oriokot (2012) involved testing geogrid-reinforced crushed aggregate to determine the improvement in the bearing capacity, and also to measure the subsequent reduction in settlement of the footing. The triangular geogrid was similar to that used by Hartley (2010) and Buratovich (2011). Different sized footings were used of widths 140 mm and 200 mm, and the geogrid was placed at varying depths in the range 0 mm – 125 mm.

The results from the plate loading tests showed that as the depth of placement of the reinforcement layer increased so did the bearing capacity, as shown in Figure 56 (a) for a 140 mm footing and Figure 56 (b) for a 200 mm footing. From the observed results the optimum depth was at 125 mm which gave depth to width ratios of 0.9B and 0.6B for the 140 mm and 200 mm footings respectively.

In conclusion, as seen in the results obtained, it was discovered that the inclusion of a geogrid layer at subsequent depths had an incremental improvement in the BCR of the sand. As such, it was confirmed that the inclusion of a geogrid layer in the Klipheuwel sand led to an improvement of the load bearing capacity and a subsequent reduction in the settlement of the footing in the sand.

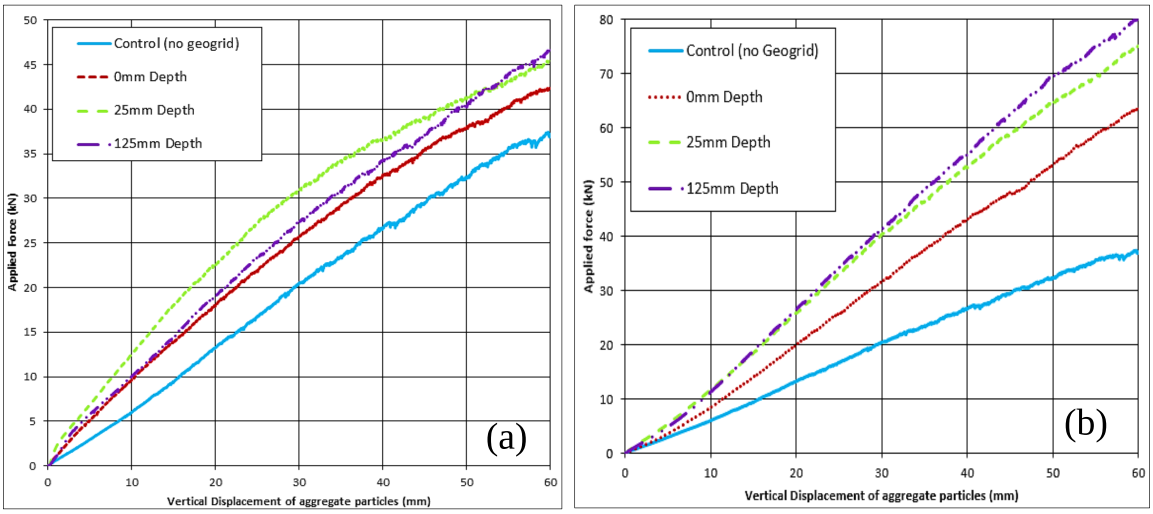

Figure 56: Graph of applied force against vertical displacement for (a) 100mm width geogrid at various depths for the 140 mm footing, and (b) 200mm length geogrid at various depths for the 200 mm footing (Oriokot, 2012).

In the study conducted by Mawer (2013), though the testing methodology was similar, a footing size of width 150 mm was used, and the soil material was Durbanville clay. A non-woven geotextile was included in the clay as the reinforcement layer. The depths of placement of the geotextile corresponded to the footing width, and were 0.25B; 0.5B; 0.75B and 1B. The results are presented in Figure 57, showing that the optimum depth of placement for a non-woven geotextile was 0.25B.

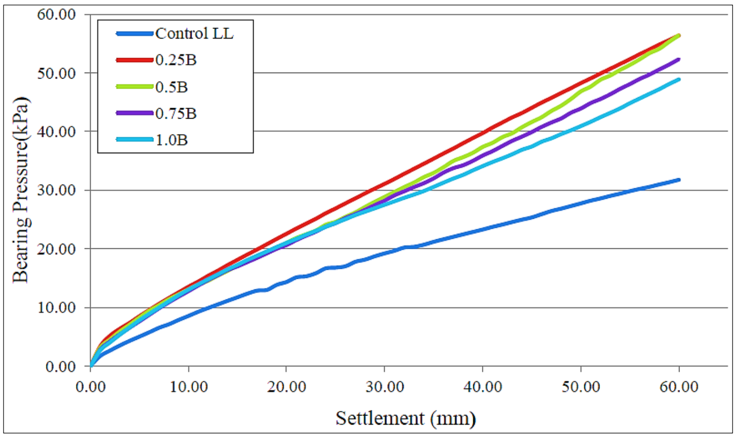

Figure 57: Graph of bearing pressure vs settlement for different depths of placement of geotextiles (Mawer 2013).

The obtained optimum depths of placement of the geosynthetic layer differed according to the soil material, the type of geosynthetic product tested and the width of the footing. Therefore it is necessary to determine the optimum depth of reinforcement when different materials and configurations are tested, to attain the maximum improvement from geosynthetic inclusion.

4.6. Comparison of geosynthetic products

Geosynthetics are manufactured differently using varying materials, and as a result the products have different properties. When applied as reinforcement layers they each provide different performance benefits. It is thus a necessity to compare the products in similar configurations to determine which would provide the most desirable reinforcement benefits.

In the study conducted by Chen (2007) and Chen et al. (2009), 7 different geosynthetic products were compared. It was observed that the geogrid with the highest tensile modulus performed best. This was attributed to the greater resistance to tensile strains and lateral spread of particles. In addition, geogrids have apertures that enable interlocking of particles to form a composite structure with higher resistance to applied stresses and strains.

Hartley (2010) compared the performance of a geogrid to that of a geotextile, and Figure 58 shows the obtained results when tested using the 100 mm model footing, and a reinforcement length of 200 mm, at a depth equivalent to 0.5B. The reinforcement materials used were a triangular geogrid and a RockGrid geotextile. It was observed that the stiffer RockGrid had greater improvement than the geogrid, and this is attributed to the RockGrid having a higher tensile strength thus having the ability to support greater stresses applied on the soil.

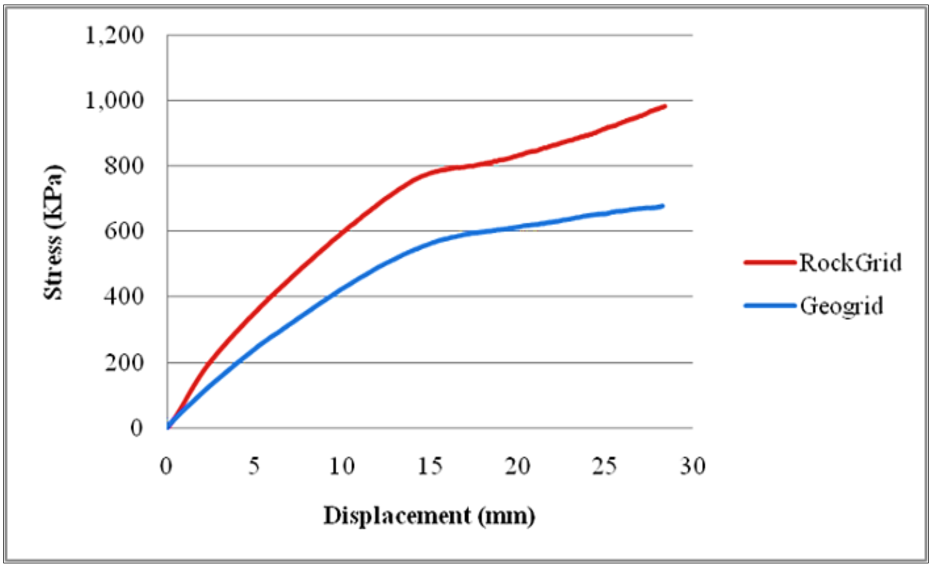

Figure 58: Stress versus displacement graph comparing the geogrid to the RockGrid for the 100mm model footing using a 200mm length of geosynthetic at a depth of 0.5B (Hartley 2010).

Moayed and Nazari (2011) compared the reinforcement benefits of a geotextile and a geogrid, with the graphs presented in Figures 59 – 61 showing that geogrids performed better than geotextiles. This is attributed to geogrids having higher tensile strength than the geotextiles, and in addition able to confine the soil particles in their apertures forming a more stable composite structure.

At 40 mm fill thickness there is an improvement due to geogrid and geotextile inclusion, with the geogrid performing better that the geotextile. The observed respective improvements were approximately 50% and 40% above the unreinforced case. At 55 mm fill thickness, there is an increase in the improvement in both geogrid and geotextile of approximately 30% and 25% respectively. At 70 mm fill thickness the benefits from the geotextile and geogrid subsequently reduce, with the improvement from both materials approximately similar at 10%. This could be attributed to failure occurring within the fill before the reinforcement from the geosynthetics could be mobilized at the interface.

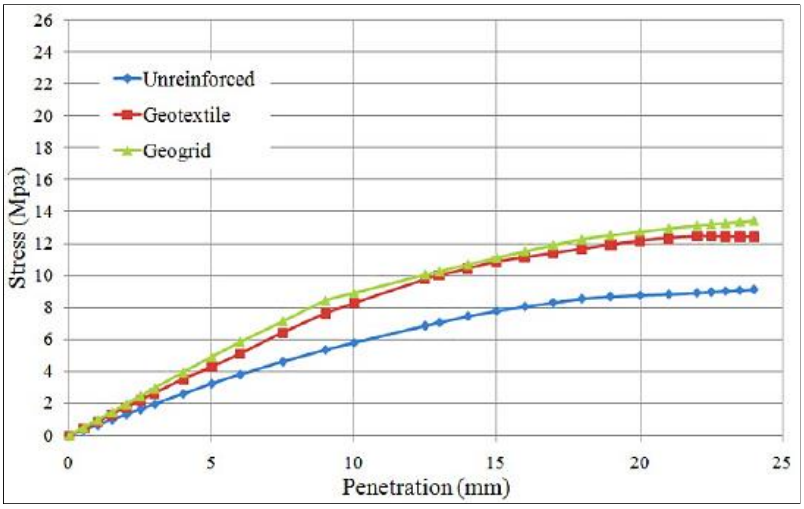

Figure 59: Graph of stress against penetration for the 40 mm thickness layer (Moayed and Nazari 2011).

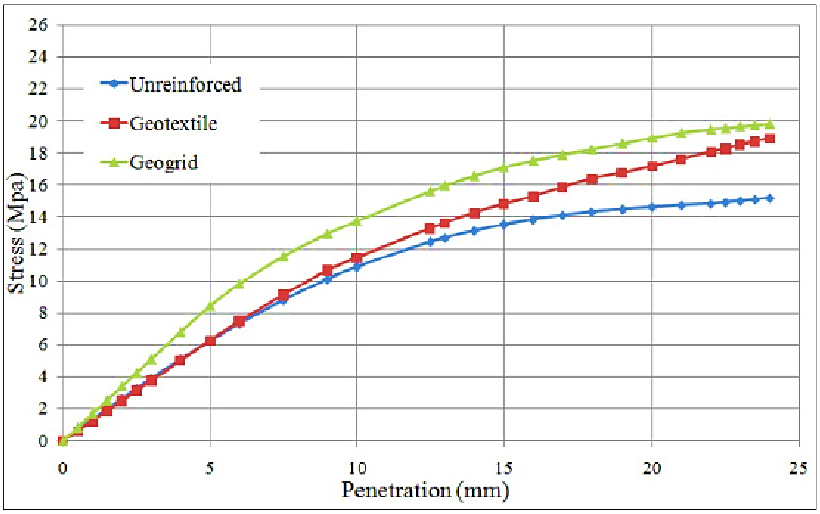

Figure 60: Graph of stress against penetration for the 55 mm thickness layer (Moayed and Nazari 2011).

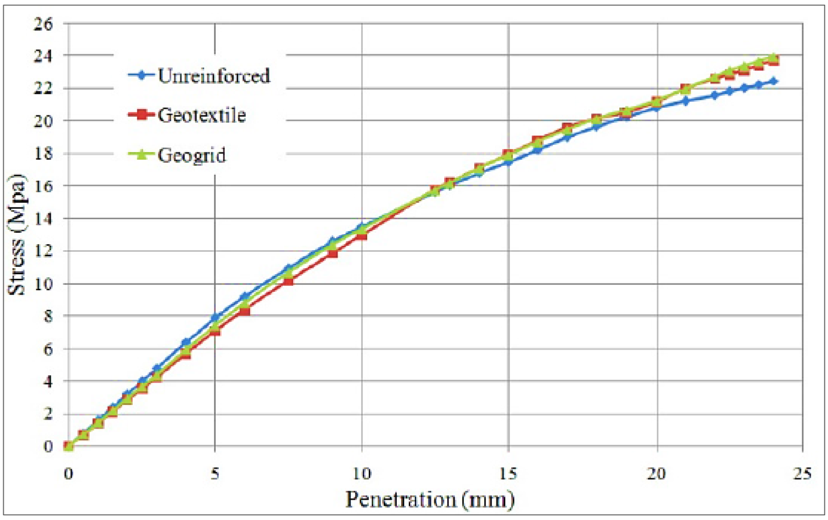

Figure 61: Graph of stress against penetration for the 70 mm thickness layer (Moayed and Nazari 2011)

4.7. Size of model footing

Regarding the model footing size, it was observed that an increase in the width led to contrasting results. In certain studies there was a subsequent increase in the load-bearing capacity, whereas in others there was an observed reduction in improvement. The increase could be as a result of a reduction in the stresses on the underlying soil, and a larger spread of the applied load; which would allow for greater exerted loads. While the reduced improvement could be associated with an increase in footing weight which leads to an increased settlement.

In the study conducted by Chen (2007), when the results from the 152 mm by 152 mm and 152 mm by 254 mm model footings were compared it was observed that there was a reduction in the bearing capacity with an increase in the footing size. This trend was observed in both the unreinforced and geosynthetic-reinforced set-ups, which is consistent with the bearing capacity formula suggested by Vesic (1973).

The study conducted by Ornek et al (2012) involved testing 7 diameter sizes in the range of 0.06 m - 0.9 m. The results of the tests conducted on the natural clay alone with the different footing diameters is presented in Figure 62, from which it was observed that as the footing diameter increased there was a reduction in the bearing capacity of the natural clay.

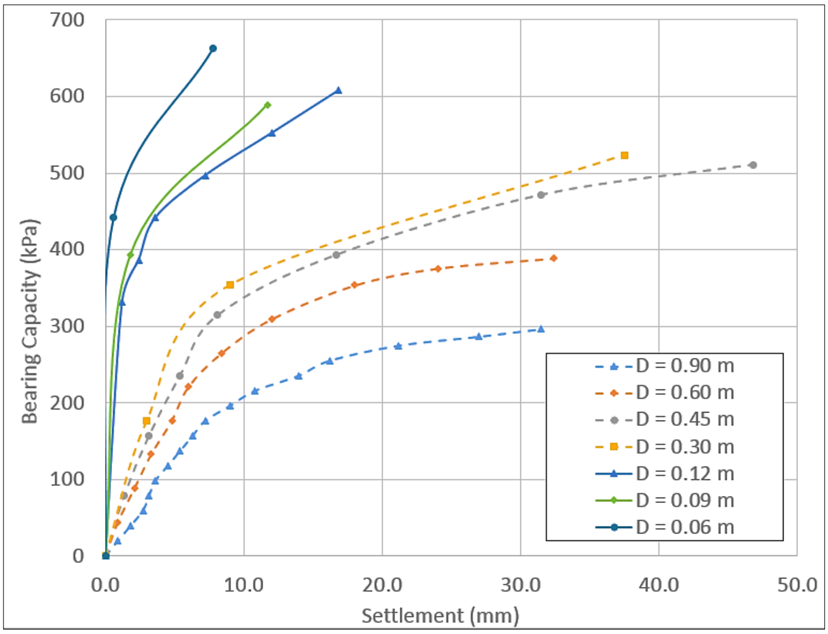

Figure 62:Bearing capacity verses settlement for unreinforced natural clay using varying footing diameters – Series 1 (Ornek et al. 2012).

Oriokot (2012) conducted tests on model footings of widths 140 mm and 200 mm, and it was observed that there was an improvement in the load with increasing footing width. The results obtained from the studies showed that an increase in the model footing width led to an increase in the load-bearing capacity of the soil. This is attributed to a greater distribution of the load applied to a larger area, which allows for more resistance from the soil particles.

Given the varying trends with increase in footing size with respect to the load-bearing capacity, it was necessary to conduct more tests with varying footing widths.

4.8. Summary from the review of previous research.

From this review it is evident that although many authors have investigated the inclusion of geosynthetics in soil structures, there are variations in the results with respect to changes in the parameters. These include: depth of placement; width of reinforcement; type or stiffness of reinforcement; size of model footing.

Overall it has been demonstrated that the inclusion of geosynthetics is applicable to soils ranging from soft soils (clay, silt); coarse grained (sand, aggregates) to multi-layered soils. A summary of these studies is presented in Table 7 showing the major findings from each.

According to Moayed and Nazari (2011), the combination of granular material (good in compression however poor in tension) and the inclusion of geosynthetics (poor in compression though good in tension) has led to an improved application in pavement design as a result of the reduced necessity for imported higher strength soils. This allows for the use of fill material of reduced strength to improve the strength of soft subgrade. This was taken into consideration as this research was conducted, and relatively low strength granular material was tested in a geosynthetic-reinforced two-layered composite structure to determine the reinforcement benefits.

Love et al. (1987) stated that for the shear stress on the soil particles to be transferred to the geosynthetic reinforcement layer, “the granular fill must interact efficiently with the reinforcement” to form a composite layer, and the grid must be well embedded within the fill that acts to interlock with the grid structure of the geosynthetic reinforcement. There was also the possibility of the fines in the subgrade extruding through the grid, thus breaking the bond between the grid and the fill. Geotextiles are thus more appropriate for separation, though this phenomenon could be overcome if adequate compaction is applied during construction. Taking that into consideration, geotextiles and geogrids were tested in this research to compare the performance benefits.

For the desired effect to be obtained, it was identified that the geosynthetic reinforcement layer should have adequate stiffness and strength to take up the tension induced by shear stress from the granular fill and subgrade, without failure occurring. The use of geogrids in reinforcement of multi-layered soils consisting of clay subgrade and granular fill would be preferable as they have higher stiffness than geotextiles. According to the authors, the granular fill forms a stable base on which construction can be carried out with larger loads exerted on the structure. In addition the geosynthetic placed at the interface acts to separate the different soil types as well as improving the load-bearing capacity due to geosynthetic reinforcement action.

The application of geosynthetics in the studies by Love et al. (1987); Ornek et al. (2012) and Moayed and Nazari (2011) was only at the interface of the soils. Although this is beneficial in providing separation of the different materials, it is worth analysing the effect of placing a geosynthetic layer within the fill layer. Therefore a different experimental configuration was investigated that involved placement of a geosynthetic layer at varying depths within the fill layer. This was compared to tests conducted with the reinforcement at the interface. In addition, the optimum depth of placement of the geosynthetic layer within the fill layer was determined.

According to Gill et al. (2012), the thickness of fill should be substantial such that the minimum thickness of fill is not less than 50 mm. This allows for sufficient anchorage of the geosynthetic layer and confinement of the soil particles. However, the fill thickness should not exceed the zone of influence, otherwise the reinforcement would not be mobilized. This is a necessity in the configurations that involve the geosynthetic at the interface of the soils. Thus there is a necessity to carry out a study to determine the optimum thickness of the granular material when geosynthetic reinforcement is provided in a multi-layered soil.

In addition, the work carried out by previous authors has shown that geosynthetics incorporated in multi-layered soils could lead to multiple benefits that include; an increase in the load-bearing capacity, a reduction in settlement, and a reduction in the amount of granular fill necessary to stabilize the subgrade soils. As a result the potential reduction in fill thickness was investigated in this research.

Table 7: Summary of previous studies.

Studies	Experimental set-up	Model Footing Size	Geosynthetic material (manufacturer)	Layer Thickness (mm)		Layer Material Description		Major Findings/Benefits
				Subbase/Base	Subgrade	Subbase/Base	Subgrade	
Guido et al. (1985)	Square Plexiglas box; 1.22 m by 0.92 m (height)	310 mm (square footing)	Geotextile	-	Not provided	-	Sand (Uniformly graded)	BCR = 1.6 – 2.8 Improvements in bearing capacity negligible when D > 1.0B
Love et al. (1987)	Rectangular box; 1000 mm by 300 mm by 600 mm	75 mm (width)	Tensar SS Geogrid (Netlon)	50, 75, 100	400	Leighton Buzzard sand and gravel mixture	Kaolin clay 6, 9, 15 kPa (CBR < 3%)	Geogrid reinforcement reduces the shear stresses transmitted to the surface of the clay subgrade.
Ramaswamy and Purushothaman (1992)	Not provided	40 mm (diameter)	Geogrid	-	Not provided	-	Clay (CL)	BCR = 1.15 – 1.7 (for N = 1 – 3) Bearing capacity reduced with increase in moisture content
Mandal and Sah. (1992)	Steel box; 460 mm (length, width and height)	100 mm (square footing)	Geogrid	-	Not provided	-	Clay (CL)	Maximum BCR = 1.36 at u/B = 0.075 Minimum settlement at u/B = 0.25 Reduction in settlement up to 45%
Yetimoglu et al. (1994)	Steel tank; 0.7 m (length and width), 1.0 m height	101.5 mm (width) by 127 mm (length)	Uniaxial Geogrid Terragrid GS1000 (Turkey)	Not provided	-	Yalikoy quartz river sand	-	Optimum depth of placement was 0.3B for single a layer and 0.25B for multi-layers Optimum number of layers = 4 Optimum geogrid spacing = 0.2B

Adams and Collins (1997)	Test pit; 5.4 m (width) by 6.9 m (length) by 6 m (depth)	300 mm; 460 mm; 610 mm; 910 mm	Biaxial Geogrid and Geocell	Not provided	-	Fine concrete mortar sand (SP)	-	Maximum BCR at depth = 0.25B Optimum number of layers = 3
Dash et al. (2003)	Test tank; 0.9 m by 0.9 m by 0.6 m (height)	20 mm (width and length)	Geocell (formed from a Biaxial Geogrid)	0.42D; 0.84D; 1.26D; 1.68D; 2.10D; 2.52D (D is footing width)	Not provided	Sand (SP)	Silty Clay (CL)	BCR = 1.06 – 6.06 Optimum width = 5D Optimum geocell height = 2.1D
Patra et al. (2005)	0.8 m (length), 0.365 m (width), 0.7 m (height)	80 mm (width) by 360 mm (length)	Uniaxial Geogrid	Not provided	-	Sand	-	BCR increases with depth of placement
Chen (2007)	Steel box; 1500 mm by 910 mm (width and height)	152 mm and 254 mm (widths)	Geogrids: Mirafi BasXgrid11 Tensar BX6100 Tensar BX6200 Tensar BX1100 Tensar BX1200 Tensar BX1500 Tenax MS330 Mirafi Miragrid 8XT Mirafi HP570 Geotextile Steel Wire Mesh Steel Bar Mesh	-	Not provided	-	Silty Clay (CL)	BCR = 1.20 – 1.81
				Not provided	-	Sand (SP)	-	BCR = 1.01 – 3.86
				Not provided	-	Kentucky crushed limestone (GW)	-	BCR = 1.03 – 2.85
	Large scale tests	457 mm (width)	Geogrid	-	On site (varies)	-	Silty Clay (CL)	BCR = 1.18 – 1.48

Basudhar et al. (2007)	Square tank; 0.44 m by 0.44m by 0.21 m (height)	30 mm; 45 mm; 60 mm (diameter)	Woven Geotextile	210	-	Ganga sand	-	Percentage increase in BCR of 150.0% - 456.3% Percentage reduction in settlement of 59.1% - 80.0%
El Sawwaf (2007)	Tank; 1.00 m by 0.50 m by 0.50 m (height)	75 mm (width) and 498 mm (length)	Tenax TT Samp Geogrid	425	Not provided	Medium to coarse sand	Clay (CL)	Optimum depth of geogrid = 0.6B Optimum width of geogrid = 5B Optimum number of layers = 3 Optimum geogrid spacing = 0.5B 50% reduction in sand thickness when 3 geogrid layers are used instead of 1
Ranadive and Jadhav (2010)	Rectangular box; 600 mm by 110 mm by 500 mm (height)	100 mm (width)	Non-woven Geotextile (Garware Wall-Ropes)	-	450	-	Not provided	Optimum number of layers = 4 Optimum geotextile spacing = 0.25B BCR = 1.27 – 1.65
Hartley (2010)	Rectangular box; 900 mm by 500 mm by 140 mm	50 mm and 100 mm (widths)	Tensar TriAx TX160 Geogrid and Rockgrid PC Geotextile (Kaytech)	400	-	Sand (Klipheuwel)	-	Optimum depth of placement = 25 mm (0.25B for 50 mm; 0.5B for 100 mm) Optimum width of geogrid; 5B for 50 mm; 4B for 100 mm Bearing capacity increase of 100%
El Sawwaf and Nazir (2010)	Soil bin; 2.0m by 0.6m by 0.6m (height)	80 mm (width) by 120 mm (length)	Tenax TT Samp Geogrid	500	-	Medium silica sand	-	Optimum width of geogrid = 5B Optimum number of geogrids = 3
Moayed and Nazari (2011)	Cylindrical mould; 152 mm diameter, 178 mm height	150 mm (diameter)	Geotextile and Geogrid	40, 55, 70	116	Sand SW	Clay (CL) CBR = 11%	BCR = 0.9 – 1.6
Buratovich (2011)	Rectangular box; 900 mm by 500 mm by 140 mm	100 mm (width)	Tensar TriAx TX160 Geogrid (Kaytech)	350	-	Sand (Klipheuwel) GW	-	Optimum number of layers = 3 Optimum geogrid spacing = 0.5B Load-bearing capacity increase of 20 – 240% Settlement reduction of 30 – 45%

Ornek et al. (2012)	Large scale tests	7 diameters (0.06 m – 0.90 m)	-	0.33D; 0.67D; 1.00D (D is footing width)	On site (varies)	Granular material	Natural clay (CH, CL)	Optimum fill thickness = 1.00D BCR = 1.21; 1.35; 1.44 for thicknesses 0.33D, 0.67D, 1.00D respectively.
Al-Qadi et al. (2012)	Large scale tests	Dual-tire assembly	Biaxial Geogrid	203; 305; 457	On site (varies)	Crushed limestone aggregate	CBR = 4%	Improvements of 22%; 28%; 45% Rutting reductions of 20%; 25%; 43%
Oriokot (2012)	Rectangular box; 900 mm by 500 mm by 140 mm	140 mm and 200 mm (widths)	Tensar TriAx TX160 Geogrid (Kaytech)	400	-	Crushed greywacke aggregate	-	Optimum depth of geogrid = 0.6 - 0.9B Optimum width of geogrid = 1.4 - 2.5B
Mawer (2013)	Rectangular box; 900 mm by 500 mm by 140 mm	150 mm (width)	BIDIM A7 Non-woven Geotextile (Kaytech)	-	400	-	Durbanville clay	Optimum geotextile depth of 0.25B – 0.3B Optimum length = 5B Optimum number of geotextiles = 3 Optimum spacing of geotextile of 0.75B – 1.0B

CHAPTER 5

5. Methodology

5.1. Introduction

This chapter details the methodology followed to achieve the necessary results for the study. The chapter begins by describing the research materials used, followed by the method by which the materials were prepared. Thereafter, the testing schedule and the test procedure followed for the bearing capacity tests are described.

5.2. Research materials

5.2.1. Soil Materials

5.2.1.1. Granular Material

The granular material used in all reinforced experiments was sourced from Contermanskloof quarry; Contermanskloof Road, Durbanville Hills, Western Cape, South Africa, and was supplied by AfriSam Western Cape, Regional Services Centre, Tannery Park, 21 Belmont Road, Rondebosch, 7700.

The material consisted of grey, angular to sub-angular particles; and had a medium dense consistency as observed when excavated from the quarry. The percentage passing the 0.075 mm sieve was approximately 25%, and the relative density of the material was 0.94. The material had a CBR of approximately 15% at 93% Mod. AASHTO; and a penetration rate ranging from 9.1 to 13.99 mm/blow was achieved from DCP tests conducted on the material in-situ. A plasticity index (PI) of approximately 11.00 was achieved for the material. The optimum moisture content of the granular material was 4.9 % with a maximum dry density of 2240 kg/m^3 as provided by the supplier.

According to materials classification methodology described by Jooste et al. (2007), which is based on the TRH14 (1996) and SAPEM (2013d), the material that was used in the tests was categorized as a G7. This granular material is used as selected fill in the subbase layer in pavement structures and road embankments according to the South African Pavement Engineering Manual (SAPEM, 2013d). As such it was used in this study to represent the fill layer.

The sieve analysis conducted on the granular material followed the testing standard presented in ASTM D6913 and is shown in Figure 63. According to the USCS, the material is classified as a well graded gravel, GW.

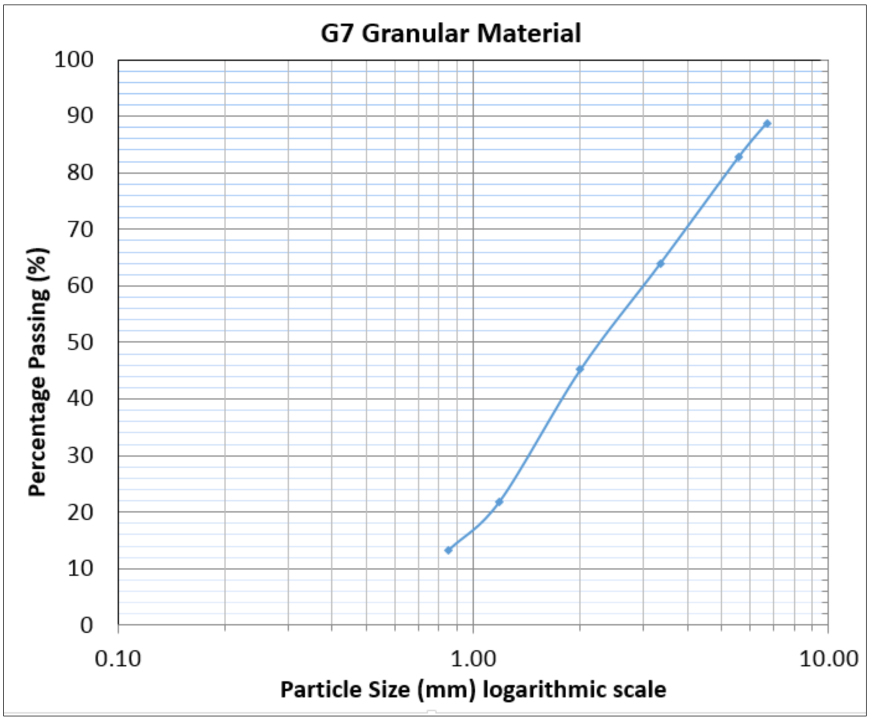

Figure 63: Particle Size Distribution for G7 Granular Material

5.2.1.2. Clay material

The clay used in all the experiments was kaolin that represented the subgrade material. This clay was specifically chosen due to the fact that its uniform mechanical and chemical properties presented great opportunity for experimentation and enhancement, and was easy to work with.

Kaolin is a good representation of soft soils in South Africa that pose a problem on construction sites. The native kaolin ore is a lamellar aluminium phyllosilicate which may be found naturally in the Eastern seaboard of South Africa, with localised deposits also around Western Cape, however, it is also located in low-lying flood plains and estuaries inland.

The kaolin clay was acquired from Serina Trading; 16 Sea Cottage Drive, Noordhoek, 7979, Cape Town, South Africa; delivered in 25 kg lined HDPE bags and stored in the Geotechnical laboratory store. The type of kaolin used was an off-white 2-micron water-washed hydrous kaolin powder that is suitable as a filler or extender in applications where whiteness is not required, and was graded as an HB powder hydrous kaolin (china clay). The refining process for the clay included water washing; hydrocyclone beneficiation and screening followed by mechanical-thermal drying.

The clay had a specific gravity of 2.6, moisture content of 4% and pH value ranging 7 – 8. The mineralogical composition consisted of kaolinite with trace mica and quartz. The HB powder had typical physical properties and the chemical analysis as presented in Tables 8 and 9.

Table 8: Physical properties of the kaolin clay

Physical Properties	Value
Abrasiveness (Einlehner tester)	64 g/m2
Particle size distribution	87% (< 10 micron)
	20% (< 2 micron)
Mean particle size (D50)	1.1 micron
Residue (> 45 micron)	1.5%
Reflectance	75% (off-white in colour)
pH value	7 – 8
Specific gravity of kaolin mineral	2.60
Mohs hardness	2.0 – 2.5
Moisture content	4%
Oil absorption (linseed oil)	45 mg/100g

Table 9: Chemical analysis of the kaolin clay

Chemical Analysis	Value
SiO_2	47.32%
Al_2O_3	36.52%
Fe_2O_3	0.56%
TiO_2	0.82%
CaO	0.31%
MgO	0.18%
$Na_2O + K_2O$	0.92%
L.O.I	13.02%

Classification tests

The classification tests conducted were in accordance with the standards. The necessity of the tests were to classify the soil using the Universal Soil Classification System (USCS) and to obtain important parameters used in the bearing capacity tests and analysis of the results. The different tests that were conducted are presented in Table 10.

Table 10: Classification tests conducted on the kaolin clay

Test performed	Standard	Method of testing	Reason for Test
Atterberg Limit Tests			
Liquid Limit Test	BS 1377: Part 2: 1990: 4.5	Cassagrande method	Used to obtain the plastic and liquid limits of the soil sample. This was vital in deciding the moisture contents that would be used during the testing procedures.
Plastic Limit Test	BS 1377: Part 2: 1990: 5.3	Plastic limit method	
Specific Gravity Test	BS 1377: Part 2: 1990: 8.3	Density bottle (small pyknometer)	Determines the density or unit weight of the soil for use in calculations.
Compaction Test	BS 1377: Part 2: 1990: 3.3	Proctor method	Calculates the optimum moisture content of the soil, which is used as a variable during the testing phase.
Direct Shear Test	ASTM D 3080	Shear box method	To calculate critical soil properties such as the shear strength, angle of internal friction as well as the cohesion values of the soil. These are critical for the calculation of theoretical bearing capacity.

Table 11 shows the results obtained from the classification tests for the kaolin clay soil, and from the Universal Soil Classification System the clay is graded as a lean clay with low plasticity, CL.

Table 11: Results from the classification tests for clay soil.

Property	Units	Value
Specific Gravity, G_s	-	2.6
Natural moisture content	%	0.5
Angle of internal friction, φ	Degrees	17.5
Cohesion, C_c	kPa	4.8
Liquid Limit, LL	%	37.0
Plastic Limit, PL	%	25.5
Plasticity Index, PI	-	11.5
Optimum Moisture Content, OMC	%	22.0
Dry Density, γ_d	kg/m³	1580

5.2.2. Geosynthetic Reinforcing Material

Four different geosynthetics were used in this study; woven geogrids, extruded geogrids, woven geotextiles and non-woven geotextiles. The geogrids and woven geotextile are conventional reinforcing geosynthetics. The nonwoven geotextile is mainly used for separation, but was evaluated in this study to assess whether considerable reinforcement could also be realized.

The geosynthetic materials were sourced from Maccaferri, Southern Africa, 24 Estmil Road, Diep River P.O. Box 22150, Fish Hoek, 7974. The details of each of the products tested are presented in the sections that follow, and the basic characteristics and mechanical properties of the geosynthetics are summarized in Appendix II, as provided by the supplier.

5.2.2.1. Extruded Geogrid (EGG)

The extruded geogrid used in this study was a MacGrid EG 40S. It was black in colour with a smooth surface, and had square apertures of size 38 mm by 38 mm. The material is shown in Figure 64 (left). MacGrid EG is a high modulus polypropylene bi-axial geogrid, produced by an extrusion process characterized by a tensile resistance both in the longitudinal and in the transverse direction. They are inert to all chemicals existing in natural soils 4 < pH < 9. These geogrids are mainly used for mechanical soil stabilization and for some kinds of soil reinforcement applications.

5.2.2.2. Woven Geogrid (WGG)

The woven geogrid that was used in this study was a MacGrid WG 8S. The MacGrid was black in colour, flexible material with a relatively rough surface, and had an aperture size of 25 mm by 25 mm. The material is shown in Figure 64 (right). MacGrid WG 8S is a bidirectional geogrid used for mechanical ground stabilization and basal reinforcement, made from high molecular weight, high tenacity polyester multifilament yarns. The yarns are woven on tension in machine direction and finished with a polymeric coating. These geogrids are engineered to be mechanically and chemically durable, and resistant to biological degradation.

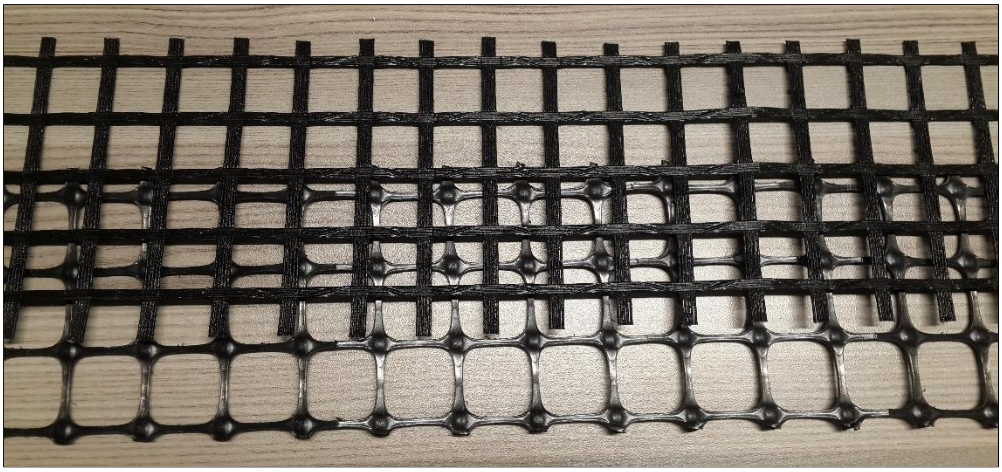

Figure 64: (left) Extruded Geogrid (EGG) and (right) Woven Geogrid (WGG)

5.2.2.3. Woven Geotextile (WGT)

The woven geotextile used in the study was a MacTex W1 8S. The woven geotextile was black in colour and flexible. MacTex W1 geotextiles are planar woven structures manufactured through weaving in the warp and the weft directions with polypropylene tapes. The material had a tensile strength of 83 kN/m and an opening pore size of 120 micro metres, which meant it was semi-permeable. The material is shown in Figure 65 (left).

5.2.2.4. Non-woven Geotextile (NGT)

The non-woven geotextile used in the study was a MacTex H40.1. The MacTex was a needle-punched & thermocalendered polypropylene nonwoven geotextile. It was grey in colour, flexible and had a thickness of 0.9 mm. The material had a tensile strength of 15 kN/m and an opening pore size of 75 micro metres, and therefore it was semi-permeable. The material is shown in Figure 65 (right).

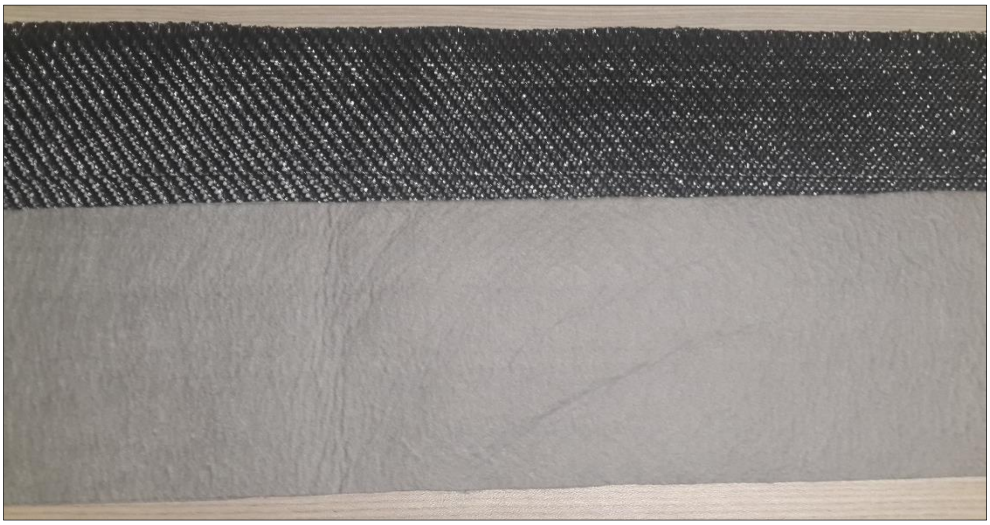

Figure 65: (left) Woven Geotextile (WGT) and (right) Non-woven Geotextile (NGT)

Tensile tests

Relative extensibility and tensile strengths of the geosynthetics alone (not in soil) was characterized using wide-width tensile tests conducted in accordance with ASTM D 4595. The tensile tests were conducted on the Zwick Universal Compression and Tension machine at a rate of 25 mm/min on the materials of dimensions 280 mm by 50 mm by 5 mm, with thicknesses dependent on the material. The dimensions chosen were such that the materials were able to be clamped, with the clamp width of 50 mm, and the length was selected such that adequate tensile strain could be subjected to the geosynthetic. The set-up of the tests are shown in Figure 66 with the results at failure shown in Figure 67.

Figure 66: Tensile strength test for (a) woven geotextile; (b) non-woven geotextile; (c) extruded geogrid; (d) woven geogrid

Figure 67: Failure during test for (a) woven geotextile; (b) non-woven geotextile; (c) extruded geogrid; (d) woven geogrid

The results of all the tensile tests on the different geosynthetics are presented in Tables 12 – 15. The graphs for the tensile tests conducted on the geosynthetics are presented in the Appendix III.

Woven Geogrid (WGG)

Table 12: Results of tensile test in cross direction for WGG.

Test	Max strain (%)	Max force (N)	Max Stress (MPa)	Tensile Strength (kN/m)
1	4.11	1418.410	5.674	5.06575
2	4.57	1533.350	6.133	5.47625
3	4.92	1455.550	5.822	5.19839
4	4.69	1498.199	5.993	5.35070
5	4.37	1440.005	5.760	5.14288
Average	**4.53**	**1469.103**	**5.876**	**5.24680**

Extruded Geogrid (EGG)

Table 13: Results of tensile test in cross direction for EGG.

Test	Max strain (%)	Max force (N)	Max Stress (MPa)	Tensile Strength (kN/m)
1	11.02	2589.19	10.357	9.24711
2	13.35	2754.73	11.019	9.83832
3	12.10	2841.26	11.365	10.14736
4	13.37	2774.48	11.098	9.90886
5	10.92	2612.46	10.450	9.33021
Average	**12.15**	**2714.424**	**10.858**	**9.69437**

Woven Geotextile (WGT)

Table 14: Results of tensile test in cross direction for WGT.

Test	Max strain (%)	Max force (N)	Max Stress (MPa)	Tensile Strength (kN/m)
1	4.47	423.701	1.695	1.51322
2	4.50	422.902	1.692	1.51036
3	4.40	424.800	1.699	1.51714
4	4.60	403.548	1.614	1.44124
5	4.31	444.053	1.776	1.58590
Average	**4.45**	**423.801**	**1.695**	**1.51358**

Non-woven Geogrid (NGT)

Table 15: Results of tensile test in cross direction for NGT.

Test	Max strain (%)	Max force (N)	Max Stress (MPa)	Tensile Strength (kN/m)
1	46.02	610.594	2.442	2.18069
2	48.83	619.130	2.477	2.21118
3	42.39	567.745	2.271	2.02776
4	44.12	530.755	2.123	1.89555
5	45.85	555.192	2.221	1.98283
Average	**45.44**	**576.683**	**2.307**	**2.05958**

5.2.3. Model Footing

Two footing sizes were used to determine the effect of footing width on the improvement of bearing capacities. The sizes that were prefabricated in the laboratory had dimensions of 75 mm x 140 mm x 20 mm and 150 mm x 140 mm x 25 mm that represented the width, length and thickness respectively. The model footings used are shown in Figure 68.

The size of the footings determines the zone of influence (H) which is an important feature as it defines the range of fill thicknesses that should be applied in the testing, so as to limit wastage of material by exceeding the H value. This phenomenon was discussed in detail in Section 2.3.3.

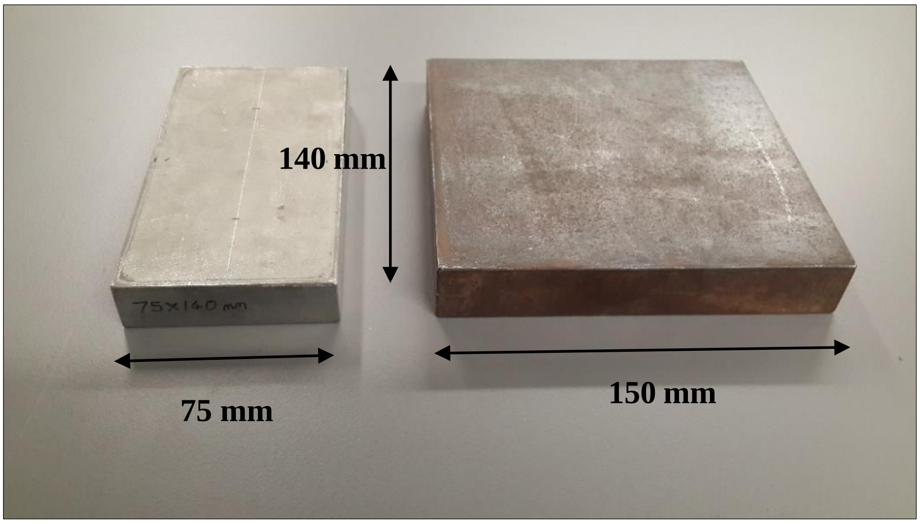

Figure 68: Illustration of the 75 mm and 150 mm model footings

5.2.4. Steel "box" model

The main apparatus used in this effort consisted of a bench scale steel "box" model (Figures 69 and 70). The rectangular configuration of the box, with internal dimensions 950 mm by 140 mm by 500 mm for the length, width and height respectively, was deemed to represent a pavement strip scenario. Metal bracings were fixed on either side of the equipment to prevent any lateral deformation of the box due to the applied loading. The occurrence of this during the testing process would have undermined the results of the investigation.

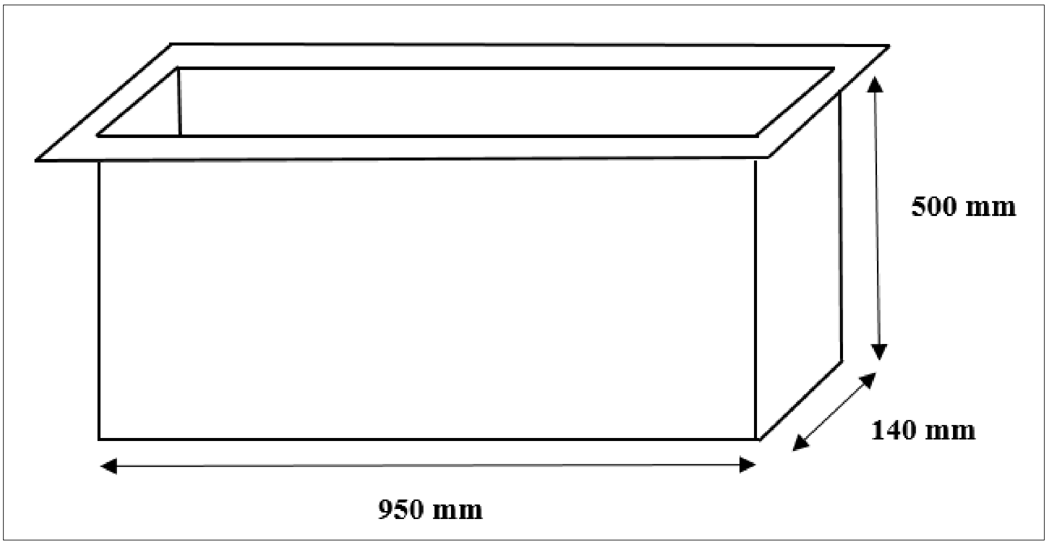

Figure 69: Illustration of the internal dimensions of the steel "box" model.

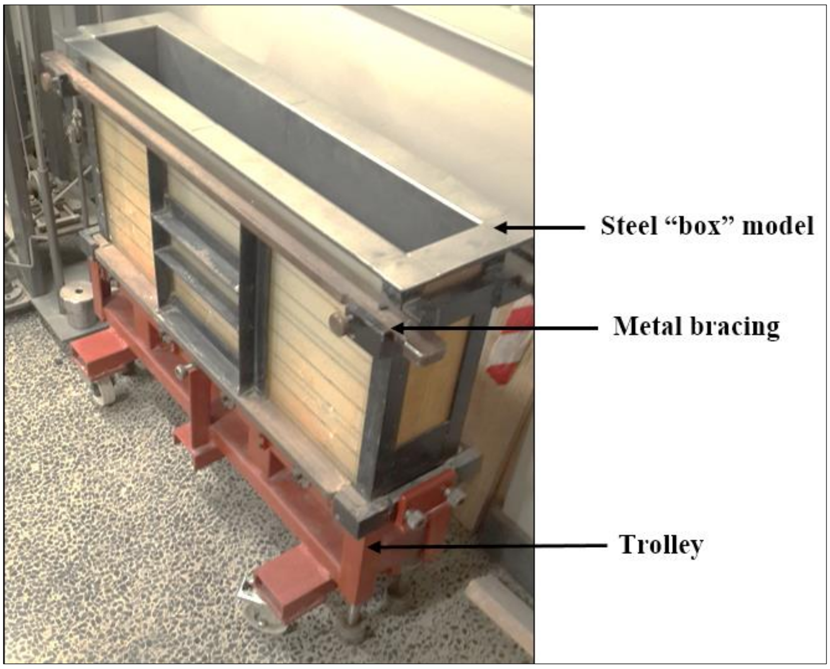

Figure 70: Steel "box" model showing the metal bracing and trolley

5.3. Testing Procedure

The methodology adopted during the testing programme is given in the sections below.

5.3.1. Material Preparation

5.3.1.1. Soil Specimen Preparation

The kaolin clay was prepared as described in Section 5.3.1.1.1, while the granular material was prepared as described in Section 5.3.1.1.2.

Correlation between CBR and Index Properties of soil

To obtain the required CBR value for the kaolin clay, correlation equations were used that related the CBR of the soil to its index properties. Attempts have been made by several researchers [Venkatraman et. el (1995), Kumar et. el. (2000), Karunaprema and Edirisinghe (2002)] to develop suitable correlations between CBR values of compacted soils at natural moisture content and results of some simple field tests.

According to Patel and Desai (2010) the California Bearing Ratio (CBR) is a method that is commonly used to evaluate the stiffness modulus and shear strength of subgrade in pavement design. However, the type of soil and the different properties possessed by the soil affect the CBR value makes it difficult for transportation engineers to obtain a representative value for design of pavements.

The study they conducted proposed a method for correlating CBR values with the liquid limit, plastic limit, plasticity index, moisture content, and maximum dry density of cohesive soils. The use of correlation equations was applied as these index property tests are much more economical and rapid than CBR test. Equations 22 and 23 were used in the comparison:

$$CBR = 53.783 - 103.571\,(LL) + 103.447\,(PL) + 103.443\,(Ip) - 0.077\,(SL) - 21.782\,(MDD) - 0.304\,(MC) \tag{22}$$

$$CBR = 43.907 - 0.093\,(Ip) - 18.78\,(MDD) - 0.3081(MC) \tag{23}$$

Where MDD is in gm/cc.

In the study it was observed that the predicted value of CBR from Equations 22 and 23 was much less than the value reported from the experimental tests. The laboratory test results were compared to the design equations and percentage errors ranging from 2.5% – 12% were obtained. As such this equation was not adopted in this research for the determination of CBR from design equations.

In the study conducted by Datta and Chottopadhyay (2011), the predicted and tested values of CBR of various soils were used to check the applicability and limitations of available methods. Some of the available correlation between CBR value and simple soil properties are described;

Vinode and Cletus (2008) correlated the value of CBR with liquid limit (LL) and gradation characteristics of soils as presented by Equation 24.

$$CBR = -0.889\,(W_{LM}) + 45.616 \tag{24}$$

Where W_{LM} is modified liquid limit and is given by

$$W_{LM} = LL\,(1 - C/100) \tag{25}$$

Where LL is the liquid limit on soil passing 425 micron sieve (in percent) and C is the fraction of soil coarser than 425 micron (percent).

It was observed that the predicted value of CBR from Equation 24 showed a wide divergence from the experimental value for most of the soils reported. As such this equation was not adopted in this research for the determination of CBR from design equations.

According to Talukdar (2014) for a given soil, the CBR value, and consequently the design, will depend largely on the density and the moisture content of the soil. It is also dependent on type of soil; and is more applicable for sandy soil than clayey soil. The limitation of the CBR test is that it is laborious and time consuming; furthermore, the results sometimes are not accurate due to poor quality of skill of the technicians testing the soil samples in the laboratory (Roy, Chattopadhyay and Roy, 2010). To overcome these difficulties, an attempt was made in their study to correlate CBR value statistically with the liquid limit (LL), plastic limit (PL), plasticity index (PI), maximum dry density (MDD) and natural moisture content of soil, because these tests are simple and can be completed with less period of time (Patel and Desai, 2010).

As the main aim of their study was to establish a relation of CBR value of soil with LL, PL, PI, MDD and MC, a multiple linear regression model was developed using Linex function of Microsoft Excel software. The mathematical relationship is shown in Equation 26:

$$CBR = 0.127\,LL + 0.00\,PL - 0.1598\,PI + 1.405\,MDD - 0.259\,MC + 4.618 \tag{26}$$

The comparison of the value of CBR on the 16 samples determined from the laboratory tests and obtained from Equation 26 is shown in Figure 71.

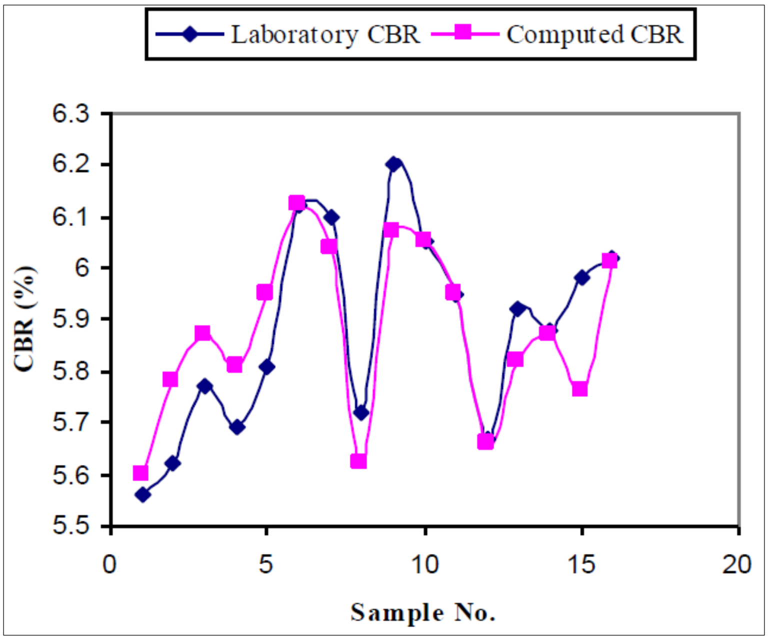

Figure 71: Comparison of Laboratory and computed CBR value (Talukdar, 2014).

Comparison of CBR value showed that in some soil samples, the laboratory and computed value of CBR have no difference. The maximum difference obtained was 3.67%, but in most cases the differences were < 3%. As there were slight differences between the CBR value determined in the laboratory and computed by using multiple linear regression model involving the soil index properties, Equation 26 was used in determination of the moisture content necessary to attain the require CBR.

5.3.1.1.1. Kaolin Clay

The clay was factory made and delivered near dry, this made it possible to determine the amount of water that needed to be added to achieve the required moisture content prior to testing.

The kaolin clay was mixed to give a CBR value less than 3%, which would have the prepared material replicate a soft subgrade. At that CBR it is deemed necessary for stabilization and reinforcement action to occur before construction can occur (SAPEM, 2013d: Table 4, page 21), which was conducted in this study using granular fill and a layer of geosynthetic.

Using the index properties obtained in Section 5.2.1.2 and the required CBR value of 3%, the amount of water that needed to be added to the soil was determined by obtaining the moisture content value using a modified equation adopted from Equation 26;

$$MC = \frac{(0.127\,LL + 0.00\,PL - 0.1598\,PI + 1.405\,MDD + 4.618 - CBR)}{0.259} \tag{27}$$

To prepare the samples at the specific moisture content, a pre-determined volume of water of 900 ml was added to 3 kg of dry kaolin HB powder and mixed thoroughly in an industrial mixer shown in Figure 72. This process was repeated until the required amount of kaolin was mixed to be used in the experiments (approximately 50 kg).

Figure 72: Industrial mixer used to prepare kaolin clay at the specified CBR.

The prepared kaolin sample was stored in a plastic container, shown in Figure 73, to allow for even moisture distribution throughout the sample. The container was kept in the Geotechnical store room for 24 hours before the tests were commenced and used within 72 hours, after which a new batch was made for the next round of testing.

Figure 73: Storage of mixed kaolin clay in plastic container.

5.3.1.1.2. Granular Material

The granular material was mixed at an optimum moisture content by weighing out 5 kg of the dry granular material and mixing 230 ml of water to the material. The mixed sample was stored in plastic containers in the Geotechnical laboratory store prior to testing, as shown in Figure 74. This was conducted to allow even distribution of the moisture throughout the sample.

Figure 74: Storage of granular material in a plastic container

5.3.1.2. Geosynthetic Preparation

The geosynthetics were delivered in rolls of varying dimensions; extruded geogrid (2 m by 3.95 m); woven geogrid (2 m by 3.9 mm); woven geotextile (2.6 m by 2 m); and non-woven geotextile (5.2 m by 2 m). The delivered rolls are shown in Figure 75.

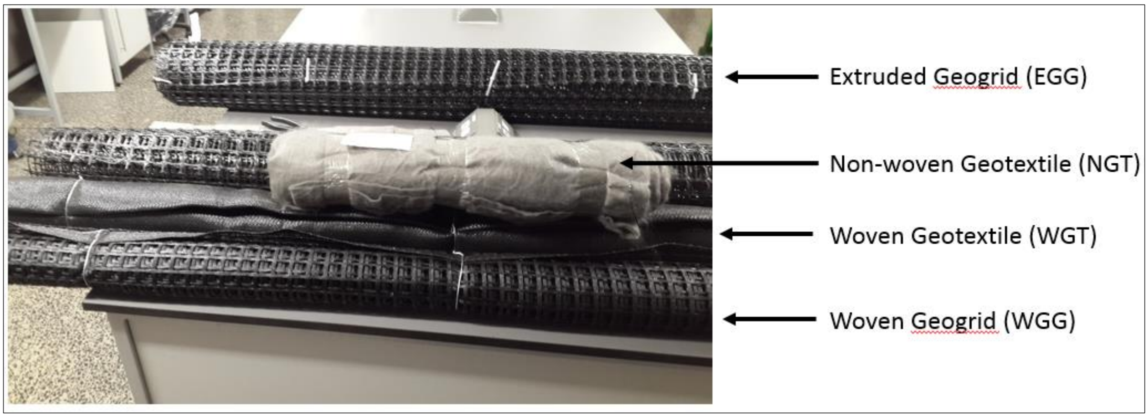

Figure 75: Geosynthetic samples shown as delivered in rolls.

The geosynthetics were cut to the specific rectangular sizes of dimensions 375 mm by 140 mm and 750 mm by 140 mm, to represent the lengths and widths respectively. The loading box had a width of 140 mm that limited the dimension of the geosynthetic to that measurement. Whereas the chosen measurements of 375 mm and 750 mm were selected to match the optimum width of the geosynthetics for all the tests of 5B, for the model footing widths of 75 mm and 150 mm respectively. A total of 64 specimen were prepared to be used in the geosynthetic-reinforced experiments.

5.3.2. Bearing Capacity Test Procedure

Testing was conducted using a large capacity Universal Testing Machine; where the strength of the geosynthetic reinforced two-layered soil was obtained by applying a compressive load on the composite in a specially fabricated loading box of internal dimensions of about 0.95 m by 0.45 m by 0.15 m to represent the length, depth and width respectively. The methodology followed in this study was similar to that conducted in previous studies, using the same equipment, by Hartley (2010); Buratovich (2011); Oriokot (2012); and Mawer (2013).

The two soil layers were hand compacted, to ensure no loose particles, and levelled in the loading box. The clay layer was prepared in all tests to a constant thickness of 250 mm, while the granular material had varying thicknesses from 50 – 150 mm depending on the footing width. Two model footings of widths 75 mm and 150 mm were used in the testing to obtain the effect of footing width on the thickness of the fill, the depth of placement of the geosynthetic layer, and the bearing capacity of the composite soil. The geosynthetic layer was placed at varying depths in relation to the footing width, from within the granular layer up to the interface of the two soils, at which the separation function was activated.

Figures 76 and 77 show the depth of placement (D) of the geosynthetic within the top layer and at the interface of the two soils for the varying thicknesses (Z) of the granular material; of which the depth of placement, D, and the thickness of fill, Z, are both related to the width of the footing (B).

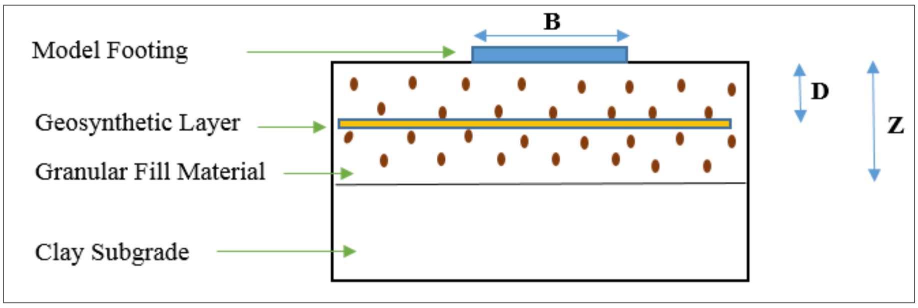

Figure 76: Geosynthetic layer placed within granular fill material.

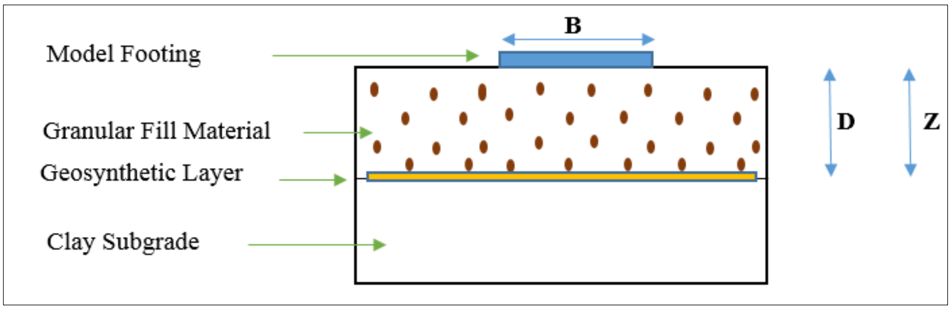

Figure 77: Geosynthetic layer placed at the interface of the two soils.

The laboratory procedure for the bearing capacity tests was as follows:

1. The clay was placed in the loading box in layers of 125 mm up to a thickness of 250 mm, which was kept constant throughout all the tests.
2. The soil was compacted and levelled for each of the layers to allow for preparation at maximum density that replicated field conditions closely. Figure 78 shows the levelling of the sample.

Figure 78: Levelling of the (a) kaolin clay soil, (b) granular material, and (c) and model footing (c) during preparation.

3. The granular material was then placed on top of the clay soil at varying thicknesses of 50 – 150 mm. In the situations that the thicknesses were greater than 100 mm, the fill was placed in 50 – 100 mm increments. Each of the layers were compacted and levelled similar to step 2.
4. The geosynthetic layer was placed at the varying depths of 50 – 150 mm, and centrally located below the model footing to allow for symmetrical reinforcement. Figure 79 shows the placement of the geosynthetic (a) at the interface of the soils and (b) within the granular fill.

Figure 79: Geosynthetic layer placed (a) at the soil interface and (b) within the granular material layer.

5. Once the sample was prepared, it was wheeled into position to be loaded into the Universal Compression Machine.
6. The model footing was placed centrally on top of the granular layer, and levelled.
7. Using the machine interface, the right settings that were necessary for each test and those to be recorded were applied. This included: the dimensions of the footing and the loading rate, which was set at 1.2 mm/min that replicated undrained conditions (ASTM D3080).
8. The test was started and was run until a vertical displacement of 30 mm was attained at which failure in a pavement structure would have occurred (Berg et al., 2000). From the SAPEM (2013d) it was stated that a rutting level of 20 mm is considered terminal for the road structure, as such running the tests to 30 mm would be sufficient.
9. The process was repeated for each of the 76 tests conducted, for the unreinforced soil and geosynthetic-reinforced composite.

Test conditions and procedures were similar throughout the tests programme; however, different granular fill thicknesses, depths of placement, and geosynthetic types were used.

5.3.3. Repeatability Assurance

It is desirable practice to check the repeatability of results for nominally identical test conditions. The reproducibility of the testing procedure and results were verified by conducting replicate experiments for one of the set-ups. The selected repetition was the unreinforced kaolin clay using the 150 mm model footing, which was conducted at the beginning and mid-point (experiment number 38) of all 76 tests. This ensured that the results were expected to be repeatable.

Furthermore, to ensure that the test conditions were properly reproduced, the following precautions were taken:

1. The weighing scale was calibrated to ensure all weight measurements were accurate.
2. The industrial mixer was washed and dried before a new mix of the clay was prepared, this was to ensure the right moisture content was attained for each of the prepared mixtures.
3. Any contaminated kaolin clay or granular material was discarded after each test. The contamination was in terms of mixing of the two soils that would alter the soil properties.
4. The clay mix was only used within 72 hours of preparation, and a new mix made for the next set of experiments, and this was to ensure that the sample had the right moisture content throughout all the tests.
5. The granular material was also replaced after 72 hours, to ensure the sample had the right moisture content too.

5.3.4. Testing Schedule

A total of 76 tests were conducted that included: the repeatability tests, the unreinforced set-ups and the geosynthetic-reinforced set-ups. Tables 16 – 19 show the investigations for the unreinforced set-up and each geosynthetic type. The full schedule is presented in the Appendix.

5.3.4.1. Repeatability Tests

The repeatability tests were conducted on the 150 mm model footing with no imported fill placed on the clay subgrade material. Three tests were conducted to ensure that the equipment was operating well, and that the testing procedure could be replicated.

Table 16: Table of the testing schedule for the repeatability tests

Model Footing Width (B)	Thickness of granular material (Z)	Thickness to Width Ratio (Z/B)	Test No.
150	0	0	RT/B150/Z0-1
	0	0	RT/B150/Z0-2
	0	0	RT/B150/Z0-3

5.3.4.2. Unreinforced (UR) set-up

The unreinforced tests were the control tests, as the strength of the subgrade could be determined and compared with the geosynthetic-reinforced soil composite. There were two series of control tests that were run. The first series involved the clay subgrade tested with neither fill material nor geosynthetic reinforcement included in the set-up. The second series involved the clay subgrade overlain by the granular material, however with no geosynthetic reinforcement.

Table 17: Table of the testing schedule for the unreinforced multi-layered soil

Model Footing Width (B)	Thickness of granular material (Z)	Thickness to Width Ratio (Z/B)	Test No.
75	0	0	UR/B75/Z0
	50	0.67	UR/B75/Z50
	75	1	UR/B75/Z75
	112.5	1.5	UR/B75/Z112.5
150	0	0	UR/B150/Z0
	50	0.33	UR/B150/Z50
	75	0.5	UR/B150/Z75
	112.5	0.75	UR/B150/Z112.5
	150	1	UR/B150/Z150

5.3.4.3. Woven Geogrid (WGG)

Table 18: Table of the testing schedule for the 75 mm model footing using the woven geogrid (WGG).

Model Footing Width (B)	Thickness of granular material (Z)	Thickness to Width Ratio (Z/B)	Depth of placement (D)	Depth to Width Ratio (D/B)	Test No.
75	50	0.67	50	0.67	WGG/B75/Z50/D50
	75	1	50	0.67	WGG/B75/Z75/D50
			75	1	WGG/B75/Z75/D75
	112.5	1.5	50	0.67	WGG/B75/Z112.5/D50
			75	1	WGG/B75/Z112.5/D75
			112.5	1.5	WGG/B75/Z112.5/D112.5

Table 19: Table of the testing schedule for the 150 mm model footing using the woven geogrid (WGG).

Model Footing Width (B)	Thickness of granular material (Z)	Thickness to Width Ratio (Z/B)	Depth of placement (D)	Depth to Width Ratio (D/B)	Test No.
150	50	0.33	50	0.33	WGG/B150/Z50/D50
	75	0.5	50	0.33	WGG/B150/Z750/D50
			75	0.5	WGG/B150/Z750/D75
	112.5	0.75	50	0.33	WGG/B150/Z112.5/D50
			75	0.5	WGG/B150/Z112.5/D75
			112.5	0.75	WGG/B150/Z112.5/D112.5
	150	1	50	0.33	WGG/B150/Z150/D50
			75	0.5	WGG/B150/Z150/D175
			112.5	0.75	WGG/B150/Z150/D112.5
			150	1	WGG/B150/Z150/D150

CHAPTER 6

6. Results

6.1. Introduction

This chapter presents in detail the results of the 76 bench scale tests performed to study the benefits of reinforcement of a pavement structure using geosynthetics. The applied loads against the vertical displacements were recorded for all of the experiments using the Zwick machine, and graphs generated from the data obtained. The results for the unreinforced kaolin clay were compared with results for the geosynthetic-reinforced composite to determine the degree of improvement.

In this chapter, however, little attempt was made to explain the trends or the observed performance implications due to geosynthetic-reinforcement of the multi-layered soil. Detailed discussions and analyses especially with respect to the parameters of interest are presented in Chapter 7.

6.2. Repeatability tests

To verify the reliability of the test procedure followed and the machinery used in the study, the experimental set-up that involved having only the unreinforced kaolin clay with the 150 mm footing was conducted three times. The results obtained from the repeatability tests are presented in Figure 80. It was assumed from the consistency of the results obtained that the experimental procedure followed was reproducible, which indicated that all the tests conducted throughout the study were repeatable.

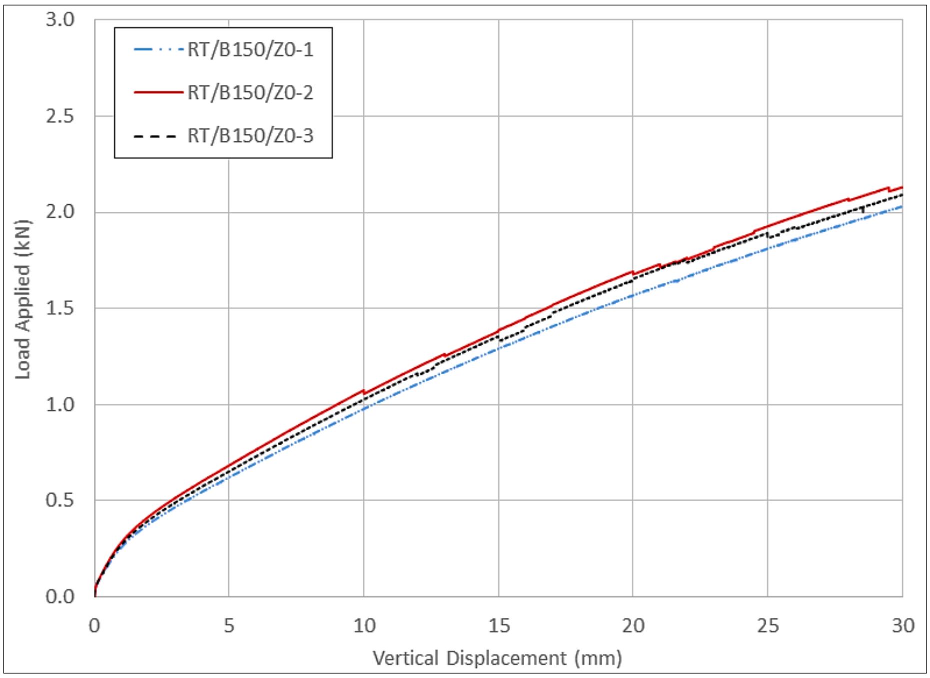

Figure 80: Graph of load applied against vertical displacement for the unreinforced clay subgrade

The analysis shown in Table 20 indicated that the maximum deviation from the average was 2.67%. This was acceptable as it falls under the maximum acceptable standard deviation of 5% that is considered to be sufficient to affect the results; hence confirming validity of the experiments.

Table 20: Repeatability results analysis.

Test Specimen	Peak Stress (kPa)	Average Peak Stress (kPa)	Deviation from Average	
			kPa	%
RT/B150/Z0-1	2.0283	2.0824	-0.0541	-2.67
RT/B150/Z0-2	2.1297		0.0473	2.22
RT/B150/Z0-3	2.0891		0.0068	0.03

6.3. Control tests

Control tests are essential for any experimental study as they provide a basis of comparison for results obtained from test procedures where parameters are changed. The main feature of these tests was that no geosynthetic reinforcement was included. The testing of the unreinforced kaolin clay with only a layer of granular material for both footing sizes (75 mm and 150 mm) was conducted.

As the applied load was increased, there was an observed increase in the vertical displacement of the material. This was attributed to the applied load exceeding the maximum that could be resisted by the soil particles, resulting in a lateral spread of the particles and the vertical displacement. The results were recorded up to a vertical displacement of 30 mm that represented the point of failure in pavement structures (SAPEM, 2013d), also known as the serviceability limit state.

The results presented in Figure 81 for the control test using the 75 mm footing showed that as granular material was overlain on the clay material there was an increase in the applied load. However, as the thickness of the granular material was increased further from 50 mm to 75 mm, there was no observed difference in the improvements obtained.

As the thickness of the granular material is increased, there was a subsequent increase in the generated shear stresses at the base of the granular material (at the interface of the soils), which directly reduces the load-bearing capacity of the clay material. Therefore, for a particular clay strength, the unreinforced soil can only maintain an approximate constant ultimate load despite the increase in thickness of the granular material used (Brocklehurst, 1993).

This phenomenon was also in the experiments using the 150 mm footing as shown in Figure 82. The applied loads for the 75 mm, 112.5 mm and 150 mm fill thicknesses were similar at all vertical displacements, and at the point of serviceability failure (30 mm vertical displacement) they all had an approximate applied load of 2.3 kN.

The benefit of the inclusion of the granular material over the clay material is the reduction in settlement, as seen at any specific applied load when the values of the vertical displacement are compared.

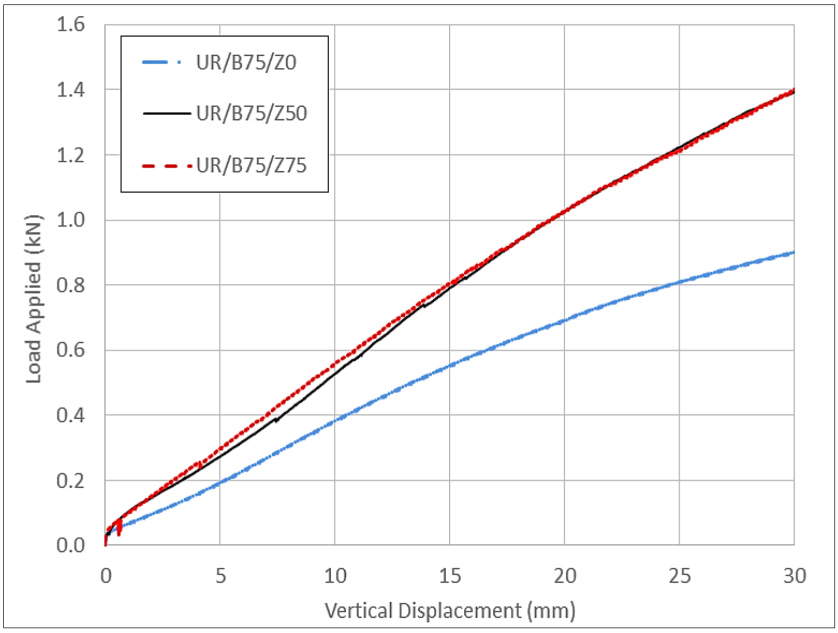

Figure 81: Load applied against vertical displacement for the control tests using a 75 mm footing.

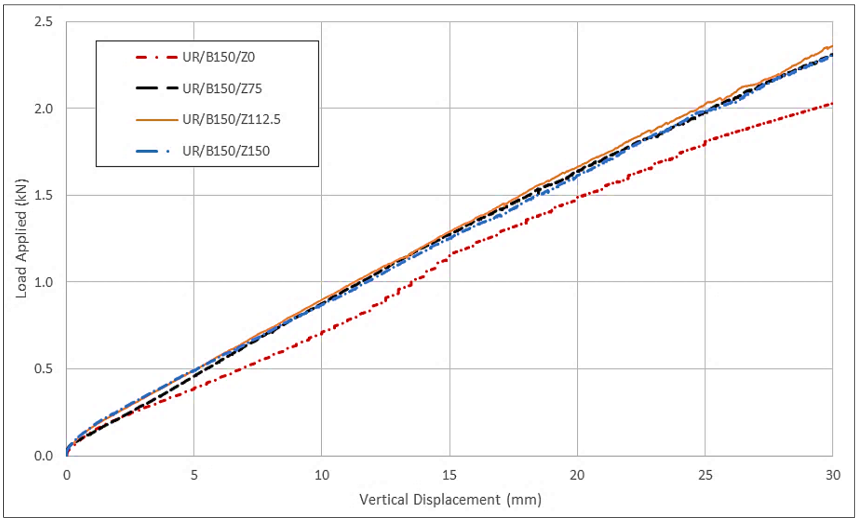

Figure 82: Load applied against vertical displacement for the control tests using a 150 mm footing.

6.4. Geosynthetic-reinforced tests

The basis of this study was to determine the benefit of reinforcement of a two-layered soil structure using a geosynthetic layer. The soil structure represented the foundation levels of pavement structures and consisted of a soft clay layer overlain by a layer of granular material. The geosynthetic layer was placed either at varying depths within the granular material or at the interface of the two soils.

The graphs presented in Sections 6.4.1 and 6.4.2 for the 75 mm and 150 mm footings respectively are the results obtained from the experiments on the geosynthetic-reinforced composites, with variations in the thickness of the granular layer and the depths of placement of the reinforcement.

6.4.1. 75 mm model footing

The graphs presented in this section are for experiments conducted using the 75 mm model footing, with different configurations for inclusion of a geosynthetic layer. It was observed that as the applied load was increased there was a subsequent increase in the vertical displacement. However, with the inclusion of a geosynthetic layer, there was a reduction in that vertical displacement at a specific load applied. The degree of reduction in settlement was dependent on the type of geosynthetic product used and the configuration. In addition, when compared at any specific vertical displacement it was observed that there was an improvement in the load applied with inclusion of the geosynthetic layer. The degree of improvement was also dependent on the type of geosynthetic and the configuration.

Figure 83 (a) are results for the woven geogrid placed at various depths for a constant granular thickness of 75 mm. Figure 83 (b) are results for the extruded geogrid placed at the interface of the soils, with the thickness of the granular material was increased. Figure 83 (c) are results for the woven geotextile placed at a constant depth of 50 mm while the granular thickness was increased. Figure 83 (d) are results for the non-woven geotextile in a similar configuration as the woven geogrid, with the reinforcement placed at various depths for a constant granular thickness of 75 mm.

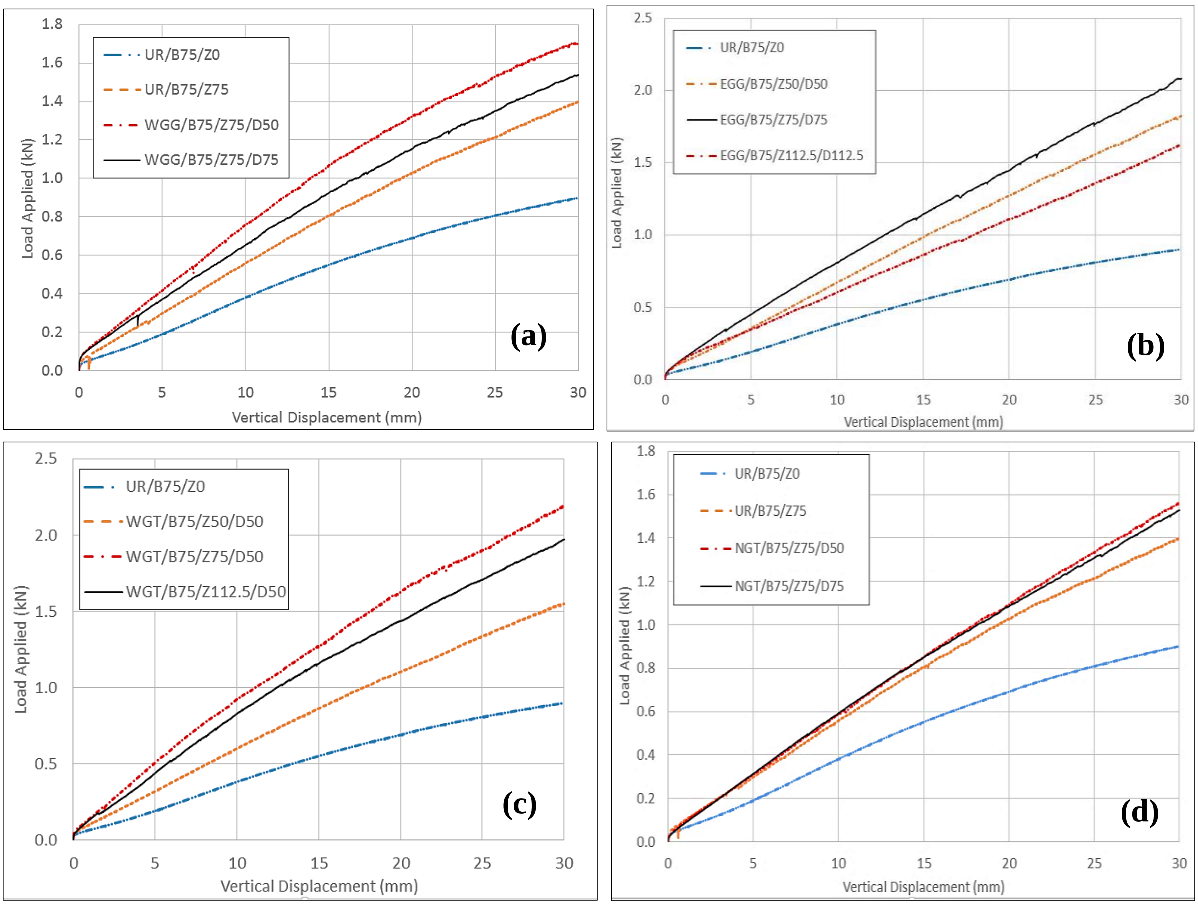

Figure 83: Load applied against vertical displacement for the (a) woven geogrid (b) extruded geogrid (c) woven geotextile and (d) non-woven geotextile, using the 75 mm model footing.

As the depth of placement for the woven geogrid was increased from 50 mm to 75 mm, there was an observed reduction in the improvement of the applied load when the two results were compared, as seen in Figure 83 (a). A similar response was observed in the results obtained from the non-woven geotextile shown in Figure 83 (d), where there was a reduction in the improvement when the reinforcement depth was increased from 50 mm to 75 mm.

For the results from the configurations with the reinforcement at the interface of the two soils, it was observed that as the thickness of the granular material was increased there was a subsequent increase in the improvement in the load applied and increased reduction in the vertical displacement. These improvements were however reduced at certain configurations. For the extruded geogrid, there was reduced performance for the granular thickness and placement depth of 112.5 mm as shown in Figure 83 (b). Whereas for the woven geotextile the reduced performance was observed for the reinforcement placed at 50 mm depth in a granular thickness of 112.5 mm, as shown in Figure 83 (c).

6.4.2. 150 mm model footing

Similar experiments of geosynthetic-reinforced soil composites were conducted on a 150 mm model footing. This was performed to determine the effect of an increased footing size on the benefits of geosynthetic reinforcement. The similar response of increased vertical displacement with increase in applied load was observed as shown in Figure 83 for the different configurations. In addition there

Figure 84 (a) are results for the woven geogrid placed at various depths for a constant granular thickness of 75 mm. Figure 84 (b) are results for the extruded geogrid placed at the interface of the soils, with the thickness of the granular material was increased. Figure 84 (c) are results for the woven geotextile placed at a constant depth of 75 mm while the granular thickness was increased. Figure 84 (d) are results for the non-woven geotextile in a similar configuration as the woven geogrid, with the reinforcement placed at various depths for a constant granular thickness of 75 mm.

The observed trend shown in Figure 84 (a) for the woven geogrid was similar to that obtained when using the 75 mm footing, in which there was a general increase in load applied and increased reduction in vertical displacement, due to inclusion of the reinforcement in the soil structure. However, there was a reduction in the improvement in the load applied and a decrease in the reduction in the vertical displacement as the depth of placement increased from 50 mm to 75 mm.

For the extruded geogrid, the results shown in Figure 84 (b) indicated a constant improvement as the granular thickness was increased, that differed from the results obtained using the 75 mm footing. The trend for the woven geotextile shown in Figure 84 (c) also differed from that observed using the 75 mm footing. This could be attributed to the difference in depth of placement of the reinforcement, which were 50 mm and 75 mm for the for the 75 mm and 150 mm footings respectively. In the latter configuration, there was a subsequent improvement in performance with increase in granular thickness. The non-woven geotextile results shown in Figure 84 (d) indicated an improvement of performance when the granular thickness was increased from 50 mm to 75 mm, though they also differed from the observed results when using the 75 mm footing.

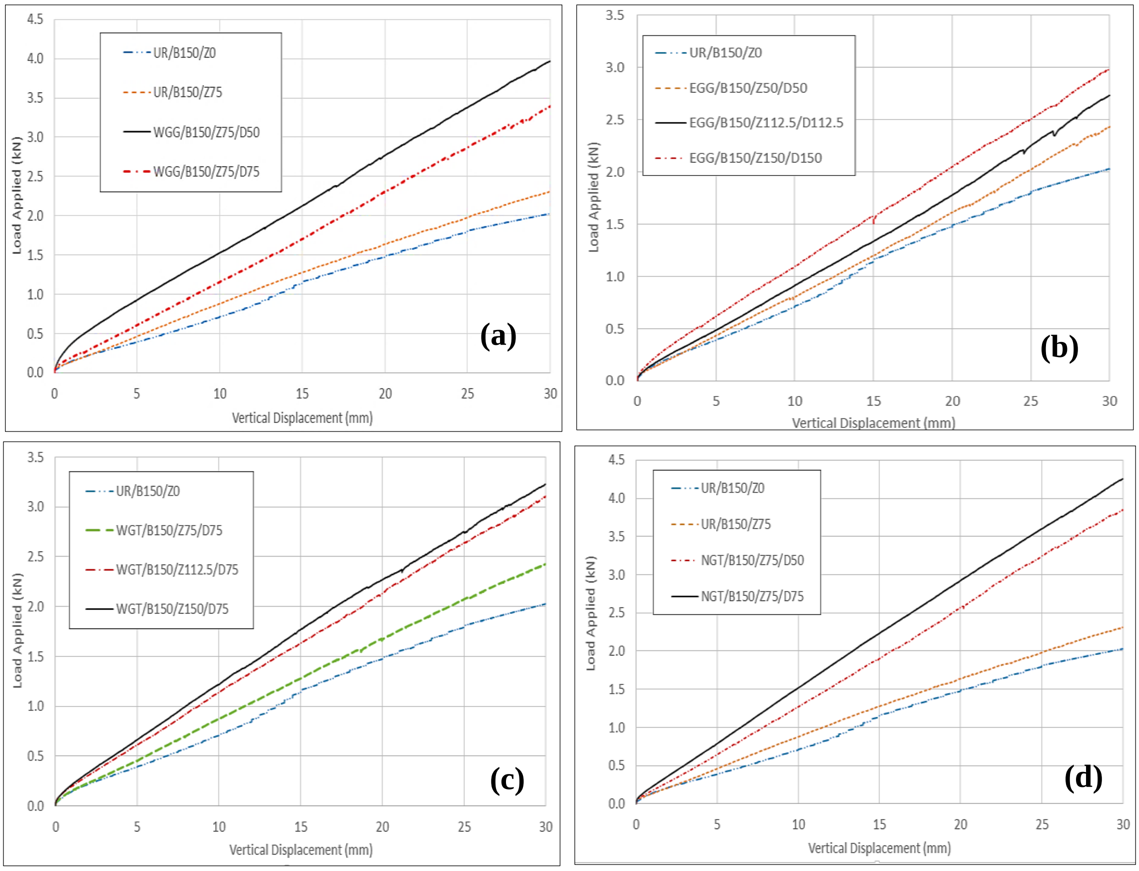

Figure 84: Load applied against vertical displacement for the (a) woven geogrid (b) extruded geogrid (c) woven geotextile and (d) non-woven geotextile, using the 150 mm model footing.

6.5. Summary of Results

This Chapter has provided results of bench-scale experiments performed to investigate the effectiveness of using geosynthetics in reinforcement of two-layered soils. In the process, different parameters such as: thickness of granular fill, depth of placement of geosynthetic layer, model footing size, and the type of geosynthetic product have been explored. The trends and changes in the results were presented in this chapter, however, these were analysed in detail in Chapter 7 to provide a better understanding of how the variations in the parameters and the reinforcement benefits are achieved.

Overall, the results of the study revealed that:

1. The inclusion of the granular fill over the soft subgrade, without any geosynthetic reinforcement, led to an increase in the applied load on the layered soil structure. However, further increases in the thickness of the fill did not result in an improvement in the bearing capacity of the two-layered soil structure.
2. The incorporation of geosynthetics in the two-layered soil led to a further increase in the applied load of the composite irrespective of the position of placement.
3. As the thickness of the granular fill was increased with geosynthetic reinforcement included in the soil structure, there was a subsequent increase in the applied load by the Zwick machine, which could be directly related to an increase in the bearing capacity of the soil. However, as the thickness was increased beyond a certain limit, there was a reduction in the improvement in strength of the composite.

4. As the depth of placement of the reinforcement layer was increased, at constant granular thickness, there was an observed reduction in the strength of composite structure. However, when different geosynthetic products were used there was also an observed increase in the strength.
5. Doubling the footing width by 50% led to both an increase in the applied load on the soil structure by approximately 10% – 30%, and also a reduction in the range of 20% – 30%, dependent on the type of geosynthetic product. The increase could be attributed to wider spread of the load applied and thus greater resistance and support by the soil particles and geosynthetic layer, while the decrease could be attributed to increased weight from the footing.

CHAPTER 7

7. Analysis of Results

This chapter discusses in detail the results presented in Chapter 6, and also puts the experimental findings into context in terms of the benefits achieved from reinforcing pavement structures with geosynthetics. The improvement in load-bearing capacity and the reduction in settlement are first discussed. This is followed by analysis of the effects of changing different parameters that included the thickness of granular material; depth of placement of the reinforcement; model footing size; and the type of geosynthetic product.

7.1. Load-Bearing Capacity

The load-bearing capacity of the soil is its ability to support applied loads. From the results obtained it was observed that the inclusion of a layer of granular material on the clay led to an increase in the load-bearing capacity of the soil. When the two-layered soil was reinforced with a layer of geosynthetic, both within the granular layer and at the interface of the soils, there was a further increase in the load-bearing capacity.

The improvement on the inclusion of only the granular layer initially was attributed to the material having a higher strength than clay thus being able to support higher loads exerted on the soil. The granular particles also transferred the loads applied to a greater area, reducing the load transferred to the weak clay. This resulted in a change of the failure response from potential punching failure in the weak clay to general shear failure. This phenomenon is shown below in Figure 85.

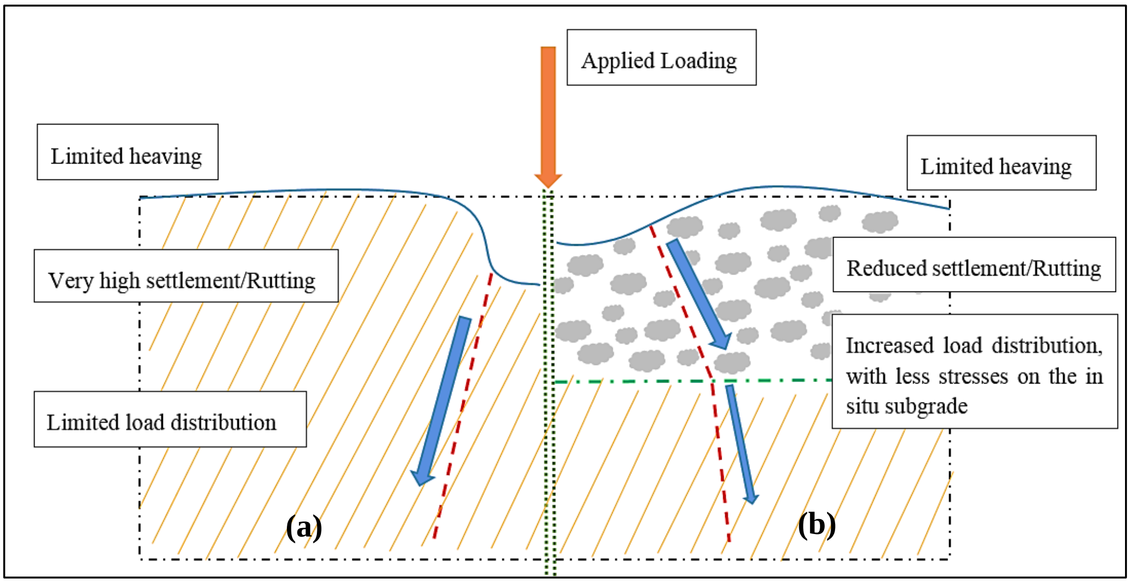

Figure 85: Transfer of applied load through clay and granular-reinforced clay.

In Figure 85 (a), for the unreinforced clay, it was observed that there was high settlement and limited heaving of the soil. There was also limited distribution of the load throughout the soil, with a more direct transfer below the load which leads to the high settlement. Figure 85 (b) is an illustration of the clay overlain by the granular fill. In this set-up there was also limited heaving, however, there was a reduction in the settlement. This was attributed to the increased distribution of the applied load through the fill, resulting in reduced transfer to the weak clay. The greater strength of the fill led to an increase in the stability of the soil structure, allowing for support of the loads.

On inclusion of the geosynthetic layer, the load-bearing capacity of the composite is increased through the transfer of the applied forces from the soil particles, granular fill, to the geosynthetic layer. This occurs as the applied load leads to lateral dispersion of the granular particles, which the geosynthetic acts to prevent forming a composite. The geosynthetic layer has a higher tensile resistance than soil, thus taking up the shear stresses that develop in the soil, and hence provides the additional support to the structure which leads to an improvement in the load-bearing capacity. Figures 86 and 87 show the distribution of the loads through the geosynthetic-reinforced composite when placed within the fill and at the interface of the two soils respectively.

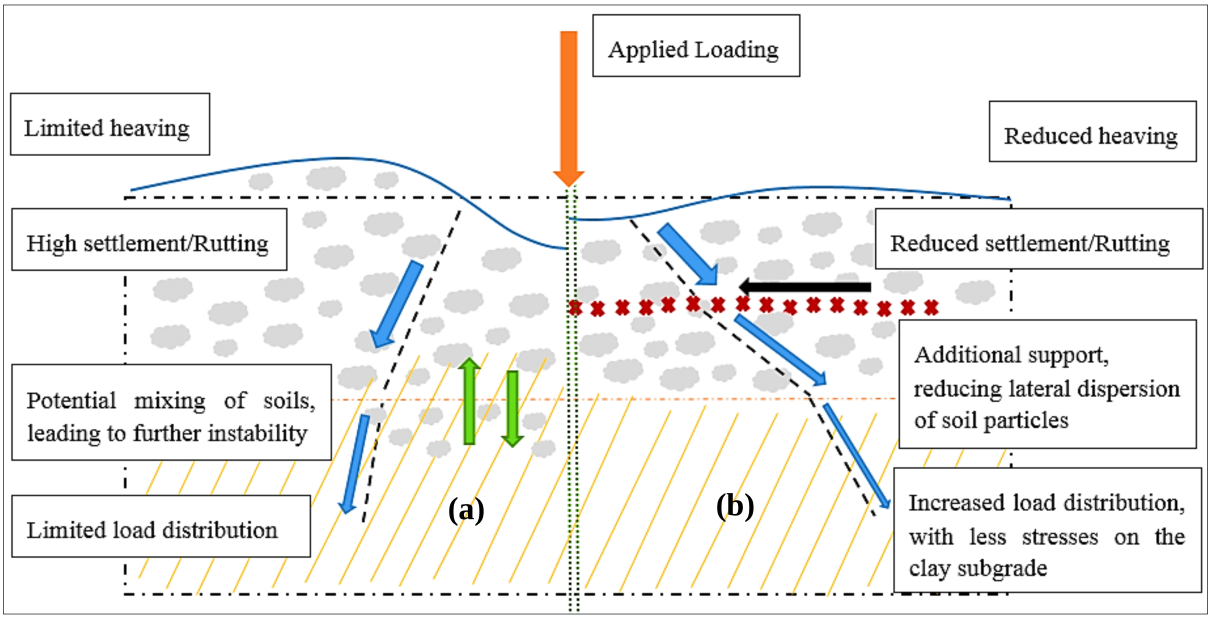

Figure 86: Load distribution for geosynthetic reinforcement within the granular material layer (subbase reinforcement)

In Figure 86 (a) it was observed that for the two-layered soil with continuous application of the load there was the potential mixing of the soil particles, with the clay fines being pumped into the fill and the granular particles penetrating into the soft clay. As this occurred the soil structure was destabilized, which led to increased settlement and heaving. There was also limited load distribution from the fill to the clay that contributes to settlement. With the inclusion of the geosynthetic layer within the fill layer, as shown in Figure 86 (b), there was an improvement in the stability of the soil structure as there was reduced lateral dispersion of the granular particles. There was also an increase in the load distribution with less stress transferred to the clay. These all contribute to an improvement in the load-bearing capacity of the soil, with a reduction in settlement and heaving.

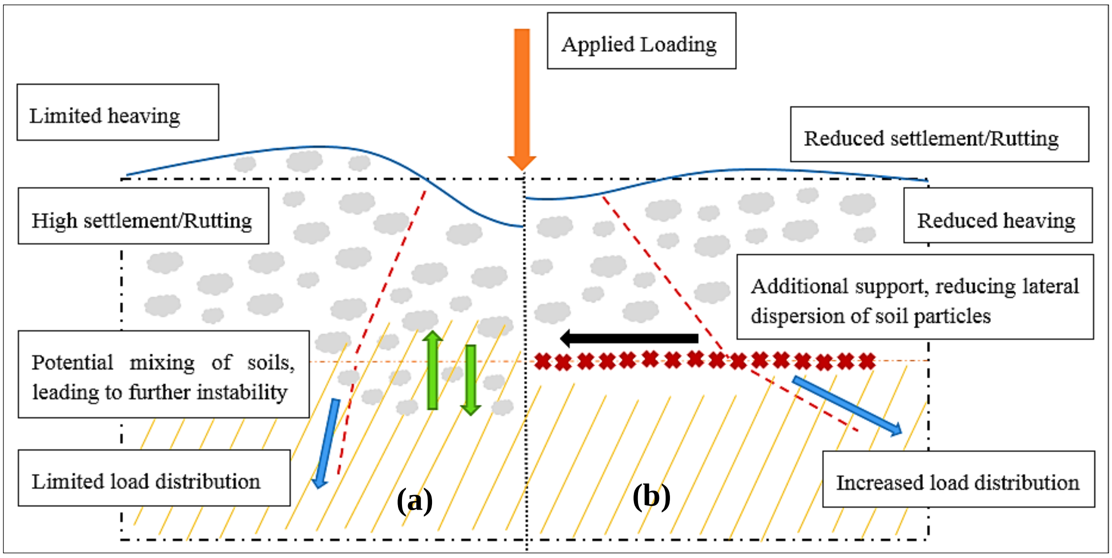

Figure 87: Load distribution for geosynthetic reinforcement at the soil interface (subgrade reinforcement).

In Figure 87 (a), the occurrences are similar to that described for Figure 86 (a). However, on inclusion of the geosynthetic layer at the soil interface, as shown in Figure 87 (b), there was the reduction in the lateral dispersion of the granular particles and an increase in the load distribution into the clay. This results in additional support of the soil structure which leads to an increase in the load-bearing capacity with reduced settlement and heaving.

It was observed that the inclusion of the granular material to the clay led to an increase in the load-bearing capacity, and on inclusion of the layer of geosynthetic either within the fill layer or at the interface of the two soils led to a further improvement in this load-bearing capacity. This is evident as shown Figure 88 in which the bearing capacity ratio increased for the granular-reinforced clay in the range of 35% - 50%, whereas when the woven geogrid was included, there was a further increase in the bearing capacity ratio in the range of 65% - 75%.

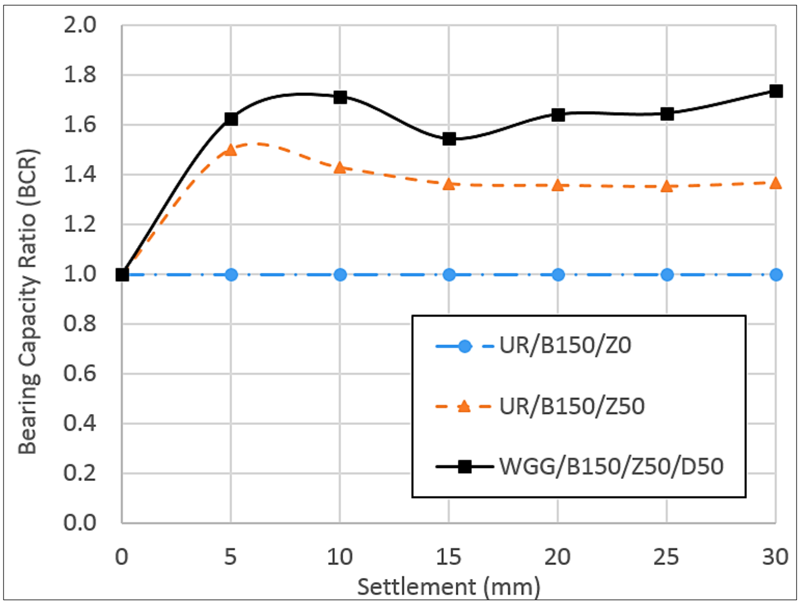

Figure 88: Bearing capacity ratio against settlement for the woven geogrid using the 150 mm footing.

7.2. Settlement

Settlement is a major factor in marginal soils, and as such a reduction of the degree of settlement is necessary in the design and construction phases. To determine if the applied design is adequate, the anticipated settlement can be measured using relevant equations. According to Ornek et al. (2012) the percentage reduction in footing settlement (PRS) is calculated using the following equation:

$$PRS = \frac{S_o - S_r}{S_o} \; x\,100 \qquad (28)$$

Where:

S_o - is the settlement of unreinforced soil at a given footing pressure.

S_r - is the settlement of reinforced soil at the same footing pressure.

Considering the pressure at 85 kPa, the settlement of the unreinforced soil was observed at 25 mm as shown in Figure 89. When this is compared to the composites of granular material only and of the combined granular material and woven geogrid, the settlement was reduced to approximately 17.5 mm and 15 mm respectively. From Equation 28, the calculated PRS for the two composites would be 30% and 40% respectively. This shows that the addition of granular material and a geosynthetic layer to the multi-layered soil has the effect of reducing the settlement faced considerably.

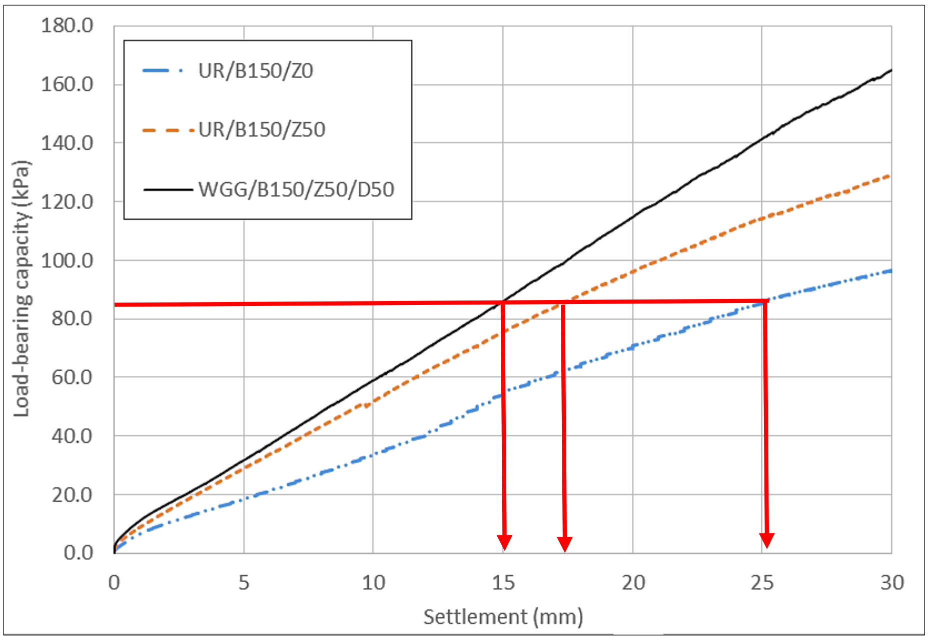

Figure 89: Load-bearing capacity against vertical displacement for the 150 mm footing and woven geogrid

When the results shown in Figures 90 and 91 were analysed, for different thicknesses of the granular material and different types of geosynthetics, it was observed that there were reductions in settlement ranging from 35% - 60%.

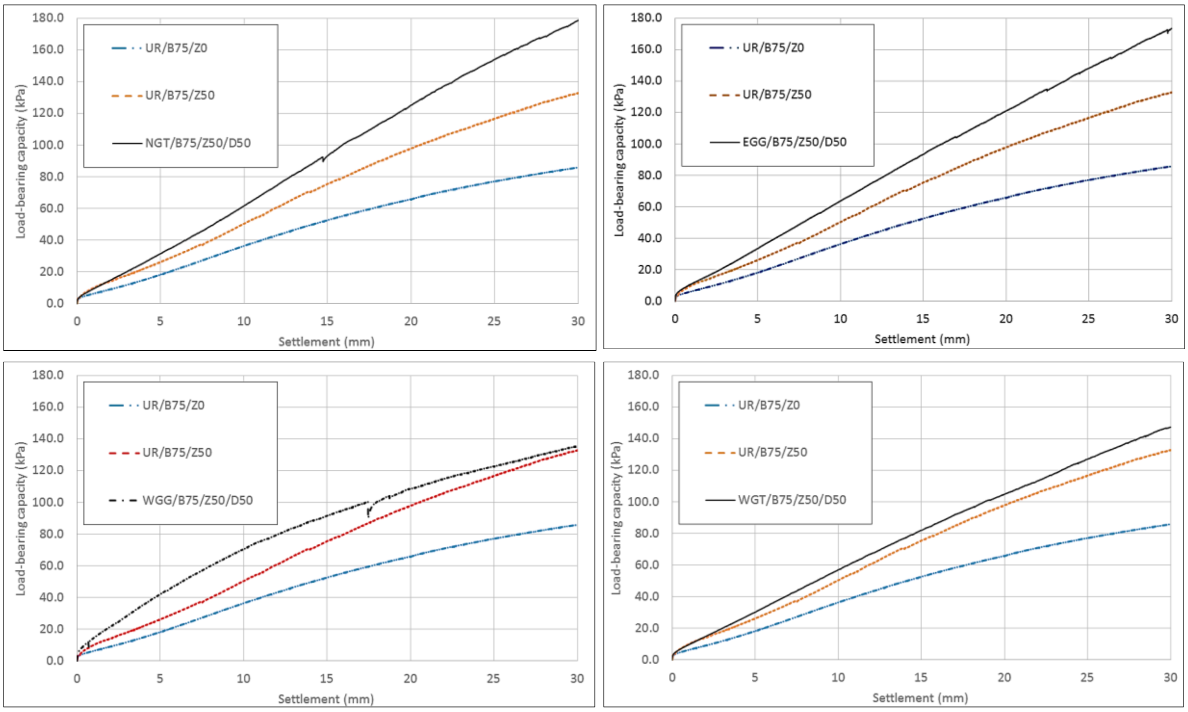

Figure 90: Load-bearing capacity against settlement for geosynthetic layer placed at the interface of soil for granular thickness of 50 mm using the 75 mm footing.

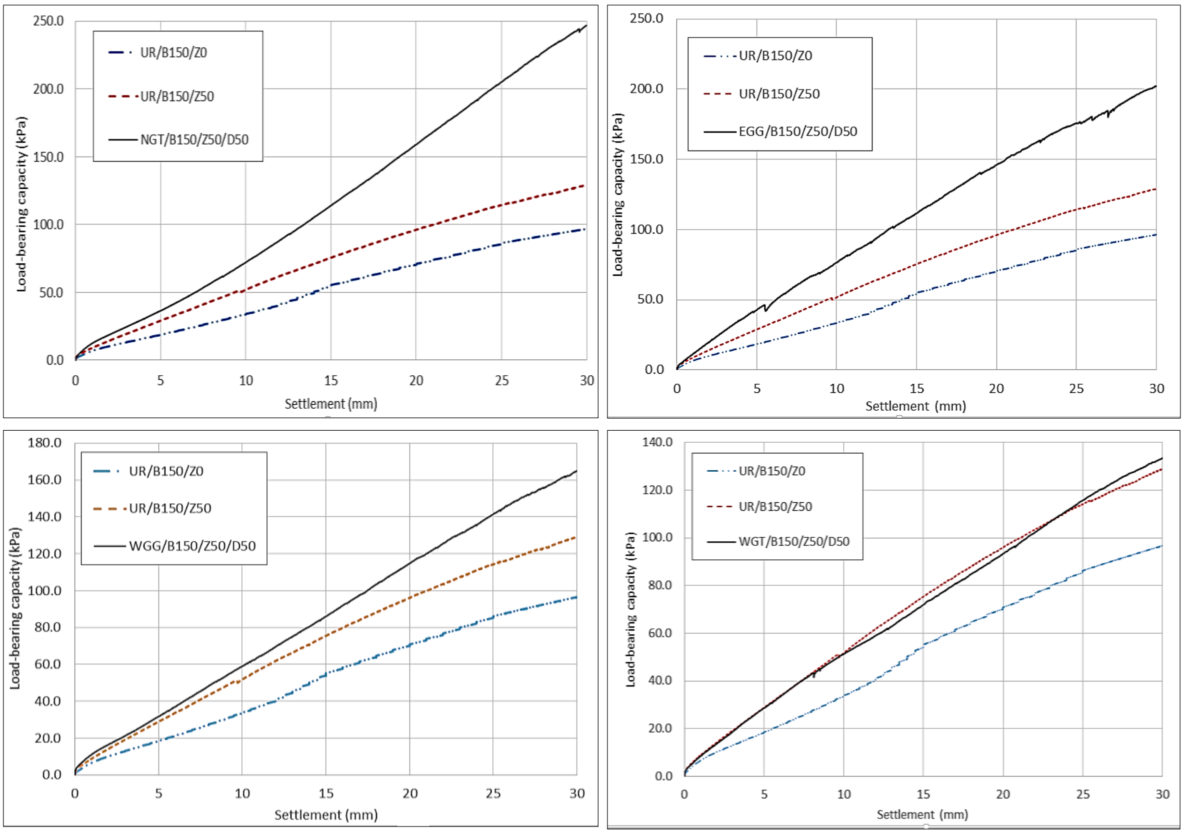

Figure 91: Load-bearing capacity against settlement for geosynthetic layer placed at the interface of soil for granular thickness of 50 mm using the 150 mm footing.

It was observed that the addition of geosynthetic reinforcement improves the performance of the roadway by decreasing deflections. For all cases studied, if the stiffness of the geosynthetic used was greater than that of the clay subgrade, the resulting deflection was less than that of the unreinforced case. In other words, adding a stiffer material to the two-layered soil decreased the deflection under a given pressure.

7.3. Thickness of granular material

The determination of the optimum thickness of the granular material was conducted by comparing the improvements in the load-bearing capacities for different tests conducted, in which the fill thickness was varied and the other parameters were kept constant.

There were two test series configurations that were compared that involved either having the geosynthetic layer placed at a constant depth with the granular thickness varied, or having the geosynthetic layer placed at the interface of the soils for all the variations in fill thickness. These two differing configurations are shown in Figures 92 and 93 respectively.

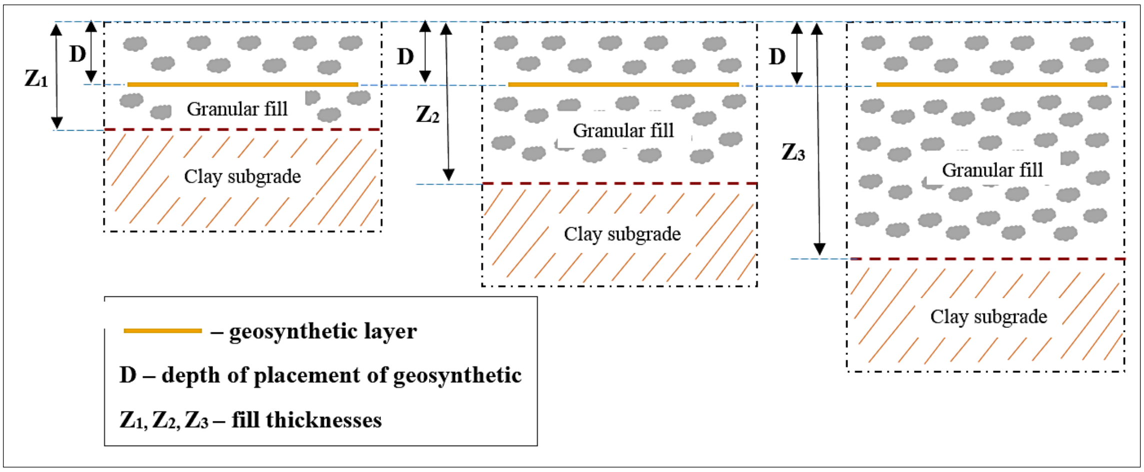

Figure 92: Series configuration 1 with the geosynthetic layer placed at a constant depth and granular thickness varied.

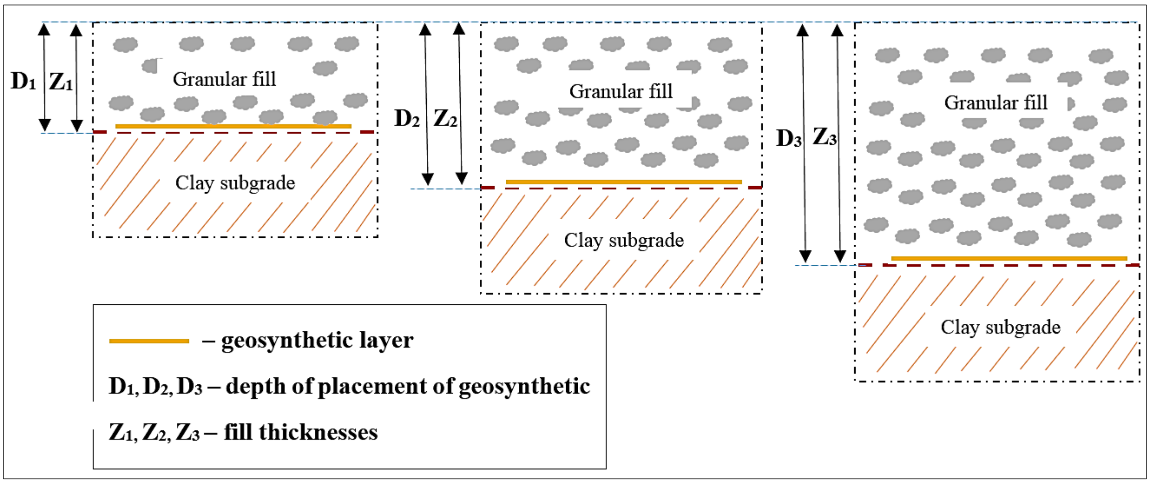

Figure 93: Series configuration 2 with the geosynthetic layer placed at the interface of the two soils as the granular thickness varied.

In the analysis of results it was observed that as the thickness of the granular material is increased the applied load increased. However, there was a point when a further increase in granular thickness did not bring about a subsequent increase in the load-bearing capacity of the composite.

7.3.1. Series Configuration 1

The result presented in Figure 94 (a) was when the depth of placement of the woven geotextile was kept constant at 50 mm below the surface and the fill thickness varied. The unreinforced clay gave a load-bearing capacity of 85 kPa, and it was observed that as the granular fill thickness was increased there was a subsequent increase in the load-bearing capacity of the reinforced composite from approximately 150 kPa to 210 kPa. However, as the thickness was increased further from 75 mm to 112.5 mm there was a reduction in the improvement in the load-bearing capacity to approximately 185 kPa.

Taking the 30 mm settlement as the point of failure, the improvement in the strength of the soil increased from 90% for the 50 mm fill thickness to 160% for the 75 mm fill thickness, with a reduced improvement of 130% for the 112.5 mm thickness.

Figure 94 (b) is a graph of the load-bearing capacity against the varying granular thickness ratios at different observed settlements of 20 mm, 25 mm and 30 mm, which were used in the determination of the optimum granular thickness. From all the graphs presented a trend was observed of an increase in the load-bearing capacity with increase in granular thickness. This was evident up to a thickness of 1B, beyond which there was a reduction in the load-bearing capacity. This showed that the optimum fill thickness was at a thickness to footing width ratio of 1.0B.

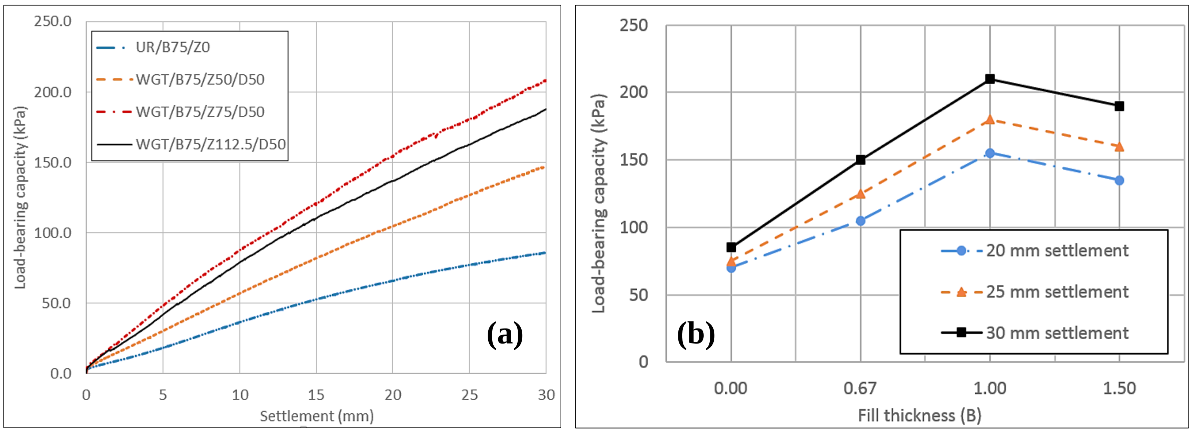

Figure 94: (a) Load-bearing capacity against settlement and (b) Load-bearing capacity against fill thickness for the woven geotextile placed at a constant depth of 75 mm using the 75 mm footing.

A similar trend was observed when the footing width was increased to 150 mm for the woven geotextile placed at a constant depth of 75 mm and the fill thickness increased. The optimum thickness to footing width ratio of 1B was also obtained. The results presented in Figure 95 (a) showed that there was an improvement in the load-bearing capacity from approximately 95 kPa in the unreinforced soil to 115 kPa when the reinforcement was placed at 75 mm depth. As the fill thickness is further increased with the depth of placement kept constant, there was an increase in the load-bearing capacity to 150 kPa for the 112.5 mm fill and to 155 kPa for the 150 mm fill.

When the load-bearing capacities were compared at different settlements, the trends in Figure 95 (b) showed that for the woven geotextile placed at a constant depth of 75 mm there was an increase in the load-bearing capaicty with an increase in the granular thickness. This gave an optimum fill thickness to width ratio of 1.0B.

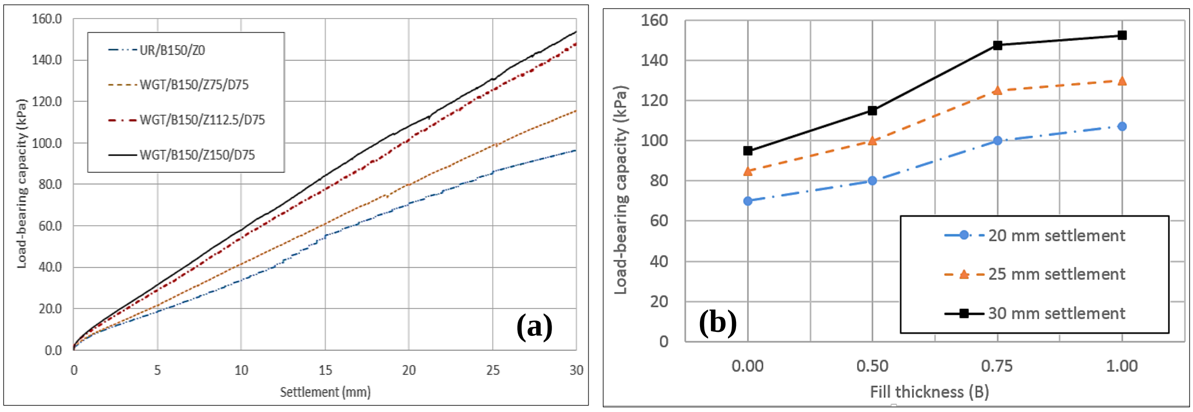

Figure 95: (a) Load-bearing capacity against settlement and (b) Load-bearing capacity against fill thickness for the woven geotextile placed at a constant depth of 75 mm using the 150 mm footing.

When the extruded geogrid was tested, a similar trend of increased composite strength with granular thickness was observed. From Figure 96 (a) it was observed that when the depth of placement of the extruded geogrid was kept constant at 50 mm, and the thickness of the fill was gradually increased, there was a subsequent increase in the load-bearing capacity. The improvement for the 50 mm, 75 mm and 112.5 mm fill thicknesses were 175 kPa, 185 kPa and 210 kPa respectively, which gave improvements of 105%, 115% and 145% respectively.

From the trends observed in Figure 96 (b), there was an observed increase in the load-bearing capacity with increased granular thickness to width ratio at each of the selected settlements of 20 mm, 25 mm and 30 mm. The optimum granular thickness to footing width ratio of 1.5B was obtained.

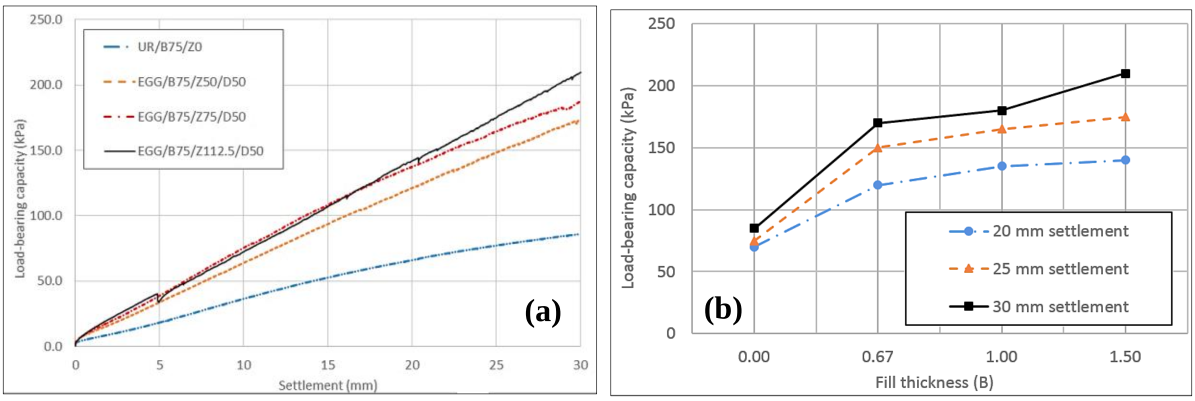

Figure 96: (a) Load-bearing capacity against settlement, and (b) Load-bearing capacity against fill thickness for extruded geogrid at a constant depth of 50 mm using the 75 mm footing.

The improvement in the load-bearing capacity could be attributed to the interlocking of the granular particles in the extruded geogrid apertures forming a composite structure with increased stability and higher strength. It was observed that the load-bearing capacities for the 75 mm and 112.5 mm thicknesses were similar up to 20 mm settlement, after which the 112.5 mm fill thickness had an increase in strength. There is also the additional support from the granular material as there is an increase in the thickness of material below the reinforcing layer, thus delaying the transfer of the applied loads to the weak clay.

7.3.2. Series Configuration 2

Figure 97 (a) shows the results for tests conducted on a woven geogrid using a 75 mm footing with varying thicknesses of the granular material and the reinforcement placed at the soil interface. There was an observed improvement in the load-bearing capacity of the composite to 135 kPa, 145 kPa and 170 kPa as the fill thickness increased to 50 mm, 75 mm and 112.5 mm respectively. This corresponded to improvement benefits of 60%, 70% and 100% respectively, with an optimum of 112.5 mm.

Figure 97 (b) shows the trend for selected settlements of 20 mm, 25 mm and 30 mm for the configuration of the woven geogrid at the interface of the two soils. It was observed that as the granular thickness to footing width ratio increased there was a subsequent increase in the load-bearing capacity. The optimum fill thickness to width ratio was 1.5B.

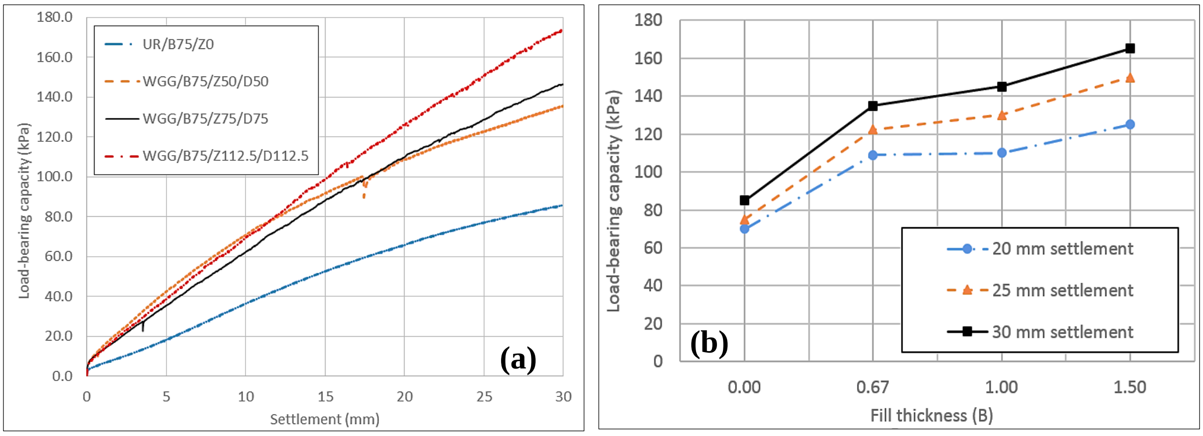

Figure 97: (a) Load-bearing capacity against settlement, and (b) Load-bearing capacity against fill thickness for the woven geogrid at the interface using the 75 mm footing.

The trend of optimum fill thickness of 75 mm was also observed in results from series configuration 2. Figure 98 (a) represents results for the woven geotextile placed at the interface of the two soils. There was an observed improvement in the load-bearing capacity from 90% for the 50 mm fill thickness to 140% for the 75 mm fill thickness. However, there was an observed reduction in improvement to 100% for the 112.5 mm fill thickness.

From Figure 98 (b) the trends of settlement showed that as the fill thickness to width ratio was increased there was a subsequent increase in the load-bearing capacity. However, beyond the thcikness to width ratio of 1.0B there was a reduction in the load-bearing capacity. As such the optimum thickness to width ratio was 1.0B, which was similar to the results for both the woven geotextiles in series configuration 1, shown in Figures 94 and 95.

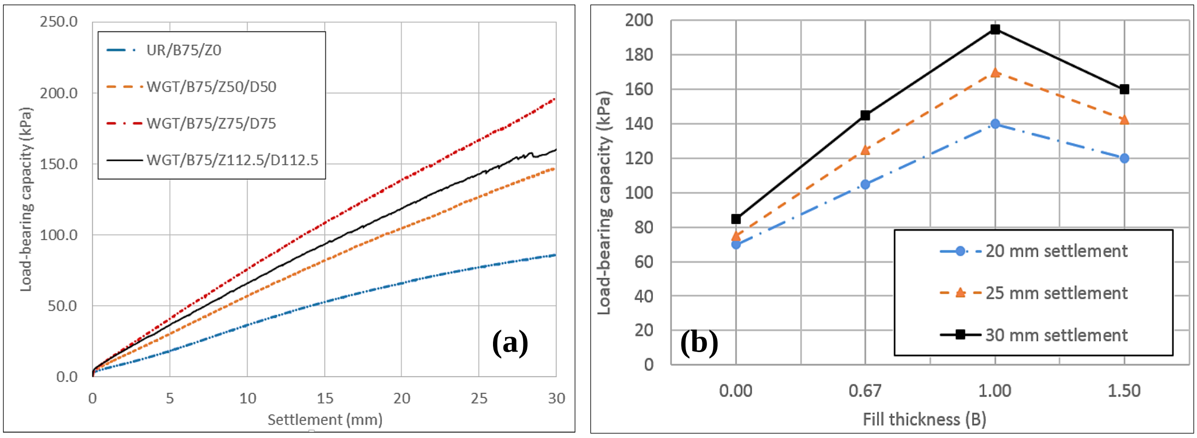

Figure 98: (a) Load-bearing capacity against settlement, and (b) Load-bearing capacity against fill thickness for the woven geotextile at the interface using the 75 mm footing.

Figure 99 (a) is a representation of the results obtained for the extruded geogrid placed at the interface of the soils as the fill thickness was varied. There was an improvement from 175 kPa to 200 kPa for fill thickness and depth of placement of 50 mm and 75 mm respectively. However, there was a reduced improvement to 155 kPa for the fill thickness and depth of placement of 112.5 mm. This gave improvement benefits of 105%, 135%, and 80%, respectively, which showed the optimum fill thickness was 75 mm.

The trends for settlements of 20 mm, 25 mm and 30 mm in Figure 99 (b) show that when the granular thickness to width ratio was increased, there was an increase in the load-bearing capacity. This was evident up to the ratio of 1.0B beyond which there was a reduction in the load-bearing capacity. As such the optimum fill thickness to footing width was 1.0B.

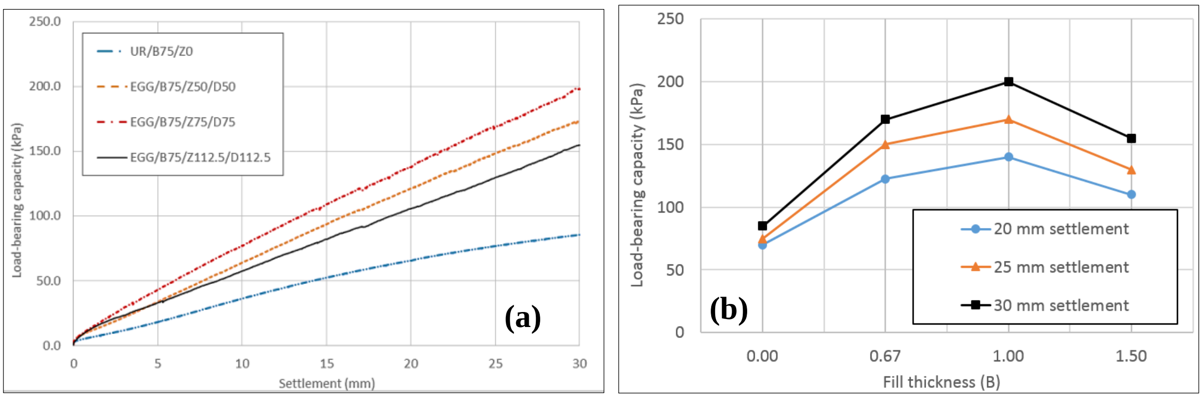

Figure 99: (a) Load-bearing capacity against settlement, and (b) Load-bearing capacity against fill thickness for the extruded geogrid at the interface for 75 mm footing using the 75 mm footing.

The observed reduction in the strength of the composite could be attributed to shear failure occurring in the granular fill before the reinforcement from the geogrid was mobilized for the case of the 112.5 mm thickness.

Figure 100 (a) shows the results for the extruded geogrid placed at the interface of the two soils, with changes in the fill thickness. There was an observed improvement in the load-bearing capacity from 95 kPa for the unreinforced clay to 115 kPa, 130 kPa and 140 kPa for increasing fill thicknesses of 50 mm, 112.5 mm and 150 mm respectively. This gave improvements of 20%, 35% and 50% respectively, showing that the optimum fill thickness was 150 mm. The trends in Figure 100 (b) show that when the granular thickness to width ratio was increased, there was an increase in the load-bearing capacity. As such the optimum fill thickness to footing width was 1.0B.

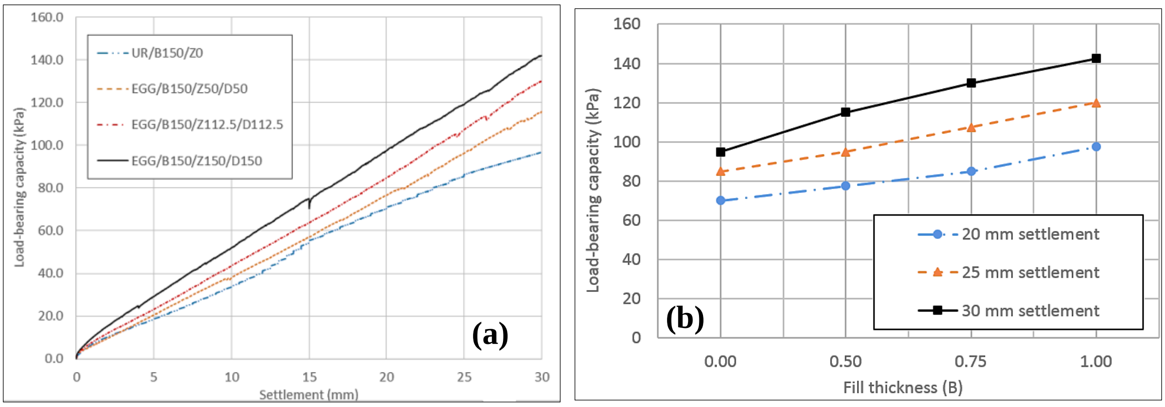

Figure 100: (a) Load-bearing capacity against settlement, and (b) Load-bearing capacity against fill thickness for the extruded geogrid at interface using the 150 mm footing.

The trend of improving strength with increasing fill thickness could be observed in the other results presented in the Appendix. The optimum fill thickness was dependent on the footing size and the geosynthetic product used as the reinforcing layer. The obtained optimum thickness to width ratio ranged from 1.0B – 1.5B.

7.3.3. Base/Subbase Thickness Reduction

The benefit of reducing the base/subbase aggregate thickness is defined by the base-course reduction ratio (BRR). The BRR defines the percentage reduction in the base course (or subbase) thickness of a reinforced pavement. The thicknesses of the base course are selected at the same bearing pressure, and the BRR determined using the equation:

$$BRR = \frac{D_{2(U)} - D_{2(R)}}{D_{2(U)}} \tag{29}$$

Where: $D_{2(U)}$ - is the thickness of the unreinforced base course

$D_{2(R)}$ - is the thickness of the reinforced base course

The range of reduction of base/subbase thickness is dependent on the initial thickness of the unreinforced fill. In the case of an unreinforced 150 mm fill, there is a possible reduction ranging from 25% - 67% that correspond to the fill thickness being reduced within the range of 112.5 mm – 50 mm as a result of geosynthetic inclusion. Figure 101 shows the reduction in thickness due to a woven geogrid inclusion at the interface of the two soils.

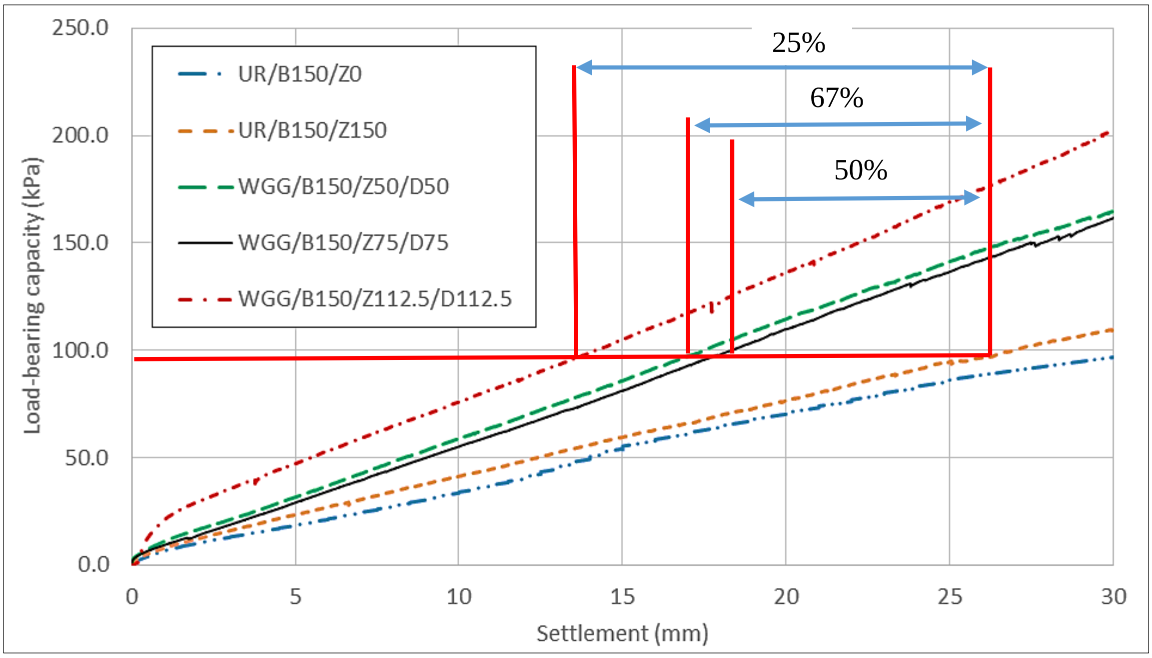

Figure 101: Load-bearing capacity against settlement for a woven geogrid placed at the interface.

For an initial unreinforced fill thickness of 112.5 mm, the range of reduction could be 33% - 56% that correspond to a reduction in the range of 75 mm – 50 mm. Figure 102 shows the reduction in thickness due to a non-woven geotextile inclusion at the interface of the two soils.

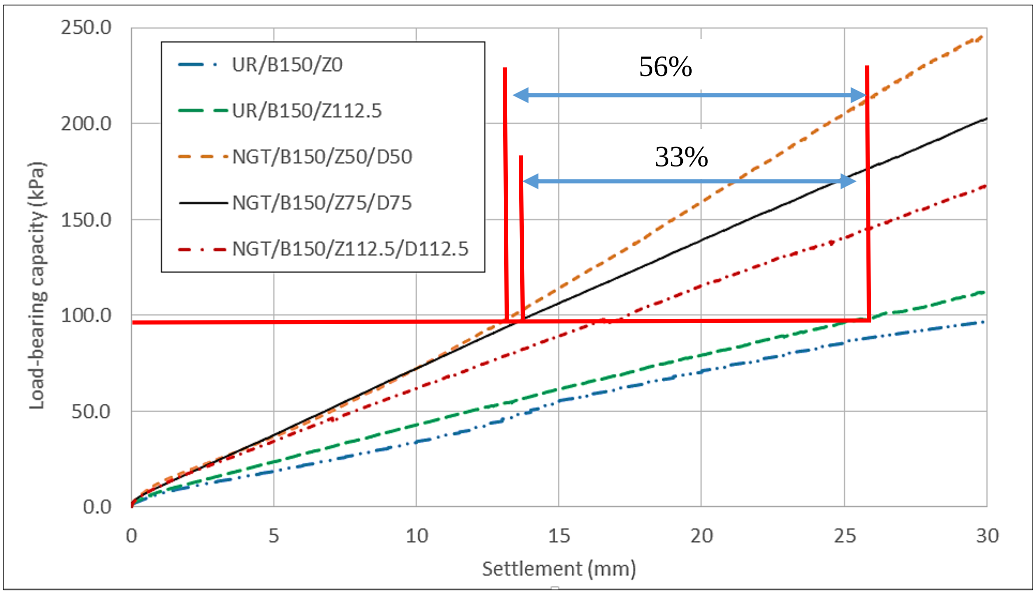

Figure 102: Load-bearing capacity against settlement for a non-woven geotextile placed at the interface.

For a 75 mm fill thickness the anticipated reduction in base/subbase would be 33% which is for a reduced fill of 50 mm with geosynthetic reinforcement included. In the case of a 50 mm fill no reduction is expected, as it is the minimum required for anchorage of the geosynthetic within the material, and also prevents extrusion of the reinforcement layer as loading was applied.

7.4. Depth of placement of geosynthetic layer

In determination of the optimum depth of placement of the geosynthetic layer, the fill thickness was kept constant and the reinforcement layer depth varied.

From Figure 103 (a) it was observed that for the fill thickness of 75 mm there was an improvement in the load-bearing capacity from 85 kPa for the unreinforced clay to 135 kPa for the unreinforced two-layered soil. On inclusion of the woven geogrid at 50 mm depth there was an increase to 160 kPa, however, as the depth of the reinforcement was increased to 75 mm, there was a reduction in the improvement to 145 kPa. This corresponds to benefits of 90% and 70% respectively, which showed that the optimum depth of placement of the woven geogrid was at the depth of 50 mm. Figure 103 (b) showed that there was an observed increase in the load-bearing capacity due to granular reinforcement (GR). There was a further increase in the strength when the woven geogrid was included in the soil structure at a depth to width ratio of 0.67B. However, there was a reduction in the improvement when the depth to width ratio was increased to 1.0B. This gave an optimum depth to footing width ratio of 0.67B.

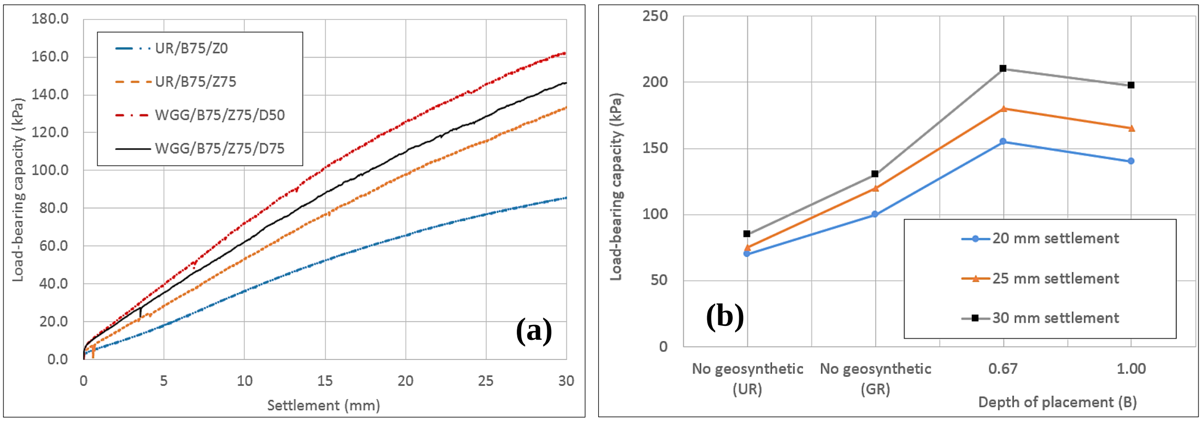

Figure 103: (a) Load-bearing capacity against settlement, and (b) Load-bearing capacity against depth of placement for the woven geogrid using 75 mm footing.

The trend of the optimum depth of placement was also observed when the results were observed for the woven geotextile and 112.5 mm fill thickness. As seen in Figure 104 (a), there was an improvement in the load-bearing capacity from 85 kPa for the clay subgrade to 125 kPa for the unreinforced two-layered soil, which gave a percentage improvement of 47%. When the woven geotextile was placed at the varying depths of 50 mm, 75 mm and 112.5 mm there were increases in the load-bearing capacities to 190 kPa, 165 kPa and 160 kPa respectively. This gave improvements of 120%, 95% and 90% respectively that showed optimum placement of the reinforcement layer at 50 mm depth.

The trends shown in Figure 104 (b) are for the selected vertical displacements of 20 mm, 25 mm and 30 mm. From the graph there is an evident increase in the load-bearing capacity due to inclusion of a granular reinforcement (GR), and a further increase when the woven geotextile is placed in the two-layered soil structure. However, as the depth of placement of the geotextile is increased there was a reduction in the load-bearing capacity. This indicated that the optimum improvement was at the depth to footing width ratio was 0.67B.

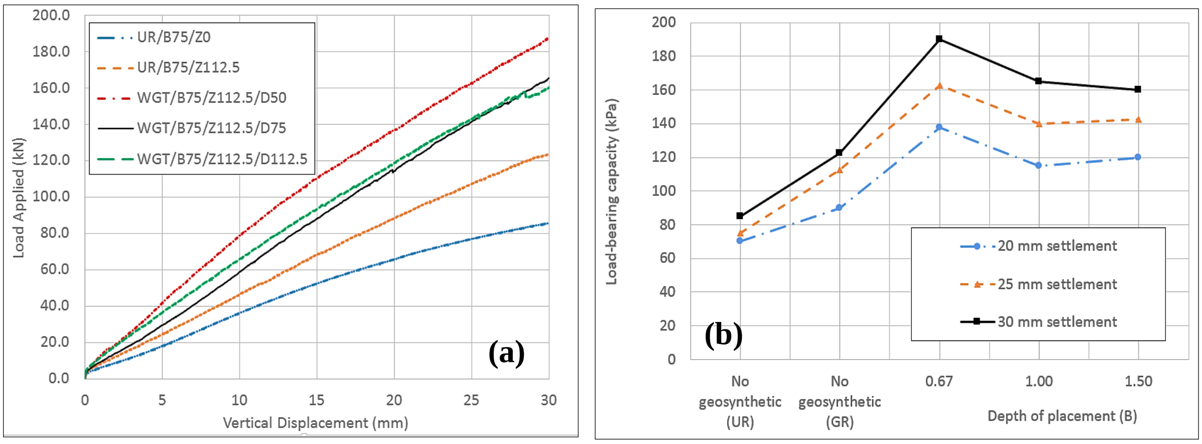

Figure 104: (a) Load-bearing capacity against settlement, and (b) Load-bearing capacity against depth of placement for the woven geotextile for a constant granular thickness of 112.5 mm using the 75 mm footing.

The observed trend of reduced improvement could be attributed to shear failure within the granular fill layer before the reinforcement benefit from the woven geotextile was mobilized. However, for the extruded geogrid there was a difference in the trend observed with the 75 mm thickness having the greatest improvement. The results in Figure 105 (a) show that there was improvement in the load-bearing capacity to 185 kPa for the extruded geogrid placed at 50 mm depth and to 200 kPa when placed at 75 mm depth. This corresponded to benefit ratios of 115% and 135 % respectively. However, from Figure 105 (b) it was observed that there was no substantial increase in load-bearing capacity with an increase in the depth to footing width ratio. Therefore the optimum depth of placement to width ratio of 0.67B.

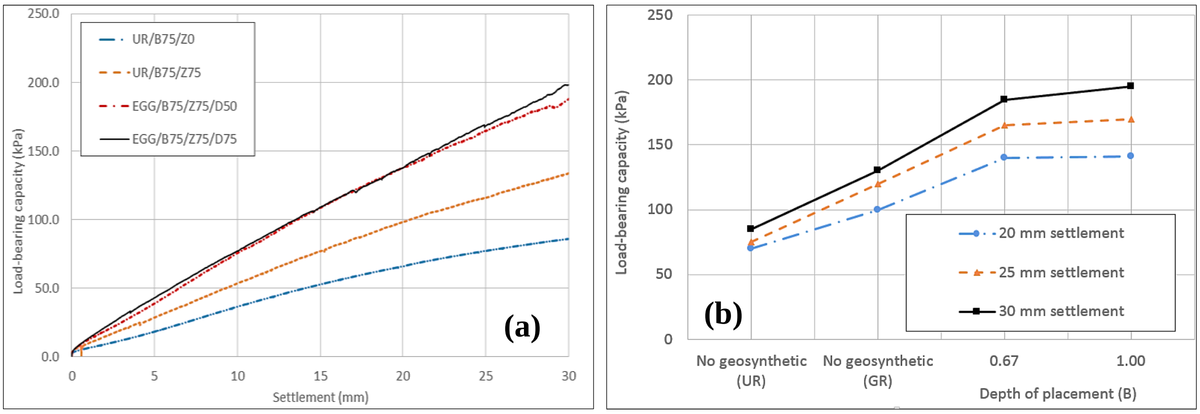

Figure 105: (a) Load-bearing capacity against settlement, and (b) Load-bearing capacity against depth of placement for the extruded geogrid for a constant granular thickness of 75 mm using the 75 mm footing

The results for the non-woven geotextile tested with the 75 mm footing, a constant fill thickness of 112.5 mm and variations in depths of placement of the reinforcement layer are shown in Figure 106 (a). The observed trend was that the load-bearing capacity for the 50 mm, 75 mm and 112.5 mm depth of placement resulted in improvements to 160 kPa, 155 kPa and 160 kPa respectively, which gave benefits of 85%, 80% and 85% respectively.

The selected appropriate depth of placement in the design and construction phase would depend on the soil materials used. If there is a high susceptibility of mixing of the two soils, then the placement at a depth of 112.5 mm that corresponds to the soil interface would be advised so as to act as a separation layer in addition to the reinforcement benefit. However, if the loads expected and the frequency of application are both high, then placement of the non-woven layer at 50 mm would be advised, so that the reinforcement benefit could be mobilized early. These would give depth to width ratios of 1.5B and 0.67B respectively.

From the trends shown in Figure 106 (b) obtained it could be observed that similar load-bearing capacities were obtained for the 0.67B and 1.5B depths of placement to footing width ratio. When the non-woven geotextile is placed at 0.67B, the increase in the load-bearing capacity could be attributed to reinforcement provided by the geotextile as the tensile strength of the layer is mobilized through the tensioned-membrane effect. As for placement of the geotextile at the interface of the two soils, at 1.5B depth, the reinforcement is provided mostly by the granular fill, with the geotextile providing a separation function.

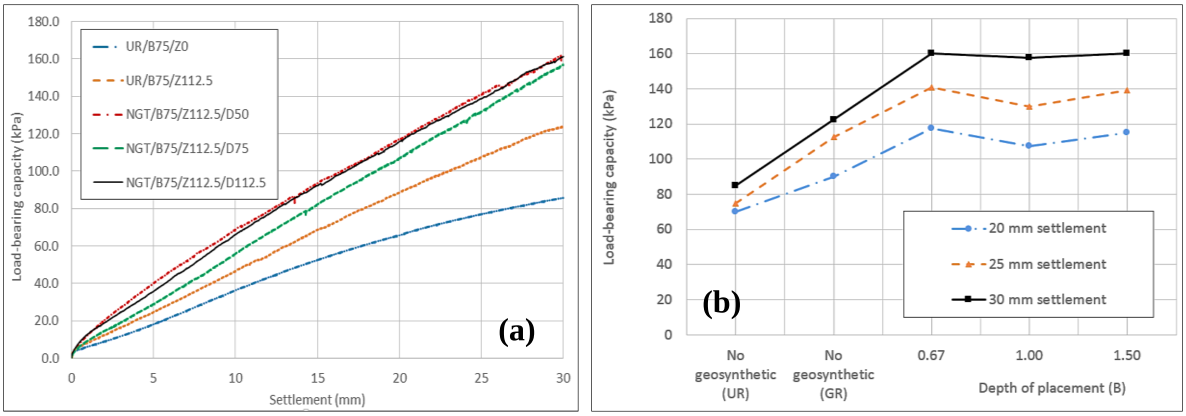

Figure 106: (a) Load-bearing capacity against settlement, and (b) Load-bearing capacity against depth of placement for the non-woven geotextile for a constant granular thickness of 112.5 mm using the 75 mm footing.

From all the results obtained and the trends seen, there was an evident greater increase in the load-bearing capacity of the reinforced soil composite when the geosynthetic is placed within the fill layer as opposed to at the interface. This could be attributed to the mobilization of the added reinforcement provided by the geosynthetic layer at an earlier stage. This would lead to spread of the applied load through the fill material that has a higher strength and thus providing more support to the structure. As such there is a reduction in the stress transferred to the weaker clay subgrade, resulting in less destabilization of the structure allowing for greater loads to be exerted on the soil. This corresponds to an increase in the load-bearing capacity of the soil. This phenomenon is illustrated below in Figure 107. The incorporation of the geosynthetic layer within the fill is to provide reinforcement through the tensioned membrane effect in geotextiles and also in woven geogrids that stabilize the soil structure.

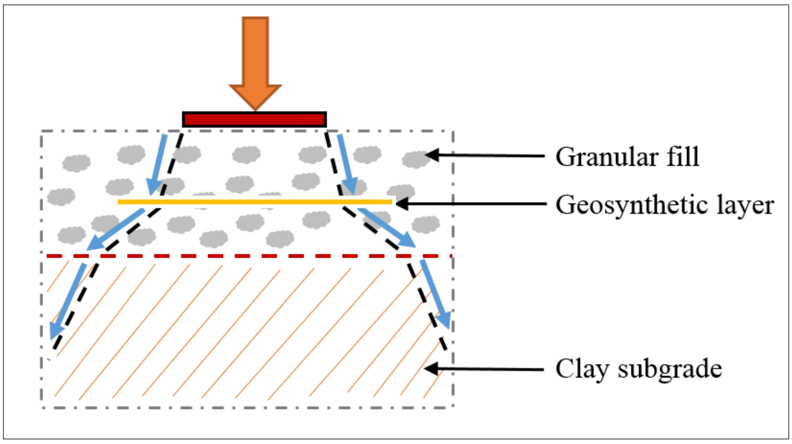

Figure 107: Illustration of the increased load distribution when the geosynthetic layer is placed within the fill layer.

Figure 108 shows the distribution of the applied load through the composite when the geosynthetic layer is placed at the interface. In the results obtained it was observed that with this configuration there was a reduced improvement in the load-bearing capacity as the fill thickness was subsequently increased. This could be attributed to the high loads resulting in shear failure within the fill layer before the geosynthetic reinforcement at the interface is mobilized to provide the additional support. Also at the time the geosynthetic at the interface is reached the failure that would have occurred in terms of shear failure in the fill section and settlement cannot be reversed. The structure could then be rendered to have failed, with maintenance necessary.

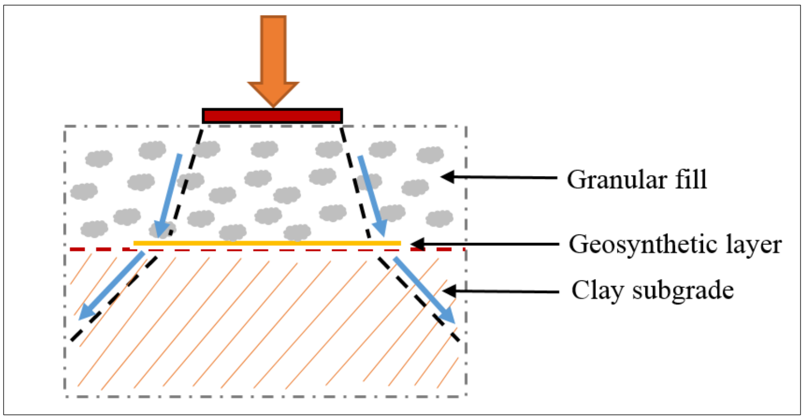

Figure 108: Illustration of the increased load distribution when the geosynthetic layer is placed at the interface

From the test results it was observed that the optimum depth of placement of the geosynthetic layer depended on the product used, and the obtained depth to width ratio was 0.67B.

7.5. Model footing size

An increase in the footing size led to a general increase in the load-bearing capacity of the reinforced soil composite. This was attributed to the greater distribution of the applied load through the soil that led to increased support. This was observed in the results obtained for the different geosynthetics tested with varying configurations.

7.5.1. Woven Geogrid

Figure 109 shows the comparison of the results for the (a) 75 mm footing and (b) 150 mm footing when the woven geogrid was placed at varying depths in a constant fill thickness of 75 mm. From the graphs it was observed that there was an increase in the load-bearing capacities from the range of 85 – 160 kPa for the 75 mm footing to 95 – 190 kPa for the 150 mm footing.

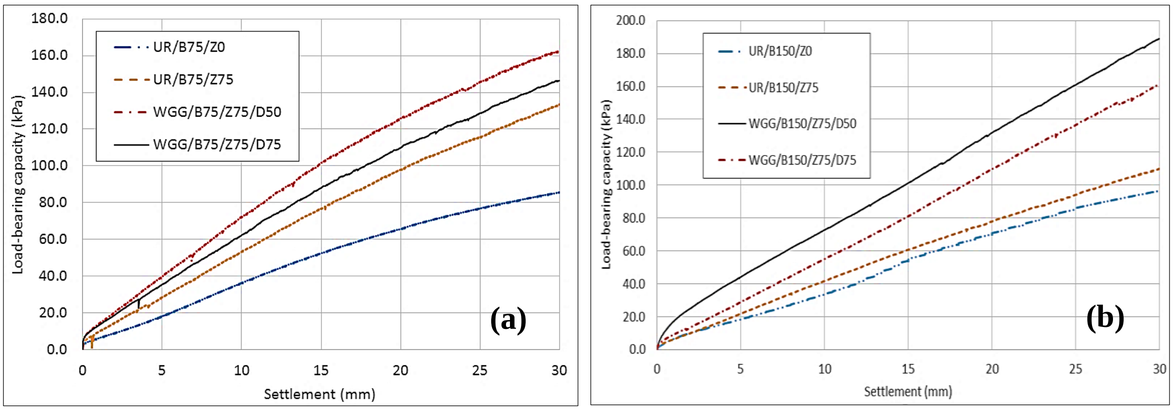

Figure 109: Load-bearing capacity against settlement showing the comparison of the results for the 75 mm and 150 mm footings when the woven geogrid was placed at varying depths in a constant fill thickness of 75 mm.

7.5.2. Extruded Geogrid

Figure 110 shows the comparison of the results for the (a) 75 mm footing and (b) 150 mm footing when extruded geogrid placed at the interface of the soils. It was observed that there was a reduction in the range of load-bearing capacities from 85 – 200 kPa to 95 – 140 kPa for 75 mm and 150 mm footings respectively.

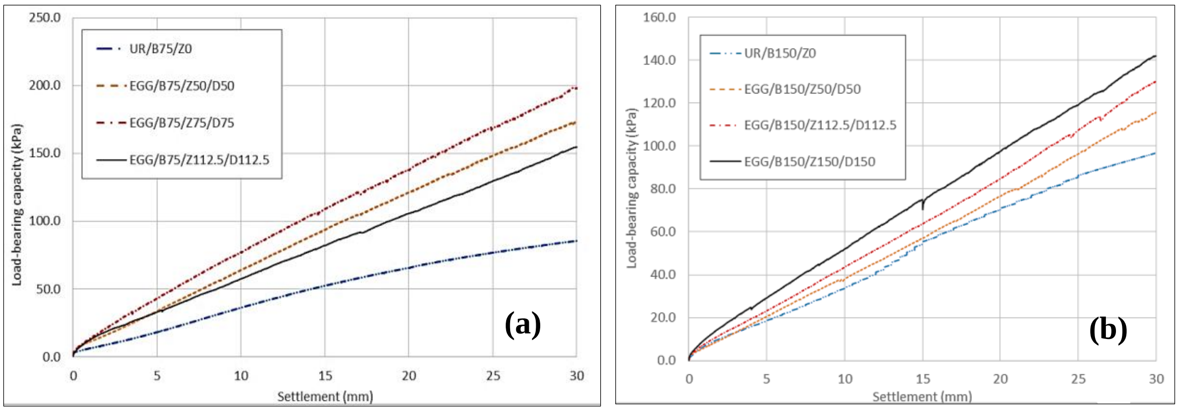

Figure 110: Load-bearing capacity against settlement showing the comparison of the results for the 75 mm and 150 mm footings when extruded geogrid placed at the interface of the soils.

7.5.3. Woven Geotextile

Figure 111 shows the comparison of the (a) 75 mm footing and (b) 150 mm footing when the woven geotextile was placed at the interface of the soils. From the graphs there was an observed improvement in the range of load-bearing capacities from 85 – 200 kPa to 95 – 155 kPa for the 75 mm and 150 mm footings respectively.

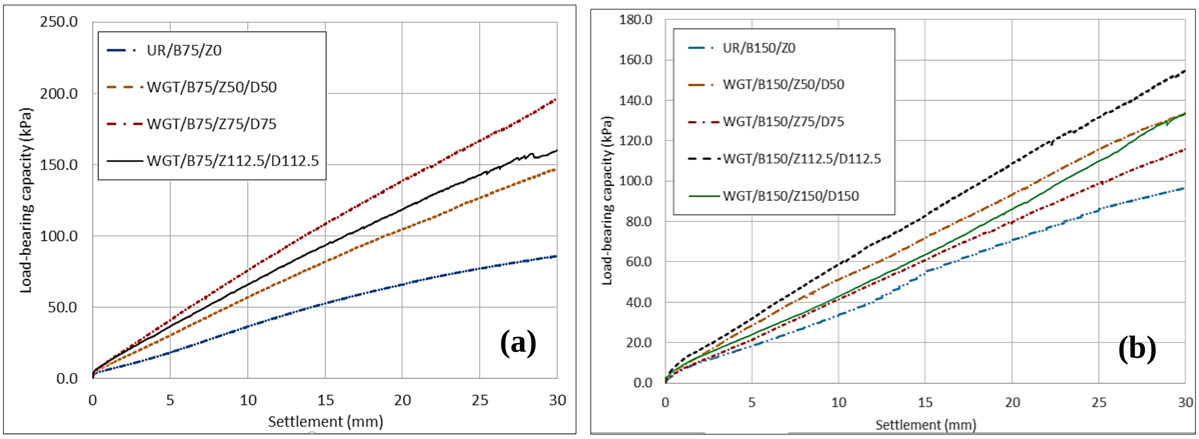

Figure 111: Load-bearing capacity against settlement showing the comparison of the results for the 75 mm and 150 mm footings when the woven geotextile was placed at the interface of the soils.

7.5.4. Non-woven Geotextile

Figure 112 shows the comparison of the (a) 75 mm footing and (b) 150 mm footing when the non-woven geotextile was placed at varying depths of a 75 mm fill thickness. From the graphs there was an observed improvement in the load-bearing capacities from the range of 85 – 150 kPa for the 75 mm footing to 95 – 200 kPa for the 150 mm footing.

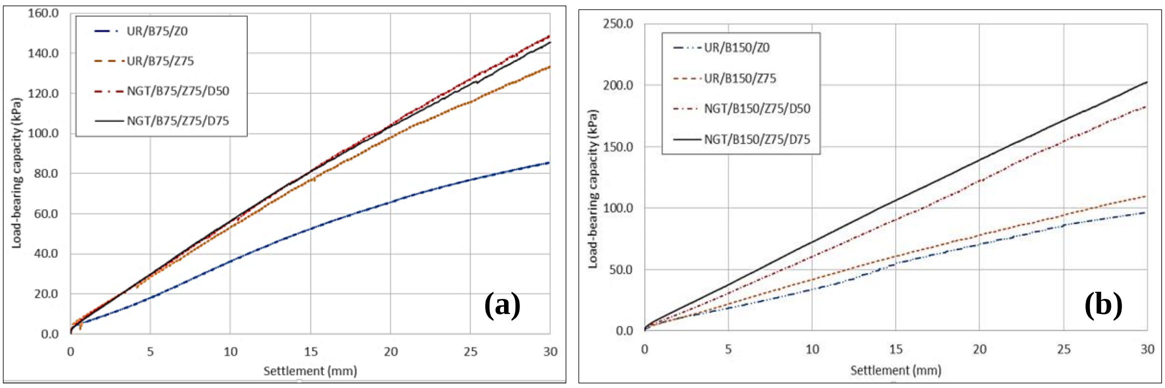

Figure 112: Load-bearing capacity against settlement showing the comparison of the results for the 75 mm and 150 mm footings when the non-woven geotextile was placed at varying depths of a 75 mm fill thickness.

When the results were compared for all the test results presented in the appendix there was an observed general improvement in the load-bearing capacity as the footing size was increased. This was attributed to increased load distribution with increase footing size, which led to an increased support from the soil particles to the applied load through resistance to failure of the composite structure. However, some of the comparisons showed a reduction in improvement in the load-bearing capacity as the footing size increased, which could be attributed to increased weight from the footing.

7.6. Type of Geosynthetic Product

There were different types of geosynthetic products tested that included woven geotextiles; non-woven geotextiles; extruded geogrids and woven geogrids. Each of these products had different ranges of reinforcement benefits as they had differing properties. The property of interest in this study was the tensile strength of the geosynthetics, as soil is strong in compression and weak in tension, while the geosynthetics are weak in tension but strong in tension. This made them desirable in providing reinforcement to the soil structure as the particles are dispersed laterally as loads are applied onto it with the development of shear failure in the structure. The inclusion of the geosynthetic would act to confine the soil particles thus increasing the stability of the structure and increasing the strength of the soil allowing for support of greater loads. To determine which geosynthetic product provided the most desirable reinforcement benefit, the results obtained were compared and analysed.

7.6.1. Geotextiles

Geotextiles are primarily used for their separation function, however, they also provide adequate reinforcement benefits when included in the soil structure. The results presented are for the woven and non-woven geotextiles tested when placed at the interface of the soils.

50 mm fill thickness and depth of placement (at interface)

From the results obtained as shown in Figure 113 it was observed that the non-woven geotextile had a greater increase in the load-bearing capacity than the woven geotextile, with improvements of 110% and 70% respectively.

The non-woven geotextile had a greater improvement at the depth of 50 mm as it provided a greater tensioned membrane effect as it was capable of undergoing higher strains than the woven geotextile. In addition, as the non-woven geotextile deformed there was the possible friction between its surface and the soil particles as they were embedded into it, which would have been lacking in the woven geotextile as it had a smoother surface.

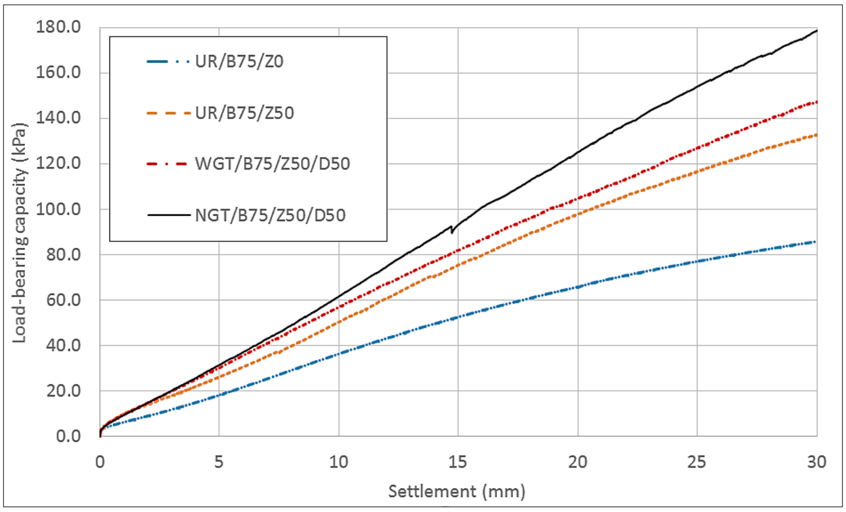

Figure 113: Load-bearing capacity against settlement for geotextiles placed at the interface of a 50 mm fill thickness

75 mm fill thickness and depth of placement (at interface)

From Figure 114 the woven geotextile had a greater increase in the load-bearing capacity than the non-woven geotextile, with improvements of 130% and 70% respectively. This could be attributed to the reinforcement having been partially provided by the granular fill of 75 mm thickness and partially by the geotextile. Given that the woven geotextile had a greater tensile modulus than the non-woven geotextile, a higher load-bearing capacity is obtained from it.

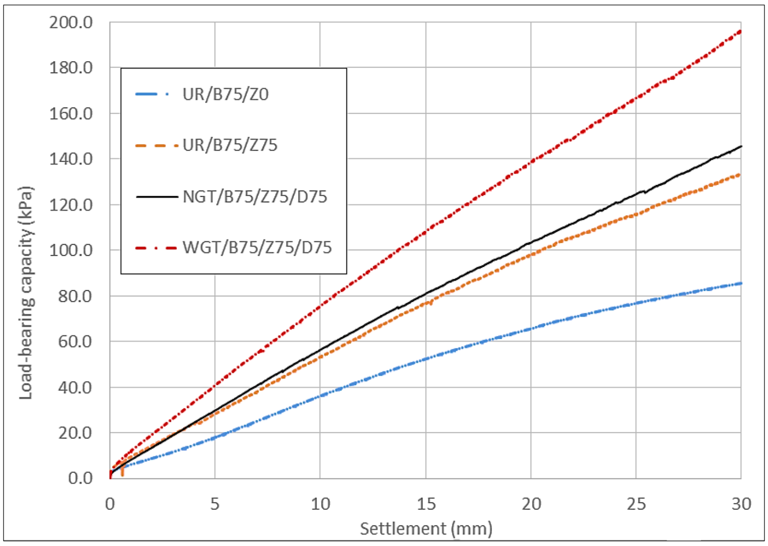

Figure 114: Load-bearing capacity against settlement for geotextiles placed at the interface of a 75 mm fill thickness

112.5 mm fill thickness and depth of placement (at interface)

From Figure 115 it was observed that the woven and non-woven geotextiles had similar improvements in the load-bearing capacity of 90%. The similar load-bearing capacities could be attributed to the reinforcement being provided by the 112.5 mm granular fill, with the geotextiles only contributing to separation of the soil layers and minimal reinforcement.

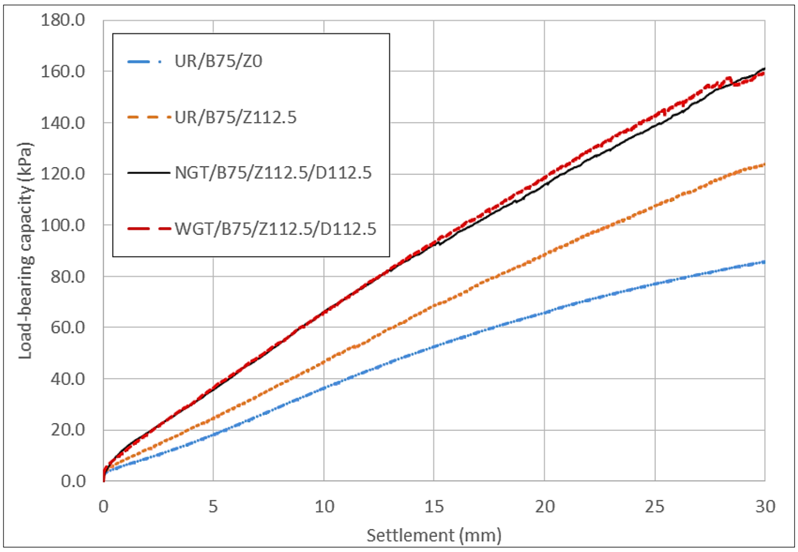

Figure 115: Load-bearing capacity against settlement for geotextiles placed at the interface of a 112.5 mm fill thickness

7.6.2. Geogrids

Geogrids are primarily used for reinforcement, however they also have some separation benefits. Performance of geogrids for reinforcement relies on the rigidity of the material; high tensile modulus; and aperture size in relation to the soil particle size, which allows for interlocking to occur. The results presented below show the comparison of tests conducted using extruded and woven geogrids.

50 mm fill thickness and depth of placement (at interface)

From Figure 116 it was observed that the extruded geogrids as compared to the woven geogrid had a greater improvement in the load-bearing capacity of 100% and 60% respectively.

The extruded geogrid had a greater improvement as it was able to interlock the granular particles in its apertures creating a composite structure. This restrained the soil particles laterally and prevented shear failure from occurring in the fill layer and the soil structure as a whole. As such it allowed the transfer of the tensile strains into the extruded geogrid, which had a high tensile modulus making it possible for it to support greater loads.

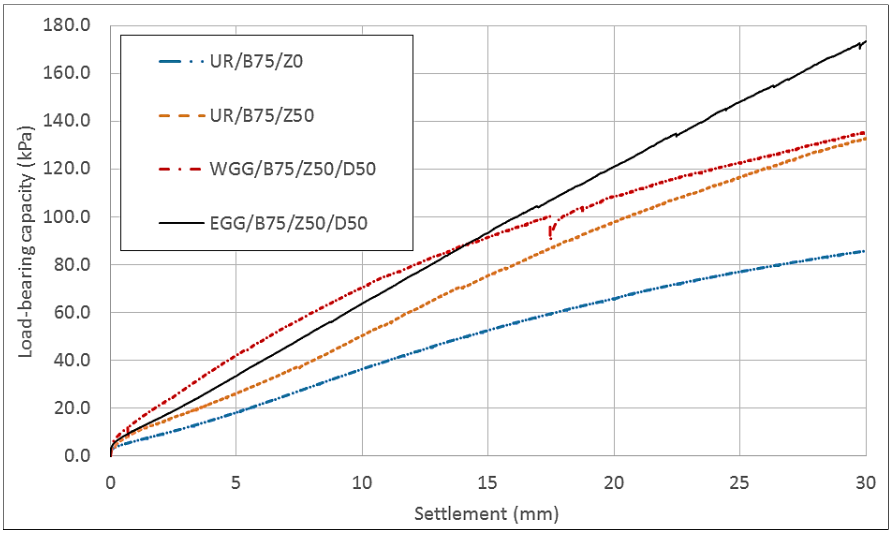

Figure 116: Load-bearing capacity against settlement for geogrids placed at the interface of a 50 mm fill thickness

75 mm fill thickness and depth of placement (at interface)

From Figure 117 it was observed that there were improvements in the load-bearing capacity of 135% and 70% for the extruded and woven geogrids respectively. The increase from the 75 mm fill thickness result could be attributed to the added reinforcement being provided partially by the granular material and the geogrids.

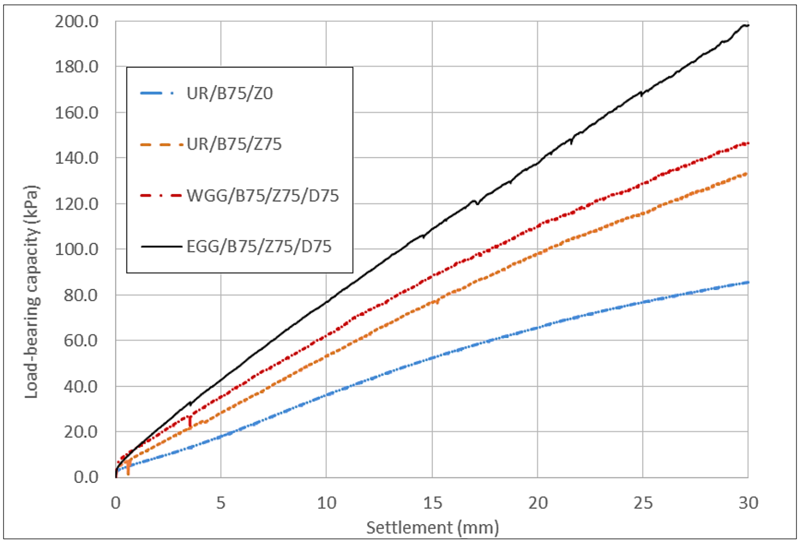

Figure 117: Load-bearing capacity against settlement for geogrids placed at the interface of a 75 mm fill thickness

7.6.3. Geotextiles vs Geogrids

When the results of the geogrids and geotextiles were compared it was observed that the geogrids generally performed better than the geotextiles. This was attributed to the stiffness of the geogrids being greater than that of the geotextiles allowing for greater tensile strains transferred to the geogrids and thus higher loads supported, which leads to an increase in the load-bearing capacity of the composite. Figures 118 – 120 show the comparison of results for tests conducted on geogrids and geotextiles placed at the interface of the soils.

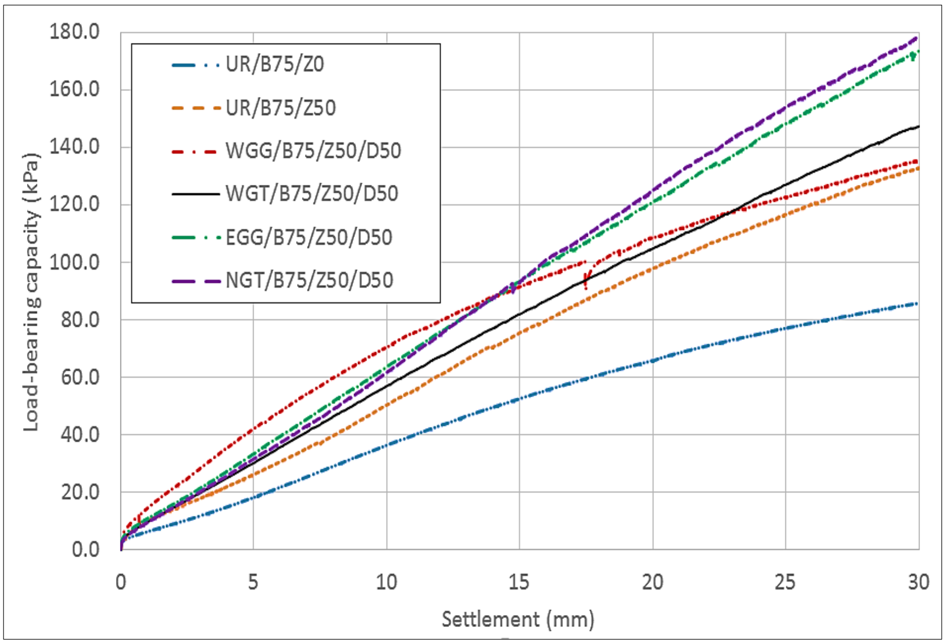

Figure 118: Load-bearing capacity against settlement for comparison of geogrids and geotextiles placed at the interface of a 50 mm fill thickness

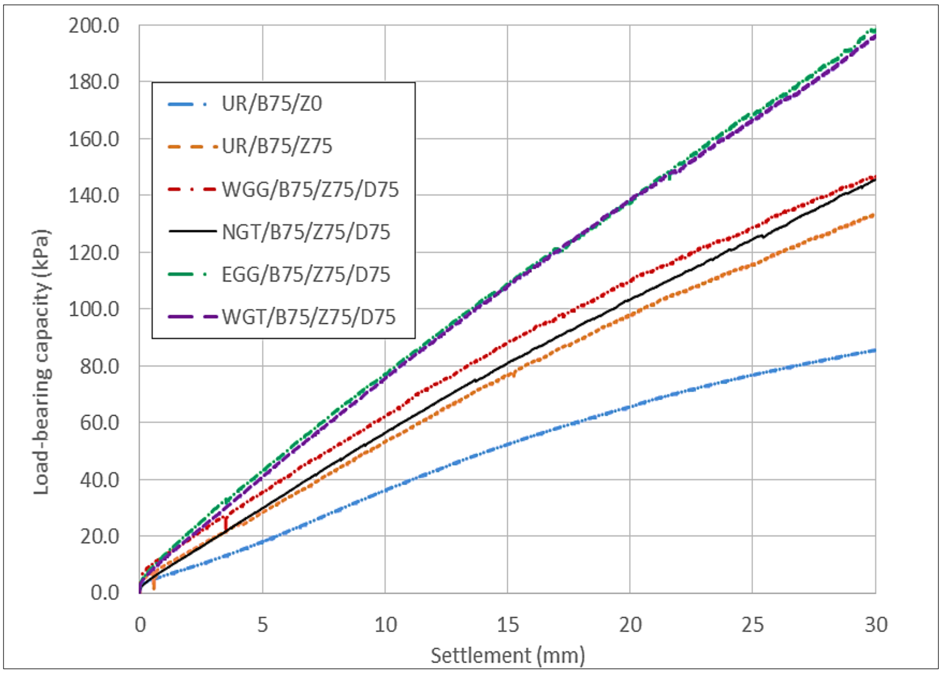

Figure 119: Load-bearing capacity against settlement for comparison of geogrids and geotextiles placed at the interface of a 75 mm fill thickness.

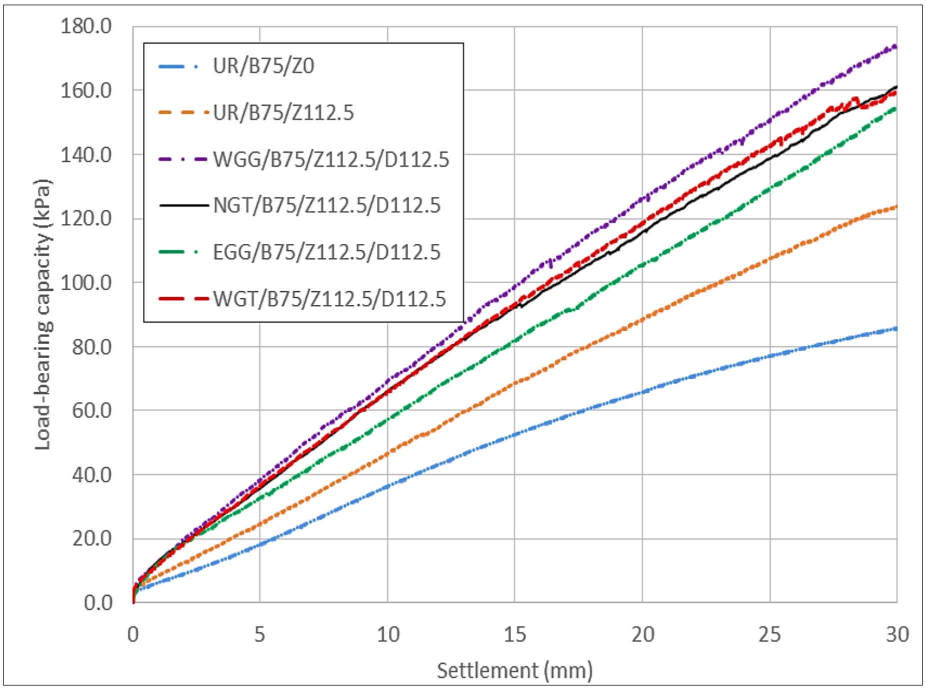

Figure 120: Load-bearing capacity against settlement for comparison of geogrids and geotextiles placed at the interface of a 112.5 mm fill thickness.

In general, the geogrids gave greater improvements in the load-bearing capacity as the tensile modulus of geogrids were greater than that of geotextiles. However, the non-woven appeared to perform better than the other geosynthetic products in some configuration comparisons, which was an anomaly as geogrids have higher stiffness thus expected to provide greater reinforcement benefits to soil. This could be attributed to the lack of anchorage of the non-woven geotextile, allowing it to move with load application, and thus the friction between the geotextile and the soil particles contributing to the additional strength gains. The non-woven geotextiles were also able to take greater strains than the other geosynthetic products, which allowed for greater deformations in the soil structure.

7.7. Summary of Analyses

The benefits of reinforcement provided by the geosynthetics could be associated with the mechanisms of soil-reinforcement interaction that include: the tension membrane action; and lateral restraint of soil particles. These mechanisms lead to the alteration of failure surfaces with a resultant improvement in the load-bearing capacity, and added benefit of reduction in settlements.

The lateral restraint of particles is attained mainly through interlocking of the soil particles in the geosynthetic apertures, which entails that this action was greater in geogrids as they possessed apertures making it possible for the interlocking to occur. The lateral restraint of soil particles is shown in Figure 121. However, in the case that the friction in the geosynthetic layer was high, there was development of interlocked particles through the friction between the geosynthetic and the soil. This was achieved in the non-woven geotextile that added to the reinforcement benefits. This phenomenon was discussed in Section 3.10 that described the soil-geosynthetic interaction.

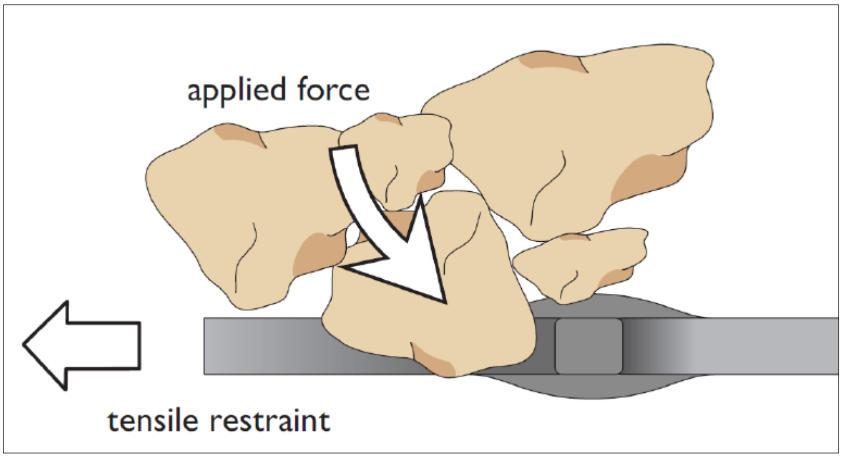

Figure 121: Lateral restraint of soil particles due to geosynthetic layer (TENSAR International)

The tensioned membrane action is mainly mobilised in geotextiles, however, woven geogrids are also able to activate this action as they are flexible. In addition, it is usually only beneficial if deformations occur in the soil structure before the tensioned membrane action can be activated. As such, the depth of placement of the reinforcement is critical to allow for this action to take place. From Figure 122 it can be seen that when there is no geosynthetic reinforcement, the stresses in the granular base and subgrade are equal, which shows that the applied load is transferred through to the softer subgrade layer below. However, on inclusion of the reinforcement there is a reduction in the stress in the subgrade as there is effective vertical support by the geosynthetic.

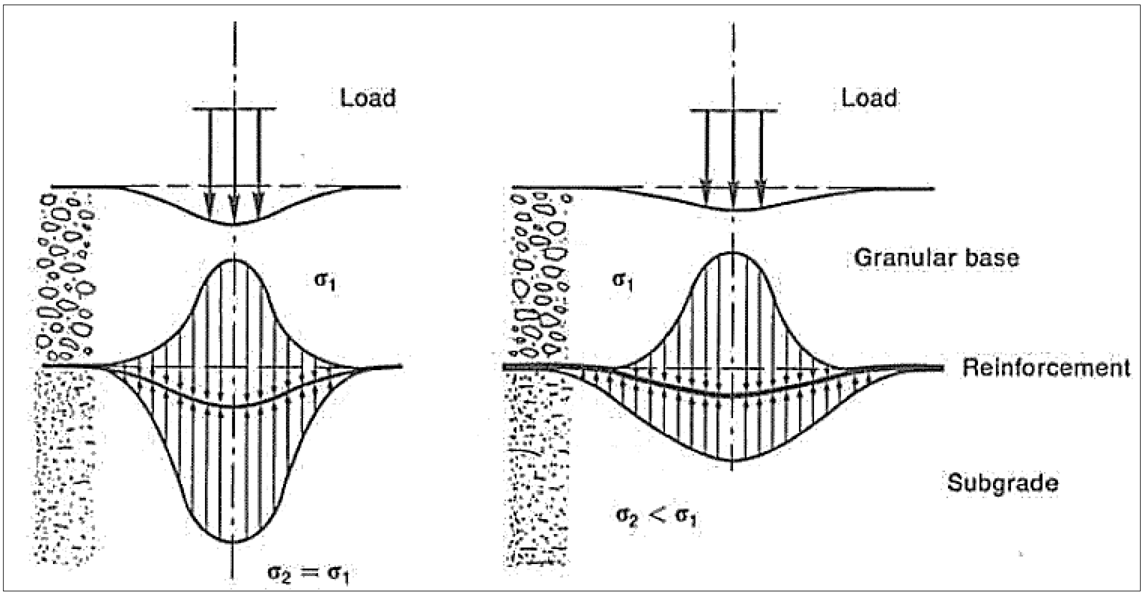

Figure 122: Illustration of the tensioned membrane action of geosynthetic reinforcement (Bourdeau and Ashmawy, 2012).

The increase in load-bearing capacity with increasing settlement is due to mobilization of more of the reinforcement mechanisms of the geosynthetic. These mechanisms include lateral restraint and tensioned membrane effect that are particularly activated with high settlements. At low vertical displacements, the aggregate particles are still intact and are providing the support to the footing as the load is applied. At higher vertical displacements, the aggregate particles are destabilized more, being laterally distributed with an increase in outward shear stress. This shear stress is transferred to the geosynthetic membrane that has high tensile strength, thus resisting the lateral distribution keeping the soil structure stable, hence leading to an increase in the load-bearing capacity.

According to Burd (1986) the restraint effect is proposed to be in two parts. When vertical displacements become appreciable, the reinforcement retrains heave deformation of the subgrade on each side of the load by membrane action associated with reverse curvature of the reinforcement. The reinforcement applies an additional surcharge loading to the subgrade which increases the vertical bearing capacity under the load. Conversely, the reinforcement reduces the tensile strain at the base of the fill, and improves the load-spread action of the layer thus reducing the magnitude of the vertical stresses at the base of the fill. This also results in an increase in the load-bearing capacity of the soil.

This chapter has shown the different benefits from geosynthetic reinforcement through the analyses of the results obtained from the tests conducted with the various parameters taken into consideration. From the results and analyses of the data, the optimum depth of placement was 0.67B. The thickness of the granular layer was also important, when the configuration involved placement of the geosynthetic at the interface of the soils. The optimum granular thickness ranged from 1.0B – 1.5B.

CHAPTER 8

8. Practical Applications of Geosynthetics

The purpose of this chapter is to show the different aspects of reinforcement with geosynthetics that can be applied practically. The equations for determination of the bearing capacity of an unreinforced soil and a geosynthetic reinforced two-layered soil composite as presented in Section 2.3.4 were used. These included; Hansen's method for two layered soils; the projected area method, and Chen's method.

Hansen's method for two layered soils

Hansen's method involved determining the average values of: cohesion ($\bar{c}$), angle of internal friction ($\bar{\varphi}$), and unit weight of the soils ($\bar{\gamma}$), and the equivalent significant depth, z_{max} for the layered soils. The equations presented in Section 2.3.4.1 were used in the verification of the Hansen's method against the experimental results.

For the 75 mm footing;

B = 0.075 m

L = 0.014 m.

h_{subgrade} = 0.25 m

z_{max} = $h_{fill} + h_{\text{subgrade}}$

$\gamma_{fill} = 21.952\ kN/m^3$

$\gamma_{subgrade} = 15.484$ kN/m³

For granular thickness of 50 mm;

$h_{fill} = 0.05\ m$;

$z_{max} = 0.05 + 0.25$ = 0.3 m

$$\bar{\gamma} = \sum_{i=1}^{n} \gamma_i\, h_i / z_{max} = \frac{21.952 \text{ x } 0.05 + 15.484 \text{ x } 0.25}{0.3} = 16.562 \text{ kN}/m^3$$

$$\bar{c} = \sum_{i=1}^{n} c_i\, h_i / z_{max} = \frac{(25\ x\ 0.05 + 4.7\ x\ 0.25)}{0.3} = 8.083\ kN/m^2$$

$$\bar{\varphi} = \sum_{i=1}^{n} \varphi_i\, h_i / z_{max} = \frac{(35\ x\ 0.05 + 17.4\ x\ 0.25)}{0.3} = 20.33°$$

$$s_c = 1 + \frac{N_q}{N_c}\frac{B}{L} = 1 + \left(\frac{5.1}{12.8228}\right)\left(\frac{0.075}{0.14}\right) = 1.2131$$

$$s_q = 1 + (\frac{B}{L}) sin\varphi = 1 + \left(\frac{0.075}{0.14}\right) sin 20.33 = 1.18612$$

$$s_\gamma = 1 - 0.4\frac{B}{L} = 1 - 0.4\left(\frac{0.075}{0.14}\right) = 0.78571$$

The bearing capacity factors used in the final equation were determined using the average friction angle calculated. The new bearing capacity factors used were: $N_c = 15.20975$; $N_\gamma = 3.22445$

Using Equation 3 the load-bearing capacity was determined;

$$q_u = \bar{c}N_c s_c + \frac{1}{2}\bar{\gamma}BN_\gamma s_\gamma$$

$$q_u = (8.083 \text{ x } 15.20975 \text{ x } 1.2131) + (0.5 \text{ x } 16.562 \text{ x } 0.075 \text{ x } 3.22445 \text{ x } 0.78571)$$

$$q_u = 153.4622 \text{ kPa}$$

The summary of the calculations for the 75 mm and 150 mm footings are presented in Appendix VI. Figures 123 and 124 for the 75 mm and 150 mm footings respectively show the comparisons of the measured bearing capacity tests with the calculated bearing capacities using Hansen's method for two-layered soils.

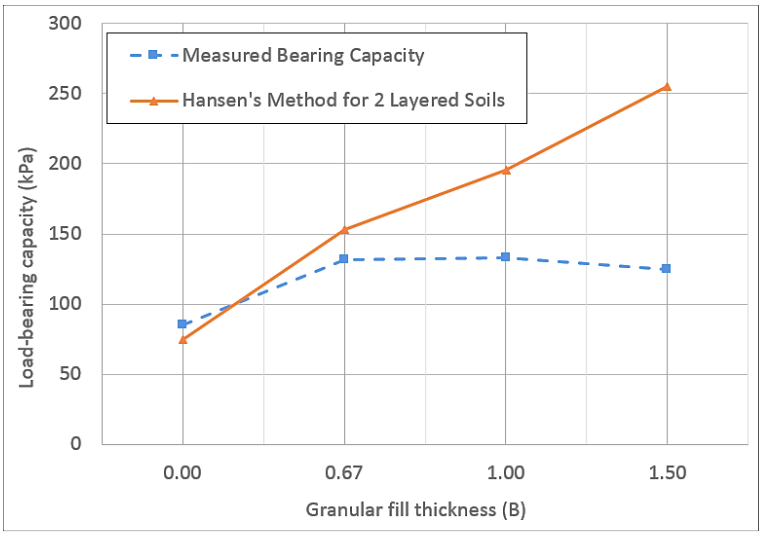

Figure 123: Load-bearing capacity against the granular fill thickness for the 75 mm footing

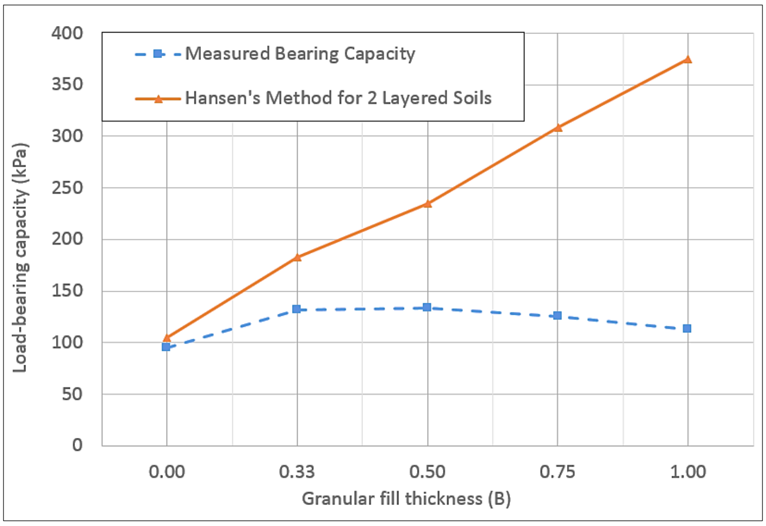

Figure 124: Load-bearing capacity against the granular fill thickness for the 150 mm footing

It can be observed that for smaller fill thicknesses there is more of a correlation between the measured and calculated values, and as the fill thickness is increased further there is a greater deviation between the values. The Hansen's equation gave higher values than the measured load-bearing capacities. Measured values shown that increase in fill thickness do not increase the bearing capacity without bound. This investigation suggested a limiting value equivalent to 0.67B for the 75 mm footing, whereas for the 150 mm footing the limiting value was observed at 0.33B. The limiting correlation with increasing fill thickness indicated that Hansen's equation for two layered soils could not be applied in determination of the load-bearing capacity.

Projected area method

The projected area method for layered soils uses the ultimate bearing capacity, q_u, and is given by the Equation 11 which is presented in Section 2.3.4.2. The load spread angle is assumed to follow Terzaghi's general shear failure and is given by the equation;

$$\alpha = 45° + \frac{\varphi}{2} \tag{30}$$

The angle of friction used in determination of the load spread angle is for the top layer, which was the granular material. The angle of internal friction of the granular material was 40°, and thus;

$$\alpha = 45° + \frac{\varphi}{2} = 45° + \frac{40°}{2} = 65°$$

For the 75 mm footing; B = 0.075 m

The bearing capacity at the base layer is determined using the Hansen's equation, as shown:

$$q_b = cN_c s_c + \frac{1}{2}\gamma B N_\gamma s_\gamma \tag{31}$$

$$q_b = (4.7\ x\ 12.8228\ x\ 1.2131) + \frac{1}{2}(15.484\ x\ 0.075\ x\ 2.016\ x\ 0.78571) = 74.0298\ kPa$$

For granular fill thickness of 50 mm; H = 0.05 m, and the bearing capacity is calculated using Equation10;

$$q_u = \frac{(74.0298(0.075 + 2\ x\ 0.05\ x \tan 65))^2}{0.075^2} = 68.2331\ kPa$$

Table 25 is a representation of the calculated values of bearing capacity using the projected area method using the 75 mm footing for the unreinforced clay with different fill thicknesses. Figure 125 shows the comparison of the results, with the maximum deviation of the results obtained of 55%. The projected are method gave values lower than the measures bearing capacity.

Table 21: Comparison of the bearing capacity from the Projected Area Method and the Experimental results.

H (fill thickness) m	Theoretical Bearing Capacity (Projected Area Method) kPa	Experimental Bearing Capacity kPa	Deviation in Bearing Capacity kPa
0.000	68.2331	85.7143	11.6845 (16%)
0.050	86.5780	132.6367	46.8858 (55%)
0.075	91.6115	133.3381	41.7266 (46%)
0.1125	100.4023	123.3469	22.9446 (23%)

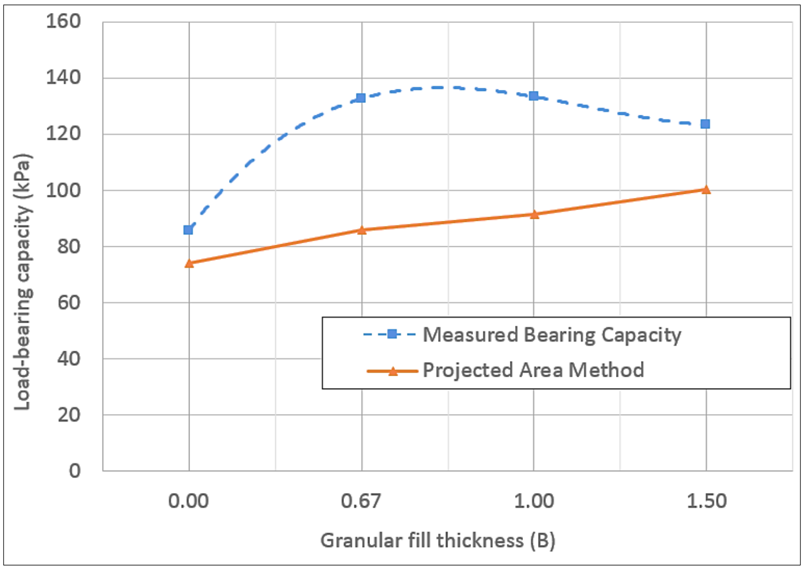

Figure 125: Load-bearing capacity against fill thickness for 75 mm footing

Comparison of Hansen's method for two layered soils and the projected area method in Figure 126 shows that the latter has a better correlation to the measured bearing capacity values, and as such should be used in further comparisons for the geosynthetic reinforced composite.

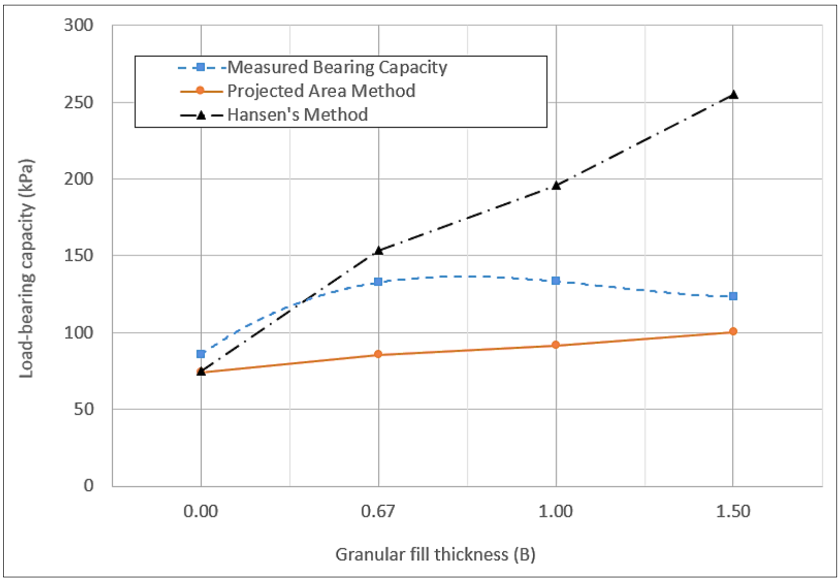

Figure 126: Load-bearing capacity against fill thickness showing comparison of methods.

Bearing Capacity Equation for Geosynthetic Reinforcement of a Two-Layered Soil.

The equation considered in the verification of the results was a combination of the equations from the studies involving the Projected Area Method and Chen's Method. The formulated bearing capacity equation for geosynthetic reinforced two-layered soil was obtained from a combination of Equations 11, 13 and 14 presented in Section 2.3.4.2 and 2.3.4.3. This led to a bearing capacity equation of;

$$q_u = \frac{q_b(B + 2Htan\alpha)^2}{B^2} + \Delta q_T \tag{32}$$

The calculations using Equation 32 for the predicted bearing capacity are presented in Appendix VI. The comparative results presented in Figures 127 and 128 are for the 75 mm footing and inclusion of a woven geogrid at various depths and fill thicknesses. From the graphs there was an observed general correlation between the measured bearing capacity and the values calculated using Equation 32.

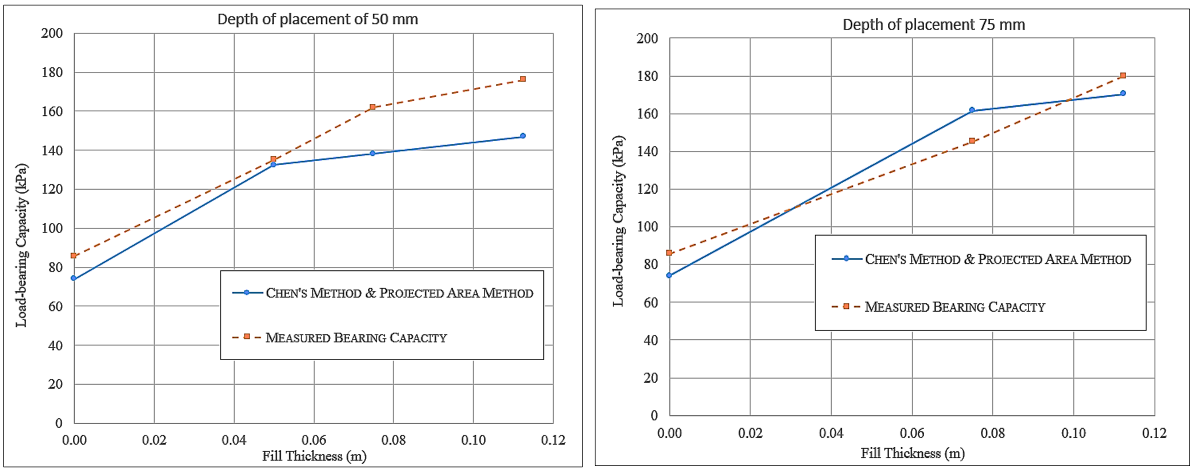

Figure 127: Load-bearing capacity against fill thickness for woven geogrid placed at a depth of (a) 50 mm and (b) 75 mm

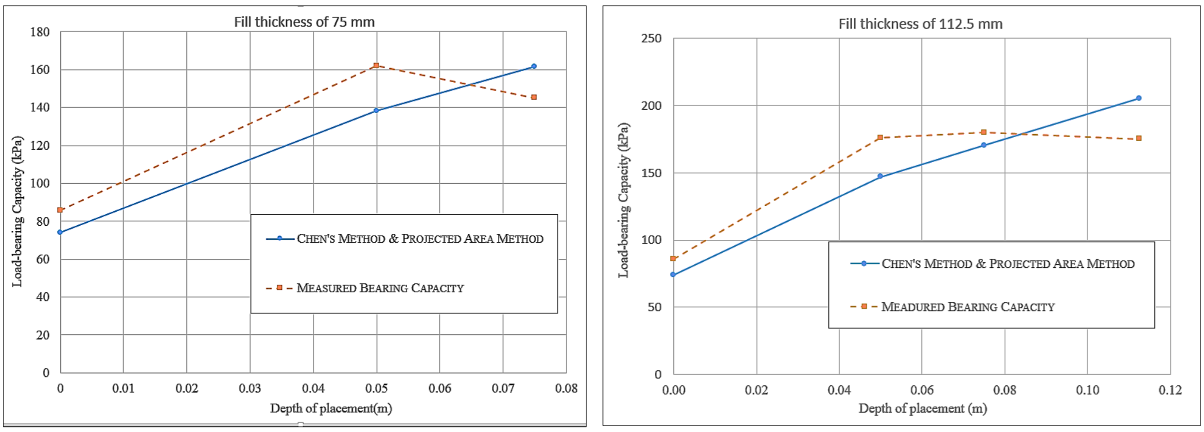

Figure 128: Load-bearing capacity against depth of placement for woven geogrid placed at various depths in a granular fill thickness of (a) 75 mm, and (b) 112.5 mm.

Incorporating Chen's Method and the Projected Area Method in comparison of the results obtained from the study conducted showed that there is a correlation between the predicted and the measured bearing capacities. This showed that the inclusion of the geosynthetic layer in the layered soil has the benefit of improving the strength of the soil. Further improvement of the equation could lead to development of a more accurate prediction that would match the obtained measured values. These equations could then be applied in the design of pavement structures that incorporate geosynthetics with the required strength obtained through altering the parameters involved.

CHAPTER 9

9. Conclusions and Recommendations

9.1. Introduction

The study investigated the reinforcement benefits of inclusion of a geosynthetic layer in a two-layered soil that represent the founding levels of pavement structures. The necessity of the reinforcement was due to the occurrence of soft subgrade soils on site that had relatively low load-bearing capacities and were susceptible to high settlements. This would lead to failure of the pavement structures during the construction phase and through their design life.

Plate load tests were conducted on the unreinforced soft kaolin clay; clay reinforced only by granular material; and on geosynthetic-reinforced two-layered soil composite using a Zwick Universal Compression and Tension machine. The types of geosynthetic products used in the experiments included: extruded geogrids, woven geogrids, woven geotextiles, and non-woven geotextiles.

9.2. Summary of Conclusions

The following conclusions can be made:

1) The study demonstrated that the inclusion of a geosynthetic layer in a two-layered soil, either within the fill or at the interface, had the benefit of improving the soil structure by increasing the load-bearing capacity and subsequently reducing the settlement.
2) The improvement in load-bearing capacity due to addition of only a layer of granular fill ranged from 20% -50%, whereas on inclusion of a geosynthetic layer either within the fill layer or at the interface of the soils there was an overall improvement that ranged from 35% - 160%.
3) The overall reduction in settlement due to geosynthetic inclusion ranged from 35% to 60% and this differed as the type of product tested changed. The least reduction in settlement was observed in the woven geotextiles, while the most reduction was observed in the extruded geogrids.
4) It was observed that when the geosynthetic was placed within the fill layer, the reinforcement from the geosynthetic was mobilized earlier than when it was placed at the interface of the two soils. This led to greater improvements in the load-bearing capacity for the former configuration than the latter. The optimum depth of placement obtained was 0.67B and the optimum range of fill thickness obtained was 1.0B – 1.5B. These were both dependent on the geosynthetic product used as they each had different reinforcement benefits.
5) Geogrids generally provided better reinforcement improvements than geotextiles. However, the non-woven geotextile was seen to have greater improvements for a few configurations. This was attributed to the lack of anchorage of the geotextile, and the generated increased friction between the geotextile and the soil as it moved.
6) The reduction in base/subbase thickness possible due to geosynthetic reinforcement was dependent on the initial unreinforced thickness, and ranged from 25% - 67%.
7) The bearing capacity of geosynthetic-reinforced two-layered soil can be closely predicted using the equation derived from combining Chen's method and the Projected Area method.

9.3. Recommendations

The following recommendations can be made for future study;

1) To reduce on the effect of the boundary conditions, the test scale should be increased. This would lead to representative results that closely match field conditions, and could be directly applied in the design phase and construction process.
2) Confidence in geosynthetic products has not been developed as yet as large-scale research and development programmes on geosynthetic applications and field demonstrations are insufficient. In addition, the inadequate field monitoring and performance studies of geosynthetic-reinforced structures has led to a lack of reporting on geosynthetics at a broader level. Measurement of strains in the geosynthetics could be conducted by means of strain gauges. This would provide the full magnitude of the tensile strains in the reinforcement layer, the membrane action, and the magnitude of the membrane force.
3) Dynamic/cyclic tests and rolling wheel tests could also be conducted to replicate actual loading conditions from traffic loads and trains, which would give the benefits from geosynthetic-reinforcement that match their applications.
4) Certain factors like pull-out failure and warping action of the geosynthetic as the loads are applied during testing could affect the results obtained. Anchorage of the reinforcement in place could prevent these from occurring and in addition aid in the tensile support necessary from the geosynthetic.
5) The design equations for geosynthetic-reinforced two-layered soil composite could be developed further to allow for better predictions of the load-bearing capacities. This would improve the design phase of construction projects.

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Images

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APPENDIX

REINFORCEMENT OF PAVEMENT SUBGRADE USING GRANULAR FILL AND A GEOSYNTHETIC LAYER

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Supervised by
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REPORT ON REVISIONS

University of Cape Town
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November 2014

Table of Contents

1. General

1.1. Typographical Errors and Minor Changes

Examiner's comment	**Page**	**Line**	**Changes made**
Chapter 1			
Last paragraph of 1.2: insert comma "…. use geosynthetics, it is found ….”	2	24	Comma inserted
Second last line, last word: change "if" to "of"	2	28	Word changed from "if" to "of"
Insert break to get Chapter 8 in a separate paragraph	3	19	Break inserted
It is suggested that a section is added to Chapter 1 titled 1.4 Research Limitations	3		Section added
Chapter 2			
Remove "of" in third last line to read …. "subcategories including ….”	4	10	"of" removed
There is no need to include the page number of cited references within the body of the test.	5	3	It was recommended that this section be referenced in that manner
Do not use above and below in reference to figures and text.	7	22	"below" removed
Third bullet: "Binder condition" is not a clear description of a failure condition	8		removed
Section 2.2.4, second line: Change "preventative-actions" to "preventative actions"??	9	2	Change made from "preventative-actions" to "preventative actions"??
Last line: Change "stresses and strain" to "stress and strain	9	16	Change made from "stresses" to "stress"
Figure 9: Title says reinforcement is at materials interface, but figure shows geogrid within base course	10		Change made from "materials interface" to "within base course"

The last sentence is not clear. It refers to range of fill thicknesses, testing and wastage of materials but no explanation of, or any other reference to, which testing is referred to or how fill thickness and wastage are involved in such testing is given. Maybe rephrase the sentence?	12	16	Sentence changed to… "The zone of influence is important as it determines the maximum thickness that should be applied in the experimental testing, so as to limit wastage of material by exceeding the zone of influence."
Second bullet: Is it Projected <u>Areas</u> Method	14	14	Changed to "Projected <u>Area</u> Method"
Equation 7: How is λ determined	15		Equation altered to show how z is determined $z_{max} = \lambda B = z_{top\ soil} + z_{bottom\ soil}$
Equation 11: Units of q_u will differ from units of q_b	15		Equation edited to $q_u = \frac{q_b(B + 2Htan\alpha)^2}{B^2}$
Equation 13: It is suggested that units of T_i are indicated	16		Units added "kN/m"
Das (2010) does not appear in References	17	21	Changed to right reference "Das (2011)"
No year for PDW is indicated	17	25	Year added (DPW, 2007)
2nd last line: Change to "….. but <u>also</u> to roads …."	17	27	Change made
Section 2.4; It states that soils are problematic because they undergo extreme changes. What changes are these? These should be explicitly stated	17	1	Addition of extreme changes "…expansion, collapse, dispersion, excessive settlement, and have a distinct lake of strength"
Byrne (2008) does not appear in References. Is it Braatvedt et al. (2008)	18	1	Changed from Byrne (2008) to Braatvedt et al. (2008).
It is suggested that you remove part on collapse potential test because no tests were mentioned or described to identify or classify other problem soils	19		Section removed (collapse potential tests)

Tables 2 and 3 show all problem soils associated with residual and transported soils, and not only expansive soils and should be moved to 2.4 Problematic Soils or to 2.4.1 Types of Problem Soils	19		No change, tables are in the right place
Line 4: Add comma: "…….marginally of highly, then precaution …."	21	4	Comma added
Paragraph 2: 2nd sentence: Change to "When there is a high exchangeable sodium percentage in the clay-water system, there is a ….."	22	9	Change from… "exchange of sodium there is…" to "exchangeable sodium percentage in the clay-water system, there is…"
Change sentence 4: "When the clay is in contact …."	22	12	Change made to sentence
2.4.1.4: First sentence is too long. Break up into two or three sentences.	22	15	Sentences split into… "The nature of the problem associated with dolomitic soils is as a result of changes in the water table and the presence of soluble bedrock (Kalumba, 2013). The dissolution of the dolomitic rock cavities made up of carbonate bedrock results in settlements and punching shear failure."
The photo is of Cape Town but the title says the sinkhole is because of dolomite. I didn't know that there were dolomites in the Cape Town area	23		There are minor deposits in the Western Cape region (Council of Geoscience - http://www.geoscience.org.za/)
Byrne (2008) does not appear in References. Is it Braatvedt et al. (2008)	24	6	Changed from Byrne (2008) to Braatvedt et al. (2008).
Pargraph 2, Line 4: "…… 30% <u>of</u> the height …"	24	9	Change made, added "of"
Last line of 2.4.1.5: Is it "a challenges in" or "challenges in" or "a challenge in"?	24	16	Changed to "challenges in"
Last line of 2.4.1.5: In reference, remove (PDW)	24	16	"(DPW)" removed
2.1.4.6: Line 6: Change "heavy <u>loadings</u>" to "heavy loads"	24	22	Changed from "loadings" to "loads"

First line of last paragraph: Change "expandable" to "expansive".	24	29	Changed from "expandable" to "expansive"
Last line: Change to "… distribution of the dispersive …."	24	32	Change made removing "the"
Line 1: "……. soil that is faced …."	25	1	Change made including "is"
Line 5: Change " … and even railways, which would be …." to "….and even railways would be …"	25	5	Change made removing "which"
Line 6: " … the driving comfort …"	25	6	Change made removing "the"
Line 6 & 7: Repeating "expansive clay" in sentence seems incorrect.	25	6	No change, sentence kept as is.
3rd last line: "…cracking with and the formation of ….."	25	15	Change made from "with" to "and"
Last line: "… occurring on these structures roads there …"	25	16	Replaced "these structures" with "roads"
Line 1: "… the treatment of these soils would have to …."	26	1	Removed "of these soils"
Table 4 title: Change to "Techniques to deal with problematic ground conditions"	26		Change made to title
2.4.4.1: Paragraph 2, line 2: "…. or fly ash for the stabilization of …."	26	16	Addition of "for the"
2.4.4.2: 2nd last line: "…. are strength and compression compressibility which could …"	27	6	Change made from "compression" to "compressibility"
Preloading: 2nd sentence: it is not clear what is meant by "…voids are filled…"	27	9	Sentence changed to …"air voids are filled by the soil particles"… to give an indication of what is filled through preloading action
Preloading: "…. settlements that would not cause prevent further problems to …."	27	14	Change made from "not cause" to "prevent"
Preloading: Remove the word "Although" from the last sentence	27	14	"Although" removed

2.4.4.3: 2nd last sentence: Change to "Using one of the many available engineering methods would strengthen the soil."	27	20	Change made to whole sentence
2.4.4.3: Last sentence: "One of these the methods of interest will be further"	27	21	Change made to "One of the methods will be further discussed in Chapter 3"
2.4.4.3: Bullet 1: "….subjected to: to improving to improve the….."	27	25	Change made removing "to: to"
2.4.4.3: Bullet 3: "Improving the slope stability ….."	27	29	Change made removing "the"
2.4.4.3: Bullet 5: Last word is spelled incorrectly.	27	35	Change made to "recompacted"
2.5.4: "Benefits and Limitations of this Method Replacement"	28		Change made to heading
Chapter 3			
3.3: 3rd line after table: "provide a passage for water but do not cause water to flow." It is not clear what is meant.	30	7	The geosynthetic acts passively by allowing water to flow along it and not actively by causing the water to flow.
3.3.1: 3rd last line: "…excess pressures into geosynthetics would result …."	30	17	"into geosynthetics" added
3.3.2: 2nd line: "…… of the soft soil ….between the particles of the granular soil …"	31	2	Changes made adding "the" to both sections
3.3.2: Paragraph 3: '…. under an unsurfaced access road …"	31	10	Addition of "unsurfaced"
Agrawal (2011) not in References.	31	16	Added to reference list
Figure 23: Not date for reference in title.	31		Date included (for when obtained from website)
Line 5: "without creating an immediate increase in its shear strength."	32	5	Changed from "creating an immediate increase" to "improving"
Figure 25 does not show soft underlying soil, only compacted granular aggregate on both sides of geotesynthetic.	32		Figure changed to represent soft underlying soil.

Figure 24 has no date in reference.	32		Date included (for when obtained from website)
Paragraph 3, line 1: "…. selection, and design and use …"	33	16	Removed "and"
Line 1 after figure: change modem engineering to modern engineering.	34	14	Changed from "modem" to "modern"
Figure 27: Alibaba.com not in references.	35		Reference added
3.6.1: line 5: "soils particles" Is it soil particles or soil's particles ?	36	8	Change made to "soil particles"
4th line after figure: Bourdeu and Ashmay (2013) is not in References.	36	15	Changed to "Bourdeau and Ashmawy (2012)"
Section 3.6.1; The load is not subjected on the structure. The structure is subjected to loading	36	5	Change made to "…soil structure is subjected to loading"
3.7.1: 5th line: " …temporaty used by …. for a limited period, …"	38	19	Removed "temporary"
Perkins et al, (2013) not in References.	39	3	Changed to Perkins et al. (2012)
Figure 32: Blogspot.co not in References, also no date	39		Changes made to "murderiseverywhere.blogspot.com" and Date included (when obtained from website)
Paragraph 2, line 1: Is it vertical elastic deformation?	39	4	The deformation is vertical and transverse
Tan and Shukla (2013) not in References, only Tan and Shukla (2012).	40	6	Changed from Tan and Shukla ("2013" to "2012")
Last line: "layer is filter stable against …."	40	9	No change, sentence is complete… "the sand layer is filter stable against the ballast under load"
Line 1: Interface of which soils? Remember that there are ballast, sand and soft material involved.	41	1	Changed to "…of the ballast and soft soil, instead of a sand layer"
Figure 37 has not reference.	42		I took the picture myself (one of the products provided for the testing)

Last lines of 3.8: Is it necessary to repeat raw materials which have been listed in Table 6 on page 33	42		Removed
Figure 38 title: Alpe-Adria Textil not in References, also has not date.	43		It's in the images reference list and Date included (when obtained from website)
Agrawal (2011) not in References.	43	5	Added to reference list
Last paragraph: Are geogrids considered as geotextiles? why are they included here if Section 3.9 deals specifically with geogrids	43		Geogrids info moved to Section 3.9.2
3.8.3: paragraph 2: Sentence cannot start with " Also" . There is also no full stop at end.	44	7	Change made to sentence, removing "also"
Raw materials are listed again!	45		Removed
3.9.1.2: Should "triaxial geogrids" be changed to "triangular geogrids" like in table? Triaxial is three-dimensional.	46	3, 5	Change made from "triaxial" to "triangular"
5th line: Change "… reinforced walls" to "reinforced concrete retaining block walls .."	46	2	Change made to sentence
Bullet 2, line1: Delete comma between "base" and "also".	47	19	Comma deleted
3.10.2, line 1: Insert comma between "interaction" and "such as".	48	9	Comma inserted
3.10.2, line 3: Start a new sentence at "The mechanisms …"	48	12	Change made
3.10.4.1: Jewel et al (1984) not in References.	50	2	Added to reference list
Bullet 1: Delete comma between "sheet-like" and "geosynthetics".	51	3	Comma deleted
Chapter 4			

Section 4.1; It is stated that soft clay soils in Cape Town area pose a problem on sites. What problems are these? Need to mention the specific problems posed by these soils	52		Added "…such as susceptibility to high settlements"
Ranadie & Jadhar (2010) not in References	52	11	Reference corrected to "Ranadive and Jadhav (2010)"
4.3: Paragraph 2, line 5: "… of water and that would …."	52	21	"and" added to sentence
4.3: What size of footing did Mawer use?	52	25	Footing size used by Mawer was already stated…"140 mm by 150 mm"
Paragraph 3: What was length of geotextile that had strain of 176 mm	52	29	Dimensions included..."280 mm by 50 mm"
Line 1: tri-axial or triangular (tri-axial is 3D)?	54	3	Change made from "triaxial" to "triangular"
Line 1: "… tests on grained soils …" Should it be coarse-grained or fine-grained	55	1	Changed to "coarse-grained"
Yetimoglu & Wu (1994) not in References	55	13	Changed to Yetimoglu et al. (1994) and Added to reference list
Paragraph 4, line 7: " …. inclusion of a geosynthetic layer …"	56	23	"a" added
Line 1: Is "saturation ratio" = "degree of saturation" indicated by the symbol S_r"	57	1	Change made from "saturation ratio" to "degree of saturation"
3rd line: "… loads passed (to) (by) (through) the geosyntehtic layer ...	58	2	Sentence changed to "…loads directly to underlying soil without mobilizing the full reinforcement benefit in the geosynthetic."
1st line below figures: change figures 49 and 50 to Figures 49 and 50	58	10	Change made
2nd sentence after figures: The 0.25B must be a mistake	58	12	Change made to "5B"
Last line : "… while the varying the width of …"	58	16	"the" removed

Line 4: Is it really a "tri-axial" geogrid	59	5	Change made from "triaxial" to "triangular"
The width of the confining box and size of the model footing was not indicated. Could the geogrid width be increase more, or was that hampered by the box's sides? Would a 3-D setup maybe have changed the results?	59	11	Dimensions of box stated as "950 mm by 450 mm by 140 mm" and dimensions of footings were already given as "140 mm" and "200mm"
Check spelling of "of". The o seems to be number zero	59	12	Change made from "0f" to "of"
59 & 60: No reference in figures' titles.	59 & 60		Work is my own, but reference added… "Oriokot, 2012"
Figure 53: Every time geotextile width was increased, bearing pressure increased as well. How was conclusion made that 5B was the optimum width? 6B would probably see the bearing pressure increase again!?	60		The conclusion was made regarding the specimen tested, of which the width of 5B showed the optimum result.
Last sentence seems to be either incomplete (text missing after D = 0.1B) or totally incorrect.	60	12	Added "…there was reduced improvement in the load-bearing capacity."
Paragraph 1, last line: "…where the (optimum)(most efficient) depth of displacement of the reinforcement is at the interface …"	61	3	"optimum" added
It is suggested that paragraph 2 is combined with the last sentence of paragraph 1. This would eliminate the previous suggestion that a word should be added.	61	4	Change made, combining the sentences into 1 paragraph.
Why repeat detail on optimum depths (0.5B and 0.25B) of placement in the paragraphs above and below figures	61		They are describing two different scenarios. The first being the optimum, and the second the reduction in improvement.
Is paragraph 2 still analysing the tests discussed in the first paragraph	62	8	Yes it does

Figure 56 (a) and (b) indicate 0.125 mm depth??? Whose results are those? There is no reference. Are you sure that 125 mm is optimum in (a)? Green curve is above rest almost all of the way	62		Reference added (Oriokot, 2012). Optimum was taken at 60 mm vertical displacement (point of failure)
Last line of paragraph 1: "… showing that the optimum depth…."	63	5	"that" added
At end of 4.5 the questions arises again whether the same results would be obtained for full-scale situations and how representative these model tests will be when determining optimum depth, width, footing size etc. on a specific sand or gravel	63		Full scale tests have been recommended to verify the results.
Last paragraph: Why spell "Tri-axial" with a capital letter? Is tri-axial really the correct term?	63	21	Change made from "triaxial" to "triangular"
2nd sentence: "… 50% and 40% <u>above the unreinforced case</u>."	64	7	"above the unreinforced case" added
Last paragraph: The 50 mm placement case was actually the only width for which the geogrid really performed better than the geotextile.	64		That was noted, and explained in paragraph 2.
Are Ornek's results contradicting those of Chen discussed on page 65? If the answer is yes, this should be discussed. The two sets of results cannot merely be discussed separately as was done here. Or does the last sentence acts in place of such a discussion	66		Yes, the last sentence addresses that.
Last line of 4.7: "… necessary to conduct <u>more</u> tests …"	66	11	"more" added
Section 4.8 seems to not only serve as a summary of past research, but also as introduction to the work you are planning to do, like "investigating different experimental configurations" etc. and also very last sentence.	66 & 67		Noted. That was the intention.

Table 7: Yetimoglu and Wu (1994) not in References; El Sawwaf and Nazir (2010) different from reference in Referenced (spelling of Nazir).	68 - 71		Yetimoglu et al. (1994) added. Spelling of "Nazir" is correct
Chapter 5			
Page 73, Line 8-10 and Page 75, Line 7-9; Not necessary to include the full address of the store from which the research materials (Kaolin and geosynthetics) were purchased	73 & 75		It was recommended to state all information.
Some of the properties, such as 93% Mod AASHTO and DCP count, were clearly stated as being in situ properties. Does that apply to the RD = 0.94 too.	72		No, RD is determined through lab tests
Table 10, right-hand side column: Since liquid limit and plastic limits are not proper names, they are usually typed in lower case letters	74		Changes made to lower case letters
5.2.2.1: Line 3: "McGrid EG <u>is</u> a high modulus … geogrid<u>s</u>, …"	75	13	Change made to "geogrid"
5.2.2.1: Line 5: " … inert to all chemical<u>s</u> existing …"	75	15	Change made to "chemicals"
Both geogrids are said to be shown in Figure 63, but it should be Figure 64	75	12, 19	Change made to "Figure 64"
Both geotextiles are said to be in Figure 64, but it should be Figure 65	76	5, 9	Change made to "Figure 65"
Why were only impermeable geotextiles used? What about drainage in real situations	76	4, 9	Change from "impermeable" to "semi-permeable" to better describe the geotextiles
Last paragraph: It should be Tables 12 – 15 and not tables 12 – 15.	77	8	Change made to "Tables"
Tables 12 – 15: Should units of strain not be mm/mm, or even %? Elongation is measure in mm.	78		Strain represented as a percentage (%)
Tables 12 – 15: If the sample width was 50 mm, how was tensile strength calculated from maximum force? E.g. 1418.41 N / 0.050 m = 28.3682 kN/m.	78		Calculation made by dividing by the thickness of the geosynthetic of 5 mm (0.005 m)

5.2.4: No lateral deformation was required, but surely lateral deformation in all directions can occur in pavement material when a wheel rolls over it	79	7	Added "…of the box…" to clarify that lateral deformation of the box was limited.
It is suggested that the first paragraph of 5.3.1.1 be moved to just before 5.3.1.1 to form part of 5.3.1.	80		No change.
3rd line of paragraph 1 on CBR value: "… suitable correlations between …"	80	7	Change made "correlations"
3rd line of paragraph 2 on CBR value: "… the soil affect the CBR value, which makes it difficult …" Remove the comma and the word "which".	80		"which" and comma removed
Paragraph 1 on CBR value (actually throughout the chapter and whole thesis!): Why type common soil properties like liquid limit, maximum dry density etc. starting with capital letters?	80		Changes made to lower case throughout the whole thesis
Paragraph 1 on CBR value (actually all over pages 80, 81 and 82): Is it natural/mixed moisture content (MC) or optimum moisture content (OMC)? Make it very clear because MC appears in all equations of this section and is used for calculations but OMC is referred to in text as well. CONFUSING!!	80		Changed to "natural moisture content"
: If SL refers to shrinkage limit, SL and MDD and OMC are all time-consuming to determine as well	81		In comparison CBR is more time-consuming, thus preferable to use correlation equations
Paragraph just before Equation 24: Sentence cannot be read nor understood	81		Change made, deleted… "Correlations based on liquid limit and Gradation"
Paragraph after Equation 25: "… LL is the liquid limit ……… and C is the fraction…"	81	12	"the" added

2nd Paragraph after Equation 24: "…that the predicted …"	81	14	"the" added
3rd Paragraph after Equation 24: Remove full stop between liquid limit and plastic limit.	81	23	Full stop removed
Last three lines of 5.3.1.2 at top of page: Is it "optimum width" or "optimum length"? The width was 140 mm. Or in which direction was the wheel travelling	85	4	It is "width" …140 mm was the length
5.3.2: End of 1st paragraph: Dates of references not in brackets	85	11	Change made
Bullet 5 at top of page: " … wheeled into place position to be loaded by into the Universal …."	87	1	Changes made
Bullet 6: "…. placed centrally on the top of the granular … "	87	3	"the" removed
Chapter 6			
6.1: Line 4: " … were compared to with results for the geosynthetic … "	90	4	Change made from "to" to "with results for"
Line 2: "… standard deviation of 5% that is the considered to be … "	91	2	"the" removed
Paragraph 2 of 6.3: Here it is mentioned that there was lateral spread of particles, but on Page 79 in 5.2.4 it was specified that the box was designed to prevent lateral deformation.	91	7	Correction made in section 5.2.4 stating that the lateral deformation was of the box and not the soil
Table 20: It is suggested that the title of 3rd column be changed to "Average Peak Stress".	91		Change made, added "Peak"
Paragraph 4 of 6.3: Spelling of Brocklethurst, 1993 different than that of the same name in References	91		Change made from "Brocklethurst" to "Brocklehurst"
(and throughout the chapter): Should all paragraphs not be in past tense?	91		Tense corrected

Bullet 5: "Doubling the footing width ….. led to both an increase in the applied load … and also a reduction … . The increase in (applied load?) could be attributed to wider spread of the load ….. , while the decrease (in applied load?) could be attributed to increased loading from the footing." This last (underlined) part of the sentence is not understood	96	9	Change made from "loading" to "weight"
Chapter 7			
1st line: "This Chapter chapter discusses …"	97	1	Change made to lower case
2nd line: " … into context in terms of the benefits achieved …"	97	2	"the" added
In Figure 85 (b) the two layer system had "limited heaving" but in Figure 86 (a) the heaving was not described as "limited" again. This is contradicting	98		Changes made to "limited"
In Figure 87 (a) the heaving for the two layer system was described as "excessive" which is not the same as in Figure 85 (b) and Figure 86 (a). This is even more contradicting. Three different descriptions for the same situation	99		Changes made to "limited"
Figure 88: How can the blue and red curves have the same number (UR/B150/Z0)	99		Change made to "UR/B150/Z50"
1st line: " … and as such a reduction of the degree … "	100	1	"a" added
1st paragraph: Is it necessary to type Percentage Reduction in Footing Settlement starting with capitals	100	4	Change made to lower case
Last sentence above Figure 89: " … of reducing the settlement faced *considerably*."	100	10	"considerably" added

Figures 95, 96 and 97 and 100: Since the maximum fill depth was 1.5B or 1.0B it may be inappropriate to conclude that the optimum fill depth was 1.5B or 1.0B if there were no other results to see if the bearing capacity would increase if the fill was increased. E.g. all three curves were steadily climbing at fill thickness = 1B or 1.5 B in all these figures. The last sentence of 7.3.2 might, therefore, be incorrect	104 to 107		For the tested range, those were the optimums obtained
7.3.3: $D_{2(U)}$ and $D_{2(R)}$ are "selected at the same bearing pressure". On Figures 101 and 102 the bearing pressure was taken as just below 100kPa. Is this value for "same bearing pressure" prescribed or was that value randomly selected	107		It was randomly selected, although in this comparison of base reduction the main interest is in the thicknesses and not the bearing pressure
Paragraph 3: Bottom line: "… that showed <u>optimum</u> placement of the …."	109	20	"optimum" added
Bottom line: Is it 0.6B or 0.67B?	109	26	Change made to "0.67B"
First sentence: It is not clear which tests the sentence refer to, because it is actually still discussing the woven geotextile, but it mentions the benefits of the geogrid. Should that sentence not have been part of the last paragraph on page 109	110		Correction made to describe trends of the "woven geotextile" and "extruded geogrid"
Bottom sentence: Why indicate two optimum depths in Figure 106 but only one in Figure 105? The results appear to be very similar in these figures, i.e. "no substantial increase" in bearing capacity.	110		Sentence with the two optimum depths removed… *"There were two optimum depths of placement obtained of 50 mm and 112.5 mm."*
Paragraph 1: Is there not even a higher chance of material intermixing if loads are high and frequently applied, indicating best reinforcement depth at interface	111		Mobilizing the reinforcement earlier, with placement of the geotextile within the base course would reduce the stresses at the interface thus limiting the intermixing.

Figures 109 – 112: The “improvement measures”, such as adding gravel layers and geotextile/geogrid reinforcement, were less successful for a larger than for a smaller footing for some and more successful for other types of reinforcement. Why is there no comment about it or effort to explain it at the end of Section 7.5 other than that last sentence (which is not clear in the meaning of “attributed to increased loading from the footing” anyway)?	114	12	Change made from “loading” to “weight”
7.6: Line 8: “… thus increasing the stability … and increase increasing the strength …”	115	8	Change made to “increasing”
Paragraph 2 Line 4: “ … loading to the subgrade which increase increases the vertical … ”	121	12	Change made to “increases”
Paragraph 3: Line 3: Is it 0.6B or 0.67B as was indicated on many figures	121	18	Change made to “0.67B”
Chapter 8			
: Line 2: “”… can be applied …	122	2	“be” added
Was the second term in the bearing capacity equation omitted because it was almost zero for the model footing	123		Yes, it was. Footing at the surface, D = 0
Line 4: “… in fill thickness does do not increase … ”	124	4	Change made to “do”
Line 7: “… fill thickness indicated that that Hansen’s equation … ”	124	7	“that” removed
First sentence of Projected are method: “ … uses the ultimate bearing capacity, q_u, and is given by Equation 11, which is presented in Section 2.3.4.2”.	124	9, 10	“and” and “which is” added
: In the calculation of the factors N and s for the terms, was the correct value for □’ used? Just asking because N_c is almost the same as for Hansen’s calculations	124		Angle changed to represent the right material

Last line on page before figures: Text refers to Equation 34 but the last equation in the thesis is numbered 32.	126	8	Change made to "32"
Chapter 9			
Conclusion #2 refers to the 20 - 50% improvement in bearing capacity of a clay when a granular layer is added. Then it refers to the 35 – 160% improvement when a geosynthetic layer is included. It should be clearly stated if the latter is an improvement in addition to, or including, the first because by just reading the conclusion and not studying Chapter 7 it is not clear.	128	15	Change made to indicate it was an overall improvement stated by the "35% - 160%"
The previous bullet applies to Conclusion #3. Is that an overall improvement in settlement of just after the geosynthetic was included in the two-layer system	128	16	Change made to indicate it was an overall improvement stated by the "35% - 60%"
Conclusion #5: 2nd Sentence: Rephrase part of sentence after , because its present form seems to be incomplete. There must be some words added before the "because".	128		Change made to sentence.
Conclusion #6: "The reduction in base/subbase thickness <u>possible</u> due to geosynthetic … "	128	30	"possible" added
Recommendation #1: Very important recommendation. That might even change all results in this thesis.	129		Noted

Recommendation #3: It might not only be the cyclic effect of the wheel loads that is important, but also the rolling effect. When a wheel nears a specific point on the pavement, the particles below that point get pushed ahead and when the wheel is past the point the particles get pushed back again, causing a repeating wave or kneading action. That action might have very important effects on the material/material interface and the material/geosynthetic interface, such as repeated forwards and backwards shear, fatigue etc.	129		Added "rolling wheel tests" in the recommendation

1.2. Review of Specific Chapters

Chapter 1

Examiners advised that a section identifying the limitations of the research should be added. As such Section 1.4 was added to indicate the limitations of the research:

"The research conducted herein concerned the reinforcement of pavement subgrade using both granular fill and a geosynthetic layer, with the determination of the benefits as a result of their inclusion in the pavement subgrade. However, the tests conducted were limited to static loads, and no cyclic loading and rolling wheel tests included in the research. In addition, a limited number of geosynthetics were investigated from a single supplier."

Chapter 2

Examiners pointed out that the last sentence on page 12 needed to be rephrased. Sentence changed to:

"The zone of influence is important as it determines the maximum thickness that should be applied in the experimental testing, so as to limit wastage of material by exceeding the zone of influence."

Chapter 4

Examiners pointed out that the sentence was not clear, as such it was rephrased. Sentence changed to:

“. If the width is not adequate, then the particles would disperse outward with transfer of applied loads directly to underlying soil without mobilizing the full reinforcement benefit in the geosynthetic.”

Examiner pointed out that the last sentence on page 60 seemed incomplete, possibly lacking some information. An addition was made to the end of the sentence, as such it changed to:

“If the depth is too great, then adequate reinforcement will not be achieved, as observed in the study conducted by Guido et al. (1985) where beyond the depth equivalent to D = 1.0B there was reduced improvement in the load-bearing capacity.”

Chapter 5

Examiner requested for clarity on how the tensile strength of the geotextiles were calculated.

The calculation was made by dividing the maximum force by the thickness of the geosynthetic of 5 mm (0.005 m)

Examiner requested that the statement regarding lateral deformation be rephrased to indicate what lateral deformation was prevented. As such it was changed to:

“Metal bracings were fixed on either side of the equipment to prevent any lateral deformation of the box due to the applied loading.”

Chapter 6

Examiners were concerned that there would be a higher chance of material intermixing if loads are high and frequently applied, indicating best reinforcement depth at interface.

However, placement of the geosynthetic within the base layer would result in mobilizing the reinforcement earlier, which would reduce the stresses at the interface thus limiting the intermixing.

2. Referencing

The examiners noted that there were a few discrepancies regarding referencing of figures, tables and the work in general. The necessary changes were made to correctly reference all the work cited.

All references were updated as follows:

1. Agrawal, B.J., 2011. Geotextile : Its Application To Civil Engineering – Overview. In *National Conference on Recent Trends in Engineering & Technology*. Gujarat, India, pp. 1–6.
2. Jewell, R. A., Milligan, G. W. E., Sarsby, R. W., & Dubois, D. 1984. *Interaction between soil and geogrids.* In Symposium on Polymer Grid Reinforcement, pp. 18-30.
3. Oriokot, J., 2012. *Reinforcement of Crushed Aggregates using Geogrids*. B.Sc. Thesis. University of Cape Town, South Africa.
4. Yetimoglu, T., Wu, J. T., & Saglamer, A. 1994. Bearing capacity of rectangular footings on geogrid-reinforced sand. Journal of Geotechnical Engineering, 120(12), pp.2083-2099.

Images

1. Manufacture process of woven geotextile http://www.alibaba.com

3. Final Comments

I am grateful for all the constructive comments made by the external examiners. The reinforcement of pavement subgrades using geosynthetic layers is a beneficial topic to the field of engineering. With more research into the topic a readily applicable solution to different problem soils can be achieved.

Signed: ……………………………………..

Date: ……………………………………….

List of Appendices

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I. Classification Tests

Atterberg Limits

Table 1: Table Showing Calculation of Liquid Limit

Description	Unit	Test		
Test number	-	1	2	3
Container number	-	L1	L2	L3
Number of bumps	-	12	28	35
Mass of container+wet soil	g	11.389	15.015	15.816
Mass of container+dry soil	g	10.43	13.195	13.873
Mass of container	g	8.106	8.078	8.234
Mass of dry soil	g	2.324	5.117	5.639
Moisture loss	g	0.959	1.82	1.943
Moisture content	%	41.265	35.568	34.456
Multiplication factor	-	0.950	1.010	1.030
Average moisture content	%	**36.872**		

Table 2: Table Showing Calculation of Plastic Limit

Description	Unit	Test		
Test number	-	1	2	3
Container number	-	P1	P2	P3
Mass of container+wet soil	g	11.148	10.862	12.593
Mass of container+dry soil	g	10.842	10.628	12.059
Mass of container	g	9.704	9.712	9.808
Mass of dry soil	g	1.138	0.916	2.251
Moisture loss	g	0.306	0.234	0.534
Moisture content	%	26.889	25.546	23.723
Average moisture content	%	**25.386**		

Table 3: Table Showing Calculation of Shrinkage Limit

Description	Unit	Test	
Tin Number	-	S1	S2
Initial Length	mm	150	150
Final Length	mm	144	145
Shrinkage	mm	6	5
Shrinkage Limit	%	4	3.33
Average Shrinkage Limit	%	**3.667**	

Plastic Index = Liquid Limit – Plastic Limit = 11.486 %

Natural Moisture Content

Table 4: Table Showing Calculation of Natural Moisture Content

Description	Unit	Symbol	Tin		
Sample No.			**1**	**2**	**3**
Mass of empty tin	g	m1	8.895	8.106	9.016
Mass of tin + soil before drying	g	m2	22.85	18.601	15.881
Mass of tin + soil after drying	g	m3	22.799	18.563	15.824
Mass of soil	g	m2-m1	13.904	10.457	6.808
Mass of water	g	m3-m2	0.051	0.038	0.057
Natural moisture content (w)	%	(m3-m2)/(m2-m1)	0.367	0.363	0.360
Average moisture content (w)	%	(w1+w2+w3)/3	**0.363**		

Specific Gravity

Table 5: Table Showing Calculation of Specific Gravity

Description	Unit	Symbol	Tin		
Tin number	-	-	1	6	24
Mass of gas jar, plate, soil and water	g	m3	86.539	86.601	86.231
Mass of gas jar, plate and soil	g	m2	34.786	35.021	33.421
Mass of gas jar, plate and water	g	m4	83.865	87.441	83.387
Mass of gas jar and plate	g	m1	29.796	30.042	28.431
Mass of soil	g	m2 - m1	4.990	4.979	4.990
Mass of water in full jar	g	m4 - m1	54.069	57.399	54.956
Mass of water used	g	m3 - m2	51.753	51.580	52.810
Volume of soil particles	mL	(m4- m1) - (m3-m2)	2.316	5.819	2.146
Particle density	(Mg/m3)	(m2-m1)/{ (m4 -m1) - (m3 -m2)}	2.155	0.856	2.325
Average value	(Mg/m3)		**1.778**		

Moisture Content vs Dry Density

Table 6: Table Showing Water Added for Each Test

Test No.	Sample Mass	Water Added	Vol. Water Added
1	1700 g	32%	544 mL
2	1700 g	28%	476 mL
3	1700 g	25%	425 mL
4	1700 g	22%	374 mL
5	1700 g	20%	340 mL
6	1700 g	18%	306 mL

Table 7: Table Showing Dry Density of Test 1

Test No 1.	Unit	Tin		
Mass of tin	g	9.704	9.712	9.808
Mass of tin+wet soil	g	24.016	23.793	30.354
Mass of tin+dry soil	g	20.461	20.321	25.235
Mass of dry soil	g	10.757	10.609	15.427
Moisture loss	g	3.555	3.472	5.119
Moisture content	%	33.048	32.727	33.182
Average moisture	%		**32.986**	
Dry density	Mg/m^3		**1.357**	

Table 8: Table Showing Dry Density of Test 2

Test No 2.	Unit	Tin		
Mass of tin	g	9.707	9.712	9.81
Mass of tin+wet soil	g	28.758	23.889	26.717
Mass of tin+dry soil	g	24.573	20.769	22.999
Mass of dry soil	g	14.866	11.057	13.189
Moisture loss	g	4.185	3.12	3.718
Moisture content	%	28.151	28.217	28.190
Average moisture	%		**28.186**	
Dry density	Mg/m^3		**1.058**	

Table 9: Table Showing Dry Density of Test 3

Test No 3.	Unit	Tin		
Mass of tin	g	8.104	8.048	7.985
Mass of tin+wet soil	g	23.367	23.53	23.535
Mass of tin+dry soil	g	20.306	20.418	20.336
Mass of dry soil	g	12.202	12.37	12.351
Moisture loss	g	3.061	3.112	3.199
Moisture content	%	25.086	25.158	25.901
Average moisture	%		**25.381**	
Dry density	Mg/m^3		**0.844**	

Table 10: Table Showing Dry Density of Test 4

Test No 4.	Unit	Tin		
Mass of tin	g	8.047	8.105	8.06
Mass of tin+wet soil	g	20.743	21.51	18.48
Mass of tin+dry soil	g	18.986	19.766	16.719
Mass of dry soil	g	10.939	11.661	8.659
Moisture loss	g	1.757	1.744	1.761
Moisture content	%	16.062	14.956	20.337
Average moisture	%		**17.118**	
Dry density	Mg/m^3		**0.721**	

Table 11: Table Showing Dry Density of Test 5

Test No 5.	Unit	Tin		
Mass of tin	g	8.047	8.105	8.06
Mass of tin+wet soil	g	20.743	21.51	18.48
Mass of tin+dry soil	g	18.986	19.766	16.719
Mass of dry soil	g	10.939	11.661	8.659
Moisture loss	g	1.757	1.744	1.761
Moisture content	%	16.062	14.956	20.337
Average moisture	%		**17.118**	
Dry density	Mg/m^3		**0.615**	

Table 12: Table Showing Dry Density of Test 6

Test No 6.	Unit	Tin		
Mass of tin	g	8.208	9.764	9.658
Mass of tin+wet soil	g	13.163	20.329	17.675
Mass of tin+dry soil	g	12.436	18.728	16.463
Mass of dry soil	g	4.228	8.964	6.805
Moisture loss	g	0.727	1.601	1.212
Moisture content	%	17.195	17.860	17.810
Average moisture	%		**17.622**	
Dry density	Mg/m^3		**0.523**	

Table 13: Table Showing Bulk Density from a Test 1 Sample

Description	Unit	Quantity
mould diameter	mm	103
mould height	mm	116
mould volume	mm^3	961
mould mass	g	4413
mould+wet soil mass	g	6147
mass of wet soil	g	1734
bulk density	Mg/m^3	1.804

Table 14: Table Showing Bulk Density from Test 2 Sample

Description	Unit	Quantity
mould diameter	mm	101.76
mould height	mm	116.36
mould volume	mm^3	948
mould mass	g	4440
mould+wet soil mass	g	6252
mass of wet soil	g	1812
bulk density	Mg/m^3	1.911

Table 15: Table Showing Bulk Density from Test 3 Sample

Description	Unit	Quantity
mould diameter	mm	102.6
mould height	mm	116.23
mould volume	mm^3	961
mould mass	g	4413
mould+wet soil mass	g	6264
mass of wet soil	g	1851
bulk density	Mg/m^3	1.926

Table 16: Table Showing Bulk Density from Test 4 Sample

Description	Unit	Quantity
mould diameter	mm	102.6
mould height	mm	116.23
mould volume	mm^3	961
mould mass	g	4413
mould+wet soil mass	g	6258
mass of wet soil	g	1845
bulk density	Mg/m^3	1.920

Table 17: Table Showing Bulk Density from Test 5 Sample

Description	Unit	Quantity
mould diameter	mm	102.6
mould height	mm	116.23
mould volume	mm^3	961
mould mass	g	4413
mould+wet soil mass	g	6067
mass of wet soil	g	1654
bulk density	Mg/m^3	1.721

Table 18: Table Showing Bulk Density from Test 6 Sample

Description	Unit	Quantity
mould diameter	mm	102.6
mould height	mm	116.23
mould volume	mm^3	961
mould mass	g	4413
mould+wet soil mass	g	6017
mass of wet soil	g	1604
bulk density	Mg/m^3	1.669

Table 19: Test Summary

Average moisture (%)	Dry density (Mg/m^3)
32.986	1.357
28.186	1.491
25.381	1.536
21.532	1.580
19.926	1.470
17.622	1.419

Table 20: Table Showing OMC and Dry Density

OMC	Dry density
22.00%	1.58 Mg/m^3

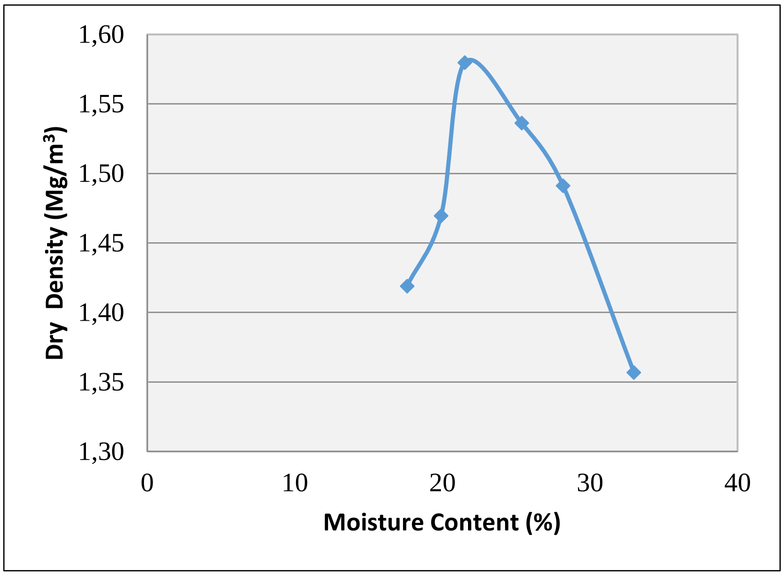

Figure 1: Graph Showing Relationship between Dry Density and Moisture Content

II. Material properties of the geosynthetics

MACTEX W1 8S (Woven geotextile)

Table 21: Summary of the material properties of the Woven geotextile (as provided by Maccaferri)

Mechanical and Hydraulic properties			**8S**
Tensile Strength (MD)	EN ISO 10319	kN/m	83
Tolerance			-3.0
Strain at max load	EN ISO 10319	%	19
Tolerance			±3
Tensile Strength (CD)	EN ISO 10319	kN/m	81
Tolerance			-5
Strain at max load (CD)	EN ISO 10319	%	15
Tolerance			±3
Static Puncture Resistance - CBR	EN ISO 12236	kN	10.0
Tolerance			-0.5
Dynamic Puncture Resistance – Cone Drop	EN ISO 13433	mm	7.5
Tolerance			+2
Permeability – normal to plane	EN ISO 11058	m/sec	0.022
Tolerance			-0.005
Opening Pore Size O_{90}	EN ISO 12956	μm	120
Tolerance			±50
Physical Properties - typical			
Warp and weft polymers		Polypropylene	
Roll width	m	Ranging from 4 to 5.2	
Roll length	m		100
Durability		The geotextile has to be covered within 1 month; it is durable at least for 25 years in natural soils with 4<pH<9	

MACTEX H40.1 (Non-woven geotextile)

Table 22: Summary of material properties for the Non-woven geotextile (as provided by Maccaferri)

PROPERTY[4]	TEST PROCEDURE	UNITS	MINIMUM AVERAGE ROLL VALUES (MARV)[2]
Mechanical			
Grab Tensile	ASTM D 4632	lb (kN)	160 (0.712)
Grab Elongation	ASTM D 4632	%	50
Trapezoidal Tear	ASTM D 4533	lb (kN)	60 (0.267)
Puncture (CBR)	ASTM D 6241	lb (kN)	410 (1.824)
Endurance			
UV Resistance	ASTM D 4355	% Retained @ 500 hrs.	70
Hydraulic			
Permittivity	ASTM D 4491	sec^{-1}	1.3
Flow Rate	ASTM D 4491	gpm/ft^2 (lpm/m^2)	110 (4482)
Apparent Opening Size (AOS)[3]	ASTM D 4751	US Sieve (mm)	70 (0.212)
Packaging (Typical)			
Roll Width	Measured	ft (m)	12.5 (3.81) / 15 (4.57)
Roll Length	Measured	ft (m)	360 (109.73) / 300 (91.44)
Roll Area	Measured	yd^2 (m^2)	500 (418)
Roll Weight	Calculated	lb (kg)	230 (104)

MACGRID WG 8S (Woven geogrid)

Table 23: Summary of material properties of the Woven geogrid (as provided by Maccaferri)

Mechanical Properties (typical values)		**8S**
Tensile Strength – MD (ISO 10319)	kN/m	80.0
Tensile strength at 2% strain – MD (ISO 10319)	kN/m	18.0
Tensile strength at 5% strain - MD (ISO 10319)	kN/m	38.0
Strain at max strength - MD (ISO 10319)	%	11
Tensile Strength – CMD (ISO 10319)	kN/m	80.0
Tensile strength at 2% strain – CMD (ISO 10319)	kN/m	18.0
Tensile strength at 5% strain - CMD (ISO 10319)	kN/m	38.0
Strain at max strength - CMD (ISO 10319)	%	11
Physical Properties		
Mesh size (± 20%)		25 x 25
Geogrid Structure		High Tenacity Polyester
Polymer coating (standard)		PVC
Roll width (standard)	m	3.9
Roll length	m	100

MACGRID EG40 (Extruded geogrid)

Table 24: Summary of material properties of the Extruded geogrid (as provided by Maccaferri)

Mechanical Properties		**40S**
Minimum Average Tensile Strength	kN/m	40.0
Tensile strength at 2% strain - Longitudinal	kN/m	14.0
Tensile strength at 5% strain - Longitudinal	kN/m	28.0
Typical strain at M.A.T.S - Longitudinal	%	13
Minimum Average Tensile Strength	kN/m	40.0
Tensile strength at 2% strain - Transverse	kN/m	14.0
Tensile strength at 5% strain - Transverse	kN/m	28.0
Typical strain at M.A.T.S - Transverse	%	10
Typical junction strength efficiency	%	95
Physical – Chemical Properties		
Grid Structure		Extruded Bi – axial
Polymer		100% stabilized UV polypropylene
Carbon Black content	%	≥ 2
Color		Black
Mesh Opening size nominal value	mm	38 x 38
Roll Length	m	50
Roll Width	m	3.95

III. Tensile Test Graphs

Non-woven geotextile

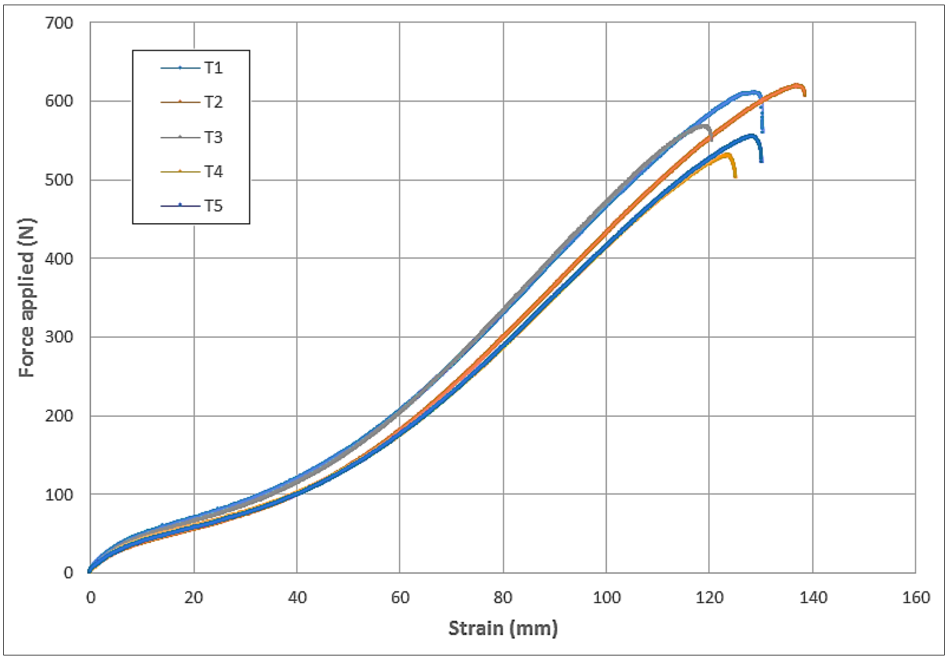

Figure 2: Force applied against strain for the tensile test of the non-woven geotextile

Extruded geogrid

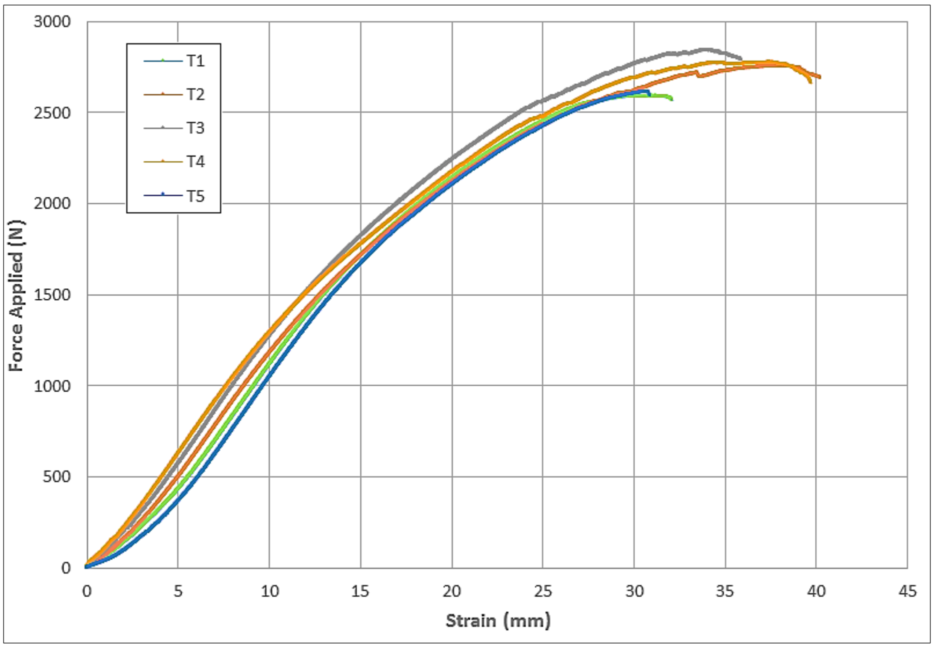

Figure 3: Force applied against strain for the tensile test of the extruded geogrid

Woven geogrid

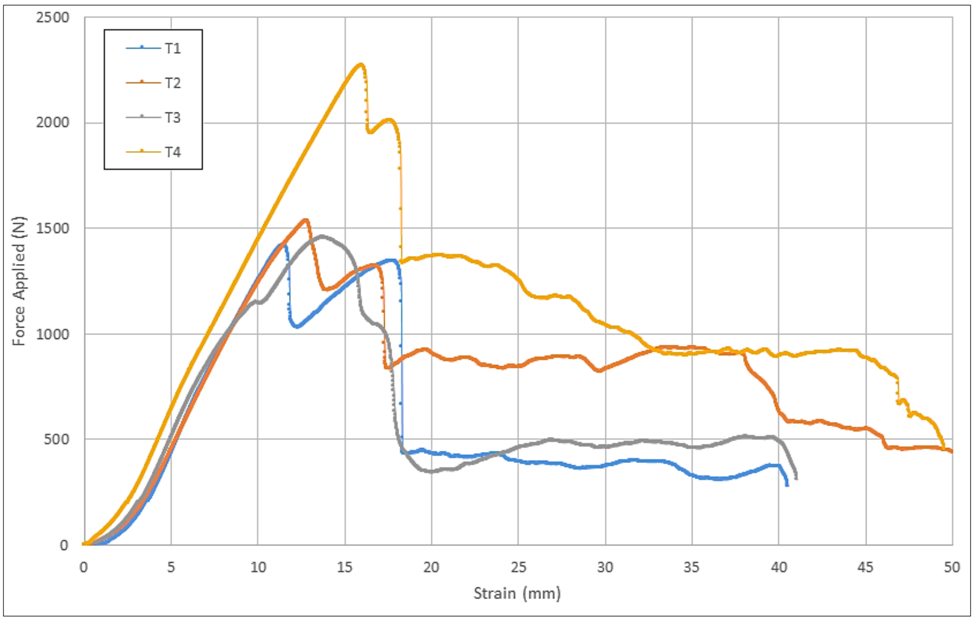

Figure 4: Force applied against strain for the tensile test of the woven geogrid

Woven geotextile

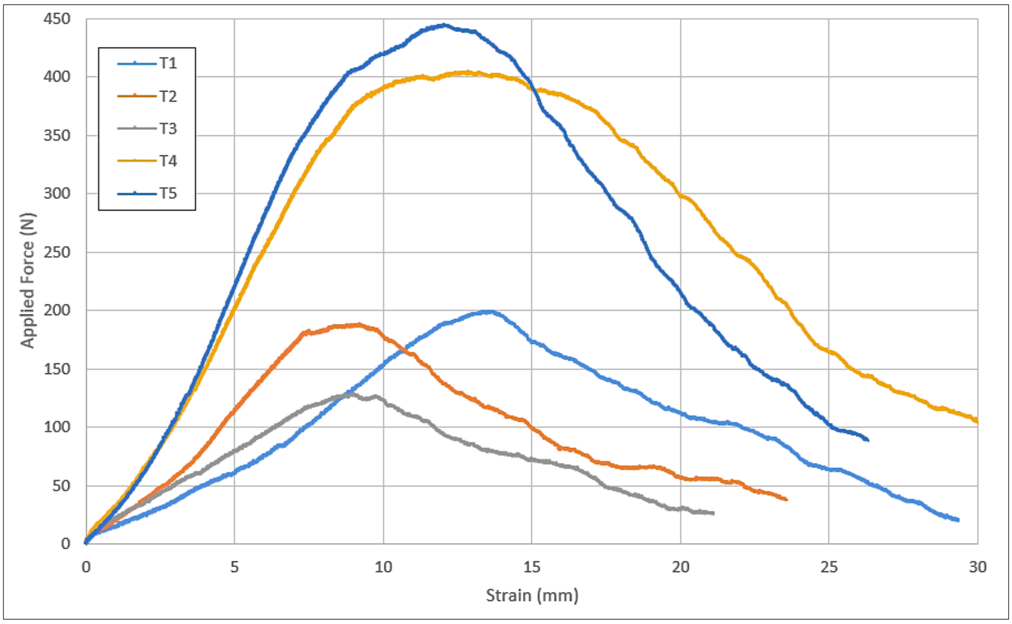

Figure 5: Force applied against strain for the tensile test of the woven geotextile

IV. Testing Schedule

Unreinforced Tests (UR)

Table 25: Table of the testing schedule for the unreinforced multi-layered soil

Base Plate Width (B)	Thickness of granular material (Z)	Thickness to Width Ratio (Z/B)	Test No.
75	0	0	UR/B75/Z0
	50	0.67	UR/B75/Z50
	75	1	UR/B75/Z75
	112.5	1.5	UR/B75/Z112.5
150	0	0	UR/B150/Z0
	50	0.33	UR/B150/Z50
	75	0.5	UR/B150/Z75
	112.5	0.75	UR/B150/Z112.5
	150	1	UR/B150/Z150

Woven Geogrid (WGG)

Table 26: Table of the testing schedule for the 75 mm base plate using the woven geogrid (WGG).

Base Plate Width (B)	Thickness of granular material (Z)	Thickness to Width Ratio (Z/B)	Depth of placement (D)	Depth to Width Ratio (D/B)	Test No.
75	50	0.67	50	0.67	WGG/B75/Z50/D50
	75	1	50	0.67	WGG/B75/Z75/D50
			75	1	WGG/B75/Z75/D75
	112.5	1.5	50	0.67	WGG/B75/Z112.5/D50
			75	1	WGG/B75/Z112.5/D75
			112.5	1.5	WGG/B75/Z112.5/D112.5

Table 27: Table of the testing schedule for the 150 mm base plate using the woven geogrid (WGG).

Base Plate Width (B)	Thickness of granular material (Z)	Thickness to Width Ratio (Z/B)	Depth of placement (D)	Depth to Width Ratio (D/B)	Test No.
150	50	0.33	50	0.33	WGG/B150/Z50/D50
	75	0.5	50	0.33	WGG/B150/Z750/D50
			75	0.5	WGG/B150/Z750/D75
	112.5	0.75	50	0.33	WGG/B150/Z112.5/D50
			75	0.5	WGG/B150/Z112.5/D75
			112.5	0.75	WGG/B150/Z112.5/D112.5
	150	1	50	0.33	WGG/B150/Z150/D50
			75	0.5	WGG/B150/Z150/D175
			112.5	0.75	WGG/B150/Z150/D112.5
			150	1	WGG/B150/Z150/D150

Extruded Geogrid (EGG)

Table 28: Table of the testing schedule for the 75 mm base plate using the woven geogrid (EGG).

Base Plate Width (B)	Thickness of granular material (Z)	Thickness to Width Ratio (Z/B)	Depth of placement (D)	Depth to Width Ratio (D/B)	Test No.
75	50	0.67	50	0.67	EGG/B75/Z50/D50
	75	1	50	0.67	EGG/B75/Z75/D50
			75	1	EGG/B75/Z75/D75
	112.5	1.5	50	0.67	EGG/B75/Z112.5/D50
			75	1	EGG/B75/Z112.5/D75
			112.5	1.5	EGG/B75/Z112.5/D112.5

Table 29: Table of the testing schedule for the 150 mm base plate using the woven geogrid (EGG).

Base Plate Width (B)	Thickness of granular material (Z)	Thickness to Width Ratio (Z/B)	Depth of placement (D)	Depth to Width Ratio (D/B)	Test No.
150	50	0.33	50	0.33	EGG/B150/Z50/D50
	75	0.5	50	0.33	EGG/B150/Z750/D50
			75	0.5	EGG/B150/Z750/D75
	112.5	0.75	50	0.33	EGG/B150/Z112.5/D50
			75	0.5	EGG/B150/Z112.5/D75
			112.5	0.75	EGG/B150/Z112.5/D112.5
	150	1	50	0.33	EGG/B150/Z150/D50
			75	0.5	EGG/B150/Z150/D175
			112.5	0.75	EGG/B150/Z150/D112.5
			150	1	EGG/B150/Z150/D150

Woven Geotextile (WGT)

Table 30: Table of the testing schedule for the 75 mm base plate using the woven geogrid (WGT).

Base Plate Width (B)	Thickness of granular material (Z)	Thickness to Width Ratio (Z/B)	Depth of placement (D)	Depth to Width Ratio (D/B)	Test No.
75	50	0.67	50	0.67	WGT/B75/Z50/D50
	75	1	50	0.67	WGT/B75/Z75/D50
			75	1	WGT/B75/Z75/D75
	112.5	1.5	50	0.67	WGT/B75/Z112.5/D50
			75	1	WGT/B75/Z112.5/D75
			112.5	1.5	WGT/B75/Z112.5/D112.5

Table 31: Table of the testing schedule for the 150 mm base plate using the woven geogrid (WGT).

Base Plate Width (B)	Thickness of granular material (Z)	Thickness to Width Ratio (Z/B)	Depth of placement (D)	Depth to Width Ratio (D/B)	Test No.
150	50	0.33	50	0.33	WGT/B150/Z50/D50
	75	0.5	50	0.33	WGT/B150/Z750/D50
			75	0.5	WGT/B150/Z750/D75
	112.5	0.75	50	0.33	WGT/B150/Z112.5/D50
			75	0.5	WGT/B150/Z112.5/D75
			112.5	0.75	WGT/B150/Z112.5/D112.5
	150	1	50	0.33	WGT/B150/Z150/D50
			75	0.5	WGT/B150/Z150/D175
			112.5	0.75	WGT/B150/Z150/D112.5
			150	1	WGT/B150/Z150/D150

Non-woven Geotextile (NGT)

Table 32: Table of the testing schedule for the 75 mm base plate using the woven geogrid (NGT).

Base Plate Width (B)	Thickness of granular material (Z)	Thickness to Width Ratio (Z/B)	Depth of placement (D)	Depth to Width Ratio (D/B)	Test No.
75	50	0.67	50	0.67	NGT/B75/Z50/D50
	75	1	50	0.67	NGT/B75/Z75/D50
			75	1	NGT/B75/Z75/D75
	112.5	1.5	50	0.67	NGT/B75/Z112.5/D50
			75	1	NGT/B75/Z112.5/D75
			112.5	1.5	NGT/B75/Z112.5/D112.5

Table 33: Table of the testing schedule for the 150 mm base plate using the woven geogrid (NGT).

Base Plate Width (B)	Thickness of granular material (Z)	Thickness to Width Ratio (Z/B)	Depth of placement (D)	Depth to Width Ratio (D/B)	Test No.
150	50	0.33	50	0.33	NGT/B150/Z50/D50
	75	0.5	50	0.33	NGT/B150/Z750/D50
			75	0.5	NGT/B150/Z750/D75
	112.5	0.75	50	0.33	NGT/B150/Z112.5/D50
			75	0.5	NGT/B150/Z112.5/D75
			112.5	0.75	NGT/B150/Z112.5/D112.5
	150	1	50	0.33	NGT/B150/Z150/D50
			75	0.5	NGT/B150/Z150/D175
			112.5	0.75	NGT/B150/Z150/D112.5
			150	1	NGT/B150/Z150/D150

V. Test Result Graphs

75 mm Base Plate

Woven Geogrid (WGG)

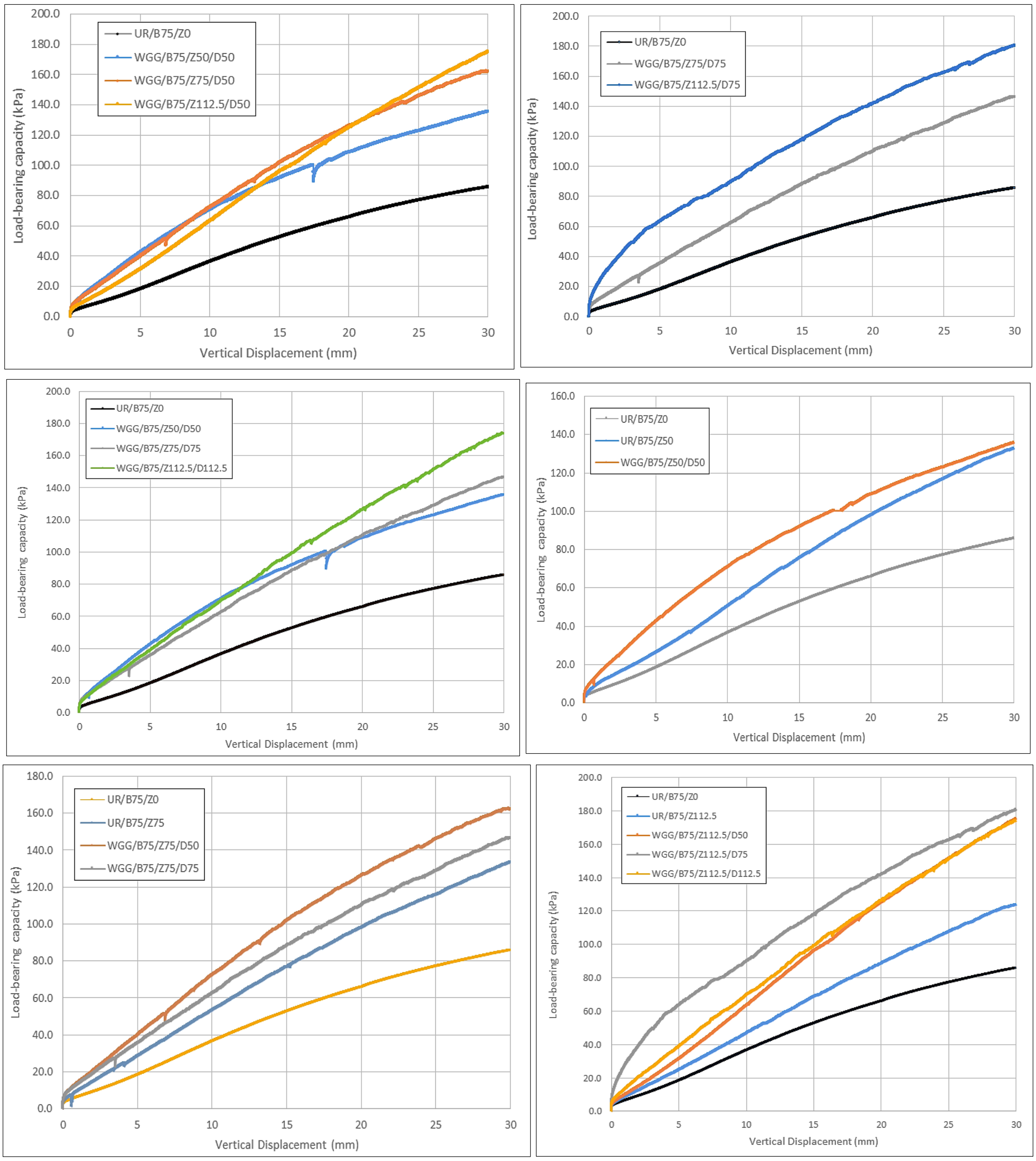

Figure 6: Load-bearing capacity against vertical displacement for the woven geogrid (WGG) placed at various depths (D) for various fill thicknesses (Z) for the 75 mm base plate.

Extruded Geogrid (EGG)

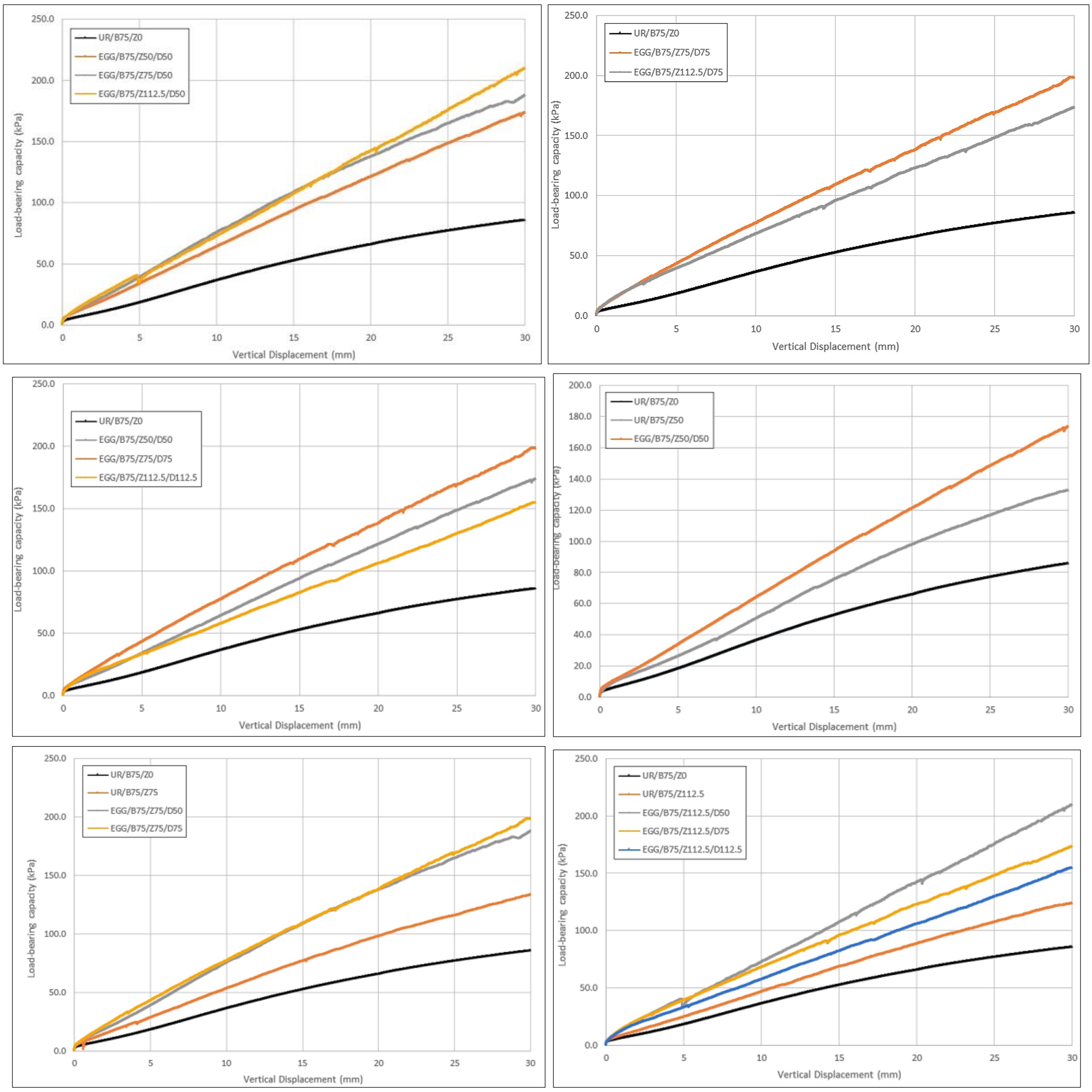

Figure 7: Load-bearing capacity against vertical displacement for the extruded geogrid (EGG) placed at various depths (D) for various fill thicknesses (Z) for the 75 mm base plate.

Woven Geotextile (WGT)

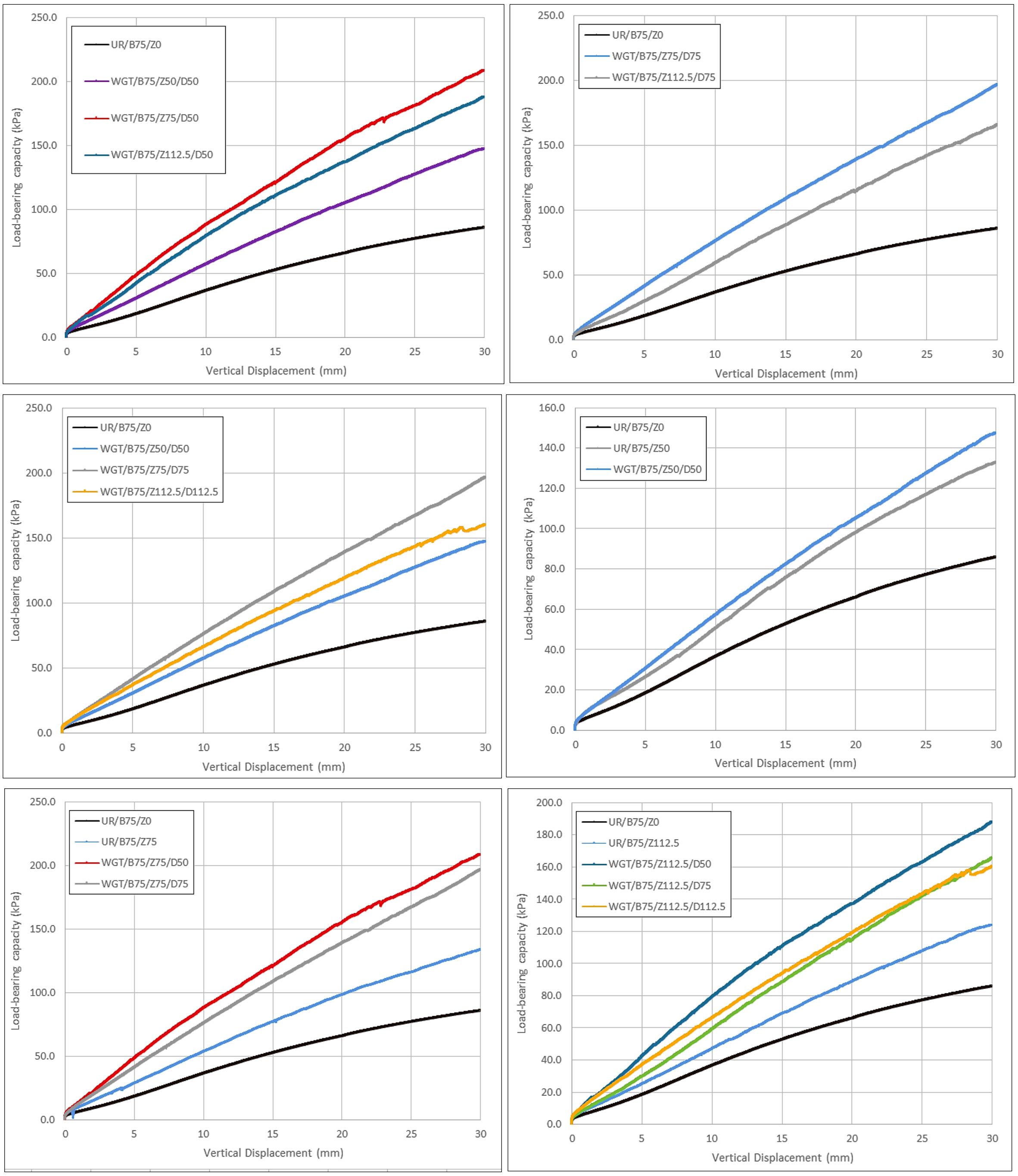

Figure 8: Load-bearing capacity against vertical displacement for the woven geotextile (WGT) placed at various depths (D) for various fill thicknesses (Z) for the 75 mm base plate.

Non-woven Geotextile (NGT)

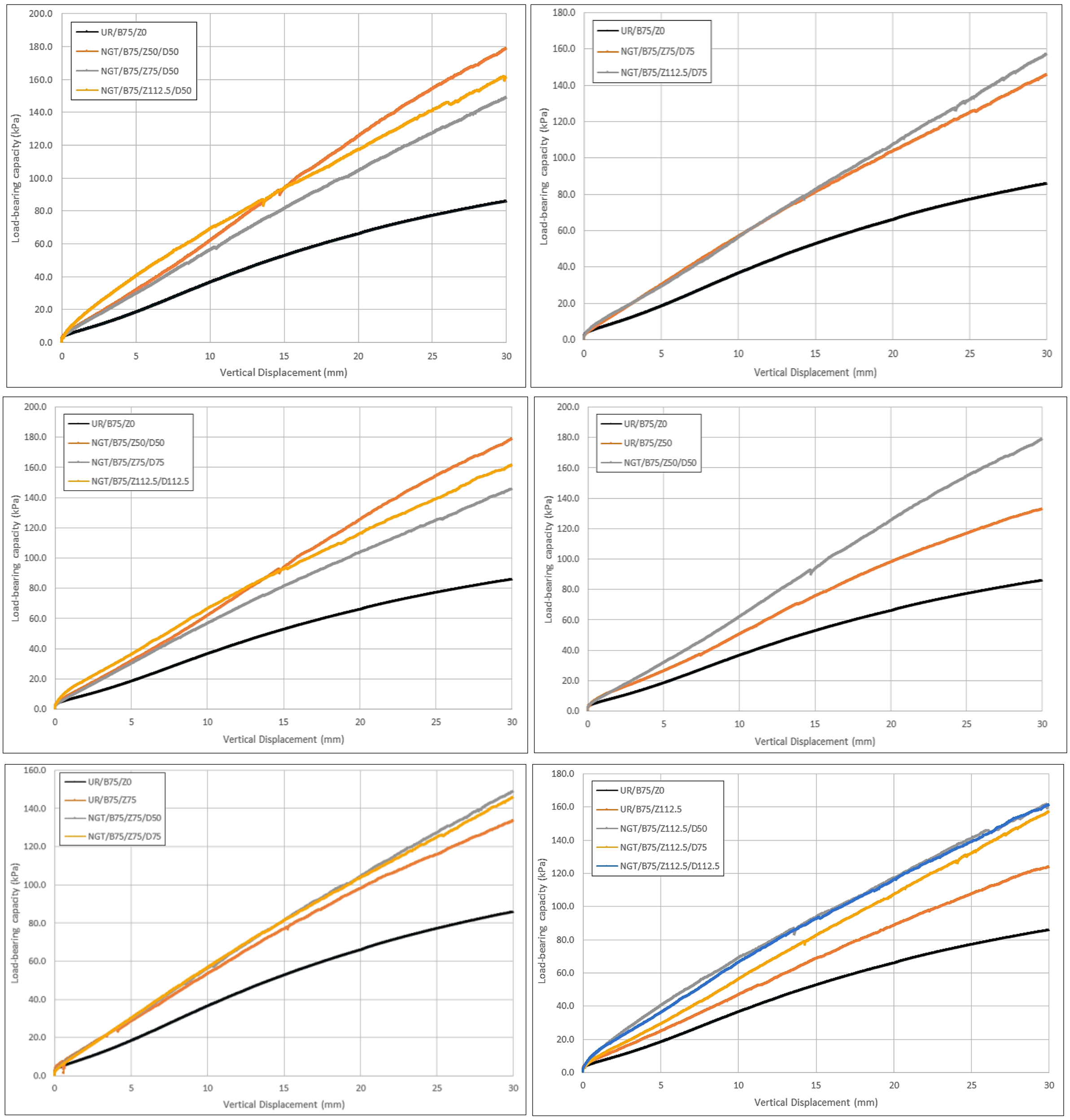

Figure 9: Load-bearing capacity against vertical displacement for the non- woven geogrid (NGT) placed at various depths (D) for various fill thicknesses (Z) for the 75 mm base plate.

150 mm Base Plate

Woven Geogrid (WGG)

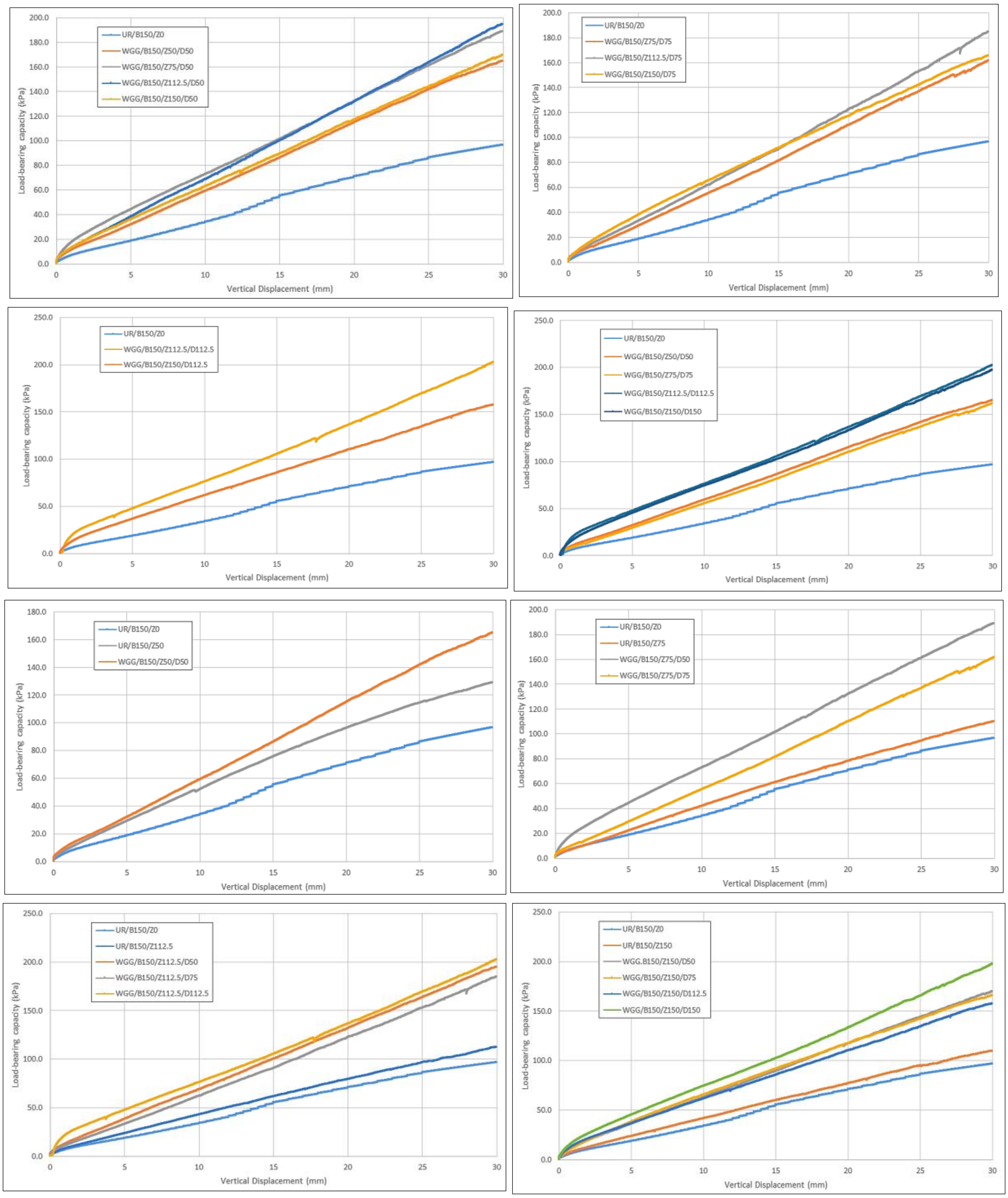

Figure 10: Load-bearing capacity against vertical displacement for the woven geogrid (WGG) placed at various depths (D) for various fill thicknesses (Z) for the 150 mm base plate.

Extruded Geogrid (EGG)

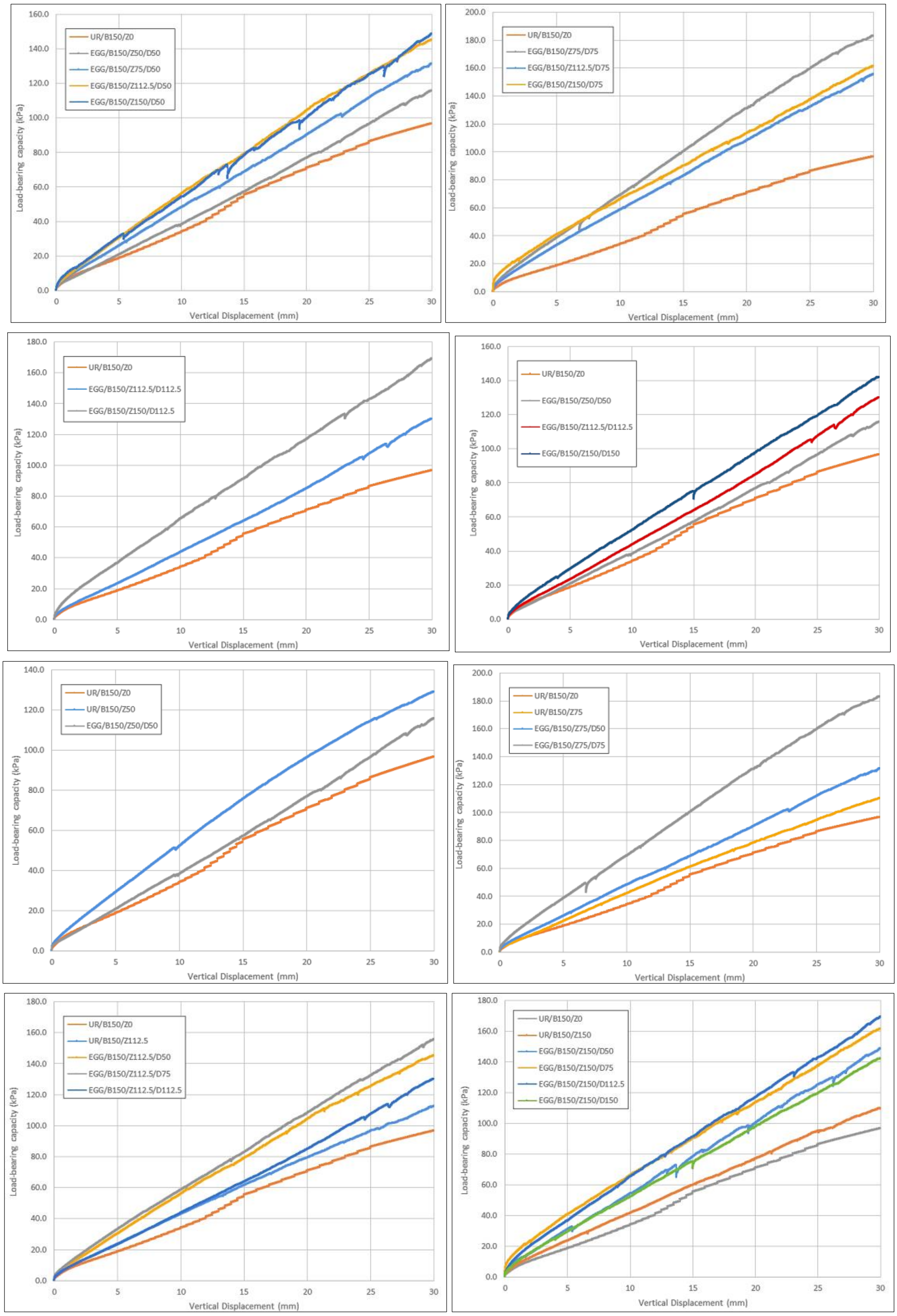

Figure 11: Load-bearing capacity against vertical displacement for the extruded geogrid (EGG) placed at various depths (D) for various fill thicknesses (Z) for the 150 mm base plate.

Woven Geotextile (WGT)

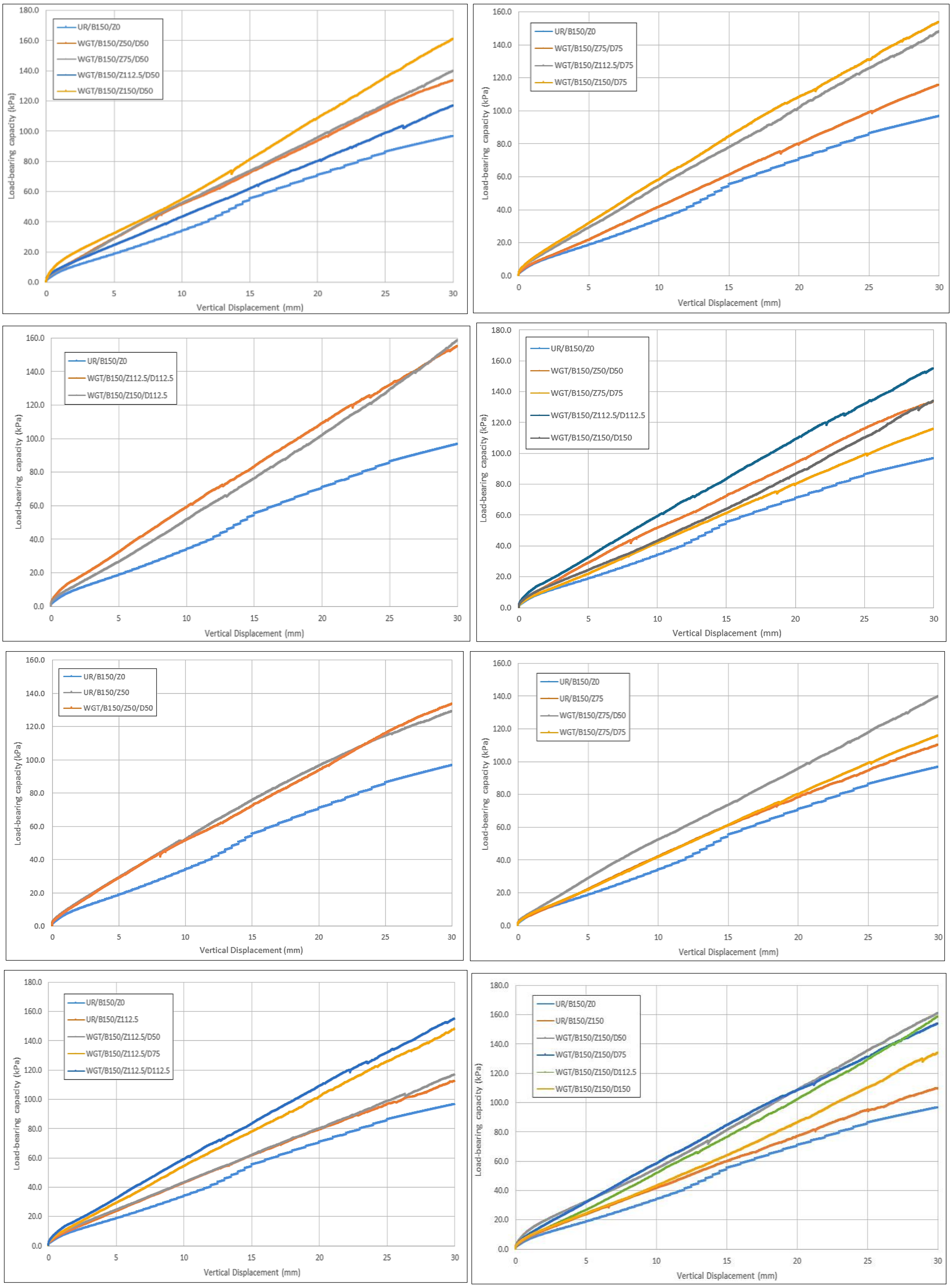

Figure 12: Load-bearing capacity against vertical displacement for the woven geotextile (WGT) placed at various depths (D) for various fill thicknesses (Z) for the 150 mm base plate.

Non-woven Geotextile (NGT)

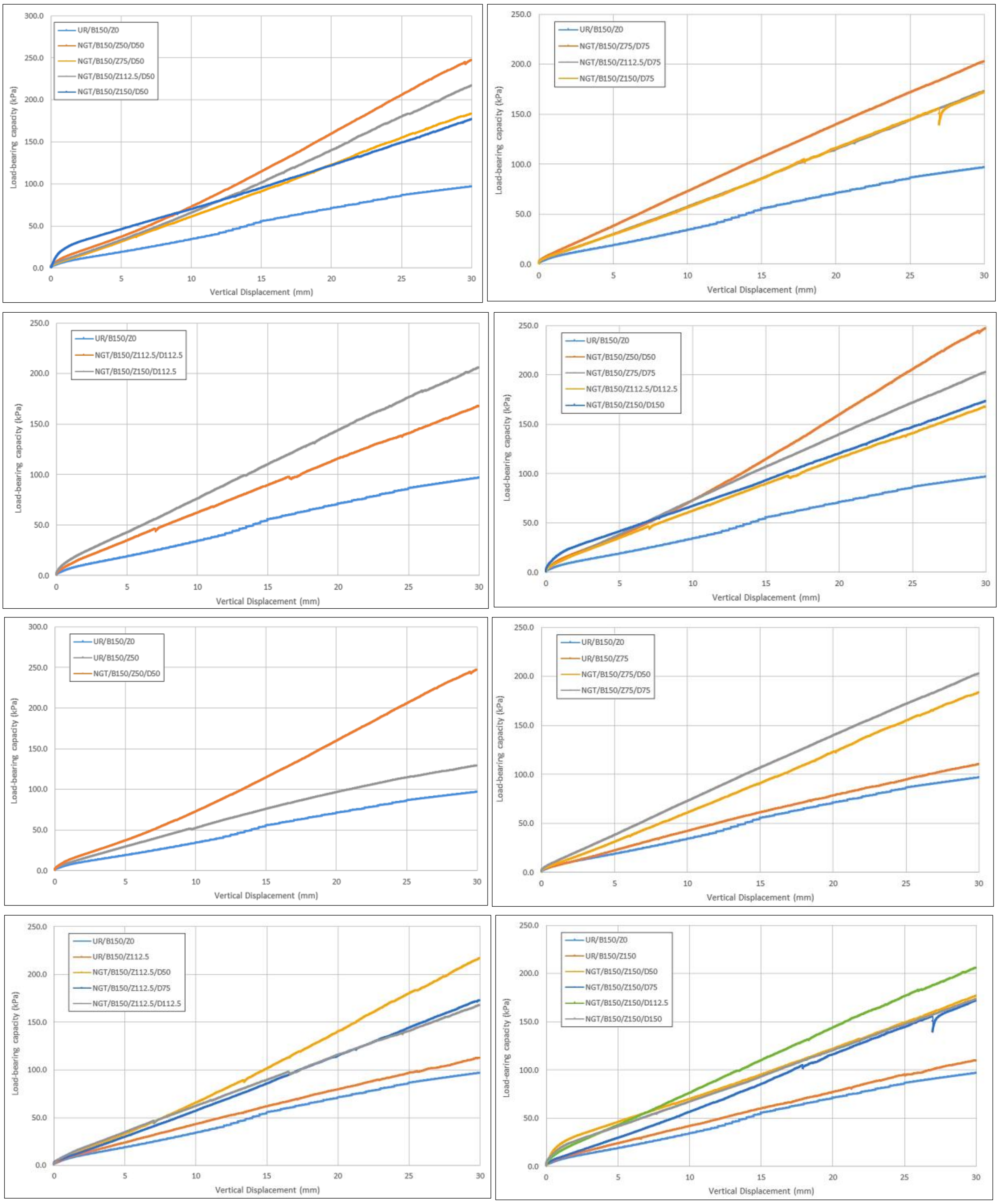

Figure 13: Load-bearing capacity against vertical displacement for the non-woven geotextile (NGT) placed at various depths (D) for various fill thicknesses (Z) for the 150 mm base plate.

VI. Sample Calculations

Hansen's Method

The calculations for the Hansen's method for two layered soils are summarized in the Tables 34 and 35, for the 75 mm and 150 mmm footings respectively.

For the 75 mm footing;

B = 0.075 m

L = 0.014 m.

h_{subgrade} = 0.25 m

z_{max} = $h_{fill} + h_{\text{subgrade}}$

$\gamma_{fill} = 21.952\ kN/m^3$

$\gamma_{subgrade} = 15.484$ kN/m³

For granular thickness of 50 mm;

$h_{fill} = 0.05\ m$;

$z_{max} = 0.05 + 0.25$ = 0.3 m

$\bar{\gamma} = \sum_{i=1}^{n} \gamma_i\, h_i/z_{max} = \frac{21.952 \text{ x } 0.05\, +\, 15.484 \text{ x } 0.25}{0.3} = 16.562$ kN$/m^3$

$\bar{c} = \sum_{i=1}^{n} c_i\, h_i/z_{max} = \frac{(25\ x\ 0.05\, + 4.7\ x\ 0.25)}{0.3} = 8.083\ kN/m^2$

$\bar{\varphi} = \sum_{i=1}^{n} \varphi_i\, h_i/z_{max} = \frac{(35\ x\ 0.05\, + 17.4\ x\ 0.25)}{0.3} = 20.33°$

$s_c = 1 + \frac{N_q}{N_c}\frac{B}{L} = 1 + \left(\frac{5.1}{12.8228}\right)\left(\frac{0.075}{0.14}\right) = 1.2131$

$s_q = 1 + (\frac{B}{L}) sin\varphi = 1 + \left(\frac{0.075}{0.14}\right) sin 20.33 = 1.18612$

$s_\gamma = 1 - 0.4\frac{B}{L} = 1 - 0.4\left(\frac{0.075}{0.14}\right) = 0.78571$

Table 34: Summary of calculations for the 75 mm footing, B = 0.075 m.

h_{fill}	z_{max}	$\bar{\gamma}$	$\bar{c}$	$\bar{\varphi}$	s_c	s_γ	N_c	N_γ	q_u
m	**m**	**kN/m³**	**kPa**	**°**	**-**	**-**	**-**	**-**	**kPa**
0.05	0.3	16.562	8.083	20.33	1.2131	0.7857	15.2098	3.2245	153.4622
0.075	0.325	16.976	9.384	21.461	1.2478	0.7857	16.5487	7.6569	195.8196
0.1125	0.3625	17.491	11.000	22.862	1.26089	0.7857	18.1957	8.8613	255.0176

Table 35: Summary of calculations for the 150 mm footing, B = 0.150 m.

h_{fill}	z_{max}	$\overline{\gamma}$	$\bar{c}$	$\overline{\varphi}$	N_c	N_q	N_γ	s_c	s_γ	q_u
m	**m**	**kN/m³**	**kPa**	°	-	-	-	-	-	**kPa**
0.05	0.3	16.562	8.08333	20.3333	15.2098	6.6838	3.2245	1.4708	0.5714	183.1206
0.075	0.325	16.9766	9.38461	21.4615	16.5488	7.6569	4.0400	1.4957	0.5714	235.2331
0.1125	0.3625	17.4913	11.000	22.8620	18.1958	8.8614	5.1324	1.5217	0.5714	308.4388
0.15	0.4	17.9095	12.3125	24.0000	19.534	9.84	6.02	1.5397	0.5714	374.941

Projected Area Method

The calculations for the "Projected Area Method" are presented below, where H is the granular fill thickness;

The angle of friction used in determination of the load spread angle is for the top layer, which was the granular material. The angle of internal friction of the granular material was 40°, and thus;

$$\alpha = 45° + \frac{\varphi}{2} = 45° + \frac{40°}{2} = 65°$$

For the 75 mm footing; B = 0.075 m

The bearing capacity at the base layer is determined using the Hansen's equation, as shown:

$$q_b = cN_c s_c + \frac{1}{2}\gamma B N_\gamma s_\gamma$$

$$q_b = (4.7\ x\ 12.8228\ x\ 1.2131) + \frac{1}{2}(15.484\ x\ 0.075\ x\ 0.78571\ x\ 2.016) = 74.02\ kPa$$

For granular fill thickness of 50 mm; H = 0.05 m, and the bearing capacity is calculated using Equation 10;

$$q_u = \frac{(74.02(0.075 + 2\ x\ 0.05\ x \tan 62.5))^2}{0.075^2} = 68.2331\ kPa$$

For H = 0.075 m;

q_u = (74.02 x (0.075+2 x 0.075 x tan 62.5)^2)/0.075^2 = 91.6115 kPa

For H = 0.1125;

q_u = (74.02 x (0.075+2 x 0.1125 x tan 62.5)^2)/0.075^2 = 100.4023 kPa

Chen's Method

Chen's method involved calculations of the bearing capacity provided by the geosynthetic reinforcement. This led to the use of the equation:

$$\Delta q_T = \sum_{i=1}^{n} \frac{4T_i(u + (i-1)h)}{B^2}$$

Which was adjusted for the configuration that had 1 layer of reinforcement, thus the equation used was:

$$\Delta q_T = \frac{4T_i D}{B^2}$$

Where: D is the depth of placement of the reinforcement, B the width if the footing, and T_i the tensile strength of the geosynthetic.

Bearing Capacity Equation for Geosynthetic Reinforcement of a Two-Layered Soil

The equation used for the calculations was a combination of the projected area method and Chen's method and was given by:

$$q_u = \frac{q_b(B + 2Htan\alpha)^2}{B^2} + \Delta q_T$$

For 75 mm footing, B = 0.075 m

$$q_b = (4.7\ x\ 12.8228\ x\ 1.2131) + \frac{1}{2}(15.484\ x\ 0.075\ x\ 3.7\ x\ 2.016) = 77.4412\ kPa$$

For granular thickness, H = 0.05 m

$$q_u = \frac{77.4412\ x\ (0.075 + 2\ x\ 0.05\ x \tan 62.5)^2}{0.075^2} = 73.6636\ kPa$$

Table 36 shows the calculation for different depths of placement for a woven geogrid using Chen' equation for geosynthetics.

For 75 mm footing, B = 0.075 m; and for the woven geogrid $T_i = 5.2468\ kN/m$

Table 36: Calculation of the bearing capacity provided by the geosynthetic

D (m)	**Δq_T (kN/m^2)**
0.05	186.5529
0.075	279.8293
0.1125	419.7440

Therefore, for a granular thickness of 50 mm and the depth of placement of a woven geogrid at 50 mm, the calculated bearing capacity of the composite would be;

$$q_u = 73.6636 + 186.5529$$

$$q_u = 260.2165\ kPa$$