PERFORMANCE EVALUATION OF GEOGRID REINFORCED PAVEMENT SUBGRADE USING NUMERICAL ANALYSIS; A CASE OF TERCHA-CHIDA ROAD PROJECT

A MASTER'S THESIS

BY

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DEPARTMENT OF CIVIL ENGINEERING

ADDIS ABABA SCIENCE AND TECHNOLOGY UNIVERSITY

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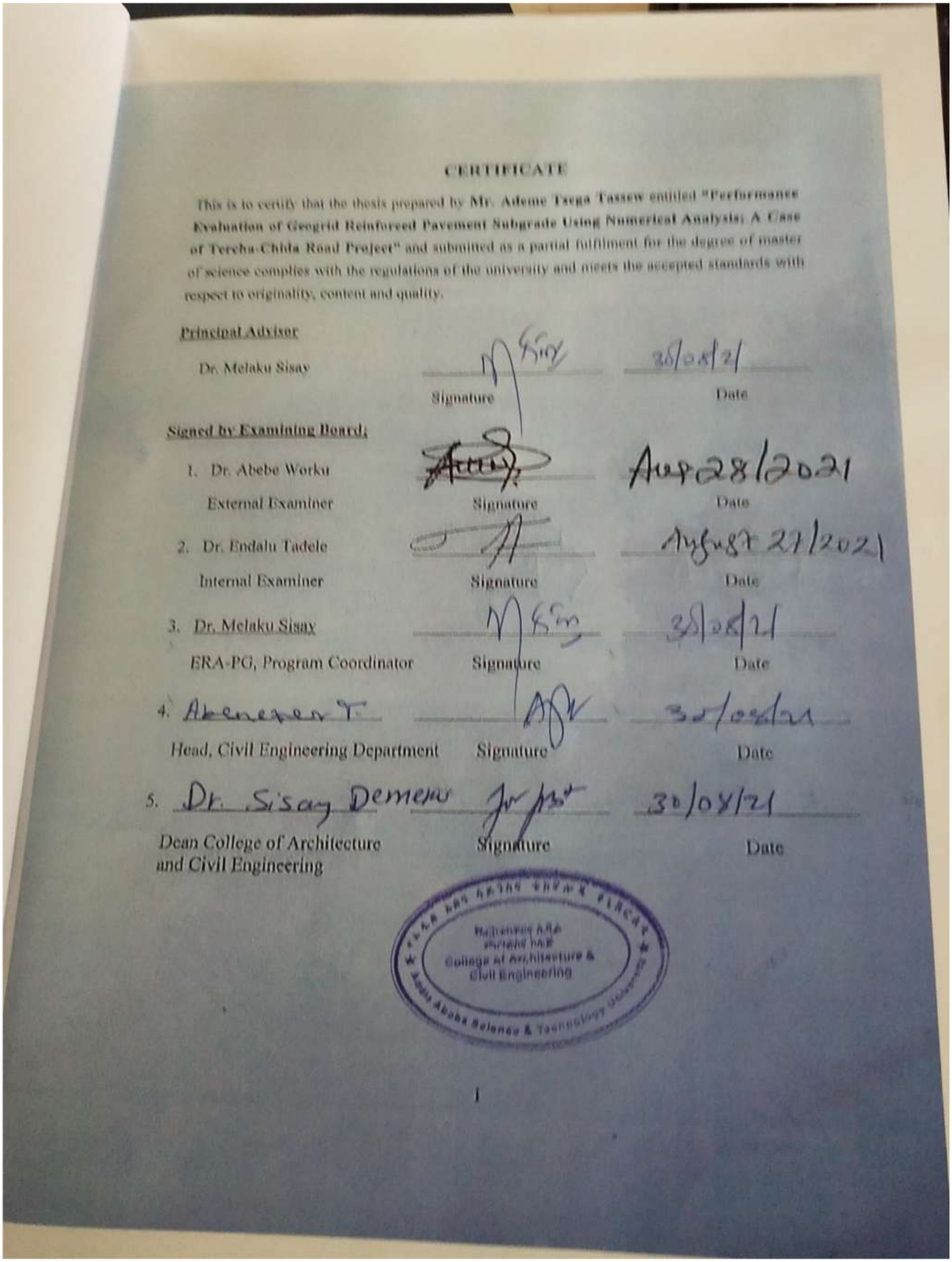

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This is to certify that the thesis prepared by **Mr. Ademe Tsega Tassew** entitled **"Performance Evaluation of Geogrid Reinforced Pavement Subgrade Using Numerical Analysis: A Case of Tercha-Chida Road Project"** and submitted as a partial fulfilment for the degree of master of science complies with the regulations of the university and meets the accepted standards with respect to originality, content and quality.

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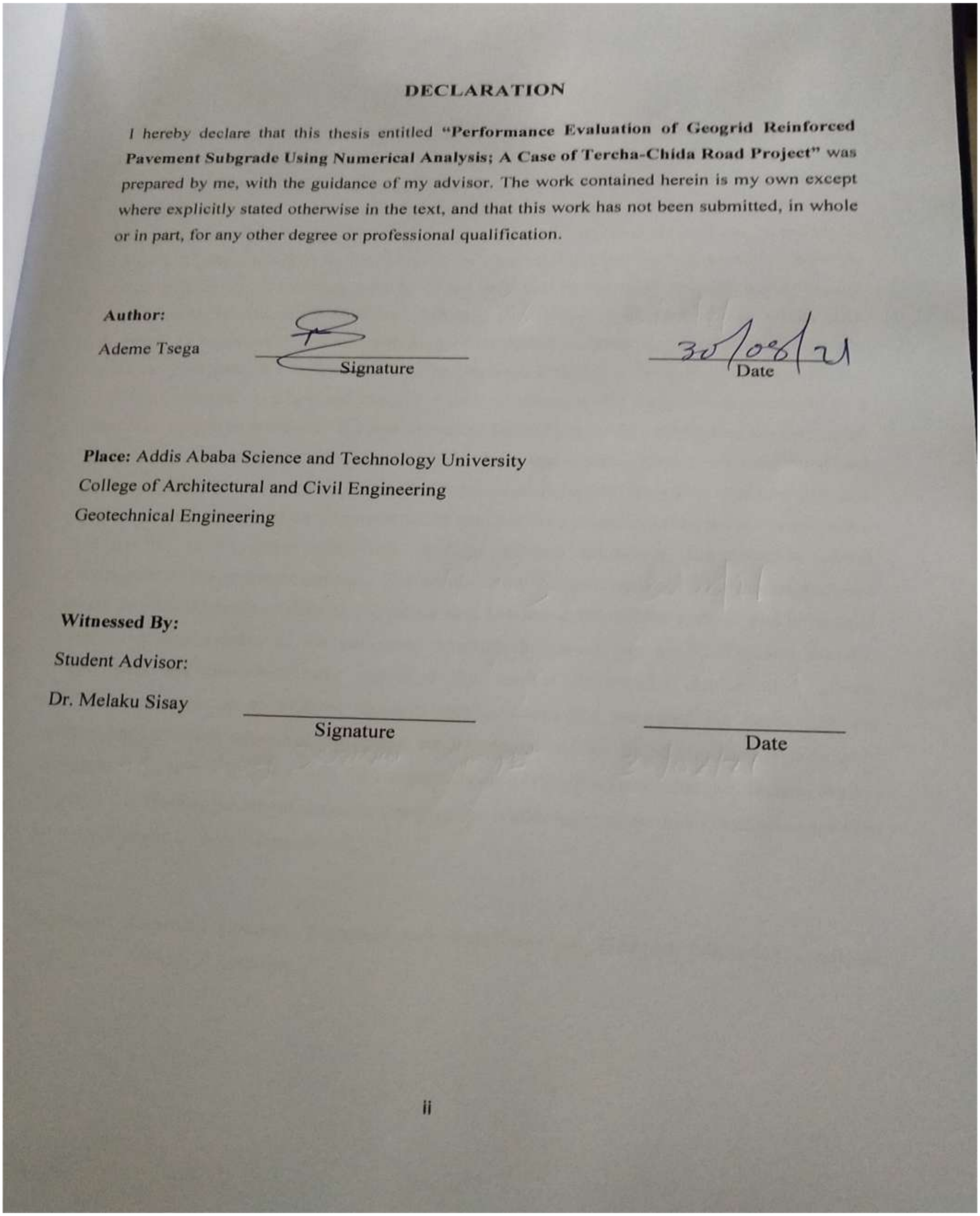

DECLARATION

I hereby declare that this thesis entitled **"Performance Evaluation of Geogrid Reinforced Pavement Subgrade Using Numerical Analysis; A Case of Tercha-Chida Road Project"** was prepared by me, with the guidance of my advisor. The work contained herein is my own except where explicitly stated otherwise in the text, and that this work has not been submitted, in whole or in part, for any other degree or professional qualification.

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ABSTRACT

One very important issue is the need for significant improvement of flexible pavement structure passing over areas of poor soil conditions due to continuous deformation and a low bearing capacity from the pavement structure foundation. One of the general problem happened on pavement structure are vertical settlement and lateral displacement of the soil layers. In order to minimize these problems enhancing the bearing capacity of the subgrade soils can be the major solution. Mostly, the subgrade soil is known by poor bearing capacity, high plasticity and high swelling factor. So, the bearing capacity of the soils can be improved by geotechnical ground improvement. In this study, reinforced subgrade performance with geogrid was numerically analyzed. Two pavement structure with geogrid reinforced subgrade and unreinforced subgrade were analyzed under the same traffic load and pavement thickness using finite element model. ABAQUS software package was used as a tool in analyzing the finite element models of a quarter of pavement structures. The distribution of the vertical surface deflection, the horizontal displacement, the stress distribution and lateral strain under static loading for reinforced and unreinforced sub-grade were studied and finally the improvement of pavement structure with and without geogrid reinforcement was numerically checked. The numerical simulations demonstrate that geogrid reinforcement effectively decrease vertical settlement and increase lateral confinement of the pavement structure. The results show that provision of geogrid reinforcement on top of weak subgrade of flexible pavement structure has stabilized the vertical and horizontal deformation/displacement of the pavement structure in general up to 74.73% and 64.08% respectively and has specifically stabilized the vertical deformation/displacement of the pavement subgrade up to 87.89%. Geogrid reinforcement has increased the stress carrying capacity of pavement structure, minimized the settlements of the foundation and significantly constrained the lateral strains within the subgrade layer of the pavement structure. Hence, the use of geogrid in flexible pavement structure stabilization could demonstrate to a sustainable solution for a problematic or poor subgrade soil.

Keyword: Pavement Structure; Subgrade Soil; Reinforcement; Geogrid; Numerical Analysis; ABAQUS Software;

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LIST OF ABBREVIATIONS

AASHTO – American Association of State of Highway and Transportation Officials
AC – Asphalt Concrete
ASTM – American Society for Testing and Materials
BCR – Base Course Reduction
CBR – California Bearing Ratio
CL – Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
ERA – Ethiopian Roads Authority
ESP – Exchangeable sodium percentage
FEA – Finite Element Analysis
FEM – Finite Element Method
GCL – Geosynthetic clay liner
GF – Geofoam
GG – Geogrid
GL – Geocell
GM – Geomembrane
GN – Geonet
GT – Geotextile
HDPE – High density polyethylene
HMA – Hot mix aggregate
MC – Mohr coulomb
MD – Machine direction
MH – Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
ML– Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
MR – Resilient modulus
MH – In organic silts, micaceous or diatomaceous fine sand or silty soils, elastic silts
OL – Organic silts and organic silty clay of low plasticity
OH – Organic clays of medium to high plasticity, organic silts

PET – Polyester

PH – Potential of hydrogen

PP – Polypropylene

PT – Peat and other highly organic soils

PVC – Poly-vinyl chloride

SC – Clayey sands, sand-silt mixtures

SNNP – Southern Nations and Nationalities People

TBR – Traffic benefit ratio

TD – Transverse direction

TIF – Traffic improvement factor

USCS – Unified Soil Classification System

XMD – Cross machine direction

3D – Three dimensional

LIST OF SYMBOLS

^0c = Degree Celsius

^0F = Degree Fahrenheit

t_e = Effective aggregate thickness

C_u = Undrained cohesion

σ_1, σ_1' = Total and effective major principal stress

σ_3, σ_3' =Total and effective minor principal stress

ε = normal strain

ε_E = Elastic strain

τ = Shear stress

σ = Normal stress

c = Cohesion

C'_R = Apparent cohesion

ø = Internal angle of friction

$Ø'_R$ = Apparent internal angle of friction

E = Young's modulus

q_{all} = Allowable foundation bearing capacity of soil

υ = Poisson's ratio

ρ = Mass density

p = Tyre inflation

P = Wheel load

A = Area

r = Radius

τ_{crit} = Critical shear stress

τ_f = undrained shear strength

% = Percentage

$\pi \sim 3.14$

& = And

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CHAPTER ONE

1 INTRODUCTION

1.1 Background Information

Flexible pavement is the most common pavement type used in Ethiopia. It has low flexural strength and transmits load to the subgrade soil through lateral distribution of stress with increasing depth. Its life of serviceability depends on various parameters such as layers thickness, quality of pavement materials used, repetition of wheel loads considered in the design, camber provided, and environment conditions.

The type of materials such as subgrade, sub-base and base course has a great effect on the quality and life of flexible pavement. Among these materials the subgrade has the most important impact, as it serves as a foundation material. Accordingly, an appropriate value of CBR is required to ensure an adequate strength for supporting the imposed traffic load. However, not all subgrade soils are able to meet up with this criterion because some have a considerably low and thus inappropriate CBR values (Ampadu, 2007).

The Engineers are often faced with the problem of constructing road beds on or with weak/problematic soils. The typical traditional approaches to construct in weak soils include: 1) replacing the top of the subgrade soils with better quality fill that exhibits superior strength properties; 2) increasing the thickness of the pavement layers, both the unbound base and asphalt concrete; 3) treating/stabilising the subgrade with a binder such as cement or lime or incorporating a reinforcement media within the soil in order to create a working platform by improving the engineering properties of the subgrade. All of these methods have a scope of applicability but are disadvantaged because of being either expensive, time consuming or both. The inclusion of geosynthetics such as geotextile and geogrids within the pavement structure can be used to address this problem. This is because these materials possess better qualities comparatively through their reinforcement and separation functions that enhance performance. Furthermore, the use of geosynthetic reinforcement has become a common solution for problems in geotechnical engineering due to their simplicity of construction (Tuna & Altun, 2012).

Geosynthetic is a planar product manufactured from polymeric material used with soil, rock, earth, or other geotechnical engineering related material as an integral part of a man-made project, structure, or system." Geosynthetics have been found to be a cost effective alternative to improve poor sub-soils in adverse locations, especially in situations where there may be non-uniform quality and/or non-availability of desired soils with applications in almost all geotechnical engineering projects such as airport and highway pavements. The geosynthetics that are routinely used in the transportation industry are geotextiles, geogrids, geomembranes, erosion control blankets and materials, geosynthetic clay liners, geocomposite and geonets (ASTM, 2001).

The typical functions of geosynthetic materials in relation with transportation engineering are separation, reinforcement, filtration, lateral drainage and acting as a liquid barrier/sealing (Babu, 2007). In providing reinforcement, the geosynthetic material structurally strengthens the pavement section by changing the response of the pavement to loading. In providing separation, it prevents contamination of an aggregate layer by the underlying subgrade and hence maintains a clean interface. In providing filtration and drainage, it aids in improving subsurface drainage and allows the rapid dissipation of excess subgrade pore pressures caused by traffic loading (Barksdale, 2006).

The Tercha-Chida Road Upgrading Project, located in the Southern Nations and Nationalities Peoples (SNNP) Regional State of Ethiopia, is mainly covered by brownish silty clay soil, highly weathered to decomposed rock and colluvium deposit. Due to the soil property, topography and frequent rainfall, the subgrade of significant stretch of Tercha-Chida road alignment is not stable to directly construct the flexible pavement structure over it.

Therefore, this study attempt to evaluate the performance of geosynthetics (geogrid) reinforced flexible pavement subgrade under static loading using numerical simulation (finite element analysis).

To meet the objective of the study, literatures have been carefully reviewed, primary and secondary data has been collected and finite element analysis were performed using ABAQUS software for typical section of Tercha-Chida Road Upgrading Project with poor subgrade soil condition which is not stable to directly construct the flexible pavement structure over it.

The analysis was comprised for models of flexible pavement structure reinforced with and without geosynthetics (geogrid) and the results were compared and finally recommendations were given.

1.2 Statement of the Problem

The Engineers are often faced with the problem of constructing road beds on or with weak/problematic soils. Problematic/Expansive soils exhibit exceptionally low strength and tend to swell when they become wet; and they are highly brittle and shrink when they become dry. This cause Fatigue cracking, rutting and subgrade shear failure due to inadequate support.

In early days, areas having weak soil deposits were avoided while fixing up the road alignment or by replacement method. But with scarcity of land and other resources, we do not have the choice of land and so roads and embankments have to be built on weak soil deposits. In such cases, it is often impossible to build a pavement structure over soft/weak subgrade due to its unfavourable properties like less bearing capacity, high vertical settlements, high liquid limits and less modulus of elasticity. Specifically, in Tercha-Chida road project there are many road sections with week subgrade soil which is uneconomical to replace the material and/or to change the route because of its environmental issues.

Therefore, it is necessary to use new techniques of construction and/or remedial measurements (mechanical stabilization, chemical stabilization, geosyntheic reinforcement…) to improve the detrimental properties of expansive soils, such as shrink-swell and low shear strength and accordingly, to provide the stable subgrade for the pavement construction.

Hence, this research is intended to investigate the performance of geosynthetics (geogrid) reinforcement as soil stiffness and load carrying capacity improvement technique of the weak subgrade.

1.3 Objectives

1.3.1 General Objective

The general objective of this study is to evaluate the performance of pavement subgrade reinforced with geogrid by numerical analysis using the ABAQUS software.

1.3.2 Specific Objectives

- Investigate the subgrade soil/material property and collect secondary data of the study area
- Identify the properties of geogrid according to its production specification
- Geometry and material modelling of the pavement structure as well as geogrid
- Analyse the engineering parameters (vertical/horizontal deformation, stress-strain distribution) of flexible pavement structure under static load with and without geogrid reinforcement using the ABAQUS software
- Compare the total deformation of the pavement structure with and without geogrid reinforcement of the subgrade
- Compare the stress, strain distribution on subgrade layer of pavement structure with and without geogrid reinforcement

1.4 Significance of the Study

This study was attempted to analyses the performance of flexible pavement structure with and without geogrid reinforcement of the poor subgrade soil using finite element method. Accordingly, the research will give clear assessment of flexible pavement performance with and without geogrid reinforcement of the poor subgrade soil and, it will be used for further studies for interdisciplinary concepts. It has also practical significance in the study area by giving peoples clear idea about using geogrid reinforcements in unstable subgrade of the flexible pavement section.

1.5 Scope and Limitation of the Study

The study was carried out by taking representative route sections with poor subgrade soil condition (S1 Subgrade class) of Tercha-Chida road project.

The scope of this study incorporates detail investigation of the study area and evaluation the performance of the selected poor subgrade soil with geosynthetics (geogrid) reinforcement. To meet the objectives of this study, the finite element analysis under static loading condition was carried-out by using the FEA software called ABAQUS. The analysis was supported by data from secondary sources and laboratory experiments.

The limitation of this study incorporates constraints of time, resource and finance. All these constraints made the work very challenging to study all the routes sections with varying stabilization techniques. The findings of this study were considered to be suggestive rather than definitive for field applications.

1.6 Organization of the Thesis

The presentation of this thesis work is organized in the following Five Chapters.

Chapter One: introduces general descriptions, problem of study, objectives, significance of the study, scope &limitation of the study and organizations of the thesis.

Chapter Two: presents the literature review, giving an overview about the types of geosynthetics and their applications in road pavement. Important and related previous studies are also included in this chapter.

Chapter Three: the methodology deals with the detail description of the study area. It briefly describes the materials and methods employed for the present study and gives detail information on the modelling and analysis procedure of the pavement structure using the ABAQUS software.

Chapter Four: is about results and discussion. It is the core of the present study and covers results and interpretations of the present research. A detailed description on results and interpretation with respect to the objectives of the study is presented; proposed remedial measures are also included.

Chapter Five: summarizes the main findings of this research study; conclusions have been made based on the result of the FEM analysis results and finally recommendations have been made and presented.

CHAPTER TWO

2 LITERATURE REVIEW

2.1 General

The major causes of failure in asphalt pavement are fatigue cracking caused by excessive vertical compressive and horizontal tensile strain at the top subgrade and bottom of asphalt layer due to repeated traffic loading and rutting deformation, caused by densification and shear deformation of subgrade. Therefore, design of flexible pavement pays particular attention to two critical locations within the pavement structure: (1) the horizontal tensile strain at the bottom of the asphalt layer, which should be minimized in order to prevent fatigue cracking, and (2) the vertical stress on the top of subgrade, which should be minimized in order to reduce permanent deformations (Gupta & Kumar, 2014).

The type of materials such as subgrade, sub-base and base course has a great effect on the quality and life of flexible pavement. The nature of subgrade soil has the most important effect among other materials. In the construction of pavements, subgrade serves as the foundation for the flexible pavement and for this purpose; an appropriate value of CBR and sufficient stability under adverse climate and loading condition is required in subgrade soil in order to ensure adequate strength to support the imposed traffic load. However, not all subgrades are able to meet up with this criterion because some have a considerably low and thus inappropriate CBR values. If pavement structures are founded on soil with low bearing capacity, they are likely to fail either during or after construction, with or without application of wheel load on them (Ampadu, 2007).

The road laid on subgrade formed of weaker soil leads to large deformations, causing increases in maintenance cost and interruption of traffic service. And if the subgrade layer of pavement consists of expansive/weak soil, due to its susceptibility to moisture change and results in subsequent high swelling and shrinkage characteristics. These soils possess less strength and bearing capacity and thereby results in increasing the thickness of pavement.

Existence of unsuitable soil for supporting structures in construction sites, lack of space and economic motivation are primary main reasons for using soil improvement techniques with poor subgrade soil conditions. There are many stabilization methods available to improve engineering properties of these types of soft subgrade soil. Use of geosynthetics over a soft subgrade found to be one of the feasible and economic solutions to strengthen road pavement and thereby increasing service life (Nagrale, et al., 2010; Bryson & Naggar, 2013; Anitha, 2017).

When considering the use of geosynthetics it is important to consider the type of geosynthetic, the intended function (reinforcement, separation, and filtration), factors affecting life span, in situ conditions, and installation (Austin & Gilchrist, 1996; Rowe & Li, 1999).

The present study focused on the application of geosynthetic (geogrid) reinforcement in flexible pavements. To highlight the significance of the current research, a literature review cantered on flexible pavements, problematic soils, methods of improvement of weak/problematic subgrade soil, concepts of geosynthetics and its function, mechanism of soil reinforcement using geosynthetics, geogrids and its application in road pavement, numerical modelling analysis and material modelling are presented below.

2.2 Flexible Pavement

According to Huang (1993), Flexible pavements are layered systems with better material on top where intensity of stress is high and inferior materials on the bottom where the intensity is low. Adherence to this design principle makes possible the use of local materials and usually results in a most economical design. This is particularly true in regions where high quality materials are expensive but local materials of inferior quality are readily available. A typical flexible pavement system is composed of four distinct layers: asphalt concrete, base course, sub base, and subgrade as shown in Figure 2-1 below.

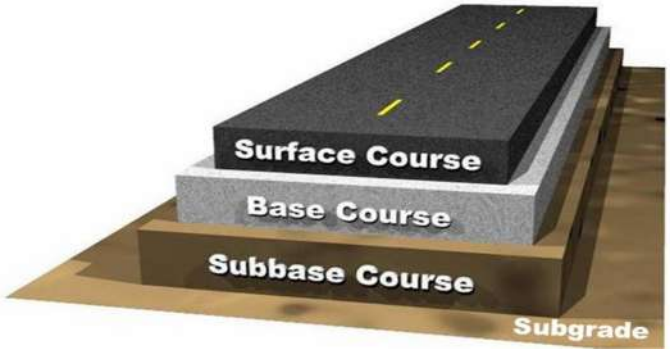


Figure 2-1: Basic flexible pavement structure (Lanham, 2006)

2.2.1 Critical Points in Pavement Design

Flexible pavements allow redistribution of traffic loads from the contact surface to the underlying layers. As the pavement flexes under the load, stresses are redistributed over a greater area than that of the tire-footprint. Design of flexible pavement pays particular attention to two critical locations within the pavement structure: (1) the horizontal tensile strain at the bottom of the asphalt layer, which should be minimized in order to prevent fatigue cracking, and (2) the vertical stress on the top of subgrade, which should be minimized in order to reduce permanent deformations (Yoder & Matthew, 1991).

2.2.2 Type of Pavement Distress

During its lifetime, a flexible pavement can experience two different types of failure modes: structural and functional. Structural failure leads to the collapse of the pavement, thereby making it incapable of sustaining the surface loads. Functional failure, on the other hand, renders the pavement incapable of carrying out its intended function, causing discomfort to passengers. Structural failure requires a complete rebuilding of the pavement whereas functional failure can be remediated by maintenance.

Pavement distress may occur due to either traffic or environmental loads. Traffic loads result from the repetition of wheel loads, which can cause either structural or functional failure. Environmental loads are induced by climatic conditions, such as variations in temperature or moisture in the subgrade, which can cause surface irregularities and structural weaknesses. Cycles of wetting and drying (or freezing and thawing) cause base course material to breakdown, generating fines in the subgrade and leading to crack development. Construction practices also affect pavement distress conditions. For example, the use of aggregates with excessive fines and inadequate inspection may lead to rapid pavement deterioration (Yoder & Matthew, 1991).

2.3 Problematic Soils

Problematic soils can be naturally occurring or man-made soils. Such soils can give rise to many geotechnical difficulties 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 problematic 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 (Slocombe, 2001). There are many types of problem soils, but some of the most remarkable are expansive soils, collapsible soils, soft clays and dispersive soils (Diop, et al., 2011).

A. Collapsible Soils

Collapsible soils can be either found naturally or formed through human activity and characterized by poor gradation with respect to particle size with a porous texture and generally exhibit low in-situ density. These soils can be partially saturated and can withstand relatively large imposed stresses with small settlements. However, if wetting occurs, the soils undergo large volume changes and exhibit an associated additional settlement with no increase in the applied loads (Jefferson & Rogers, 2012; Das, 2015).

The one-dimensional response-to-wetting test, performed using conventional consolidation equipment, represents the most used laboratory collapse test. A specimen, at in-situ moisture content, is first subjected to a total stress corresponding to that anticipated for the field conditions. Then, with the total load in place, the specimen is given free access to water, and the collapse settlement is observed. The advantage of this simple laboratory test is that the test interpretation is simplified due to the relatively uniform stress state within the specimen, so that reasonable stress-strain relationships can be developed for estimating collapse settlements.

Compacted soils may be susceptible to compression upon wetting. The response to wetting of a compacted fill depends on the soil type, compactive effort, compaction water content, and stress level at the time of wetting (Houston & Houston, 1997)

B. Expansive Soils

Expansive 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. On wetting, the clay minerals absorb water molecules and expand; conversely, as they dry, they tend to shrink, leaving large voids in the soil (Chen, 2012). The challenge with expansive soils is that the magnitude of soil movement is not often recognized in a timely manner. This is because structural damage can occur even when as little as 2 to 3% of soil expansion or contraction occurs (Diop, et al., 2011).

Expansive soils are common in arid and semi-arid climate regions of the world and cause severe problems on civil engineering structures. The Swelling potential of the expansive soil mainly depends upon the properties of soil and environmental factors, and stress conditions. Swelling pressure is a key parameter used in designing structures in and on expansive soil. The swelling pressure of soil is measured in the laboratory using a representative soil samples and by a number of testing methods. An odometer testing method is extensively used to determine the swelling pressure due to its simplicity and operational ease (Jayalath, et al., 2016).

C. Dispersive Soils

Dispersion occurs in any given soil that has a high percentage of Exchangeable Sodium Percentage (ESP), causing internal erosion and eventual piping through embankments. The tendency for the dispersive erosion in a given soil is subject to variables such as mineralogy and chemistry of the clay and the dissolved salts in the soil water and the eroding water. Extra care should be taken when designing earth dams, drainage channels and lateral support where the soil structure is dispersive because the soils are susceptible to erosion and piping (Franki, 2008).

Visual classification, Atterberg's limits and particle size analysis do not provide a basis for differentiation between dispersive clays and ordinary erosion resistant clays (Mitchell & Soga, 2005). The conventional laboratory tests performed to determine the dispersive clays include pinhole test and double hydrometer test.

In the pinhole test, distilled water is allowed to flow through a 1.0 mm diameter hole drilled through a compacted specimen. The water becomes muddy and the hole rapidly erodes in dispersive clays. For non-dispersive clays the water is clear and there is no erosion. The pin-hole test is considered most reliable but it is important that the samples correctly simulate the soil state and the water composition expected in the field (Mitchell & Soga, 2005).

The double hydrometer test is one of the first methods developed to assess dispersion of clay soils. The particle size distribution is first determined using the standard hydrometer test in which the soil specimen is dispersed in distilled water with a chemical dispersant. A parallel hydrometer test is then made on a duplicate soil specimen, but without a chemical dispersant. The percent dispersion is the ratio of the dry mass of particles smaller than 0.005 mm diameter of the test without dispersing agent to the test with dispersing agent expressed as a percentage (Mitchell & Soga, 2005). Procedures for performing the test are outlined in USBR 5405, Determining Dispensability of Clayey soils by the Double Hydrometer Test Method.

D. Soft Clays

Soft soils exhibit low shear strength, high compressibility, and lead to severe time related settlement problems. They are clays that are partially or fully saturated, frequently have high organic content and are highly compressible. Thus, they are associated with low shear strength, compressibility and severe time related settlement problems.

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 & Davies, 1985).

Depending on the geographic origin and training of the engineer or geoscientist involved, a "peat/ soft soil" may be defined as soil with an organic content greater than anywhere from 20% to 70% of the total weight. At worst "peat/soft soil" may be used interchangeably with the term "organic soil" to describe any soil that appears to have some organic content. In the Unified Soil Classification System (USCS) peats are described as soils consisting "predominantly" of plant remains, often with a distinctive smell. Organic clay, silt or sand contains "substantial amounts" of vegetable matter (Rohayu & Rashid, 2000).

The following parameters were determined to characterize the soft soil:

- Water content: The water content is measured using procedures specified in ASTM D2974 or BS 1377.

- Organic Content: As a percentage of dry weight. The organic content is measured in the laboratory using a Loss on Ignition Test, ASTM D2974 or BS 1377 Part 3(4), or a Chemical Oxidation Test, BS 1377 Part 3(3).
- Degree of Hurnification (Decomposition) of the organic material: The degree of humification represents the degree to which the organic remains have decayed. The range lies between fresh plant remains and a completely decayed visibly amorphous material with no recognizable plant structure. Where a soft soil/peat lies within this spectrum radically affects its engineering behavior.
- In the field it may be assessed by the Von Post Squeeze Test. A sample of the peat is squeezed in the hand. The color and form of fluid that is extruding between the fingers is observed together with the pressed residue remaining in the hand after squeezing. The degree of humification on a 10-point scale, Hl to H10, is obtained by comparing the observations t those described in Table 1. Atterberg Limits: The fibers in peat make determination of the Atterberg limits difficult, and results depend strongly on the methods used to prepare the samples (Rohayu & Rashid, 2000).

E. 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 cohesion less 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. 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 (Madabhushi, et al., 2010).

Four consecutive laboratory testing procedure for the assessment of the likelihood of post-liquefaction deformation are a constant stress amplitude cyclic test, a constant strain amplitude cyclic test, a monotonic shear test, and a drainage test (Jongkwan, et al., 2017).

2.4 Methods of Improvement of Subgrade

2.4.1 Soil Replacement

Construction sites are usually faced with problematic soils that need to be excavated and replaced to achieve the required strength for construction 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 (Byrne, et al., 2008).

The advantage 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.

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 & 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, et al.). There is also the issue of time consumption through all the processes needed to carry out this method, such as replacing the unsuitable material (Momes, et al., 2011).

2.4.2 Soil Stabilization

Taking in to account 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 investigating other alternatives.

Soil stabilization can be taken as alternate to borrow selected materials and it has advantage that the effect to the environment is reduced and in areas where selected/granular materials are scarce, stabilization have comparative economic advantage.

Soil stabilisation is a method that has been used to improve the bearing characteristics, hence creating a stable working platform and reducing settlement. It is the alteration of one or more soil properties, by mechanical or chemical means, to create an improved soil material possessing the desired engineering properties (Guyer, 2018).

Mechanical stabilization can be defined as a process of improving the stability and shear strength characteristics of the soil without altering the chemical properties of the soil. The main methods of mechanical stabilization can be categorized in to compaction, mixing or blending of two or more gradations, applying geo-reinforcement and mechanical remediation (Makusa, 2012; Guyer, 2018).

Chemical stabilization of soil is mixing of soil with one or a combination of admixtures of powder, slurry, or liquid such as lime, Portland cement and asphalt for the general objective of improving or controlling its volume stability, strength and stress-strain behaviour, permanently and durability (Winterkorn & Pamukcu, 1991).

In road construction, all the naturally available material cannot be utilized as construction material as there exists some problematic soils and soils with limitations to meet specifications and design standards. The problematic nature and limitations of such soils can be improved by application of stabilizing agents. The application of stabilizing agents can improve (Guyer, 2018):-

- Strength (stability and bearing capacity) of the soil
- Durability and resistance to the effect of water
- Volume stability
- Permeability
- Wet soils can be dry out
- The workability of clay soils
- Load spreading capacity of pavement layers

2.5 Concept of Geosynthetics

2.5.1 General

American Society for Testing and Materials (ASTM) (2015), defines geosynthetics as planar products manufactured from polymeric material, which are used with soil, rock or other geotechnical engineering related material as an integral part of a man-made project, structure or system.

With the advent of polymers in the middle of the 20th Century, a much more stable group of materials became available. These groups of polymer materials, called geosynthetics, have been employed in civil engineering works due to their stability and durability. Geosynthetics were first employed in the 1960s as filters in the United States and as reinforcement in Europe (Momes, et al., 2011).

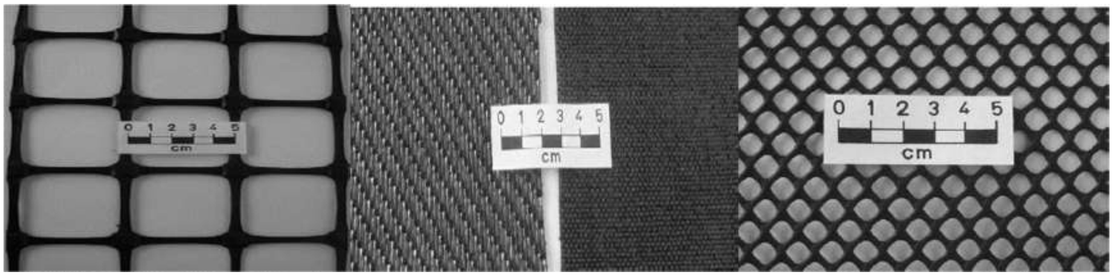
Geosynthetics have been formulated and are available in a wide range of forms to suit various engineering applications, particularly useful in road pavement construction and the earthworks associated with road construction which significantly increase the safety factor, improve performance, and reduce costs in comparison with conventional construction alternatives. It is manufactured from synthetic polymers such as polypropylene, polyesters, polyethylene, polyamide (nylon), poly-vinyl chlorides (PVC), and fibreglass. Polypropylene and polyester are the most used (Olukayode, 2011). Geosynthetics is becoming rapidly popular in construction because of their ability to perform certain necessary function such as (Anitha, 2017; Okunade, 2010) :

- A wide availability of products from the global market
- The relative ease of shipping and field handling (flexibility)
- Rapid installation techniques
- Lightweight in comparison with other construction materials
- Durability and long life when properly selected
- Environmental Sensitivity – Geosynthetic systems reduce the use of natural resources and the environmental damage associated quarrying, trucking, and other material handling activities.

- 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.

2.5.2 Types of Geosynthetics

The types of geosynthetics depend on the function, application and manufacture process, and these include: Geogrids; Geotextiles; Geosynthetic clay liners (GCLs); Geonets; Geocells; Geomembranes; and Geofoams (ASTM, 1993). Figure 2-2 below shows the different types of geosynthetics that could be manufactured for different applications and functions.



a) Geogrid, b) Geotextile c) Geonet

d) Geocell e) Geomembrane f) Geofoam

Figure 2-2: Types of Geosynthetics (Pokharel & Ochiai, 1997; Shukla & Yin, 2006; Federal Highway Administration, 2006)

A. Geogrids

Geogrid is a polymeric mesh-like planar product formed by intersecting elements, called ribs, joined at the junctions. The key feature of geogrids is the apertures, which are openings between adjacent transverse and longitudinal ribs. The ribs of geogrids are normally stiffer in comparison to the fibres of geotextiles. The ribs are linked by extrusion, bonding or interlacing, and the resulting geogrids are called extruded geogrid, bonded geogrid and woven geogrid respectively. Within the groups, the geogrids can either be uniaxial, biaxial or triaxial depending on the direction of stretch during manufacturing. Biaxial geogrids are those that exhibit the same strength in both machine and cross machine directions while uniaxial geogrids exhibit the primary strength in the machine direction with minimal strength, enough to maintain the aperture structure, in the cross machine direction Geogrid is mainly used for reinforcement purposes in soil applications (Holz, et al., 1998).

B. Geotextiles

Geotextiles are permeable, polymeric textile products in the form of flexible sheets. Among the different geosynthetic types, geotextiles are the ones that present the widest range of properties. They are classified into woven geotextiles, non-woven geotextiles and knitted geotextiles depending on the manufacturing process.

They are commonly used to provide separation, reinforcement and filtration in soil and rock. Based on experience, people found out that when the subgrade condition consists of poor soil, low untrained shear strength, a high-water table and high sensitivity, the primary function of geotextiles in stabilizing the sub-grade is separation (Koerner, 2012).

C. Geonets

Geonets is a geosynthetic material consisting of parallel sets of intersecting ribs that form a three-dimensional net-like material. They are used for drainage function and erosion control (Kercher, 2010).

D. Geocells

Geocells are three dimensional, permeable, polymeric, interconnected honey comb cells or web structure, which are ideal for soil and rock confinement. They are mainly used for basal reinforcement and any other civil engineering works requiring confinement (Pokharel, 2010).

E. Geomembranes

Geomembranes are relatively impermeable sheets of plastic. The materials may be asphaltic or polymeric, or a combination of both, making them impervious. They are commonly used in landfill applications for base and cover liner systems and barriers for liquid and solid wastes containment (Holz, et al., 1998).

F. Geofoam

Geofoam is a light weight product in slab or block form with a high void content and is used primarily as a lightweight fill, thermal insulators and drainage channels. They are used within embankments built over soft soils under the road and airfield pavements subject to freeze and thaw, and beneath on grade storage tanks containing cold liquids (ASTM, 1993).

2.5.3 General Functions of Geosynthetics

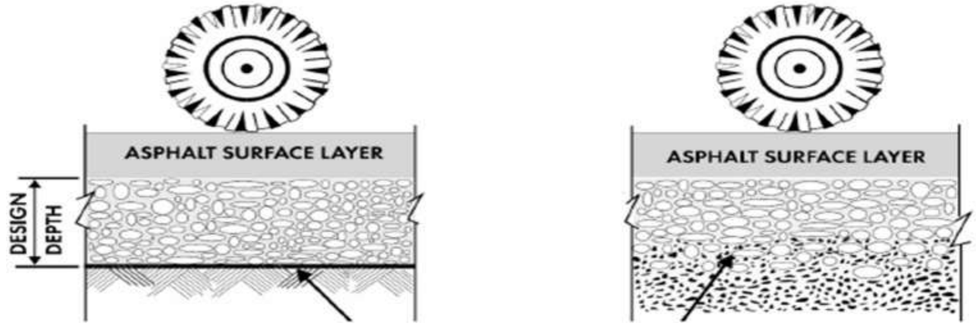
Geosynthetics are generally designed for a particular application by considering the primary function that can be provided. The multiple functions of geosynthetics are dependent on the material they are manufactured from and also on the application intended. The different functions include; separation, reinforcement, filtration, drainage, and containment as shown in Table 2-1 below (Koerner, 2012).

Table 2-1: Identification of the primary functions for each type of geosynthetic product (Koerner, 2012).

Types of Geosynthetic	Separation	Reinforcement	Filtration	Drainage	Containment
Geotextile (GT)	X	X	X	X	
Geogrids (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

I. Separation

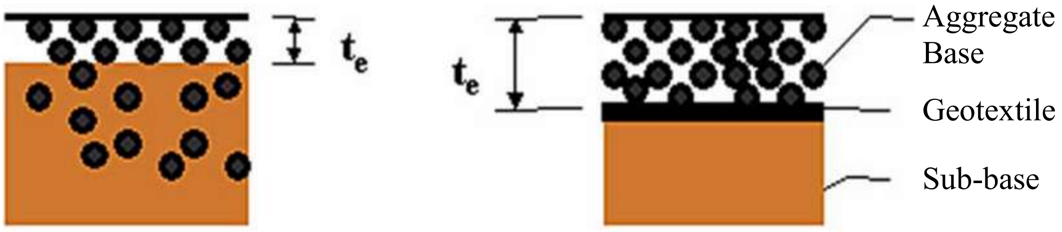
Separation is the inclusion of a permeable geosynthetic layer at the interface between different materials (i.e. subgrade/base interface) so that the integrity and the functioning of both materials can remain intact or even be improved (Koerner & Soong, 2005). In pavement design and applications, separation means the prevention of subgrade soil intruding into base layer (or sub-base), and also prevention of aggregate base (or sub-base) contamination into the subgrade (Drescher & Erickson, 2001). For instance, a main cause of failure of sections constructed over weak subgrade is migration of the base course aggregate with the underlying poor-conditioned soil as shown in Figure 2-3 below. A geosynthetic layer can be placed at the subgrade-base interface to perform as a separator and prevent the subgrade and base course aggregate from being mixed as shown in Figure 2-4 below.



Pavement with geotextile reduces contamination of expansive base course

Pavement without geotextile has subgrade intrusion into the base course

Figure 2-3: Separation function of a geosynthetic layer placed between the base aggregate and a soft subgrade (Zornberg & LaRocque, 2008)



t_e= Effective Aggregate Thickness

a) Aggregate Loss due to lack of separation

b) Separator prevents Aggregate loss

Figure 2-4: Geosynthetic separator preventing aggregate loss (Kercher, 2010)

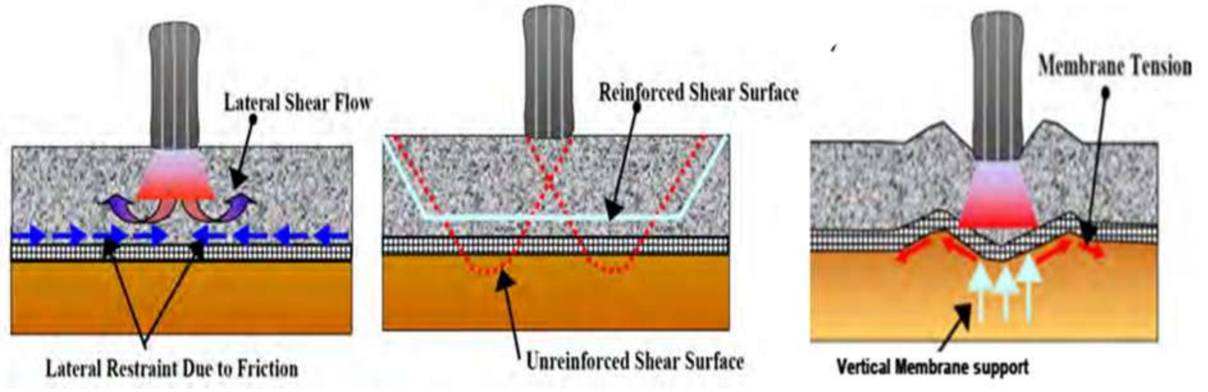
II. Reinforcement

Reinforcement is the interactive improvement in pavement strength caused by geosynthetic inclusion. Reinforcement in pavement system has two benefits. It can (1) increase the service life of the pavement and/or (2) obtain same performance with a reduction in pavement layers thicknesses. The reinforcement function is developed primarily through the following three mechanisms (Holz, et al., 1998):

a) Lateral restraint through interfacial friction between geosynthetic and aggregate. By applying load on an aggregate base layer, the aggregate tends to move laterally unless it is restrained by the subgrade or geosynthetic reinforcement. Poor condition subgrade soil provides little lateral restraint, which results in rutting development when the aggregate moves laterally. Interaction between the base course layer and the geosynthetic, transfers shear load from the base layer to a tensile load in the geosynthetic (Perkins & Ismeik, 1997). The geosynthetic being stiff in tension can limit the extensional lateral strains in the aggregate base layer. Moreover, a geosynthetic layer confines the aggregate base layer, thereby increasing the mean stress and leading to improve its stiffness and shear strength. Frictional and interlocking characteristics between the subgrade and geosynthetic are required to recognize this mechanism. Particularly, for a geogrid, this implies that the geogrid apertures and subgrade soil particles distribution should be considered properly (Figure 2-5-a).

b) Increased bearing capacity, i.e., by forcing the potential bearing surface failure plane to develop at alternate higher shear strength surface (Figure 2-5-b).

c) Membrane type of support of the wheel loads. The tension membrane effect develops as a result of vertical strain causing a concave shape in the reinforcement layer. The tension developed in the geosynthetic can help to distribute the wheel load and reduce the vertical stress on the soil, but remarkable rutting depths are required to realize this mechanism (Figure 2-5-c).



a) Lateral restraint b) Increased bearing capacity c) Membrane-type support

Figure 2-5: Reinforcement mechanisms induced by a geosynthetic layer (Zornberg & LaRocque, 2008).

III. Filtration

Filtration is defined as the equilibrium geosynthetic-to-soil system that allows for appropriate liquid flow with a little soil loss across the plane of the geosynthetic layer over a service life time compatible with the application under consideration (Koerner & Soong, 2005). In other words, it is limiting the movement of soil particles, and at the same time allowing water to move from the filtered soil to the coarser soil adjacent to it during the performance life of the road structure (Figure 2-6).

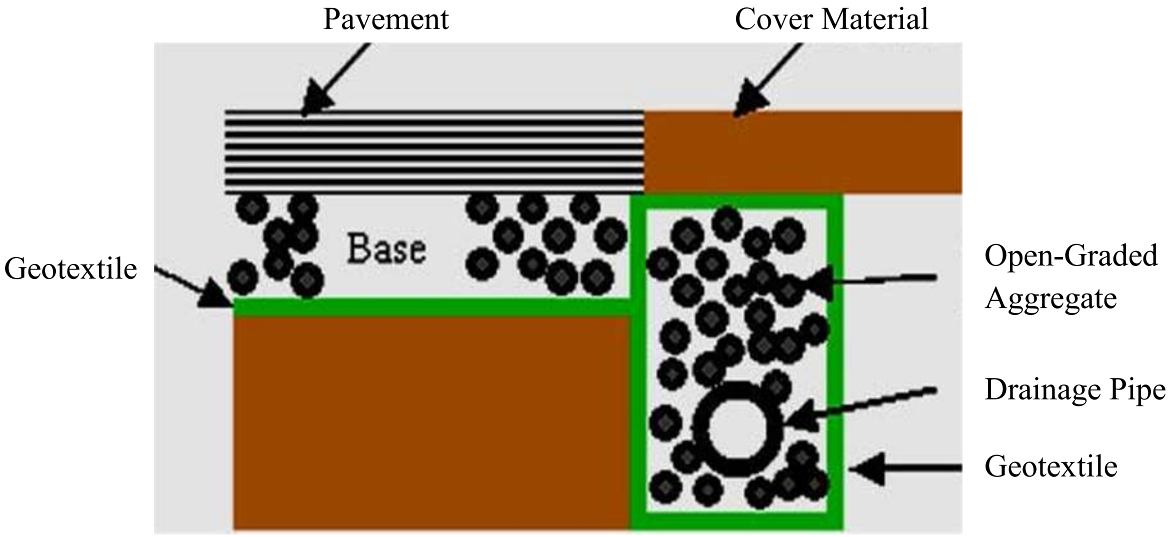


Figure 2-6: Edge drain wrapped with geotextile (Kercher, 2010)

2.5.4 Geosynthetic Reinforcement of Road Pavement

The determinant as to the need of reinforcement geosynthetic within a pavement is normally the subgrade, it being the foundation upon which the road pavement structure is supported.

2.5.4.1 Subgrade Conditions for Geosynthetic Reinforcement

Based on experience and several case histories, the following subgrade conditions have been found to be most appropriate for geosynthetic use in roadway construction as determined by (Holz, et al., 1998):

- ✓ Poor soils (Unified soil classification system: SC, CL, CH, ML, MH, OL, OH and PT)
- ✓ Low undrained shear strength soils ($\tau_f = Cu < 60\ kPa;$ CBR < 3; Resilient Modulus MR=30)

Under these conditions, separation is the primary function of the geosynthetic and the subgrade is improved through stabilisation hence allowing for long term strength improvements. However, if large ruts develop upon the application of the load, then some reinforcing effect gets mobilised. Also, AASHTO M288-96 infers that when the subgrade soil has a CBR of 1-3 or undrained shear strength of 30 to 90 kPa, reinforcement of the subgrade is needed.

Reinforcement benefits have been observed for subgrade strengths up to a CBR of 8 (MR of 80 MPa) as shown in Table 2-2. In addition, there appears to be a relationship between decreasing reinforcement benefits with increasing subgrade strength as shown in Table 2-2 below.

Table 2-2: Reinforcement benefits for paved permanent roads (Berg, et al., 2000)

Benefit	Permanent paved road Subgrade Conditions		
	Low CBR<3 (MR<30 MPa)	Moderate 3<CBR<8 (30<MR<80 MPa)	Firmer CBR>8 (MR>80 MPa)
Reducing undercut	✓	○	◇
Reducing distribution of the subgrade during construction	✓	○	◇
Reinforcement of the base aggregate in a roadway to reduce the section	○	✓	○

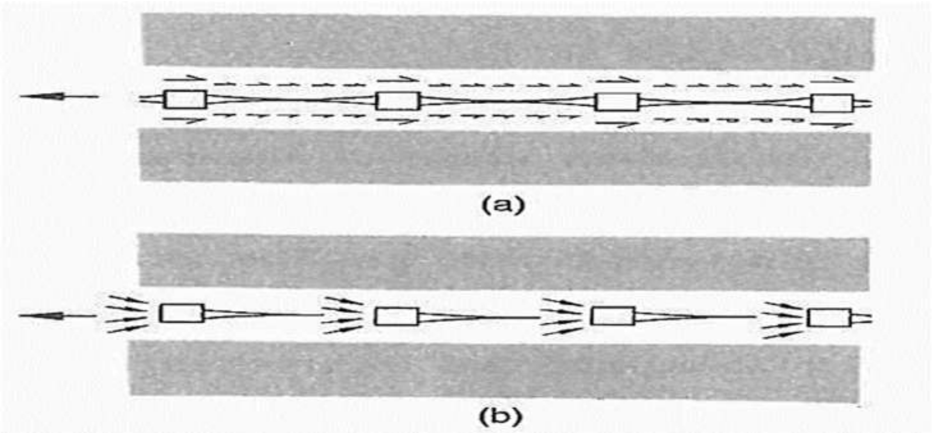
Reinforce net of the base aggregate in a roadway to increase the design life of the pavement	✓	✓	○
KEYS: ✓- usually a benefit, ○ - a known benefit in certain (various) conditions, ◇ - usually not a benefit			

2.5.4.2 Soil–Geosynthetic Interaction

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. 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 2-7(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 2-7(b) below (Lopes, 2012).



(a) Shear between soil and plane surfaces and (b) soil bearing on reinforcement surfaces

Figure 2-7: Soil-geogrid interaction mechanisms: (Lopes, 2012)

There are many factors that could have an effect on the soil-geosynthetic interaction, such as the Soil particle size, Confinement stress, Soil density and Geosynthetic structure are the main factors.

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 2-1 below.

$$\frac{Minimum\ Aperture\ Dimension}{Averagve\ Soil\ Particle\ Size} \geq 3 \qquad \text{-----------} \ (2\text{-}1)$$

Confinement stress: As shown in equation 2-1 above, 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).

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).

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 (Lopes, 2012).

2.5.5 Mechanisms of Soil Reinforcement Using Geosynthetics

2.5.5.1 Reinforcing mechanism associated with static loading

Different studies (Giroud , et al., 1984; Gupta & Kumar, 2014) have identified the Three modes of geosynthetic reinforcement in roadways, namely: lateral confinement, increased bearing capacity, and tension membrane effect. The reinforcement function is produced when the geosynthetic is placed either within the base or at the interface of the base and subgrade. The mechanisms were initially based on observations from static loading. Similar reinforcement mechanisms were also observed by studies done under cyclic loading (Webster, 1991).

i. Lateral restraint/confinement

The main function of the base of a road is to reduce the loads induced by traffic to such an extent that the underlying subgrade is protected from deformation. A vertical load will induce lateral loads that spread the aggregates, leading to local deformations of the base. However, as a result of the frictional interaction and the interlocking between the geotextile-soil and geogrid-soil respectively, the aggregates are restrained between the interfaces of the subgrade and fill. The geosynthetic is then able to take the additional shear stresses between the subgrade and fill that would have otherwise been applied to the subgrade, hence acting as a buffer. This eventually leads to an improvement in load distribution of the subgrade and reduction of the fill thickness (Figure 2-8) (Hufenus, et al., 2006).

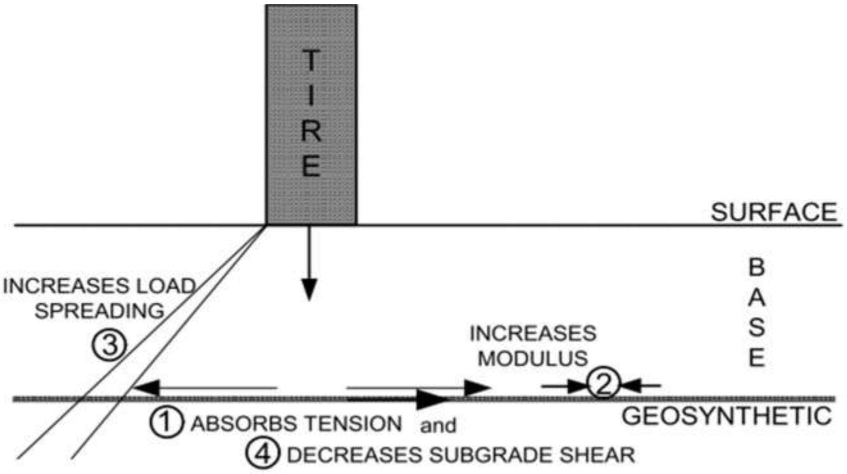


Figure 2-8: Lateral Restraint (Maxwell, 2005)

Also, in the case of subgrades, when the vertical stresses on the subgrade exceed the elastic limit in an unreinforced unpaved road, local shear and large deformations ensue. These deformations cause accelerated deformation of the base layer which furthers fatigue of the subgrade soil due to the increased stress levels (i.e. an increased ratio between applied and allowable stress). After a relatively small number of vehicle passages, the plastic limit (ultimate bearing capacity) is exceeded and general shear failure occurs. From experience, if the subgrade is confined, the local shear failure does not become large and the subgrade soil can support a vertical stress close to its elastic limit. The effectiveness of a reinforcement geosynthetic does not only rely on its ability to adequately transmit loads to the fill material through interlocking and friction, but it is also improved by the stiffness of the geosynthetic (Giroud, 1985).

ii. Improved load distribution

Load distribution is a function of the mechanical properties and thickness of the base. The presence of a geosynthetic layer in the base generally leads to a change in the stress and strain relationship in the subgrade. For a layered system with a weak subgrade underlying a base, the increase in the modulus of the base layer results in a more improved, broadly distributed vertical stress on the subgrade (Figure 2-8 above). This means that the surface deformation will be less and more uniform as well (Perkins, 1999). Therefore, a confined base layer is able to provide better applied load distribution than is possible with unreinforced base layers. A better performance is expected for a geogrid than geotextile reinforced base owing to the different nature of their interactions with the granular base (Giroud , et al., 1984).

iii. Increase of bearing capacity

Bearing capacity is the ability of a soil to support imposed load without undergoing shear failure or excessive settlement. The ultimate bearing capacity is the theoretical maximum pressure the soil can support without failure. In considering the ultimate bearing capacity, the pavement structure is assumed to fail under shear when subjected to sufficiently high traffic stresses.

The inclusion of reinforcement geosynthetics shifts the failure envelope of the pavement system, from the relatively weak subgrade to relatively strong base by forcing the potential bearing surface plane to develop at an alternate higher shear strength surface (Figure 2-9 below). This tends to increase the bearing capacity of the roadway.

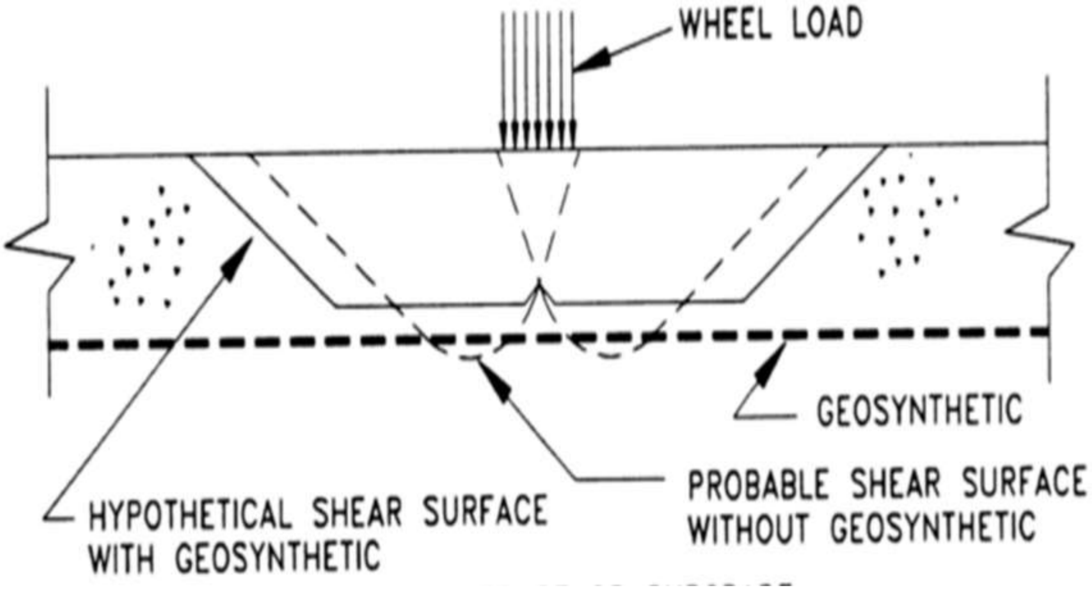


Figure 2-9: Improved bearing capacity (Holz, et al., 1998)

In Figure 2-9 above, the reinforcement action of the geosynthetic decreases the shear stresses transferred to the subgrade, thus providing vertical confinement on the subgrade outside the loaded area where the heave occurs. These decreases shear strain at the top of the subgrade and minimises subgrade deformation and upheaval. The bearing failure model may change from punching failure without reinforcement to general failure with an ideal reinforcement as established by (Gupta & Kumar, 2014). The increase in bearing capacity additionally results in the effect of reduction of settlement as indicated in Figure 2-10 below.

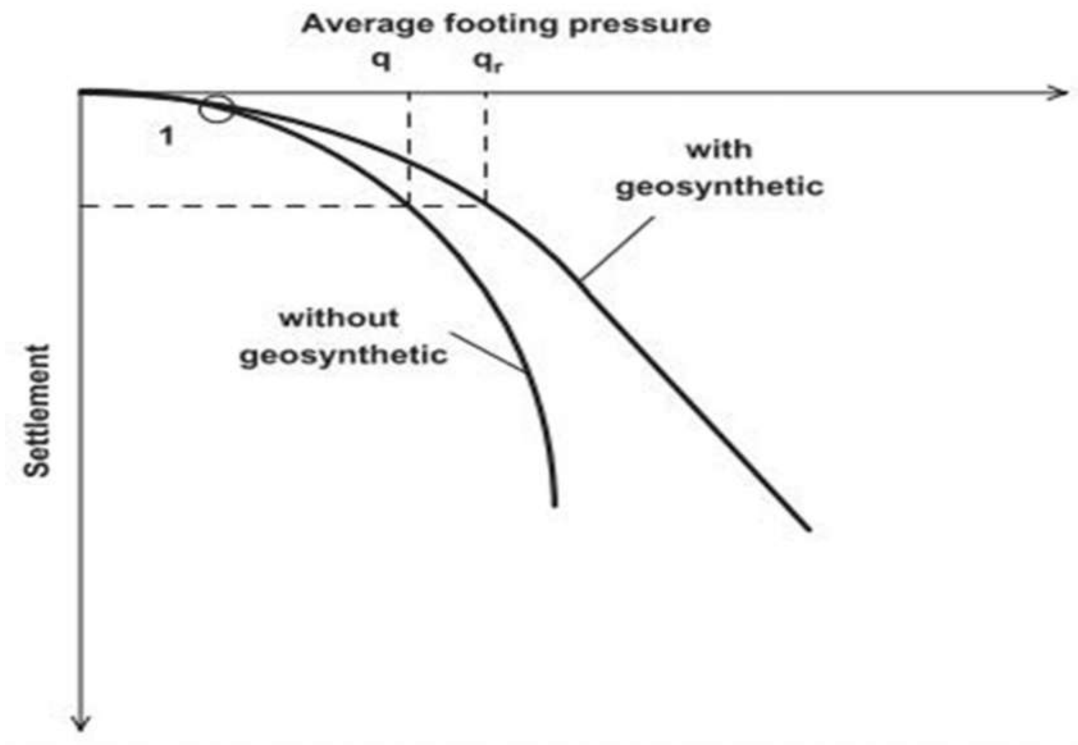


Figure 2-10: General nature of the load–displacement curves for unreinforced and reinforced subgrade (Kazimierowicz-Frankowska, 2007)

The improvement in bearing capacity due to the use of geosynthetic has been demonstrated using triaxial tests. Two phenomena have been established and demonstrated using the Mohr circle: Concept of apparent cohesion and concept of increase of apparent confining pressure.

Concept of apparent cohesion: Reinforcement assists in the introduction of an apparent cohesion to a granular soil which initially had no cohesion (Figure 2-11 below). The reinforcement in the soil increases the major principle stress at failure from σ_1 to σ_{1R} (with an apparent cohesion C'_R) as shown in the Mohr stress Figure 2-11 (Pham, 2009). When reinforcement is provided in the direction of σ1, interaction between the reinforcement and the soil generates frictional forces at the interface. Tensile stresses will be generated by the reinforcement and a corresponding compression as long as there is no slippage between the soil and reinforcement.

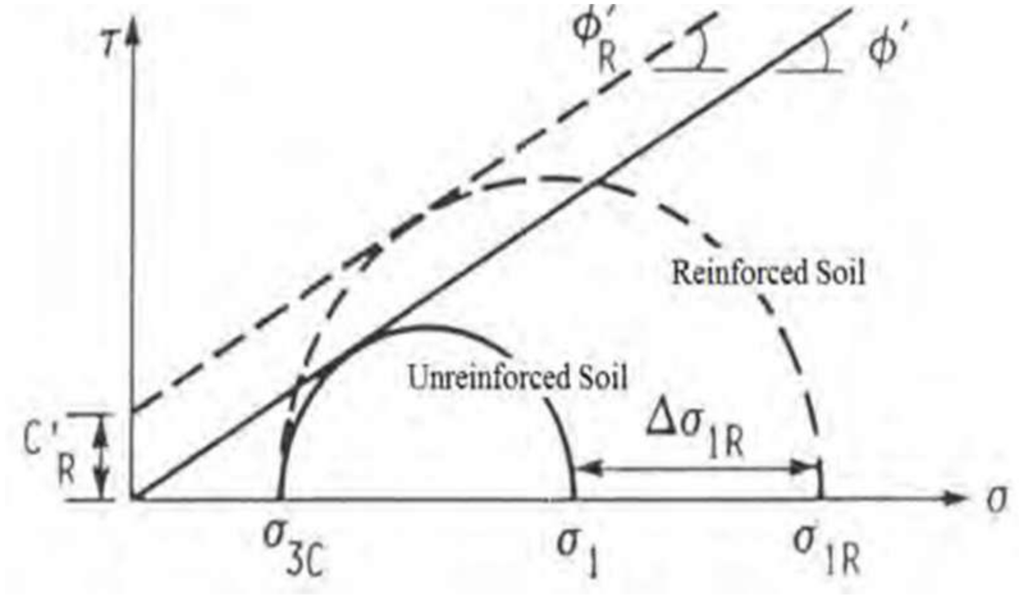


Figure 2-11: Concept of apparent cohesion (Pham, 2009)

Concept of apparent confining pressure: The inclusion of a tension member increases the axial strength from σ1 to σ_{1R} with an increase of confining pressure of $\Delta\sigma_{3R}$ (Figure 2-12 below). Therefore, the increase in strength due to reinforcement can be equated by a change in the stress state of the soil that resulted in an enhancement of the confining stress $\Delta\sigma_{3R}$ (Ruiken & Ziegler, 2008).

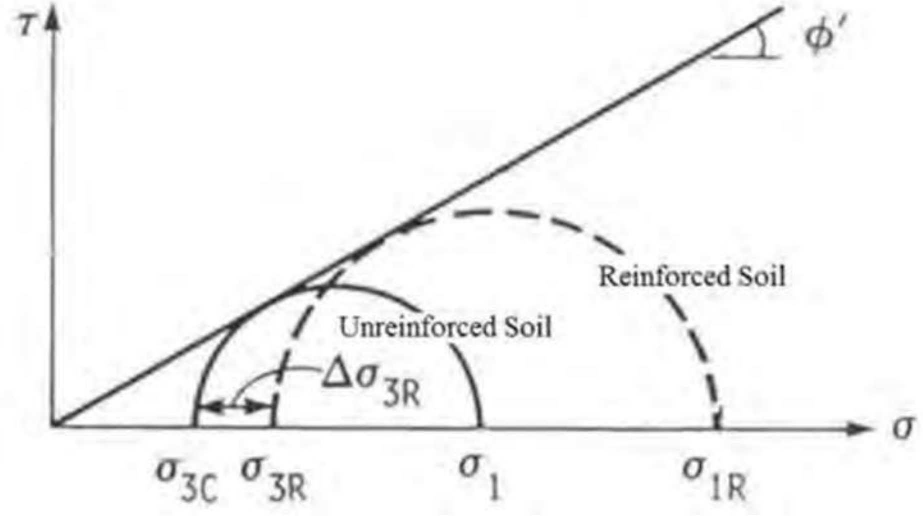


Figure 2-12: Concept of apparent confining pressures (Pham, 2009)

iv. Tension membrane effect

For tensioned membrane to be triggered, the wheel loads should cause plastic deformation and ruts in the subgrade. The geosynthetic should have a sufficient high tensile modulus for the tensile stresses to develop within the reinforcement. The action of the load leads the geosynthetic to exhibit a wavy shape that causes it to stretch (Figure 2-13 below).

The membrane effect counteracts the traffic load, hence limiting the vertical component of the load, and the reinforcement in tension distributes the load over a larger area leading to a reduction in settlement (Bloise & Ucciardo, 2000). Therefore, the mobilisation of the membrane effect requires that the geogrid and geotextile be deformed and tensioned through the development of ruts.

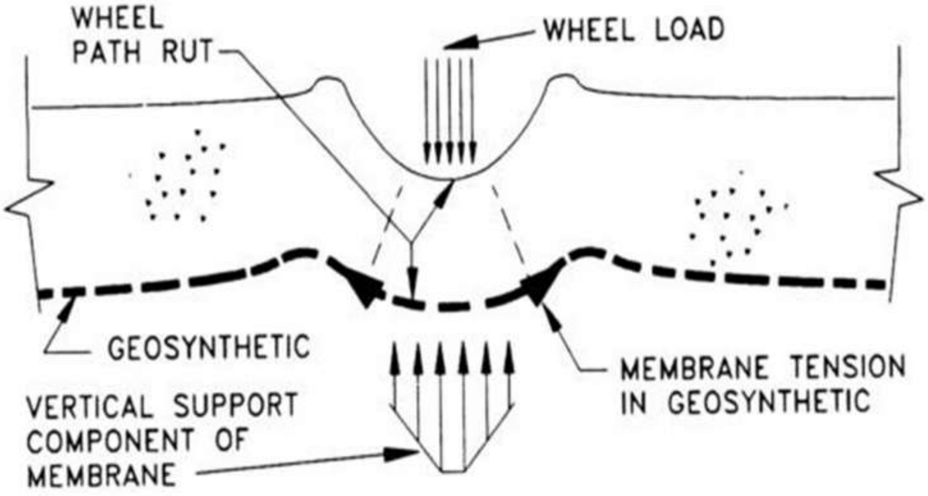


Figure 2-13: Tensioned membrane effect (Holz, et al., 1998)

2.6 Geogrid

A geogrid is defined as a geosynthetics formed by a regular network of tensile elements with apertures of sufficient size to allow strike-through of surrounding soil, rock or other geotechnical materials. Geogrids are polymeric products formed by joining intersecting ribs. Geogrids are mainly made from polymeric materials, typically polypropylene (PP), high density polyethylene (HDPE) and polyester (PET). They have large open spaces also known as "apertures". The directions of the ribs are referred to as machine direction (MD), orientated in the direction of the manufacturing process or cross machine direction (XMD) perpendicular to the machine direction ribs. Geogrids are principally used for reinforcement purposes; they can also help to provide effective separation between two soil and granular fill layers (AREMA, 2013).

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, 2011).

The ribs of a geogrid are defined as either longitudinal or transverse. The direction which is parallel to the direction that geogrid is fabricated on the mechanical loom is known as roll length direction, Machine Direction (MD), or longitudinal direction. On the other hand, the direction which is perpendicular to the mechanical loom and MD in the plane of geogrid, is known as Transverse Direction (TD) or cross machine direction. In other words, the longitudinal ribs are parallel to the manufactured direction (the machine direction); the transverse ribs are perpendicular to the machine direction. Some mechanical properties of geogrid such as tensile modulus and tensile strength are dependent on the direction which geogrid is tested. Also, geogrid installation in pavements is usually in a way that traffic path is parallel with the ribs produced in machine direction (Kwon & Al-Qadi, 2009).

In a geogrid, the intersection of a longitudinal rib and a transverse rib is known as a junction. Junctions can be created in several ways including weaving or knitting. Position of ribs and junctions could make various aperture types. Figure 2-14 below shows a section of geogrid in plain view and labels the different grid components. Geogrids can also be categorized in three main groups based on their aperture: uniaxial, biaxial and triaxial (Figure 2-15).

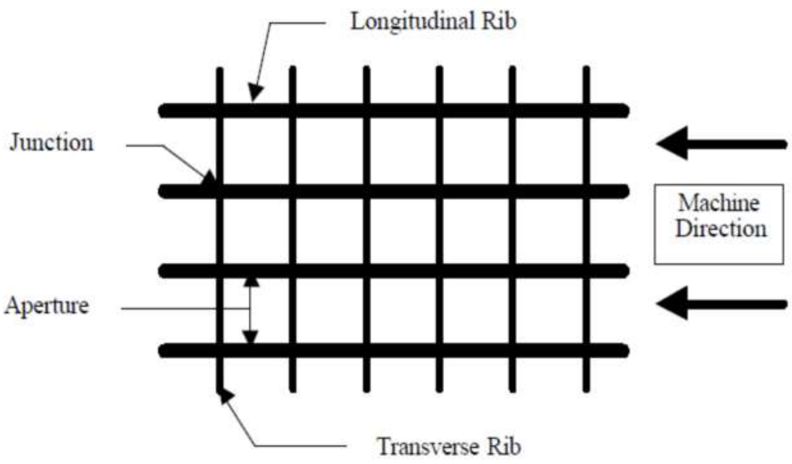


Figure 2-14: Geogrid component nomenclature (Stadler, 2001)

A. uniaxial B.| Biaxial C. Triaxial

Figure 2-15 : Types of geogrid (Ellis, et al., 2008)

Uniaxial geogrids usually exhibit a high stiffness in the machine direction [MD] with low to negligible stiffness in cross-machine direction [XMD]. However, it should be noted that there are products which have their maximum properties in the XMD. Uniaxial geogrids are intended for use in plane strain conditions where the secondary direction has a minimal loading. They are used to reinforce retaining walls, steepened slopes, dams, levees, landslide repairs, and roadway embankments. In comparison, biaxial geogrids have strength in both the longitudinal and transverse direction. By having tensile strength in two directions, they can distribute load forces, making them ideal for basal reinforcement and subgrade improvement (Kupec & Mcgown, 2004). Triaxial geogrids, which are a recent development in the industry, have strength in multiple directions.

Geogrids are also categorized in two main groups based on their rigidity. Geogrids made from polyethylene (PE) or polypropylene (PP) fibres are usually hard and stiff and they have a flexural strength more than 1,000 g-cm (ASTM, 1993). Flexible geogrids, are often made from polyester (PET) fibres by using a textile weaving process (Koerner, 1998).

Common geogrid types currently available in market include welded geogrid, extruded geogrid, and woven geogrid. Extruded geogrid is produced from a polymer plate which is punched and drawn in either one or more ways. Various aperture types are shaped based on the way the polymer sheet is drawn (Das, 2015).

The main physical properties of geogrid which have a significant effect on the application of geogrid in civil works are structures, junction type, aperture size, thickness, shear test, mass per unit area, flexural rigidity and stiffness.

Owing to the two orthogonal directions of stresses, the geogrid used in this study is a bi-oriented geogrid that is made of polypropylene and manufactured by extrusion and biaxial orientation to enhance its tensile properties. It is generally used for soil stabilization and embankment reinforcement. This geogrid has high tensile strength, high elastic modulus, and strong resistance to construction damage and environmental exposure.

2.7 Applications of Geogrid in Road Pavement

The uses of geogrid in a pavement system are to (a) aid construction over soft subgrades, (b) improve or extend the pavement service life, and (c) reduce the structural cross section of the pavement for a given service life (Christopher & Holtz, 1985; Giroud, 1985).

A subgrade is typically reinforced by placing a geogrid at the subgrade/sub-base or subgrade/base interface to improve the ability of the weak subgrade to withstand traffic loads without excessive deformation. Geogrids provide reinforcement by laterally restraining the base or sub-base and improve the bearing capacity of the system, thus decreasing shear stresses on the weak subgrade. In addition, the confinement provided by geogrids improves the distribution of the vertical stress over the subgrade and decreases vertical subgrade deformation." (Monica, et al., 2012). Geogrids are able to improve the performance of subgrade soils through Four mechanisms; (Moayedi , et al., 2009).

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.

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.

Geogrid reinforcement reduces permanent deformations in flexible pavement systems and allows up to a 50% reduction in the required thickness of a granular base based on equal load-deformation performance (Carroll, et al., 1987; Perkins & Ismeik, 1997). Webster (1991), performed studies on geo-grid reinforcement of flexible pavements for light aircraft and indicated that geogrid reinforcement, which should be placed between the aggregate and subgrade layers for best performance, improves the performance of the pavement systems as a whole. Full-scale tests have verified that for California Bearing Ratio (CBR) strengths in the range of 1.5 to 5.0%, geogrid reinforced pavements can carry about 3.5 times more traffic load repetitions than non-reinforced pavements before a rut depth of 37 mm is reached.

Traffic loads applied to the roadway surface create a lateral spreading motion of the base aggregate. Lateral tensile strains are created in the base just below the applied load as the material moves down and away from the load (Berg, et al., 2000). Lateral movement of the base allows vertical strains to develop, leading to a permanent rut in the wheel path (Houlsby & Jewell, 1990). Placement of a geo-grid layer at the bottom of the base course allows shear interaction to develop between the aggregate and the geogrid as the base attempts to spread laterally. The mobilized shear load is transferred from the base aggregate to the geogrid. The relatively high stiffness of the geogrid helps delay the development of lateral tensile strains in the portion of the base adjacent to the geogrid. Lower lateral strains in the base produces less vertical deformation of the roadway surface (Monica, et al., 2012).

The presence of a geogrid at the bottom of the base or sub-base can also lead to a change in the state of stresses and strains in the subgrade. The geogrid layer increases the stiffness of the base or sub-base. It distributes and decreases the vertical stresses on the subgrade beneath the base or sub-base. As a result, geogrid reinforcement reduces shear strains in the subgrade. Use of geogrid over soft subgrade helps with the transfer of stresses from the relatively weak subgrade to the relatively strong base course material. The result is an improvement in the bearing capacity of the subgrade resulting from transfer of stresses at the geogrid subgrade interface (Perkins & Ismeik, 1997).

Nader & Mohammad (2016) were carried out an experimental laboratory program to assess the effectiveness of biaxial and triaxial geogrid reinforced flexible pavements and Performances of biaxial-reinforced and triaxial-reinforced sections were compared with that of companion unreinforced sections. The test results revealed that inclusion of both biaxial and triaxial geogrids in flexible pavement reduced the surface rutting and vertical stresses in the subgrade-base interface. For the studied geogrid-reinforced pavement sections, no tensile strain was experienced by the strain gauges installed on the ribs of the geogrids, the vertical pressure at the centre of subgrade-base interface reduced by an average of 18 and 24% for biaxial and triaxial geogrid-reinforced pavement sections, respectively. Using the results of rutting depth, it was found that use of geogrid increased the number of load applications by a factor of 1.5 to 7 depending on the test section and geogrid type, as well as rutting depth experienced at different loading applications. Moreover, Inclusion of geogrid resulted in the base thickness reductions of 11 to 44 percent depending on the above-mentioned variables.

The selection of fabric for a particular construction application must necessarily depend upon adequate and suitable fabric properties and characteristics. The effective important properties of geogrids may be concerned in three main properties:

- Physical properties
- Mechanical properties
- Endurance properties

Many of the physical properties of geogrids including the weight (mass), type of structure, rib dimensions, junction type, aperture size, and thickness can be measured directly and are relatively straightforward. Other properties that are of interest are mass per unit area, which varies over a tremendous range from 200 to 1000 g/m2, and percent open area, which varies from 40 to 95%. And, the mechanical properties of geogrids including the geogrid stiffness, the peak tensile strength, modulus of elasticity, upper yield strength, lower yield strength, tensile strength, non-proportional extension strength, total extension strength, fracture elongation, elongation at maximum load and total elongation (Koerner, 2012).

Mohammed (2012) was tested 90 samples of 7 types of geogrid (Netlon CE121and Tensar SS2, manufactured in Britain, a type of geogrid which manufactured in Iraq, manufactured in China, SQ12, SQ15 and CE131 manufactured by Pars Mesh Polymer in Iran) by computer controlled electronic universal testing machine and this machine gave the information of peak tensile strength, modulus of elasticity, upper yield strength, lower yield strength, tensile strength, non-proportional extension strength, total extension strength, fracture elongation, elongation at maximum load and total elongation after completing test for each geogrid type. Based on the experimental study the following conclusions had been made:

- The geogrids Netlon CE121 and Tensar SS2 have tensile strength and elastic modulus higher than other geogrids made by different manufactures.
- The tensile strength and elastic modulus are not dependent on the density of materials but dependent on type of polymer used and method of manufacture.
- The dimensional properties such as rib thickness, junction thickness, longitudinal and transverse rib width of geogrid play important role in the mechanical properties such as tensile and elastic modulus.
- The effect of tensile strength (stiffness) is more significant than elastic modulus when geogrids are used as reinforcement in the soil

2.8 Numerical Modelling of Geogrid Reinforced Flexible Pavement

Numerical modelling of geosynthetic reinforced pavement is difficulty and uncertainty task. These uncertainties are related to simulation of the reinforcement mechanism of geosynthetic layers. Long-term performance is defined as pavement responses such as permanent surface deformation, stress, and strain after application of a large number of load cycles. Application of the reinforcement benefit is that a reinforced section can tolerate a higher number of load cycles than an unreinforced section before failure occurs. In addition, long-term reinforcement benefits are significantly more apparent when compared with short-term application (Perkins, 2001).

Reinforcement benefits are clearly apparent in several numerical modelling studies. The 1972 AASHTO design method was modified by Barksdale & Brown (1989) to determine design thickness of pavement sections with geosynthetic reinforcement. The study showed that reinforcement reduced lateral pressure at the bottom of the AC layer by 4 to16% and vertical pressure on top of the subgrade by 6 to 18%.

Dondi (1994) used three-dimensional finite element model to analyse geosynthetic reinforced pavement under wheel load of 1500 kPa separated by distance of 120mm. It evaluates stress and strain in base and subgrade layer. The result indicates that base layer experienced moderate increase load carrying capacity for reinforced case while strain in subgrade layer was decrease for reinforced case. The vertical displacement for the loaded area is reduced by 15% to 20% by the inclusion of geosynthetic. Finally; the horizontal tensile strain in AC is evaluated to analyse fatigue life of the section, showing that the life of reinforced section could increase by a factor of 2 to 2.5 as compared to unreinforced section.

Nazzal, et al. (2006) developed a two dimensional axisymmetric finite element model to investigate the benefits of reinforced base course layer in a flexible pavement structure with geogrid, and to evaluate the effects of different variables, such as the thickness of the base course layer, strength of subgrade soil, and the stiffness of the reinforcement layer on the performance of flexible pavements. Five different reinforced base course thicknesses and three different types of subgrades “weak, moderate, and stiff” were used in their study. Four different geogrid types were used by placing them at the bottom of the base layer. The pavement system was subjected to cyclic loadings. The parameter that was used to quantify the degree of improvement achieved by the geogrid reinforcement was the depth of rut after application of two million load cycles by using regression models. AC layer and geosynthetic material was modelled using a linear elastic model. Their study concluded that the permanent deformation (rutting) of pavement sections was reduced when geosynthetic was used. The amount of reduction depended on the subgrade stiffness, geogrid stiffness, and thickness of the base layer. In addition, the effect was found to be more profound for a weaker subgrade than for a stiffer one. The effect of geogrid reinforcement was reduced when the thickness of the base layer increased, and improved when the stiffness of subgrade layer increased.

Howard & Warren (2006) used axisymmetric finite element model to analyse geosynthetic reinforced pavement. Perfectly plastic Mohr-coulomb model was used to model the properties of natural soil and compacted subgrade and linear elastic model was used to capture the behaviour of AC and geosyhthetic.

2.8.1 Materials Behaviour and Constitutive Laws

A. **Asphalt Concrete**

The contribution of the AC layer to surface rutting is dependent on the material properties of the system layers. Harold (1994) indicated that the AC layer behaves elastic or viscoelastic at low temperature the plastic response of bituminous mixtures can be neglected. Also, Benedetto and Roche (1998) concluded that bituminous mixtures exhibit a complex elasto-viscoplastic response but at small strain magnitude the plastic component can be neglected. In this study, where the AC properties are considered at low temperature and for the given load amplitude, the vertical permanent deformation of the AC layer is considered to have insignificant contribution to the total surface deflection. Furthermore, for a load affecting a structure, when the time duration of this load is small, which is the case beforehand, the viscoelastic behavior of this structure becomes almost equivalent to an elastic structure. Therefore, in view of the asphalt properties and characteristics of the loading system (relatively short time duration) and for simplicity, the asphalt concrete layer is treated as linear elastic in this study.

B. Granular Materials

The pavement layers are subjected to both compression and tension under moving load, and only compression due to static loading. Irrespective of loading, both elastic and plastic deformations are developed depending upon the imposed stress level. For the stress greater than yield stress, elastic as well as plastic deformation occurs.

The granular material and subgrade soil in pavements behaves as a combination of the elastic and plastic deformation through the cycles of loading. This behavior is referred to as elasto-plastic behavior as shown in Figure 2-16 below. When the loading extends beyond the elastic limit into the plastic behavior, it results in accumulation of non-recoverable or plastic strain.

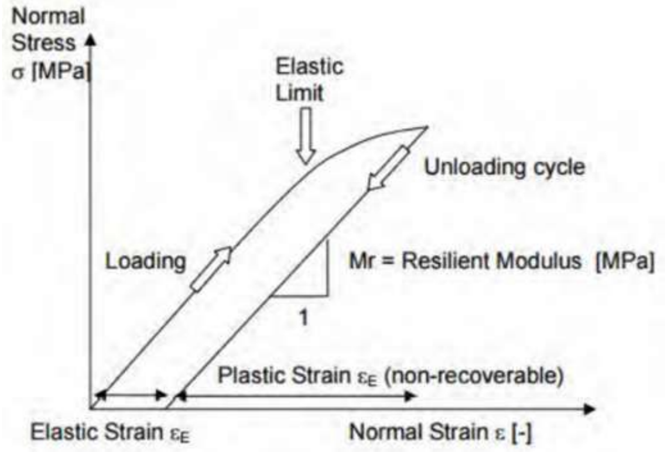


Figure 2-16: Elasto-plastic behavior of granular material (Jenkins & Rudman, 2013)

When the embankment is described by finite elements, the most widespread constitutive law is isotropic linear elasticity (55%), followed by perfect elasto plasticity (36%) and non-linear elasticity (9%) (Ti , et al., 2009).

Due to limitation on material information and different literatures recommendation the present analysis deals with elastic deformation of layered pavement system based on mechanistic approach and three dimensional (3D) finite element analyses. The mechanistic approach works well when the pavement subgrade system behaves as a linear elastic system and the loading is considered to be static (Uddin , et al., 1994).

C. Geosynthetic (Geogrid)

Perkins (2001) conducted a literature review showing the constitutive laws implemented in previous finite element analyses of geosynthetic reinforced flexible pavement systems. In his review, Perkins (2001) demonstrated that in most of these analyses the geosynthetic reinforcement membrane is considered as an isotropic elastic material. Such model proved efficient when used by other researchers; e.g., Ling & Liu, (2003); Kedir (2015); Damiso (2017). Therefore, in this study the geogrid is assumed to act as a linear isotropic elastic material.

It is assumed that no slippage occurs between the material layers. The full bonding of the geosynthetic and the surrounding layers is an acceptable assumption for the case of a paved system where the allowed surface rutting of such a system surface is small and the slippage is not likely to occur unless excessive rutting takes place (Espinoza, 1994).

2.8.2 Nature of Traffic Loading

When a vehicle is moving on the pavement, the surface experiences both static and dynamic effects. Static load is imparted through vertical (axle, wheel and tire) loads on the pavement; because of the force of gravity, these are constant.

Due to unevenness of the road pavement, the vehicle will move up and down causing a dynamic variation of the loads on the pavement, above and below their static values. The magnitude of this dynamic variation depends on various factors such as static loading, the spring and damper characteristics of the vehicle and the road roughness (i.e. the unevenness of the road surface in the longitudinal direction) and the vehicle speed. Generally, the dynamic variation increases with both speed and road unevenness (Hjort , et al., 2008).

In reality pavements are subjected to both static and moving loads. However, Saad , et al., (2005) have found that the static loading condition is more detrimental to the pavement system as compared to the dynamic condition causing almost two times higher maximum vertical surface deflection. Similar trend of results have also been reported by Singh & Sahoo (2020) and Uddin et al. (1994) between static and dynamic analysis solutions. Therefore, static loading condition is considered for this study.

The wheel load P (kN), is the load applied by one of the wheels in the case of single wheel axle loads and is the load for two wheels in the case of dual wheel axles. The wheel load P is considered to be half of the axle load. The relationship between wheel load and tire contact pressure is shown in equation 3-2 below (Mallik & El-Korchi, 2013).

$$P = pA \text{------------------ (2-2)}$$

Where, A = Tire contact area in m^2; p = Tire contact pressure in kPa. For practical reasons, tire contact pressure p is normally taken as equal to the tire inflation pressure. The tire contact area is represented as a circular area called the equivalent tyre contact area, with radius r. Replacement of the circular area formula into Equation 3-3 and making the radius (r) subject of the formulas yields;

$$r = \sqrt{\frac{P}{p*\pi}} \text{--------------------- (2-3)}$$

For instance, considering a standard axle of 80kN, Wheel load P of 40 kN and a tyre inflation *p* of 550 kPa as used in AASHO road test, the radius (r) of an equivalent contact area as r = 152 mm is obtained. The loading is considered as static.

2.9 Summary

The pavement structure, subgrade soil, functions of geosynthetics in general and geogrid specifically were discussed.

The geosynthetics (geogrid) material characteristics and requirements, significance and use, application and reinforcement principles are reviewed and summarized as follows:

- A geosynthetic reinforced soil is stronger and stiffer than soil without reinforcement.
- Inclusion of geosynthetic ensures a long lasting pavement structure by reducing excessive deformation and cracking.
- Addition of geosynthetic in form of geotextile, geogrid reduces pavement thickness significantly.
- Placing of Geosynthetic (geogrid) material in soil improves bearing capacity and therefore implies that geosynthetic (geogrid) increase load carrying capacity of soil.
- Geogrid improve subgrade restraint and base reinforcement applications.
- Inclusion of geogrid improves the shear resistance at the interface by offering interlocking resistance and reduce the lateral movement of the soil.
- Reinforced soil shows better resistance under repeated loads.

CHAPTER THREE

3 METHODOLOGY

3.1 Introduction

This chapter deals with modeling of the 3D geometry of the flexible pavement unreinforced, reinforced with geogrid, selection of the material properties, loading and finite element analysis (FEA). A brief summary of the routines that have been used to achieve the objective of this study are presented. Tercha-Chida Road Project which is located in Southern Ethiopia having 58km length was selected for this research work.

The finite element method (FEM) is the dominant discretization technique in structural mechanics. The basic concept in the physical interpretation of FEM is the subdivision of the mathematical model into disjoint (non-overlapping) components of simple geometry called finite elements. The response of each element is expressed in terms of a finite number of degree of freedom characterized as the value of unknowns functions at a set of nodal points. For the purpose of this finite element analysis a finite element solver software package, ABAQUS was selected.

ABAQUS is used as a finite element solver software package to formulate the finite element model for section of flexible pavement structure with and without geogrid reinforced subgrade. Parameters observed during the numerical simulation include vertical displacement, subgrade stress and strain and lateral displacement. The results of the analysis were compared with an unreinforced section of flexible pavement structure at the same geometry and material properties. At the end, the comparisons between reinforced and unreinforced subgrades of pavement structure summary have been given.

3.2 Description of the Study Area

3.2.1 Project Location and Accessibility

Tercha – Chida Road Project is located in the Dawro and Kulo Zone, Southern Nations and Nationalities Peoples Regional State (SNNPRS) of Ethiopia, which is approximately 474km from Addis Ababa on Tercha side and 482kms away from Addis Ababa on Chida side. The project road mainly traverses through a terrain of mainly mountainous and few sections of rolling nature. The road is going to be constructed to ERA's DS4 standard.

The start of the project Tercha, can be accessed by travelling 474km from Addis Ababa on the Addis Ababa - Butajira - Alaba - Sodo - Jimma. The end of the project Chida can also be accessed by travelling 482km on the Addis Ababa – Weliso – Welkite – Jimma – Sodo road. The location map of the Project road with respect to the map of Ethiopia is shown in figure 3-1 below.

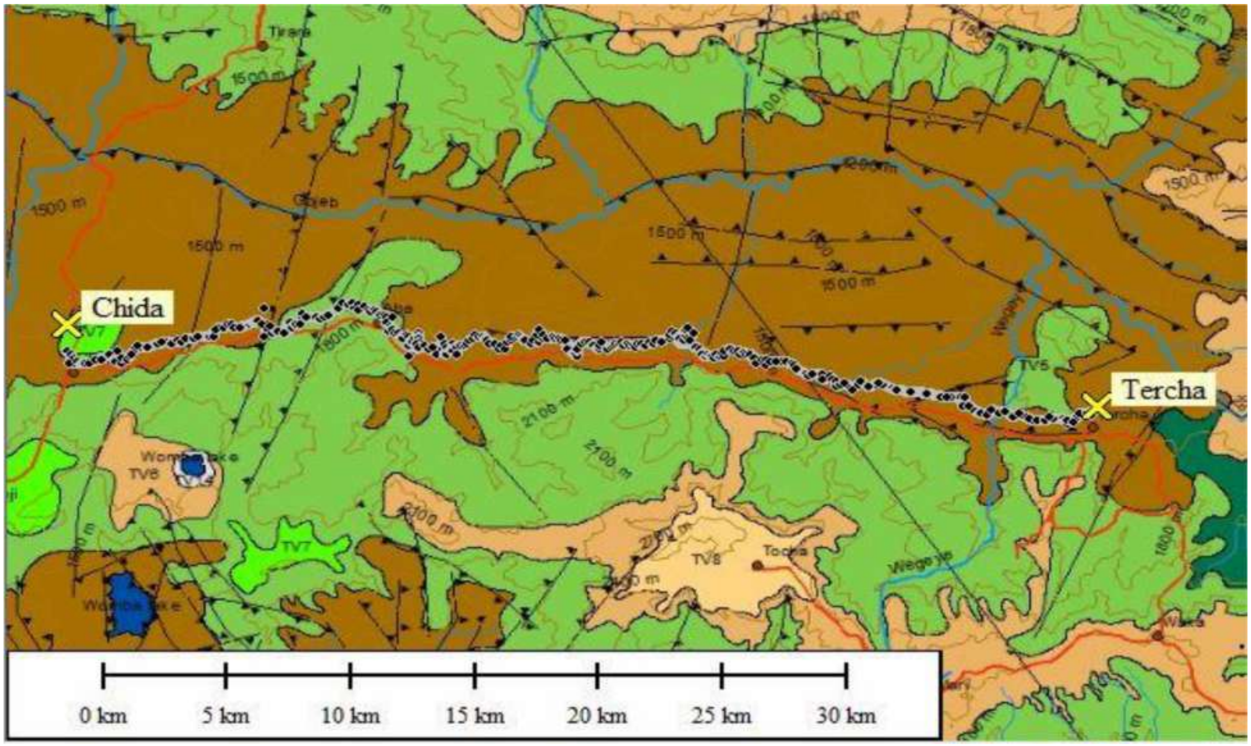
Figure 3-1: Location map of the study area (Stadia and G&Y, 2018)

Figure 3-2: Up-close project location map of the study area (Stadia and G&Y, 2018)

3.2.2 Geology

According to the geological map of the area (extracted from Jima sheet, 1:250,000), the Tercha - Chida Road Project traverses on primarily volcanic rocks of Triassic age (refer Fig 3-3 below). The regional geology of the area is part of the south western Ethiopian highlands and associated lowlands. It is found in sequence from the oldest at the bottom to the youngest at the top. All of these Triassic rocks do not encountered in the road corridor. Four geologic units among the total are traversed or found near the route corridor. These are the lower and upper trachyte (TV3 and TV8) and basalt (TV5 and TV7) rocks units found layered in sequences from top to bottom. Majority parts of the road corridor lays on the lower trachyte flows with rare intercalation of pyroclastic and coal bed deposit followed by lower layered basalt flows with rare trachyte. Deep weathering can be best describes the rocks found almost in entire route alignment. However, some places possible to get moderately weathered rock units. Silty clay soil and conglomerates also found at some localities of the road corridor.

Colluvial and alluvial deposits are also encountered in the area. Colluvial deposits cover the slope as a talus deposit at some and alluviums are found near the river and streams in the project area.



TV3	Lower trachyte flows; Stratified massive trachyte with rare pyroclastic and coal beds at places
TV5	Middle basalt flows; mainly basalts with rare intercalation of trachyte
TV7	Upper basalt flows; it consists mainly of layered basalt with rare trachyte
TV8	Upper trachyte flows; thickly layered trachyte flows

Figure 3-3: Geological map of the project area (Stadia and G&Y, 2018)

3.2.3 Soil Type

Detail description of the sub-grade soil type of the project route corridor and laboratory tests had been carried out by the Design Consultant (DANA, et al., 2014) and presented in the original design.

As per the design consultant (DANA, et al., 2014) finding and site visit observation, the sub-grade material throughout the project corridor can be generalized as:

- Reddish clay soil with scattered gravel being dominant
- Light brown clayey silt soils
- Dark brown clay mixed with gravel and rock.

3.2.4 Climatic condition

As it can be seen from ERA Drainage Design Manual, the daily mean average temperature of the Project area is about between 17.5^0C and 22.5^0C. The average maximum daily temperature of the project area is in between 22.5^0C & 27.5^0C while the average minimum daily temperature is in between 12.5^0C & 17.5^0C.

According to the map shown on ERA Drainage Design Manual 2002, the Project area is located in B1 & B2 Rainfall Regions which receive high annual rainfall. It lies in one of the wettest zones of Ethiopia and the mean annual rainfall of these regions is between 1201 - 1600mm per year. Based on mean annual rainfall estimate from Chida Station (1477mm), the project corridor is categorized under Weyna Dega to Dega Climate Zone where an extended period rainfall likely to exist from March to October as shown in the graph bellow.

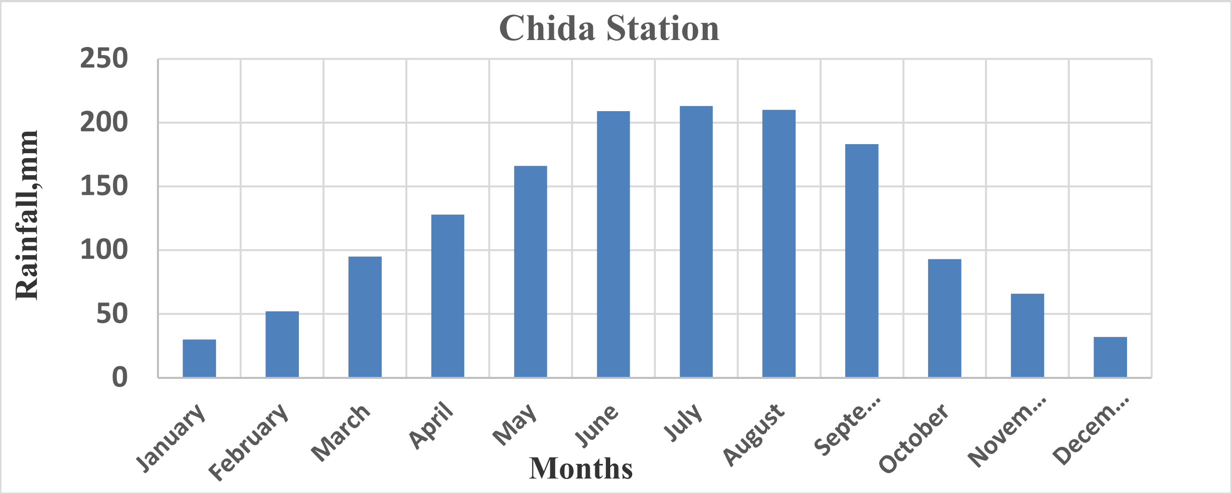


Figure 3-4: Monthly mean rainfall pattern (Stadia and G&Y, 2018)

3.2.5 Vegetation cover and Land use

Most of the project corridor is covered with cereal crops. The remaining land is covered with scattered trees of small leaves and shrubs.

3.3 Geometry of the Pavement Structure

Table 3-1: Pavement thickness of the project (DANA, et al., 2014)

Asphalt Concrete	50mm
Road base	200mm
Sub base	200mm

3.4 Material

According to (ERA, 2013) The Material characteristics for mechanistic analysis of flexible pavement structure is as shown in table 3-2 below:-

Table 3-2: the material characteristics for mechanistic analysis of flexible pavement structure (ERA, 2013)

Material	**Parameter**	**Value**	**Comment**
Asphaltic concrete wearing course and binder course	Elastic modulus (MPa)	3000	A balance between a value appropriate for high ambient temperatures and the effect of ageing and embrittlement
	Volume of bitumen	10.5%	
Asphaltic concrete road-base	Elastic modulus (MPa)	3000	
	Volume of bitumen	9.5%	
Granular road-base	Elastic modulus (MPa) Poisson's ratio	300 0.30	For all qualities with CBR > 80%
Granular sub-base	Elastic modulus (MPa) Poisson's ratio	175 0.30	For CBR ≥30%
Capping layer	Elastic modulus (MPa) Poisson's ratio	100 0.30	For CBR ≥15%
Subgrades S1 S2 S3 S4 S5 S6	Elastic modulus in MPa	 28 37 53 73 112 175	Poisson's ratio for all subgrades was assumed to be 0.4
Hydraulically stabilised material	Elastic modulus (MPa)	CB1 = 3500 CB2 = 2500 CS =1500	Poisson's ratio assumed to be 0.25 The modulus of CS is assumed to decrease with time hence a conservative low value of 1000MPa has been used

A. Subgrade Soil

The strength of the road subgrade for flexible pavements is commonly assessed in terms of the California Bearing Ratio (CBR) and this is dependent on the type of soil, its density, and its moisture content (ERA, 2013).

According to (ERA, 2013) the subgrade strength for design be assigned to one of six strength classes reflecting the sensitivity of thickness design to subgrade strength as indicated in table 3-3 below.

Table 3-3: Subgrade strength classes (ERA, 2013)

Class	CBR Range (%)
S1	<3
S2	3,4
S3	5,6,7
S4	8-14
S5	15-30
S6	>30

Testing of the subgrade materials for grain size distribution, liquid and plastic limits, moisture-density relationship, CBR and swell was mainly required to determine the material classification and identification of the bearing capacity of the soil in terms of the CBR. Hence, the laboratory test result of subgrade materials of the study area had been conducted by the design consultant (DANA, et al., 2014) is summarized under Appendix-A Table 7-2 and used as secondary data.

Relatively soft subgrade soil that exists at the chainages 12+100, along the route of Tercha-Chida road project which have different subgrade strength classes was randomly selected for this study to exhibit the effect of geogrid reinforcement for different strength class subgrade type. The detail subgrade soil material characteristics of the study area which are taken from soil test results and literatures are summarized in the table 3-4 below:-

Table 3-4: Summary of subgrade soil material characteristics

No.	Chainage	Subgrade soil test Results **(1)**					AASHTO Soil Class	Subgrade Class **(2)**	Elastic modulus (MPa) **(2)**	Poisson's ratio, μ**(2)**	Internal Friction angle(Ø) **(3)**	Cohesion (c) (KPa) **(3)**
		LL	PI	Swell	CBR	Density (g/cm3)						
1	12+100	49	23	4.82	1	1.500	A-7-5	S1	28	0.4	28	86

<u>NB: </u>*Sources of subgrade soil material characteristics are as follows*

(1) (DANA, et al., 2014)

(2) (ERA, 2013)

(3) (Nuevvo, 2013)

B. Sub-base and Base-course

The specifications and other properties of base-course (crushed stone) material and sub-base (natural gravel) are as follows in table 3-5 & 3-6 below:-

Table 3-5: Base-course material specification and characteristics

Description	Grading Requirement		Liquid limit (LL)	Plastic Index (PI)	Swell (%)	CBR (%)	Internal Friction angle(Ø) (^{0})	Cohesion (c) (Mpa)
	Sieve size (mm)	% age by mass passing						
Base-course Material (Crushed Stone)	37.5	100	-	None Plastic	-	>100%	55	Approx.0
	25.4	95 - 100						
	12.5	60- 80						
	4.25	40 - 50						
	2.36	5 - 25						
	1.18	0 - 8						
	0.30	0 - 8						
	0.075	3 - 8						
Sources	(DANA, et al., 2014)						(Kedir, 2015)	

Table 3-6: Sub-base material specification and characteristics

Description	Grading Requirement		Liquid limit (LL)	Plastic Index (PI)	Swell (%)	CBR (%)	Internal Friction angle(Ø) $(^0)$	Cohesion © (Mpa)
	Sieve size (mm)	% age by mass passing						
Sub-base Material (Natural Gravel)	63.0	100	<35%	<6	<1%	>30%	55	Approx.0
	50.0	90-100						
	25.0	51-80						
	9.5	-						
	4.75	35-70						
	2.0	-						
	0.425	-						
	0.075	5-15						
Sources	(DANA, et al., 2014)						(Nuevvo, 2013)	

C. Asphalt Concrete

The type and grade of bituminous binder to be used for Tercha-Chida Road Project and the aggregate strength requirement are as follows in table 3-7 below:-

Table 3-7: Grade of bituminous binder and aggregate strength requirement (DANA, et al., 2014)

Description	**Specification**
Grade of bituminous binder	80/100 penetration grade
Aggregate crushing value (ACV)	< 25%
Aggregate Impact Value (AIV)	< 25
Los Angeles Abrasion Value (LAA)	< 30

D. Geogrid

Owing to the two orthogonal directions of stresses, the geogrid used in this study is a bi-oriented geogrid that is made of polypropylene and manufactured by extrusion and biaxial orientation to enhance its tensile properties. It is generally used for soil stabilization and embankment reinforcement. This geogrid has high tensile strength, high elastic modulus, and strong resistance to construction damage and environmental exposure.

Based on the recommendations of manufacturers and different literatures Tensar geogrid SS2 manufactured by the British Company Netlon ltd. and has better tensile strength and elastic modulus been selected for this research work. The physical and mechanical properties of the selected Tensar SS2 geogrid are summarized as follows in the table 3-8 below:-

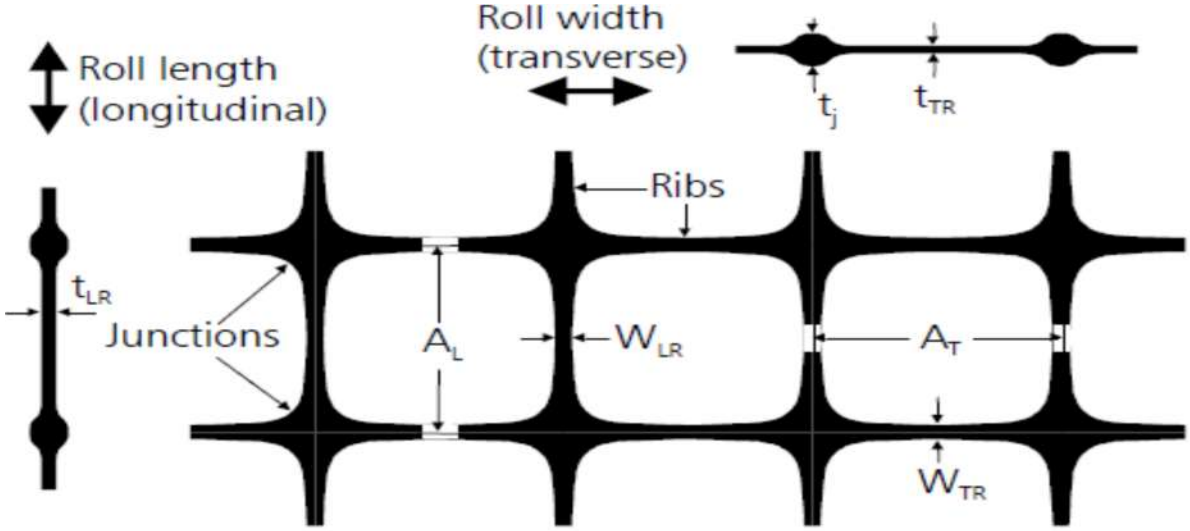


Figure 3-5: Geogrid dimension (Tensar, 2015)

Table 3-8: Geogrid dimension and material characteristics

Property	Unit	Value	Remark
Polymer	Polypropylene		
Aperture Size, ALXAT	mm	50/50	(T. Kütük-Sert, n.d.; Kedir, 2015)
Rib Thickness, tTR/tLR	mm	1.27	(Tensar, 2015; Kedir, 2015; DANA, et al., 2014)
Rib Width, WTR/WLR	mm	4	
Roll Length	m	30-50	
Roll Width	m	3.8-4	
Mass density, p	g/cm3	≥0.94	(Kedir, 2015)
Elastic Modulus, E	MPa	990	(Mohammed, 2012)
Poisons Ratio, υ	-	0.35	(Erickson & Drescher, 2001)
Peak Tensile Strength MD/XMD	KN/M	17.5/31.5	(Tensar, 2015)
Junction strength	%	90	

3.5 Finite Element Analysis

3.5.1 Introduction

Finite element modeling of reinforced flexible pavement foundation includes geometry modeling, load modeling and material modeling. The commercial FEM program ABAQUS was used in this study. ABAQUS is a powerful finite element software package. It has been used in many different engineering fields throughout the world. ABAQUS software performs static and/or dynamic analysis and simulation of complex engineering and non-engineering problems and it can deal with bodies with various loads, temperatures, contacts, impacts, and other environmental conditions.

3.5.2 Assumptions

- ✓ 3D Finite element analysis using ABAQUS
- ✓ The analysis is completely quasi-static and considers vertical track force only
- ✓ All the pavement layers are assumed as homogenous and isotropic.
- ✓ linear elastic behaviour of granular base course, sub base and subgrade
- ✓ The asphalt concrete layer is treated as linear elastic
- ✓ The geosynthetic reinforcement is treated as linear elastic material
- ✓ The rib and web of the geogrid is made same dimension
- ✓ For ease of simplification symmetry of the line is considered
- ✓ For ease of comparison the subgrade is made to be soft

3.5.3 Analysis using ABAQUS

ABAQUS is a suite of powerful engineering simulation programs, based on the finite element method that can solve problems ranging from relatively simple linear analyses to the most challenging nonlinear simulations. It has an equally extensive list of material models that can simulate the behavior of most typical engineering materials including metals, rubber, polymers, composites, reinforced concrete, crushable and resilient foams, and geotechnical materials such assoils and rock (ABAQUS, 2012).

ABAQUS is designed as a general-purpose simulation tool; it can be used to study more than just structural (stress/displacement) problems. The element functions are gathered in the global equation system containing material and geometrical data. The forces applied on the element geometry are represented by load vectors that act in the nodes. The matrixes quickly increase in size and demand high computer performance to be solved. The nodal deflections are the solution to the equation system. The values between the nodes are received by interpolation with either linearly approximations or polynomials of n degrees.

A complete ABAQUS analysis usually consists of three distinct stages: preprocessing, simulation, and Post - processing. These three stages are linked together by files as shown in figure 3-6 below. (ABAQUS, 2012):

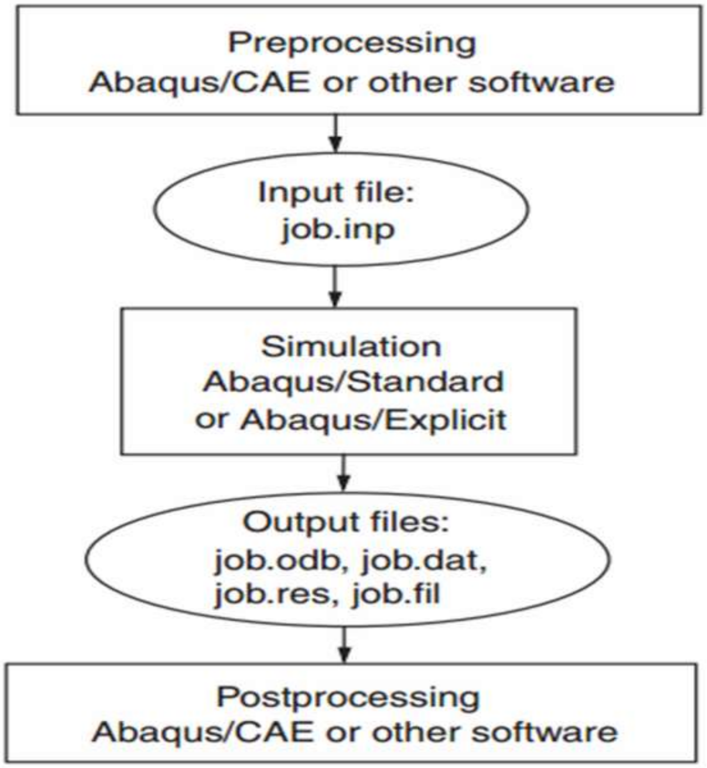


Figure 3-6: ABAQUS analysis stages (ABAQUS, 2012)

3.5.3.1 Pre-processing

In this stage the model of the physical problem must be defined and Abaqus input files shall be created. The model is usually created graphically using Abaqus or another preprocessor, although the Abaqus input file for a simple analysis can be created directly using the text editor as required.

All the steps to create the model with ABAQUS, The following principal steps are taken sequentially:-

- ✓ Creating a part/defining the model geometry
- ✓ Defining the material and section properties.
- ✓ Creating an assembly.
- ✓ Configuring the analysis.
- ✓ Assigning interaction properties.
- ✓ Applying boundary conditions and loads
- ✓ Designing the mesh.
- ✓ Creating, running, and monitoring a job.

Below are discussions of procedures and assumptions made for the preceding steps in modeling the flexible pavement layered system. Before starting of ABAQUS/CAE, the dimension can be set, because ABAQUS/CAE has no its own Unit by default.

Before starting to define this or any model, deciding which system of units used. Because of ABAQUS has no built-in system of units. All input data must be specified in consistent units. Thus, in this model SI (mm) is used as shown in Table 3-9 below.

Table 3-9: Consistent units used in the model input and outputs (Endalemaw, 2016)

Quantity	Length	Force	Mass	Elastic Modulus	Time	Stress	Density
SI (mm)	mm	N	Ton (10^3Kg)	MPa	S	MPa	10^3kg/mm^3

a. Creating a Part /Defining the Model Parameters of Flexible Pavement Geometry

The model is created with a three-dimensional, deformable body with a solid, extruded base feature. Parts can be created as native to ABAQUS, or can be imported created by other applications either as a geometric representation or as a finite element meshes. For this thesis a model native to ABAQUS using the sketcher, by a deformed 3D extrusion option is created. The next step in creating the model involves defining and assigning material and section properties to the part. Each region of a deformable body must refer to a section property, which includes the material definition.

b. Material Properties

The numerical analysis considered elastic linear behavior for all material including the subgrade soil which is characterized by the elastic modulus (E) and the Poisson ration (μ). Table 3-10 represents the summarized mechanical properties of the materials. The material properties used for FEA listed under are from literature, manuals, manufacturer's specifications and design specification of Terch-Chida Road Project.

Table 3-10: FE material properties of wearing course, base-course, sub-base and subgrade

Layer		1	2	3	4
Material	Unit	AC(asphalt concrete)	BC(base course)	SB(Sub-base course)	Subgrade (S1)
Model type	-	Isotropic	Isotropic	Isotropic	Isotropic
Thickness	mm	50	200	200	∞
Mass Density, ρ	kg/mm^3	$721*10^{-9}$	$1,600*10^{-9}$	$1,600*10^{-9}$	$1500*10^{-9}$
Elastic Modulus, E	MPa	3000	300	175	28
Poisson's ratio, μ	-	0.3	0.3	0.3	0.4

Table 3-11: FE material properties of geogrid

Property	**Unit**	**Value**	**Remark**
Polymer	Polypropylene		
Mass density, ρ	Kg/mm^3	$\geq 940*10^{-9}$	(Kedir, 2015)
Elastic Modulus, E	MPa	990	(Mohammed, 2012)
Poisons Ratio, μ	-	0.35	(Erickson & Drescher, 2001)

c. Creating an Assembly

Each part created is oriented in its own coordinate system and is independent of the other parts in the model. Although a model may contain many parts, it contains only one assembly. The geometry of the assembly is defined by creating instances of a part and then positioning the instances relative to each other in a global coordinate system. Thus, the AC, Base-course, Sub-base and Subgrade are assembled together with and without Geogrid Reinforcement. Therefore; based on the above geometric parameters, payment structure reinforced and unreinforced with geogrid are modeled with ABAQUS.

For numerical modeling purposes, the pavement system response to only a single wheel load which is to be at the center of the road cross section was considered. Moreover, due to the double symmetry of geometry, boundary conditions, and load about the horizontal x and y axes, only a quarter model is considered.

(Alex, 2000) Indicted that the nodal radial strains were assumed to be negligible at approximately10 times R (radius of loaded area) from the area applied wheel load. Also, the nodal stresses and displacements were assumed to be negligible at 20 times R below the pavement surface. Therefore, the width and the length of the model were set at 1.7m, and the total thickness of model is 3m. Total pavement structure thickness is 0.45 m above subgrade depth of 2.55 m. The thickness of AC surface course is 0.05 m, the thickness of base course is 0.20 m and the thickness of granular sub base course is 0.20 m as shown in Figure 3-7 below.

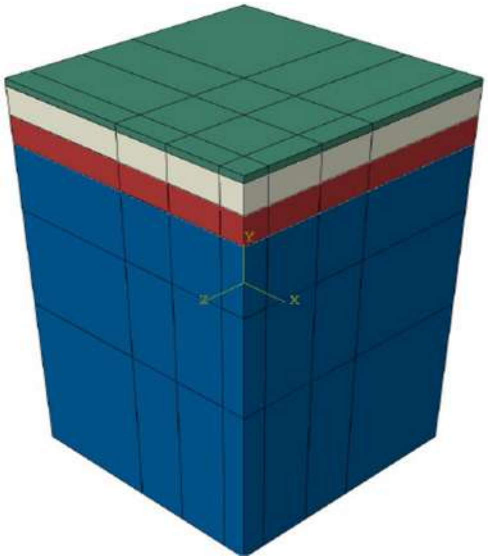


Figure 3-7: Parts of flexible pavement structure ABAQUS 3D-model, ABAQUS/CAE

d. Configuring the Analysis

Analysis steps can be mostly characterized as an initial step and analysis steps. A cross-section of pavement structure was modeled with a finite element mesh refined to observe important behavior of the foundation under loading, with or without geogrid reinforcing the subgrade-sub base interface.

➢ The initial step

ABAQUS/CAE creates a special initial step at the beginning of the model's step sequence and names it *"Initial"*. It allows defining boundary conditions, predefined fields, and interactions that are applicable at the very beginning of the analysis.

➢ Analysis steps

The initial step is followed by one or more analysis steps. Each analysis step is associated with a specific procedure that defines the type of analysis to be performed during the step. In this thesis, a static linear perturbation step is used; where in the first analysis step the wheel load is applied.

e. Assigning Interaction Properties

The interaction between contacting surfaces consists of two components: one normal to the surfaces and one tangential to the surfaces. The tangential component consists of the relative motion (sliding) of the surfaces.

The contact constraint is applied in ABAQUS when the clearance between two surfaces becomes zero. The surfaces separate when the contact pressure between them becomes zero or negative, and the constraint is removed. The system is subjected to a small force which does not induce slip. Thus, for the tangential component, rough interaction is assumed as there is no slip between the surfaces.

For normal behavior and the tangential behavior "hard" and rough contacts respectively are used in all interactions. Then interaction is created between all materials i.e. between the AC and the Base course, Base course and Sub-base, Sub-base and Geogrid and Geogrid and Subgrade with their interaction properties.

For geosynthetic–soil interface, a full bonding between the geosynthetic and the soil surrounding is assumed. For the case of a paved system, the allowed surface rutting is small and large slippage is not likely to occur unless excessive rutting takes place (Barksdale & Brown, 1989), (Espinoza, 1994). As such, full bonding assumption should be considered acceptable. The ABAQUS contact interaction feature was used in this study to model the geogrid–soil interface. With this feature, one surface definition provides the 'master' surface and the other surface definition provides the 'slave' surface. The master surface is used for rigid body surface, while the slave surface is used for deformable body surface. The interaction simulation consists of two components: one normal to the surfaces and one tangential to the surfaces. The interface in the normal direction is assumed to be 'hard contact' and no separation is allowed. While in the tangential direction, full interlocking was assumed between the geogrid layer and material surrounding it. This was done by using the tie-condition in ABAQUS interaction feature, where each node of the slave surface is tied to the nearest node on the master surface.

Basic Coulomb friction model was used to model the shear interaction, which relates the maximum allowable frictional (shear) stress across an interface to the contact pressure between the contacting bodies. The general form of the coulomb friction model is shown in figure 3-8 below:

$$\tau_{crit} = \mu\sigma \text{---------------------} (3.1)$$

Where: τ_{crit} is the critical shear stress along the interface; σ is the normal stress along the interface;μ is the interface friction coefficient ($\mu=\tan\delta$, where δ is the interface friction angle).

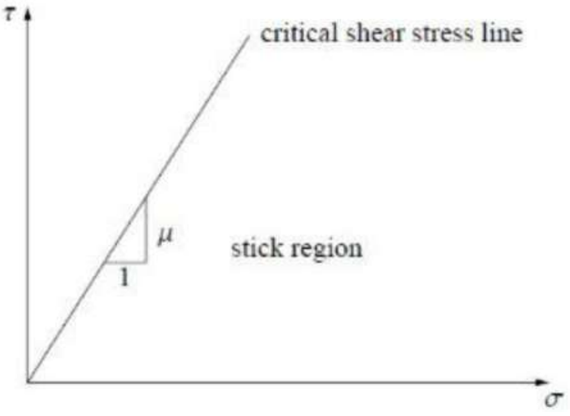


Figure 3-8: Basic coulomb friction model (Jie, 2011)

f. Loading

Standard axle loading consists of a dual-wheeled single axle, applying a load of 80KN considered on this flexible pavement structure. The wheel load P (KN), is the load applied by one of the wheels in the case of single wheel axle loads and is the load for two wheels in the case of dual wheel axles. The wheel load P is considered to be half of the axle load. The relationship between wheel load and tyre contact pressure is shown in equation 3-2 below (Mallik & El-Korchi, 2013).

$$P = pA \text{----------------------- (3-2)}$$

Where, A = Tyre contact area in m^2; p = Tire contact pressure in kPa. For practical reasons, tyre contact pressure p is normally taken as equal to the tire inflation pressure. The tyre contact area is represented as a circular area called the equivalent tyre contact area, with radius r. Replacement of the circular area formula into Equation 3-2 and making the radius (r) subject of the formulas yields;

$$r = \sqrt{\frac{P}{p*\pi}} \text{-------------------------- (3-3)}$$

For instance, considering a standard axle of 80kN, Wheel load P of 40 KN and a tyre inflation p of 550 kPa as used in AASHO road test, the radius (r) of an equivalent contact area as r = 152 mm is obtained. The loading is considered as static.

g. Applying boundary conditions

Prescribed conditions, such as loads and boundary conditions, are step dependent, which means that the step or steps in which they become active is specified accordingly. Conventional kinematic boundary conditions are adopted, i.e., roller support on all four vertical boundaries of the mesh and fixed support at the bottom of the mesh are used to prevent horizontal and vertical movement as shown in the figure 3-9 below. Such boundary conditions have been successfully used by (Zaghloul & White, 1993; Kuog, et al., 1995).

[redacted]

Figure 3-9: Boundary conditions of ABAQUS 3D-model, ABAQUS/CAE

h. Designing the Mesh

The Mesh module contains tools that allow generating meshes on parts and assemblies created within ABAQUS. In the model, a structure meshing is used. Structure meshing is a technique that gives the most control over the mesh because it applies pre-established mesh patterns to particular model topologies.

The modeled domain must be large enough to avoid any edge error. On the other hand, considerable care is taken to optimize the mesh size so as to get reliable results and the problem for computation time and storage requirements. A particularly stable and successful element, which is usually used in modeling the layers of a pavement system and employed in this study, is the eight-node isoparametric element.

As the loading on the pavement surface is localized, the finest mesh is required near the loaded area to capture the step stress and strain gradient in these areas. The subdivision is carried out so that the element aspect ratio remains close to one where the strain and stress gradients are high to achieve faster convergence in these areas. A particularly stable and successful element, which is usually used in modeling the layers of a pavement system and employed in this study, is the eight-node isoparametric element. The number of finite element meshes with different degrees of refinement was tried first in order to obtain an appropriate mesh for the analysis of pavement that converges to a unique solution as shown in the figure 3-10 below.

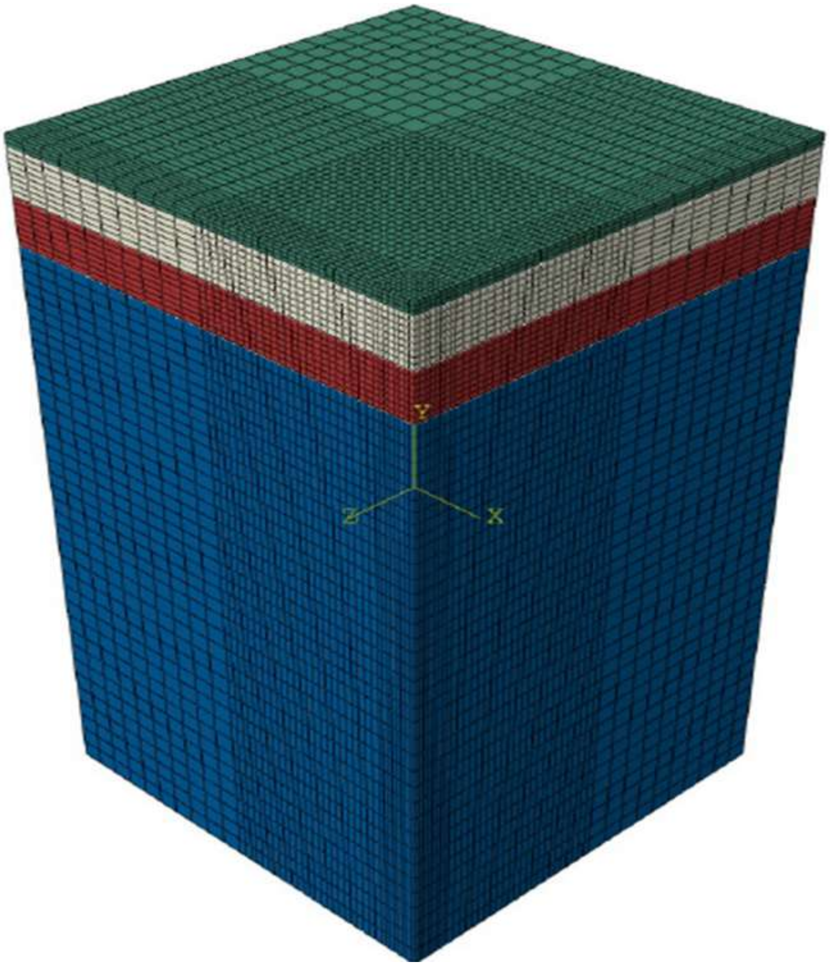


Figure 3-10: Meshing of unreinforced pavement structure of ABAQUS 3D-model, ABAQUS/CAE

i. Creating, Running and Monitoring a Job

Once defining a model is finished, the model is analyzed using the Job module. The Job module allows interactively submitting a job for analysis and monitoring its progress.

3.5.3.2 Simulation (ABAQUS/Standard or ABAQUS/Explicit)

The simulation which normally is run as a background process is the stage in which ABAQUS/standard or ABAQUS/Explicit solves the numerical problem defined in the model. Examples of output from a stress analysis include displacement and stresses that are stored in binary files ready for post-processing. Depending on the complexity of the problem being analyzed and the power of the computer being used, it may take anywhere from seconds to days to complete an analysis run.

3.5.3.3 Post Processing

The Visualization module provides graphical display of finite element models and results. It obtains model and result information from the output database; it is controlled what information is written to the output database by modifying output requests in the step module.

CHAPTER FOUR

4 RESULTS AND DISCUSSIONS

A series of simulations were performed on the Flexible pavement geometry in order to determine some of the selected important parameters. During the simulation, vertical and lateral deformation/displacement, stresses and strains were observed throughout the pavement structure in order to determine the behavior and improvement due to geogrid reinforcement. And, a comparison is made between unreinforced pavement structure and those of pavement structure reinforced with Geogrid.

The results from the finite element modeling in ABAQUS are presented. The purpose of the modeling is to investigate some important parameters of the pavement structure. The following Two static analyses have been carried out:-

- Unreinforced flexible Pavement structure,
- Flexible Pavement structure reinforced with geogrid on the top of subgrade

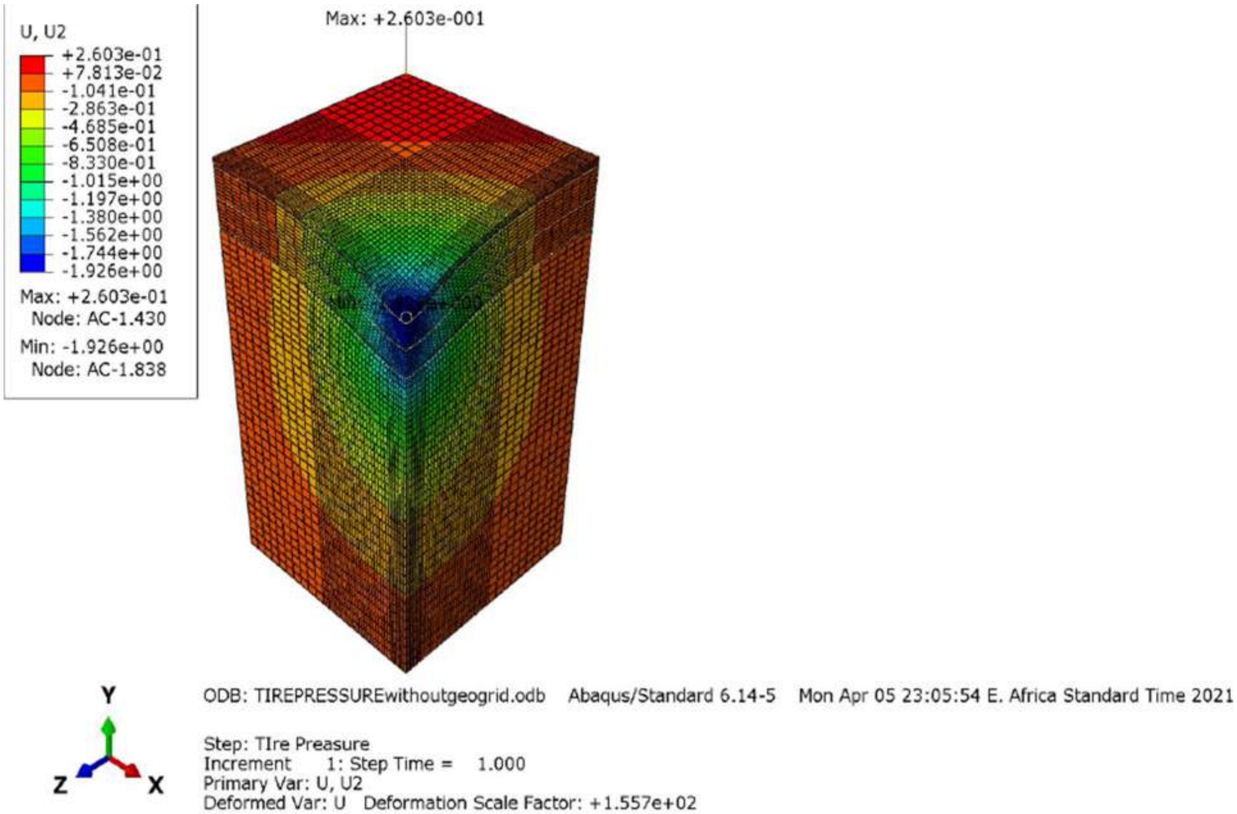
This thesis shall be limited only on the analysis and comparison of the followings parameters of reinforced and unreinforced pavement structure which most of the time cause failure of pavement structure:

- Vertical deformation (U2),
- Lateral deformation (U3) and
- Subgrade stress-strain distribution results

4.1 Vertical Deformation of the Pavement Structure

a. Vertical Deformation (U2, mm); without geogrid reinforcement

Figure 4-1 below shows the distribution of vertical deflection along the pavement structure cross-section for the case of an unreinforced pavement structure system having weak subgrade layer.



```
ODB: TIREPRESSUREwithoutgeogrid.odb    Abaqus/Standard 6.14-5    Mon Apr 05 23:05:54 E. Africa Standard Time 2021

Step: TIre Preasure
Increment      1: Step Time =    1.000
Primary Var: U, U2
Deformed Var: U   Deformation Scale Factor: +1.557e+02
```

Figure 4-1: Distribution of vertical deflection along unreinforced pavement structure cross section, ABAQUS/CAE

b. Vertical Deformation (U2, mm); with Geogrid reinforcement

Figure 4-2 below shows the distribution of vertical deflection along the pavement structure cross-section for the case of reinforced pavement structure system having weak subgrade layer.

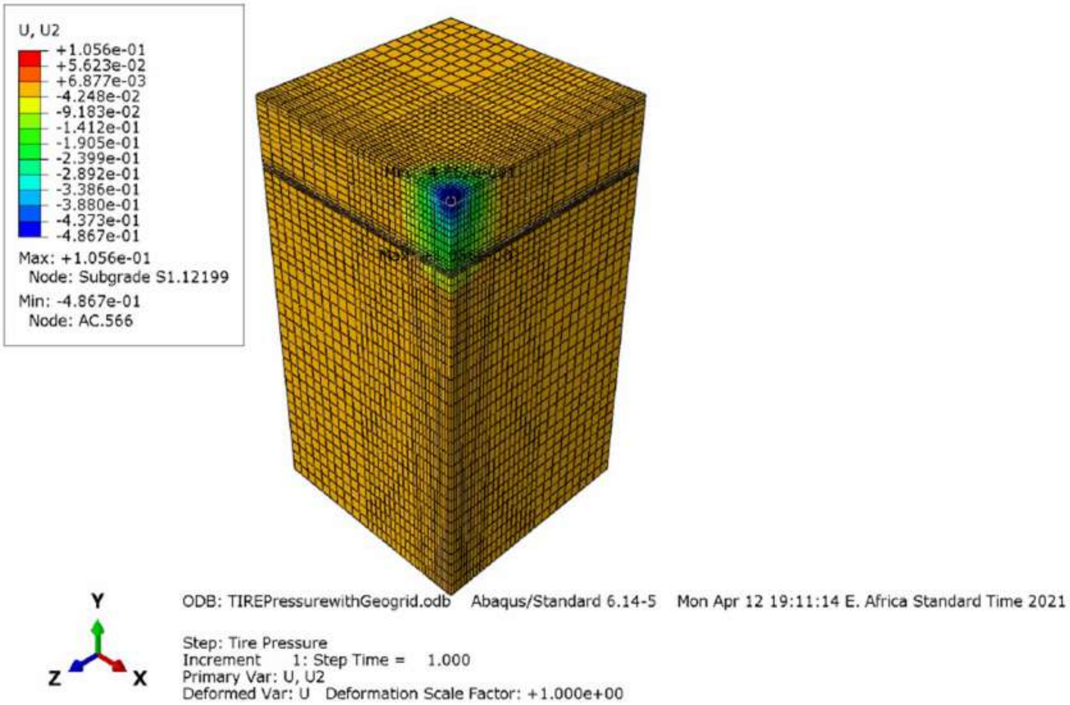


Figure 4-2: Distribution of vertical deflection along reinforced pavement structure cross section, ABAQUS/CAE

In here the analysis results are compared and verified based on reinforced and unreinforced pavement subgrade. As it has been shown above, there are different kinds of deformation of the assembled pavement structure due to the inclusion of reinforcement. By comparing the deformation, the minimum deformation is occurred on pavement structure, when the pavement subgrade is reinforced with Geogrid as shown in the table 4-1below.

Table 4-1: Vertical settlement comparison of unreinforced and reinforced pavement structure

Maximum Vertical deformation, U2(mm)	Unreinforced Pavement Structure	When subgrade of the Pavement structure reinforced with Geogrid	% Reduction
	1.926	**0.4867**	**74.73**

$$\text{Vertical settlment (\%)}: \text{Rs} = \frac{(\text{S (unreinforced)} - \text{S(reinforced)}}{\text{S(unreinforced)}} * 100 \text{ ------------- } (4\text{-}1)$$

$$\text{Rs} = \frac{(1.926 - 0.4867)}{1.926} * 100 = 74.73\%$$

In this regard, reinforcing with geogrid minimize the rate of settlement. On this thesis work, when geogrid is reinforced on the top of the weak subgrade soil; the rate of pavement structure vertical settlement is stabilized by 74.73%.

4.2 Pavement Structure Horizontal Displacement (U3, mm)

The pavement structure lateral deformation means the dispersion of the pavement structure materials to both sides of the pavement structure. Due to the hard interaction between the geogrid and pavement structure material the movement of the aggregates is minimized while load applied on it. The simulation result of ABAQUS is shown below with its numerical results.

a) Lateral Deformation (Horizontal Displacement) (U3, mm); without reinforcement

Figure 4-3 below shows the distribution of lateral deflection (horizontal displacement) along the pavement structure cross-section for the case of an unreinforced pavement structure system having weak subgrade layer.

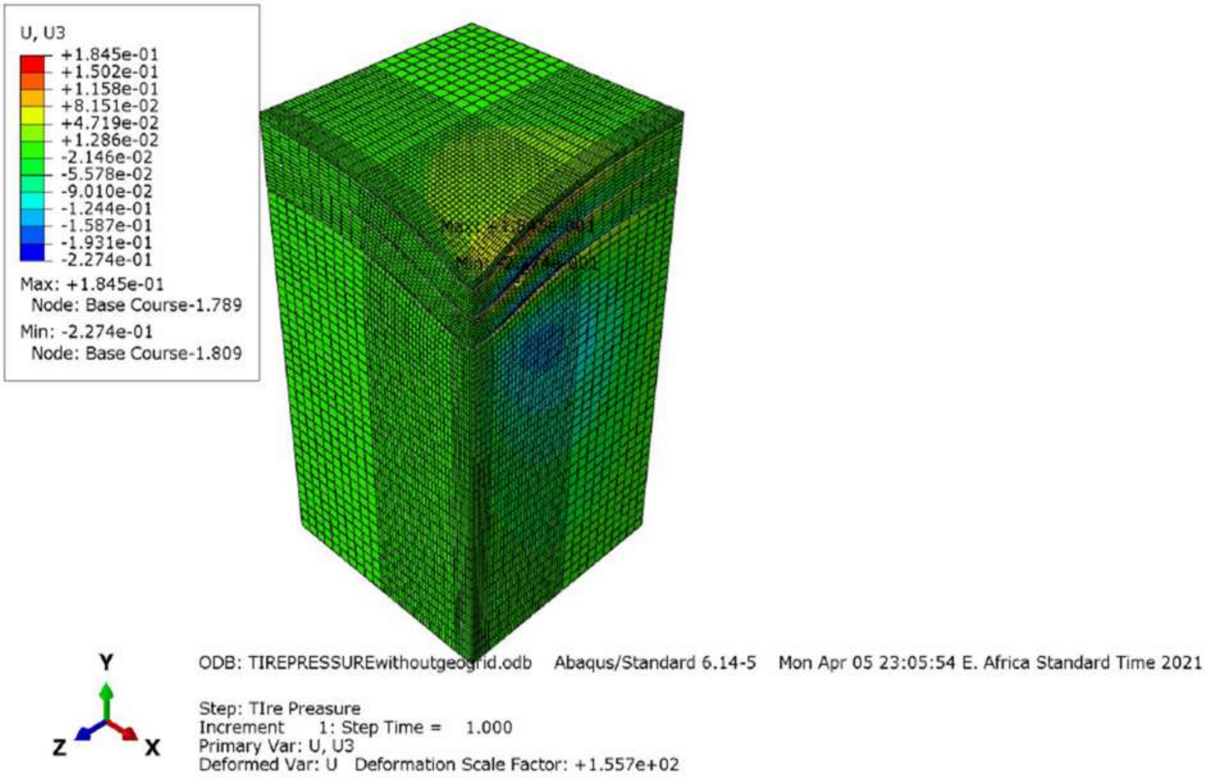


Figure 4-3: Distribution of lateral deflection along the pavement structure cross-section for unreinforced pavement structure, ABAQUS/CAE

b) Lateral Deformation (Horizontal Displacement) (U3, mm); with Geogrid Reinforcement

Figure 4-4 shows the distribution of lateral deflection (horizontal displacement) along the pavement structure cross-section for the case of reinforced pavement structure system having weak subgrade layer.

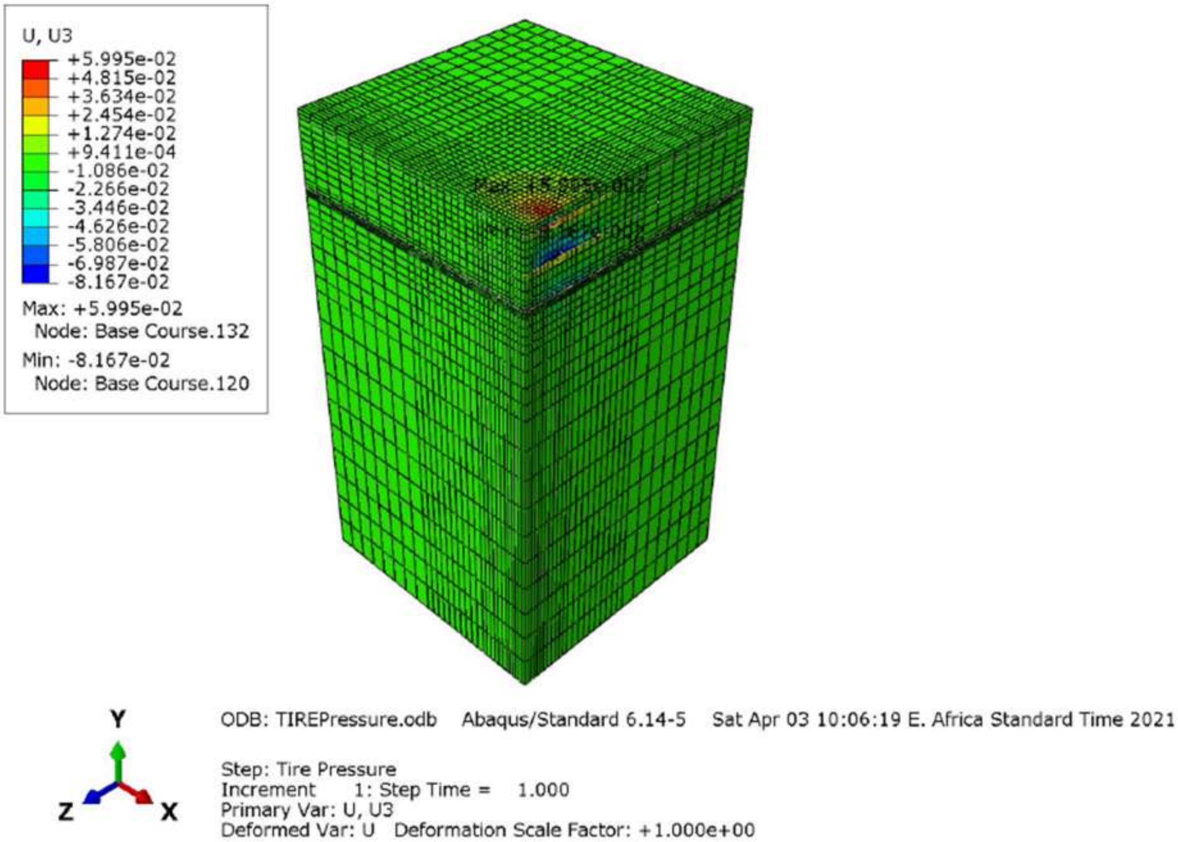


Figure 4-4: Distribution of lateral deflection along the pavement structure cross-section for reinforced pavement structure, ABAQUS/CAE

Table 4-2: Lateral deformation/displacement comparison of unreinforced and reinforced pavement structure

Maximum lateral displacement, U2(mm)	Unreinforced Pavement Structure	When subgrade of the Pavement structure reinforced with Geogrid	% stabilized laterally
	0.2274	**0.08167**	**64.08**

$$\text{Stabilized due to geogrid (Sg)} = \frac{(0.2274 - 0.08167)}{0.2274} * 100 = 64.08\%$$

In this regard, reinforcing with geogrid restrained the lateral deformation (displacement) of the pavement structure. According to this thesis work, when geogrid is reinforced on the top of the weak subgrade soil; it increases stability of lateral deformation of the pavement structure by 64.08%.

4.3 Results of parameters on Subgrade for unreinforced and reinforced pavement structure

4.3.1 Subgrade Vertical Deformation (U2, mm)

i. Subgrade Deformation (U2, mm) results of unreinforced pavement structure

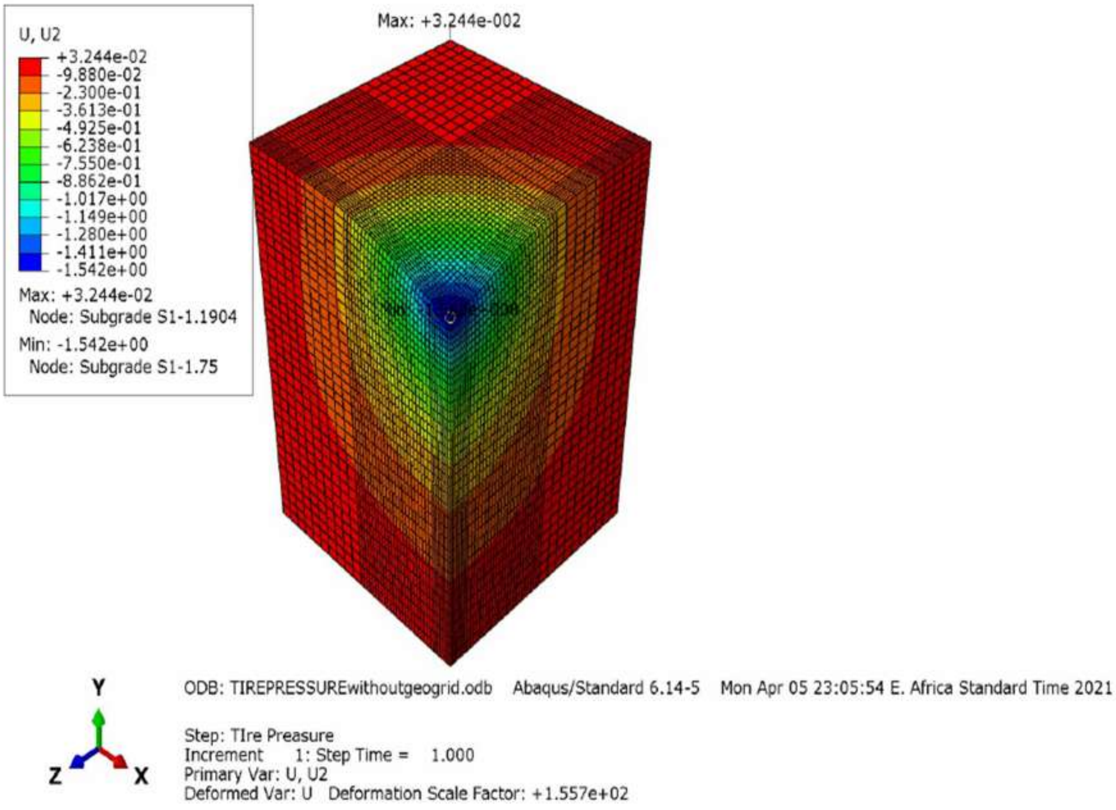


Figure 4-5: Subgrade deformation for unreinforced pavement structure, ABAQUS/CAE

ii. Subgrade deformation (U2, mm) results of geogrid reinforced pavement structure

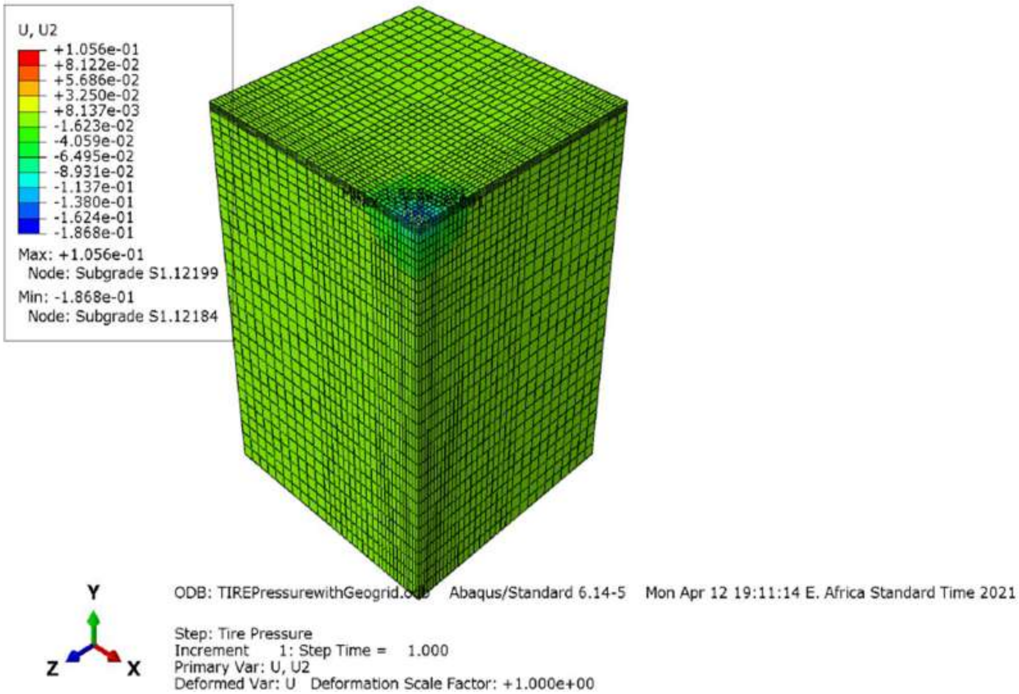


Figure 4-6 Subgrade deformation for reinforced pavement structure, ABAQUS/CAE

Table 4-3: Vertical subgrade deformation comparison of unreinforced and reinforced pavement structure

Maximum vertical subgrade deformation, U2(mm)	Unreinforced Pavement Structure	When subgrade of the Pavement structure reinforced with Geogrid	% Reduction
	1.542	**0.1868**	**87.89**

$$Rs = \frac{(1.542-0.18\ 6)8}{1.542} * 100 = 87.89\ \%$$

In this regard; reinforcing with geogrid results in significant reduction of permanent deformation of the layer. On this thesis work, due to geogrid reinforced on the top of the weak subgrade soil; the rate of vertical settlement of the subgrade can be stabilized/minimized by 87.89%.

4.3.2 Stress-Strain Distribution Results of Subgrade

The preliminary point of stress analysis is a geometrical explanation of the structure, the properties of the materials used for its parts, how the parts are joined, and the maximum or typical forces that are expected to be applied to the structure. The output data is typically a quantitative description of how the applied forces spread throughout the structure, resulting in stresses and strains of the entire structure and each component of that structure. The parameters of reinforced and unreinforced subgrade are compared/evaluated through Path 1-2 as shown the figure bellow:-

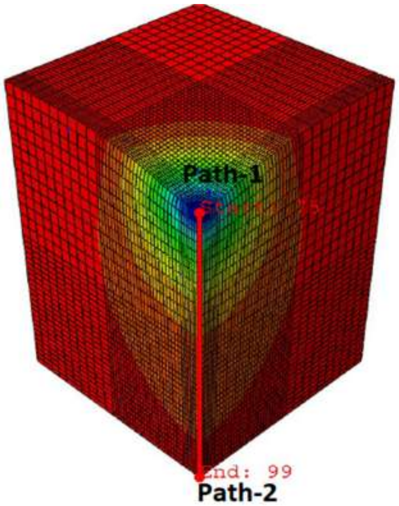


Figure 4-7 Path1-2 on subgrade for parameters comparison of unreinforced and reinforced pavement structure, ABAQUS/CAE

a) Stress Distribution of Unreinforced and Reinforced Subgrade

Table 4-4 : Stress distribution of reinforced and unreinforced subgrade of flexible pavement structure through path 1-2

Depth (mm)	Unreinforced Subgrade Stress (x10^{-3}MPa)	Reinforced Subgrade Stress (x10^{-3} MPa)
0	82.7558	25.3037
255	58.3932	6.6635
510	36.1413	1.2175
765	23.7462	0.2452
1020	16.5044	0.0510
1275	12.1811	0.0118
1530	9.6553	0.0028
1785	8.0811	0.0005
2040	7.1185	0.0002
2295	6.4925	0.0001

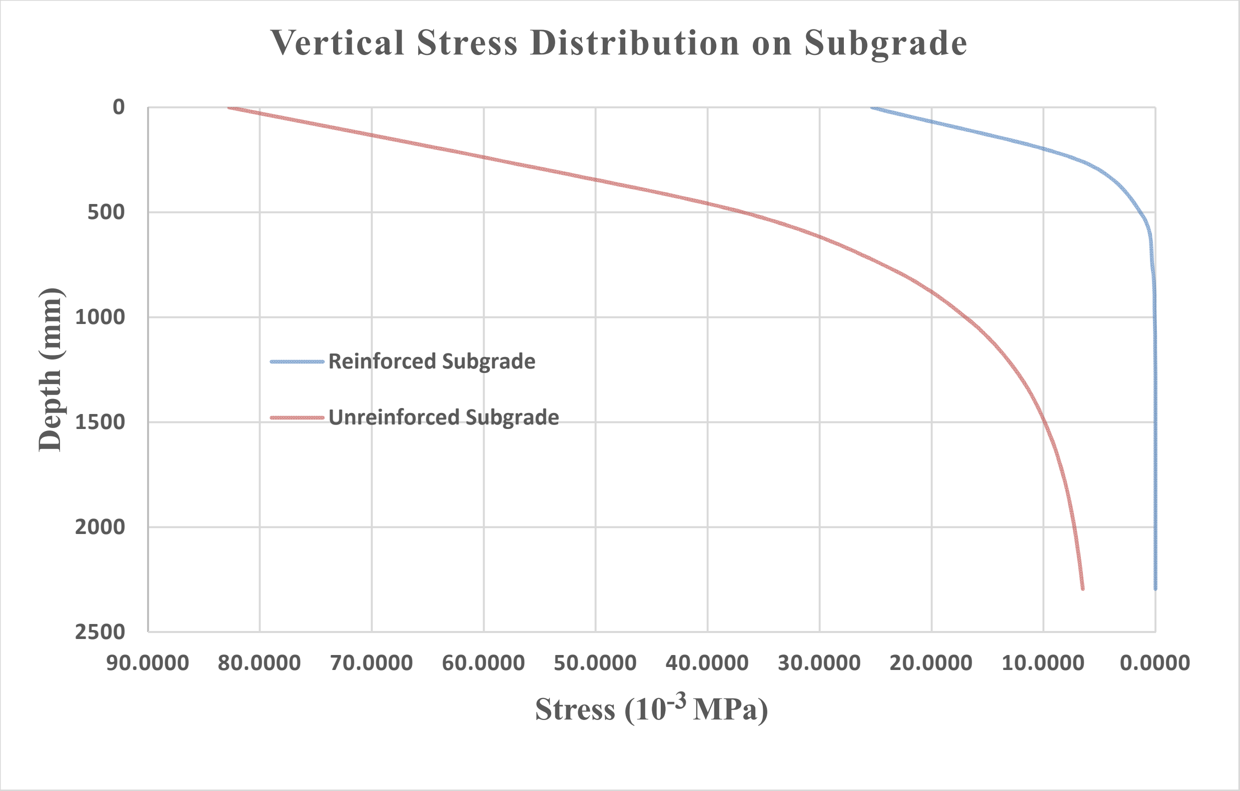


Figure 4-8 Stress distribution of reinforced and unreinforced subgrade of flexible pavement structure through path 1-2

b) Strain Comparison Results Between Unreinforced and Reinforced

The lateral strains profiles at different distances from the center of the wheel load predicted from the finite element analysis within the subgrade layer for unreinforced and reinforced. In this study, the geogrid layer was placed at the top of subgrade layer. It can be seen that the geogrid layer significantly constrained the lateral strains within the subgrade layer as shown below in figure 4.9 and figure 4.10. And, the percentage improvement on lateral strains will depend on the geogrid tensile modulus as stated on different previous studies.

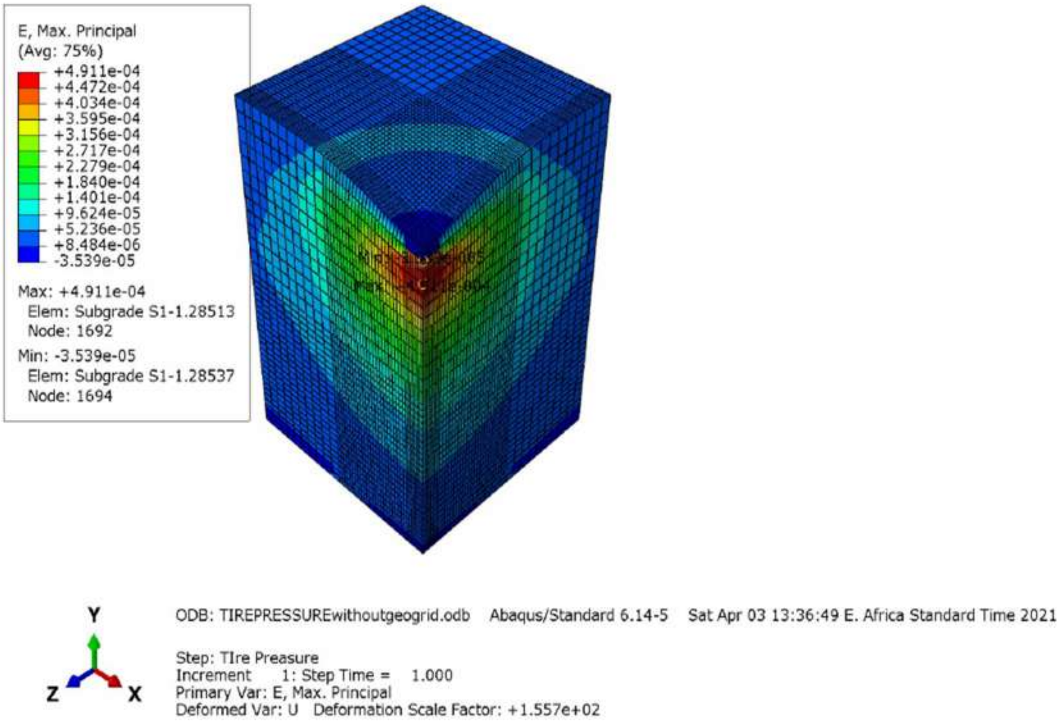


Figure 4-9: Lateral strain of unreinforced subgrade, ABAQUS/CAE

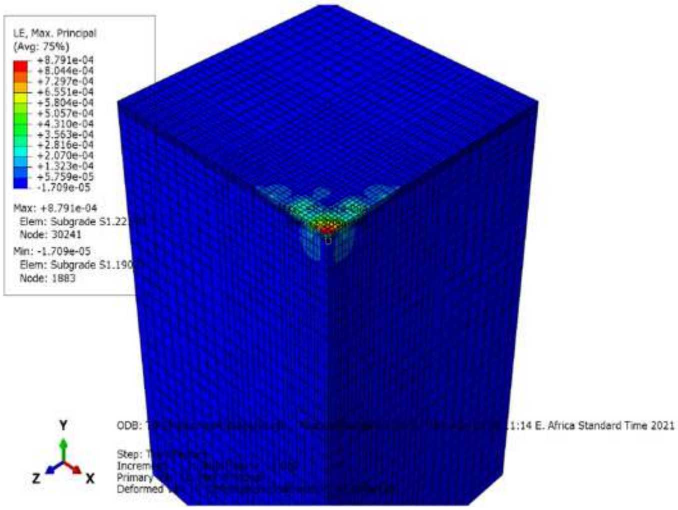


Figure 4-10: Lateral strain of geogrid reinforced subgrade, ABAQUS/CAE

4.3.3 Stress versus Deformation Results

Stress versus displacement/deformation of reinforced and unreinforced subgrade of flexible pavement structure is as follows in Table 4-5 below:-

Table 4-5: stress versus deformation of reinforced and unreinforced subgrade of flexible pavement structure at path 1-2

Stress (x10^{-3}MPa)	Unreinforced deformation (mm)	Reinforced deformation (mm)
82.76	1.54241	0.153341
58.39	1.14576	0.022071
36.14	0.79148	0.003621
23.75	0.54887	0.000999
16.50	0.38394	0.000038
12.18	0.26861	0.000046
9.66	0.18497	0.000044
8.08	0.12145	0.000034
7.12	0.07128	0.000005
6.49	0.03092	0.000007

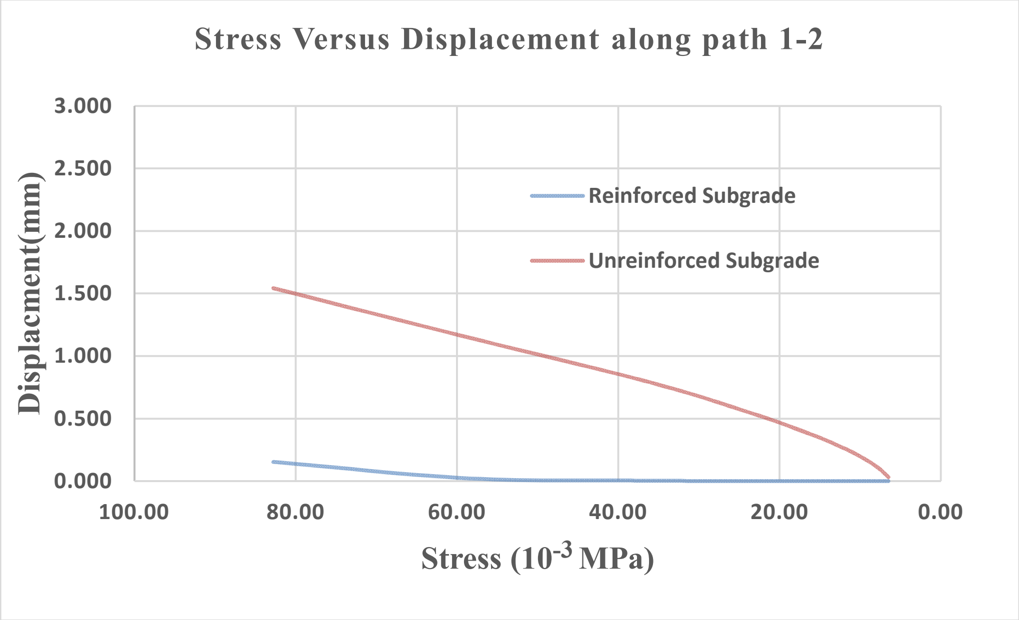


Figure 4-11 Stress versus deformation of unreinforced and reinforced subgrade results through path 1-2 of fig.4-7

Comparing the two results above in Table 4-5 and Figure 4-11; at the same stress the deformation of unreinforced subgrade is higher than the reinforced one. This means the reinforced subgrade soil is stiffer to deform than unreinforced subgrade soil. This result shows that it is possible to improve the poor subgrade soil by using geogrid reinforcement method easily.

CHAPTER FIVE

5 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

The purpose of this thesis work was to evaluate the performance of pavement structure reinforcing the weak subgrade with geogrid by numerical analysis. Accordingly, a serious of Three dimensional finite element analysis using ABAQUS software were performed on flexible pavement structure geometry with and without geogrid in order to determine some of the selected important parameterise i.e. vertical and lateral deformation/displacement, stress distribution and lateral strain of the pavement geometry. Subsequently, evaluation of the behaviour and performance improvement of the pavement structure due to geogrid reinforcement of the weak subgrade has been conducted comparing the analytical results of the reinforced and unreinforced pavement structure models. As a result, the following conclusions have drawn:

- Comparison of the analysis results of the vertical and horizontal deformation/displacement of the reinforced and unreinforced flexible pavement model were conducted and noted that provision of geogrid reinforcement on top of weak subgrade of flexible pavement structure has stabilized the vertical and horizontal deformation/displacement of the pavement structure up to **74.73%** and **64.08%** respectively. Hence, using geogrid as reinforcement of the weak subgrade of the pavement structure can be effectively reduced/stabilized vertical and lateral deformation/displacement of the pavement structure as stated in the previous different studies.

- Comparison of the analysis results of vertical deformation/displacement of the subgrade with and without Geogrid Reinforcement were conducted and noted that provision of geogrid reinforcement on top of weak subgrade has stabilized the vertical deformation/displacement of the pavement subgrade up to **87.89%.** Hence, reinforcement of weak subgrade with geogrid can be effectively reduced/minimized the subgrade settlement and improve structural performance of the pavement structure.

- Comparison of the Stress distribution on subgrade of the pavement structure with and without geogrid reinforcement were carried out and noted that the stress distribution throughout depth of the subgrade is reduced/minimized due to reinforcement of the subgrade with geogrid (as shown on figure 4.11) i.e. the stress on top of the subgrade has reduced from **$82.75 * 10^{-3}$** MPa to **$25.30 * 10^{-3}$** MPa due to geogrid reinforcement. Hence, reinforced pavement structure can carry a stress more than unreinforced pavement structure.

- The geogrid reinforcement of the weak subgrade significantly constrained the lateral strains within the subgrade layer of the pavement structure as shown on the figure 4.9 & 4.10 above. But, the percentage improvement on lateral strains will depend on the geogrid tensile modulus as stated on different previous studies.

- Generally, provision of geogrid reinforcement has improved the allover performance of the pavement structure and hence, in order to solve the problem of poor subgrade condition, using geogrid reinforcement is one of the key stabilization solutions.

5.2 Recommendation

- The result of this study can be a starting point to develop a methodology for the implementation of reinforcement method to stabilize the poor subgrade soil rather than the usual replacement method widely used in Ethiopia.

- In this study, two pavement structures with and without reinforcement subgrade under static loading were analyzed due to time and budget constraint. Additional pavements with varying thickness, material properties and with cyclic loading are suggested to be conducted.

- On this study the performance of the pavement structure is evaluated only by providing the geogrid on the top of subgrade. Hence, extending the finite element analysis and evaluating the performance and economy of the pavement structure by providing geogrid reinforcement at different depth of pavement structure as well as providing more than one layer of reinforcement at different position also suggested to be conducted.

- Economic analysis and comparison of the subgrade reinforcement method with other key subgrade soil stabilization methods is required before implementation of the same.
- The principle of reinforcement of the geogrid lies on the interlock of the geogrid and the pavement structure material. Hence, before selection of the aperture size of the geogrid, the grain size distribution investigation of the specific site is highly recommended.

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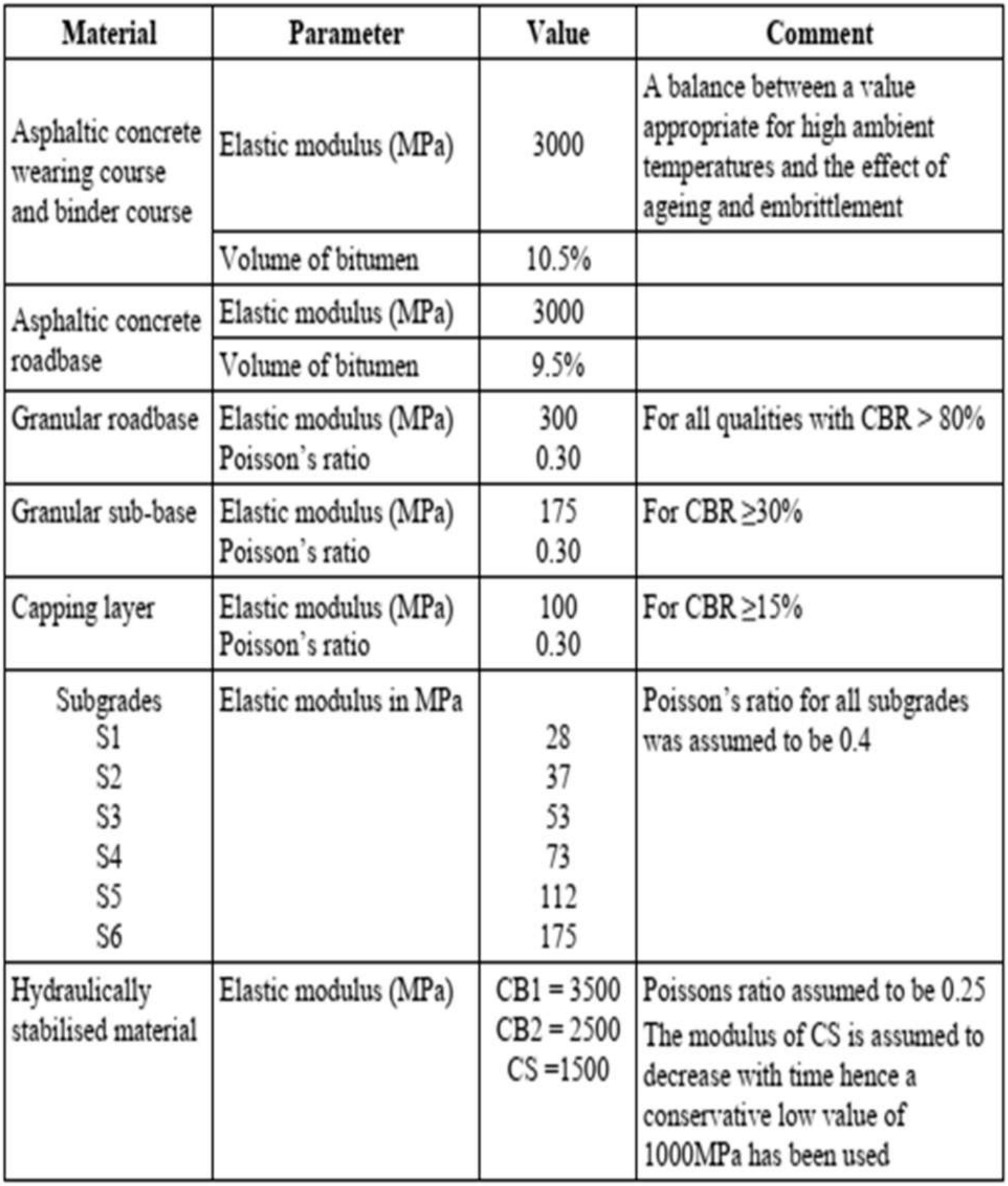
7 APPENDIX-A

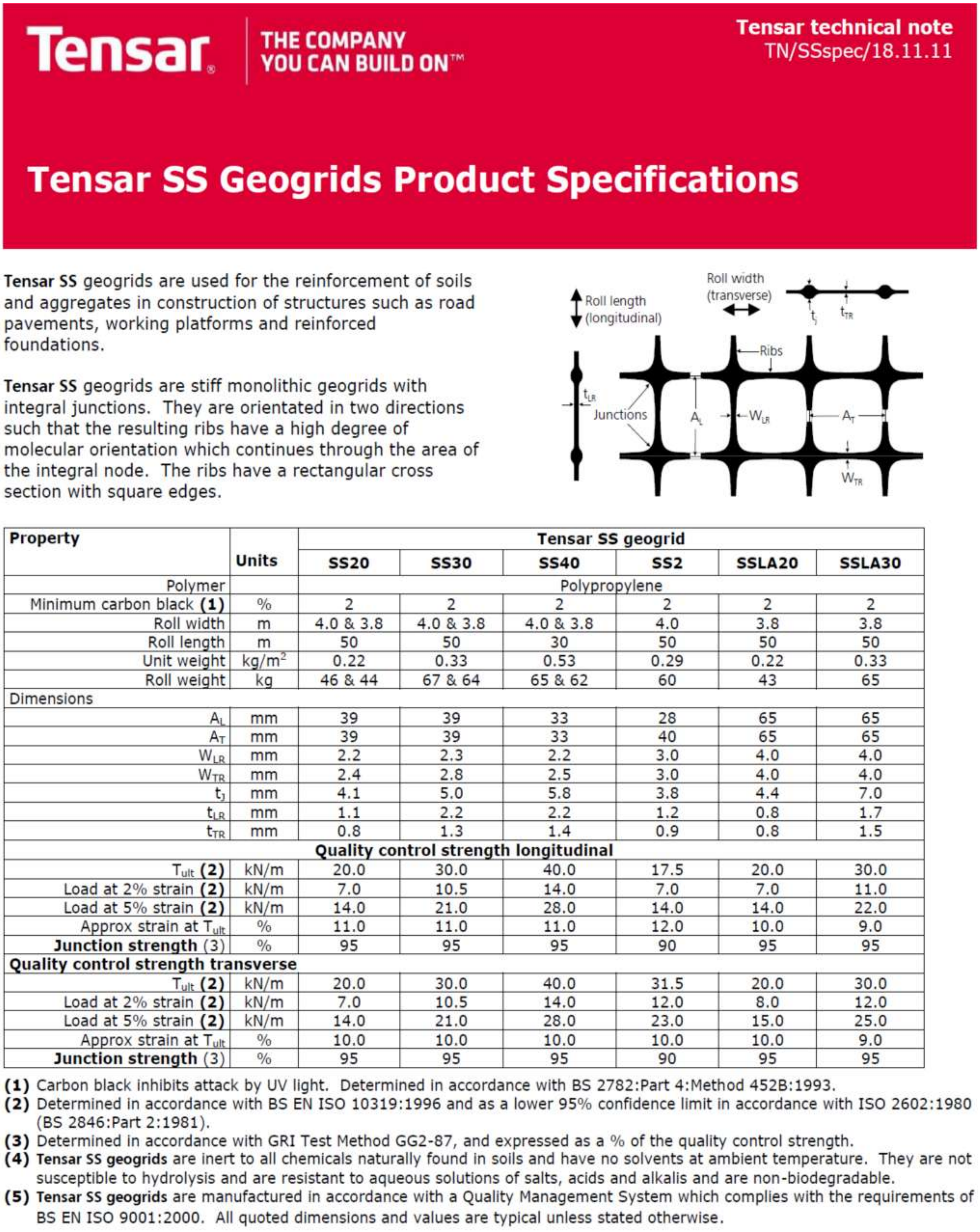
Table 7-1: The material characteristics for mechanistic analysis of flexible pavement structure (ERA, 2013)



Material	Parameter	Value	Comment
Asphaltic concrete wearing course and binder course	Elastic modulus (MPa)	3000	A balance between a value appropriate for high ambient temperatures and the effect of ageing and embrittlement
	Volume of bitumen	10.5%	
Asphaltic concrete roadbase	Elastic modulus (MPa)	3000	
	Volume of bitumen	9.5%	
Granular roadbase	Elastic modulus (MPa) Poisson's ratio	300 0.30	For all qualities with CBR > 80%
Granular sub-base	Elastic modulus (MPa) Poisson's ratio	175 0.30	For CBR ≥30%
Capping layer	Elastic modulus (MPa) Poisson's ratio	100 0.30	For CBR ≥15%
Subgrades	Elastic modulus in MPa		Poisson's ratio for all subgrades was assumed to be 0.4
S1		28	
S2		37	
S3		53	
S4		73	
S5		112	
S6		175	
Hydraulically stabilised material	Elastic modulus (MPa)	CB1 = 3500 CB2 = 2500 CS =1500	Poissons ratio assumed to be 0.25 The modulus of CS is assumed to decrease with time hence a conservative low value of 1000MPa has been used

Table 7-2 Atterberg limits, CBR, Swell and classification subgrade soil (DANA, et al., 2014)

Findings by the Design Consultant											
Station	LL	PI	CBR	Swell	classification	Station	LL	PI	CBR	Swell	classification
2+000	58	21	7	1.24	A-7-5(16)	27+700	46	18	4	2.7	A-7-6(4)
2+500	49	21	3	1.86	A-7-6(13)	28+700	86	44	1	6.94	A-7-5(20)
3+700	50	21	2	4.58	A-7-6(9)	30+400	70	31	2	3.31	A-7-5(20)
4+700	77	40	2	4.12	A-7-5(20)	32+600	79	37	2	4.28	A-7-5(20)
5+700	43	5	5	0.85	A-5(9)	33+700	42	17	4	3.1	A-7-6(2)
7+400	103	62	1	7.12	A-7-5(20)	34+600	49	21	1	4.64	A-7-6(9)
8+400	66	49	1	6.27	A-7-5(13)	36+200	27	18	20	0.27	A-2-6(0)
9+000	54	21	2	4.33	A-7-5(15)	37+000	66	34	2	4.72	A-7-5(12)
9+600	32	9	6	0.99	A-2-4(0)	38+800	36	8	12	0.81	A-4(0)
10+100	64	30	2	4.54	A-7-5(13)	39+800	52	20	2	1.93	A-7-5(9)
11+100	38	12	2	6.09	A-6(9)	41+600	50	20	5	2.52	A-7-5(6)
12+100	49	23	1	4.82	A-7-6(15)	42+600	50	22	3	1.93	A-7-6(12)
14+200	35	11	30	0.16	A-2-6(0)	44+900	46	22	2	4.92	A-2-7(2)
15+200	49	19	6	1.41	A-2-7(1)	46+200	77	41	2	4.07	A-7-5(19)
16+200	66	25	11	1.36	A-2-7(2)	47+200	69	35	2	5.83	A-2-7(4)
17+000	53	21	16	0.16	A-7-5(4)	49+300	77	43	1	6.94	A-7-5(20)
18+000	56	19	4	3.81	A-7-5(15)	49+700	40	19	4	1.55	A-2-7(1)
18+900	58	19	11	1.64	A-7-5(13)	50+200	56	26	4	2.89	A-7-5(15)
20+500	56	23	1	3.79	A-7-5(16)	52+300	55	25	2	4.11	A-7-5(15)
21+500	60	27	5	2.31	A-7-5(19)	53+800	85	51	1	6.22	A-7-5(20)
22+100	43	15	12	0.54	A-2-7(0)	54+500	42	15	15	0.45	A-2-7(0)
23+100	36	10	21	0.09	A-2-4(0)	55+000	60	25	2	3.72	A-7-5(8)
24+700	54	25	2	2.7	A-7-6(17)	56+000	56	27	2	5.77	A-7-6(18)
25+700	55	27	2	2.4	A-7-6(18)	58+100	68	38	1	7.67	A-7-5(20)



Tensar

THE COMPANY
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Tensar technical note
TN/SSspec/18.11.11

Tensar SS Geogrids Product Specifications

Tensar SS geogrids are used for the reinforcement of soils and aggregates in construction of structures such as road pavements, working platforms and reinforced foundations.

Tensar SS geogrids are stiff monolithic geogrids with integral junctions. They are orientated in two directions such that the resulting ribs have a high degree of molecular orientation which continues through the area of the integral node. The ribs have a rectangular cross section with square edges.

Property	**Units**	**Tensar SS geogrid**					
		SS20	**SS30**	**SS40**	**SS2**	**SSLA20**	**SSLA30**
Polymer		Polypropylene					
Minimum carbon black **(1)**	%	2	2	2	2	2	2
Roll width	m	4.0 & 3.8	4.0 & 3.8	4.0 & 3.8	4.0	3.8	3.8
Roll length	m	50	50	30	50	50	50
Unit weight	kg/m^2	0.22	0.33	0.53	0.29	0.22	0.33
Roll weight	kg	46 & 44	67 & 64	65 & 62	60	43	65
Dimensions							
A_L	mm	39	39	33	28	65	65
A_T	mm	39	39	33	40	65	65
W_{LR}	mm	2.2	2.3	2.2	3.0	4.0	4.0
W_{TR}	mm	2.4	2.8	2.5	3.0	4.0	4.0
t_J	mm	4.1	5.0	5.8	3.8	4.4	7.0
t_{LR}	mm	1.1	2.2	2.2	1.2	0.8	1.7
t_{TR}	mm	0.8	1.3	1.4	0.9	0.8	1.5
Quality control strength longitudinal							
T_{ult} **(2)**	kN/m	20.0	30.0	40.0	17.5	20.0	30.0
Load at 2% strain **(2)**	kN/m	7.0	10.5	14.0	7.0	7.0	11.0
Load at 5% strain **(2)**	kN/m	14.0	21.0	28.0	14.0	14.0	22.0
Approx strain at T_{ult}	%	11.0	11.0	11.0	12.0	10.0	9.0
Junction strength (3)	%	95	95	95	90	95	95
Quality control strength transverse							
T_{ult} **(2)**	kN/m	20.0	30.0	40.0	31.5	20.0	30.0
Load at 2% strain **(2)**	kN/m	7.0	10.5	14.0	12.0	8.0	12.0
Load at 5% strain **(2)**	kN/m	14.0	21.0	28.0	23.0	15.0	25.0
Approx strain at T_{ult}	%	10.0	10.0	10.0	10.0	10.0	9.0
Junction strength (3)	%	95	95	95	90	95	95

(1) Carbon black inhibits attack by UV light. Determined in accordance with BS 2782:Part 4:Method 452B:1993.

(2) Determined in accordance with BS EN ISO 10319:1996 and as a lower 95% confidence limit in accordance with ISO 2602:1980 (BS 2846:Part 2:1981).

(3) Determined in accordance with GRI Test Method GG2-87, and expressed as a % of the quality control strength.

(4) **Tensar SS geogrids** are inert to all chemicals naturally found in soils and have no solvents at ambient temperature. They are not susceptible to hydrolysis and are resistant to aqueous solutions of salts, acids and alkalis and are non-biodegradable.

(5) **Tensar SS geogrids** are manufactured in accordance with a Quality Management System which complies with the requirements of BS EN ISO 9001:2000. All quoted dimensions and values are typical unless stated otherwise.

Figure 7-1: Tensar SS geogrid products specifications

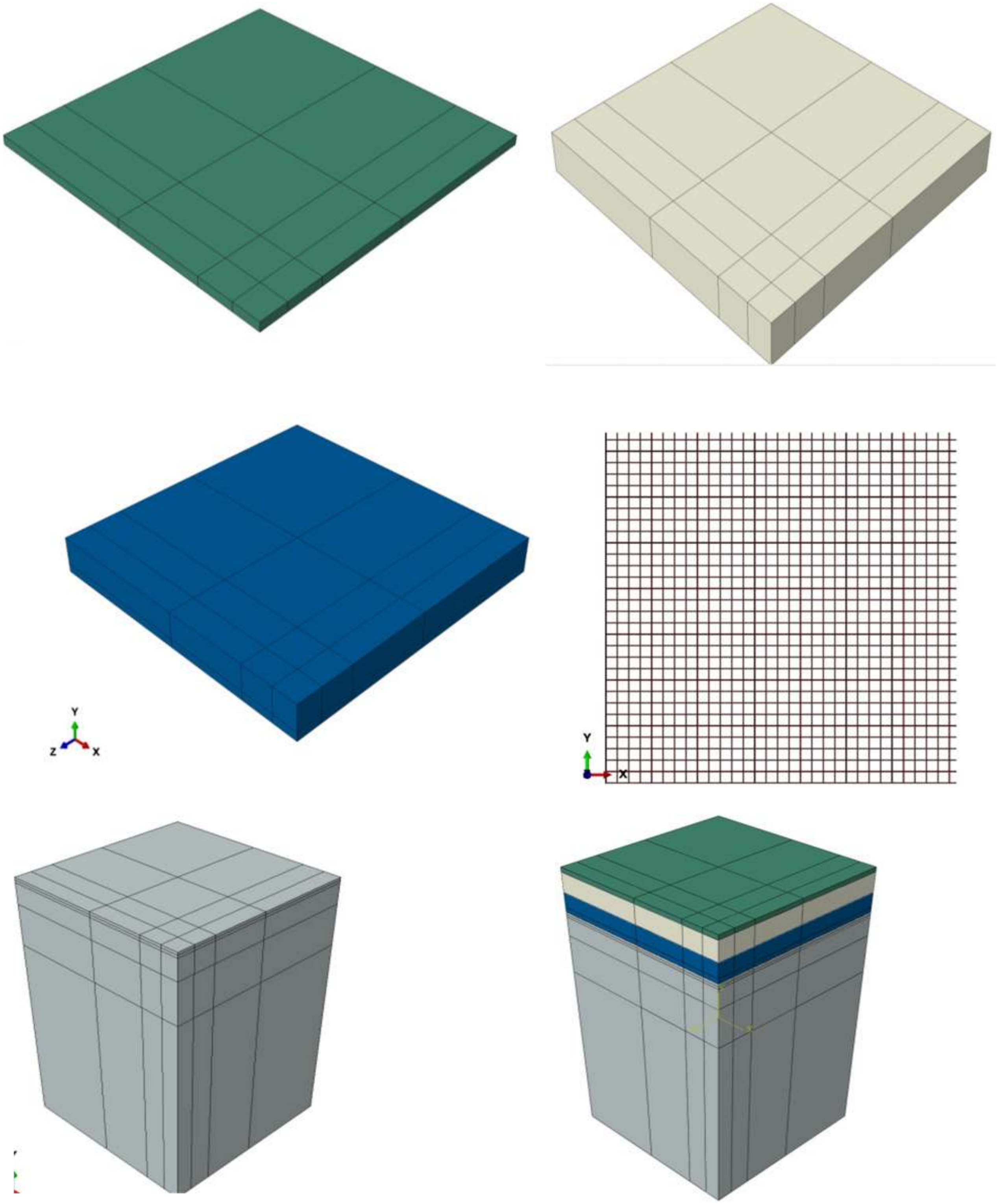

Figure 7-2: Parts and assembly of the pavement structure model, ABAQUS/CAE