HASAN KALYONCU UNIVERSITY
GRADUATE SCHOOL OF
NATURAL AND APPLIED SCIENCES

A PARAMETRIC STUDY ON THE BEHAVIOUR OF MECHANICALLY STABILIZED EARTH (MSE) WALL USING FINITE ELEMENT METHOD

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IN
CIVIL ENGINEERING

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A Parametric Study onthe Behaviour of Mechanically Stabilized Earth (MSE) Wall Using Finite Element Method

Hasan Kalyoncu University
Civil Engineering
Master Thesis

Supervisor
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May 2014

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ABSTRUCT

A Parametric Study on The Behaviour of Mechanically Stabilized Earth (MSE) Wall Using Finite Element Method

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116 page

Mechanically stabilized earth (MSE) walls offer simple construction techniques, pleasing aesthetics, and cost-effective solutions as an alternative to conventional gravity walls.This dissertation presents the analysis of reinforced soil retaining walls were carried out to review the engineering properties of the MSE wall and perform parametric studies various factors that may govern the behaviour of the MSE wall by using finite element program PLAXIS 2D. The geometric parameters such as reinforcement length, vertical spacing between the reinforcement layers, reinforcement stiffness, wall height, face element thickness, position of traffic load, and finally the type of reinforced soil were examined to investigate their effects on the forces developed in the reinforcement and the wall deformation. It is shown that the forces developed are largely independent of reinforcement length, reinforcement stiffness, and face element thikness. Generally, axial force increases with increasing the wall height, vertical spacing between the geogrid layers, and angle of internal friction for backfill soil. While, wall deformation are largely dependent on geogrid length and an acceptable resultes were obtained when using L/H equal to 0.7. Wall deformation also decreasing with increasing geogrid stiffness and therfore geogrid with higher strength is recomended to be used. Beside, the vertical spacing (Sv) between geogrid layers which also affect on the horizontal deformation and the minimum value can be used in the design of MSE walls equal to 0.5 m.

Keywords: Geosynthetic, Horizontal displacement, Finite element method, Reinforcement stiffness, MSE wall.

ÖZET

Sonlu Elemanlar Metodu Kullanılarak Mekanik Açıdan Stabilize Edilmiş Dolgu Duvar Üzerinde Parametrik Çalışma

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Mayıs 2014
116 sayfa

Mekanik olarak stabilize edilmiş dolgu duvarlar, geleneksel olarak kullanılan ağırlık duvarlarına göre basit yapım teknikleri, estetik açıdan görüm ve maliyet açısından daha iyi çözümler sunar. Bu çalışma güçlendirilmiş istinat duvarlarının analizini sunmaktadır. PLAXIS 2D lisanslı sonlu elemanlar programı kullanılarak mekanik olarak stabilize edilmiş dolgu duvarların mühendislik özellikleri ve bu duvarların davranışında etkili olan çeşitli parametreler incelenmiştir. Donatı uzunluğu, donatı tabakaları arasındaki dikey boşluklar, donatının sertliği, duvar yüksekliği,yüzey elemanlarının kalınlığı, trafik yükünün pozisyonu ve güçlendirilmiş zeminin tipi gibi geometrik parametreler analiz edilmiş ve etkiyen yüklerin donatılar ve duvar deformasyonları üzerinde ki etkileri incelenmiştir.Sonuçlar göstermiştir ki; etkiyen yükler genellikle donatı uzunluğundan, donatı sertliğinden ve yüzey elemanlarını kalınlığından bağımsızdır. Genellikle eksenel kuvvetler, duvar yüksekliğinin, geogrid tabakalar arasında ki dikey boşlukların ve dolgu zeminin içsel sürtünme açısının artmasıyla artış göstermiştir. Duvar deformasyonları genellikle geogrid uzunluklarına bağlı olmakta ve L/H oranı 0.7 olduğunda kabul edilebilir sonuçlar elde edilebilmektedir. Duvar deformasyonları ayrıca, geogrid sertliğinin artışına bağlı olarak azalma eğilimi göstermiştir ve bundan dolayı kullanım için yüksek dayanımlı geogridler önerilmiştir. Bunlara ek olarak, geogrid tabakalar arasında ki dikey boşluklar yatay yöndeki deformasyonlarda da etkili olmuştur ve minimum değer olarak bu dikey boşluklar 0.5 m olarak, mekanik olarak stabilize edilmiş dolgu duvarların tasarımında kullanılabileceği önerilmiştir.

Anahtar Kelimeler: Geosentetik, Yatay yerdeğiştirme, Sonlu eleman metodu, Donatı sertliği, Mekanik olarak stabilize edilmiş dolgu duvarlar.

TO MY BELOVED FAMILY AND MY MOTHER

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LIST OF SYMBOLS/ABBREVIATIONS

W	The weight of sliced blocks.
b	The length of sliding plane in sliced block.
α	Inclination of sliding surface with horizontal.
EA	Axial Stiffness of Soil Reinforcement.
EI	Bending Stifness of Soil Reinforcement.
Sv	Vertical Spacing between Reinorcement Layeres.
L/H	Reinforcement Length to Hight Ratio.
H	Height of the Wall.
D	Thickness of Facing Elements.
σ_1	Major Principle Stress.
σ_{1R}	Major Principle Stress in Reinforcement Soil.
C'_R	Apparent Cohesion.
$\Delta\ \sigma_{3R}$	Confining Pressure.
φ_{ps}	Plane-Strain Angle of Friction.
ψ	Angle of dilation.
c'	Effective Cohesion.
Φ'	Effective Internal angle of Internal Friction.
b	Width of each slice.

u	Water Pressure at the base of each slice.
γ_{unsat}	Saturated Unit Weight of Soil.
γ_{sat}	Unsaturated Unit Weight of Soil.
kx	Horizontal Permeability.
ky	Vertical Permeability.
E_{ref}	Young's Modulus.
ν	Poisson's Ratio.
ψ	Dilatancy Angle.
R_{inter}.	Interface Strength Reduction Factor.
L_P	Extent of active zone.
B	Width of reinforced soil mass.
q	Traffic Surcharge.
ϕ	Angle of Internal Friction of Sliding Surface.
c_{ref}	Cohesion of Sliding Surface.
F	Factor of safety.

CHAPTER I

INTRODUCTION

1.1 General

Mechanically Stabilized Earth (MSE) walls are earth retaining structures that are constructed by placing alternating layers of reinforcement and compacted soil behind a facing element to form a composite material which acts integrally to restrain lateral forces (Alzamora and Anderson, 2009). MSE walls are relatively flexible gravity structures that can accommodate differential settlement.

MSE wall typically constructed using three structural components which work together to form the composite structure referred to as MSEW. These three components are: 1) Reinforced fill: the soil that is used behind the facing of the wall. Theoretically all the types of soil can be used as a reinforced fill, a well graded granular soil is the preferred material due to its strength, drainage, durability properties, and easy in construction. 2) Reinforcement elements: in the construction of a MSE wall, reinforcements are placed in layers in the backfill soil, and this reinforced mass resists the earth pressure caused by the retained soil using the relative motion between reinforcement and soil (Kibria et al., 2014). Reinforcing elements are made out of steel or geosynthetic materials. 3) Facing elements: used to prevent the soil from raveling out between the reinforcement layers. Common facings include precast concrete panels, dry cast modular blocks, metal sheets and plates, gabions, welded wire mesh, and shotcrete (Elias et al., 2001).
The earth pressure coming from the retained fill is resisted by the reinforced soil mass by using relative motion between the reinforcement and soil.

Therefore, the performance of MSE wall depends on the interaction between soil and reinforcement (see Figure 1.1).

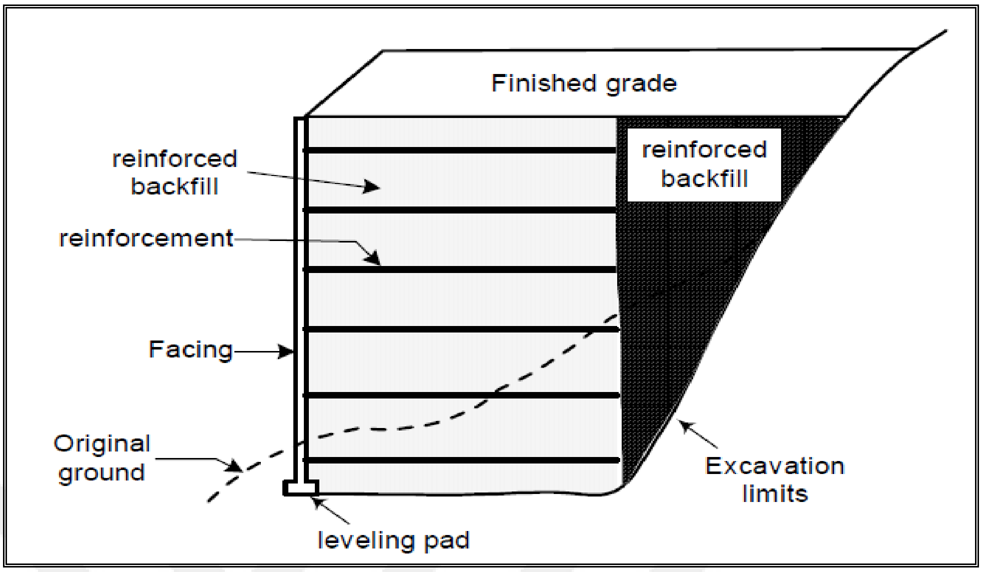


Figure 1.1 Generic cross section of a MSE structure (Liang, 2004)

MSE retaining walls are routinely designed for a 75-year service life; those supporting bridges are typically designed for 100 years (Anderson et al., 2012). Reinforced earth retaining wall have gained substantial acceptance as an ideal alternative solution to conventional masonry, gravity, and cantilever retaining wall structures in most applications due to their 1) simplicity, rapidity and easy of construction, 2) less site preparation, 3) less space requirement for construction operation, 4) technically feasible to heights in excess of 25 m, 5) do not require experienced craftsmen with special skills. In addition to technical and performance advantages, another primary reason for the acceptance of reinforced earth retaining wall it has been economical alternatives for reinforced concrete in most applications, such as wing walls, bridge abutments, and elimination the right-of-way in some cases where an embankment or excavation with stable side slopes cannot be constructed. Figures (1.2 & 1.3) shows MSE wall applications abutments, marine, and urban application.

They are suited to economical construction in steep-sided terrain, in ground with unstabil slope, also it can be used at sites with poor foundation conditions that may be required for foundation improvements. In such cases, using MSE walls resulted in cost savings of greater than 50 percent from the total cost of the project. MSE walls have been used in public and private projects for at least three decades.

There are Some additional successful uses of MSE walls include:

- Temporary structures for highway reconstruction projects.
- Reinforced soil dikes, which have been used for containment structures for water and waste.
- Dams and seawalls, including increasing the height of existing dams.
- Bulk materials storage using sloped walls.

Extensive researches have been conducted to investigate the reinforcement mechanisms. In spite of widespread usage of MSE wall, the long- term behavior of geogride - reinforced MSE walls remain uncertain. In particular, the tendency of polymeric geogride to creep under sustained loading at high temperature poses a potential risk to the performance of MSE walls in warm climate (Reddy et al., 2003).

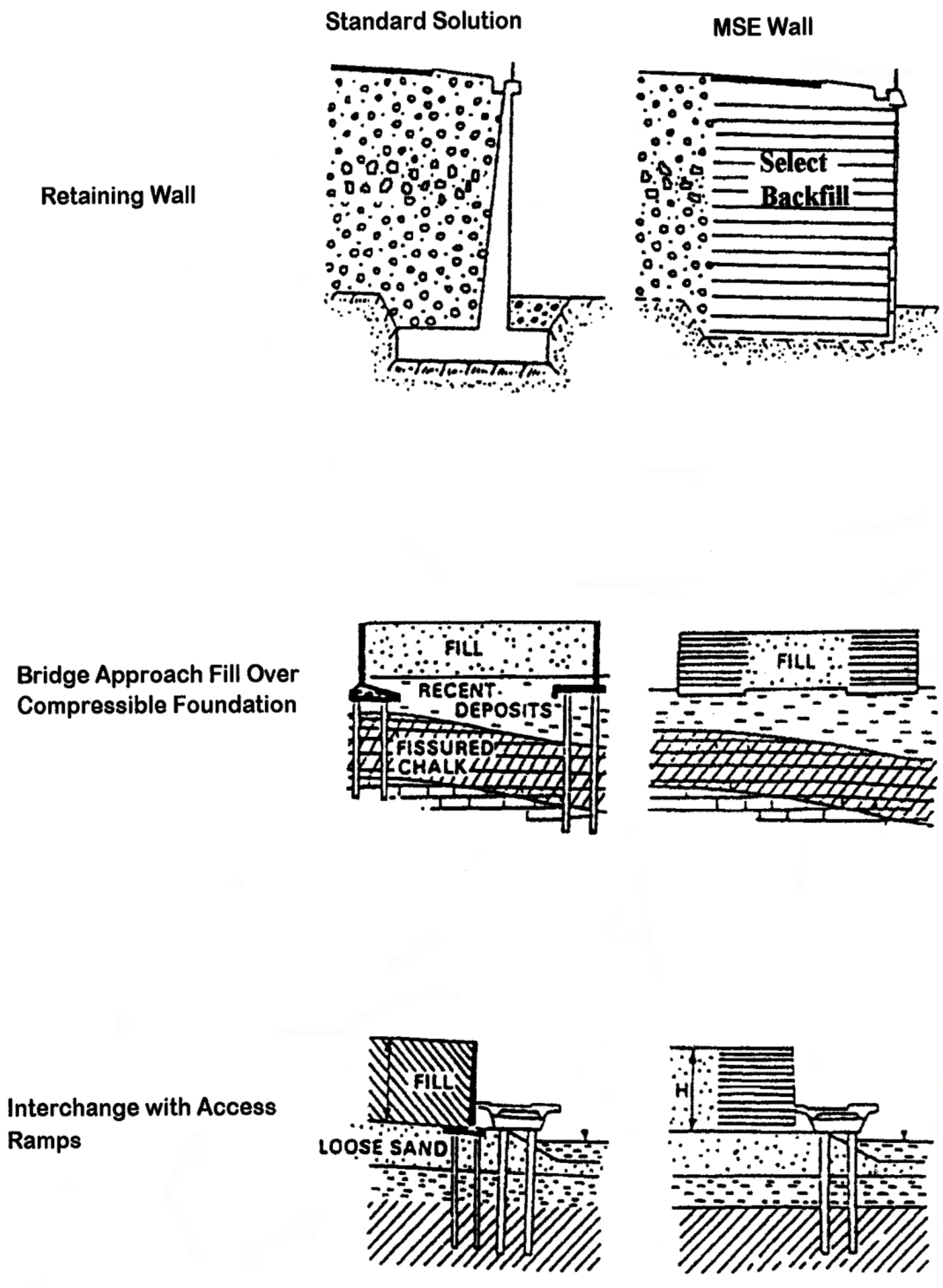


Figure 1.2 MSE walls, urban applications (Elias et al., 2001)

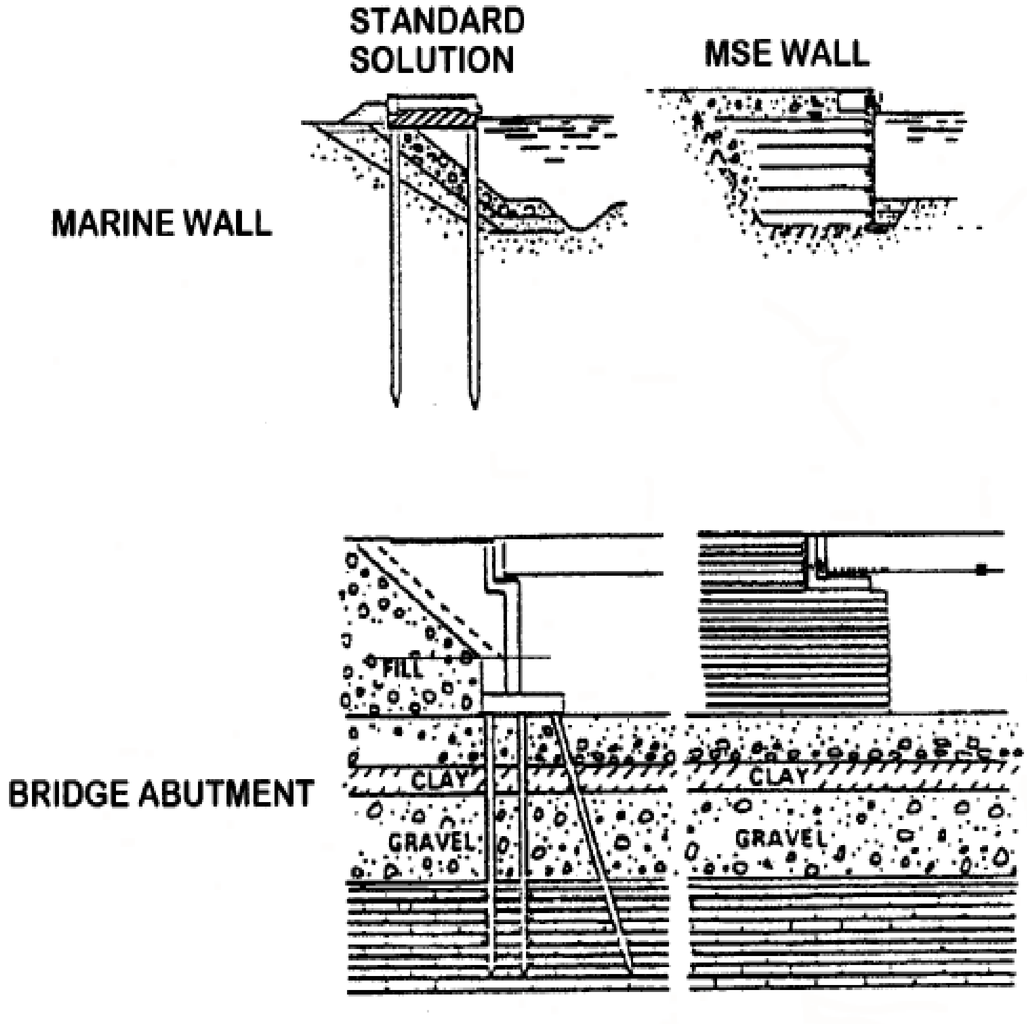


Figure 1.3 MSE wall applications, abutments, and marine (Elias et al., 2001)

Numerical modelling of reinforced earth retaining wall and other structures has been increasingly adopted in researches since their outstanding cost- time effectiveness. This research work employs by using the computer program PLAXIS 2D to develop and analysis a finite element (FE) model in each of the cases considered.

A parametric study was conducted to identify the effects of various structural components of a MSE wall which can positively or negatively influnce in the general performance of the MSE wall. These factores include Soil and reinforcement properties: 1) backfill soil (granular and clayey soil), 2) geogrid strength (EA), 3) vertical spacing between reinorcement layeres (Sv), 4) reinforcement length to height (L/H) ratio, 5) height of the wall (H), 6) thickness of facing elements (D), and 7) the position of traffic surcharge live load.

The soil model used to characterize the site was the elasto-plastic Mohr-Coulomb model. The basic Mohr-Coulomb input parameters for the all layers of soil are fully described in Chapter three of this dissertation. The soil-geogrids wall was modelled as an elasto-plastic material.

1.2 Objectives and Scope of Study

The main objective of the current research is to make further studies on the behaviour of the reinforced earth wall when the mechanical properties and geometry of its composite materials changes to achieve the design under static load.

Parametric studies were performed including various factors that may govern the behaviour of MSE wall, and investigate the effect of this parameters on the horizontal extreme displacement of the MSE wall and the axial force on the geogrid in different layers.

An adequate amount of graphs are provided to represent the displacement of the MSE wall. Moreover, the difference between the axial force in all geogrids layers are also presented and compared under different conditions. Results are reported in graphical forms.

1.3 Organization of the thesis

This thesis devided into five chapters which are arranged as follows:

- Chapter One contain an introduction on MSE wall. It also presents the objectives of this research work and provides the details of various chapters and their contents as well.
- Capter Two provides the literature review on MSE wall.
- Chapter Three presents the numerical modeling of the MSE.
- Chapter Four presents the analysis and results of numerical modeling. Initially the MSE wall was simulated and calibrated using PLAXIS 2D. Then the results of various parametric studies are presented in graphical forms.
- Chapter Five is summation of the main conclusions from the current research and recommendations.

CHAPTER II

DEVELOPMENT OF REINFORCED EARTH STRUCTURES AND THEIR UNDERLYING PRINCIPLES

2.1 General

The use of reinforcement soil can be traced to prehistoric times. The earliest form of reinforced soil was the Ziggurats in Iraq and the Great Wall of China. The Ziggurates are the large religiouse towers were built by Babylonian's during the Early Bronze Age (about 2500 to 5000 years ago). It was reinforced with woven mats made from reeds laid horizontally with plaited ropes of the same material embedded in layers of gravel and sand (see Figure 2.1).

The Great Wall of China (see Figure 2.2) for at least 1000 years (e.g., western portion of the Great Wall) and were used along the Mississippi River in the 1880s (Reddy et al., 2003). It has tamarisk branches as reinforcement embedded in a mixture of gravel and clay (Elton and Patawaran, 2005).

Many primitive people used straw to improve the quality of adobe bricks, also they used sticks and branches to reinforce mud dwellings dates back to earliest human history.

During the 17th and 18th centuries, French settlers along the Bay of Fundy in Canada and they used sticks to reinforce mud. In England people used wooden pegs for erosion and land slide control, also they used bamboo or wire mesh for revetment erosion contol. Soil reinforcing can also be achieved by using live plant roots (Berg et al., 2009). The concept of reinforced earth was introduced in France in 1960s by Henri Vidal. The use of geogrid as reinforcement was introduced in 1970.

The problem of corrosion of metal reinforcements and the emergence of geosynthetics later led to the latter's use as reinforcement.

Many hypotheses have been assumed in the last 25 years about the mechanism of load transfer between the soil and reinforcement and their interaction. Many researchers have also carried out studies to find suitable method for the analysis and design of reinforced soil structures.

Figure 2.1 The ziggurats in Iraq (http://www.atlanteagardens.blogspot.com.tr)

Figure 2.2 Great Wall of China (http://www. shedexpedition. com)

2.2 History and Development of Reinforcing Systems

Here is some important timelines in the development of the reinforced earth techniques:

1- The French architect Henry Vidal research led to development of the reinforced soil in 1966, which used steel strips for reinforced. The first wall used this technology in the United States was built in 1972 on Califorina state highway 39, northeast of Los Angeles.

2- Using geogride for soil reinforcement has been used since 1970. Design methods was developed around 1980.

3- The use of geotextiles in soil reinforcement started in 1971 in France after their beneficial effect in the construction of embankment over weak subgrades (Reddy et al., 2003). The first use of this type in the United States was construted in 1974 (Berg et al., 2009).

4- The fundamental researches on the mechanism and design of the reinforced earth which including 15-full scale experiments, were realized from 1967 to 1978 by the "Laboratoire Central des ponts et Chausses" in Paris.

5- Polymer - reinforced permanent walls have been approved by AASHTO in the USA in 1997.

The reinforced earth technique has been quickly accepted all over the world because it is an economical and efficient solution. It has been extensively used in retaining walls and bridge abutments for highways, expressways and railroads lines. As well as for other structures as industrial, civil, defence, and water works projects.

Some applications of the techniques are shown in Figure (2.3) .

Figure 2.3 Application of reinforced wall as Retaining wall & Abutments (http://www.geosynthaticsmagzine.com)

2.3 Components of MSE Wall

Reinforced soil has three main components: the backfill, reinforcement, and the facing. Other components that are required are the foundation, drainage elements, and the connections between the facing and the reinforcement. Additional components may be needed depending on the function of the reinforced soil being constructed.

2.3.1 Backfill

Theoretically any type of soil can be used as backfill since the introduction of reinforcement would cause an increase in strength. Generally; prior to the mobilization of the reinforcement tensile strength, the soil alone carries the entire load. Soil contributes to the strength of the structure and how much load the soil can carry alone before the mobilization tensile strength of the reinforcement is important. Thus, clean course grained soils (sand & gravel) are specified and preferred since they are:

- stronger and providing better durability for metallic reinforcement (see Figure 2.4).
- drainage controls are essentially not necessary because it is free draining.
- Good soil reinforcement interaction because of hight friction characteristics, which include an increased rate of wall erection and improved maintenance of wall alignment tolerances (Berg et al., 2009).
- There are also significant handling, placement, and compaction advantages in using granular soils.
- Are generally less susceptible to creep (Blaise, 2001).

These performance requirements generally eliminate soils with high clay contents. Silts and/or clays were used in the reinforced soil zones and clearly, these situations were not “free draining” and hydrostatic pressures should have been considered in the design stage when using such low permeability soils.

That is not mean that silts and/or clays can not be used in the reinforced soil zone, it is meant to imply that if these poorly draining soils are used, they must be used with proper drainage components (Koerner and Koerner, 2011), and special attension to be paid to the creep potential (Blaise, 2001).

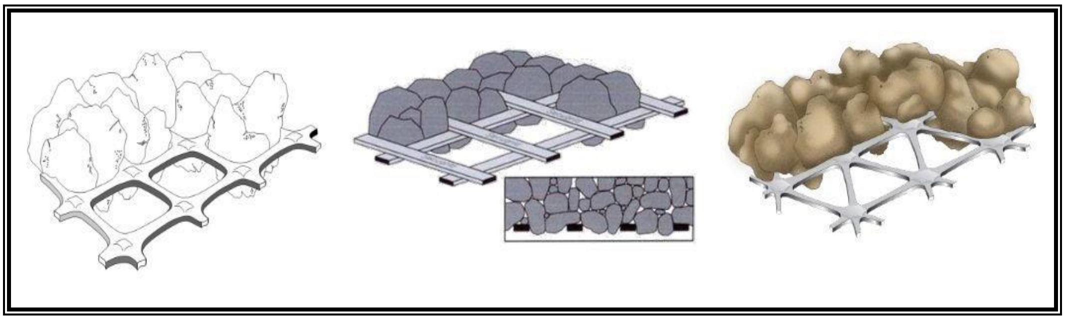


Figure 2.4 Aggregate interlocking with geogrid (http://www.windfarmbop.com)

2.3.2 Reinforcement elements

Different types of materials have been used as reinforcement. The reeds and branches used in ancient times have been replaced by metal and geosynthetics. Reinforcement can be in the form of strips, grids, and sheets. Berg et al., (2009) descriped MSE systems by the reinforcement geometry, reinforcement material, extensibility of the reinforcement material, and stress transfer mechanism.

2.3.2.1 Reinforcement Geometry

Three types of reinforcement geometry can be considered :

- Linear unidirectional: strips, including ribbed or smooth steel strips, or coated geosynthetic strips over a load-carrying fiber.
- Composite unidirectional: bar or grids mats which have grid spacing greater than 150 mm.
- Planar bidirectional: continuous sheets of geosynthetics or, welded wire mesh, and woven wire mesh. The spacing between mesh element are less than 150 mm.

2.3.2.2 Reinforcement Element

Below the characteristics of metallic and nonmetallic reinforcements:

- Metallic reinforcements: typically of mild steel. Usually the steel is galvanized or some time epoxy coated. The performance and durability considerations for this type of reinforcement vary considerably and are detailed in the companion Corrosion (Berg et al., 2009).

- Nonmetallic reinforcements: polymeric materials consisting of polypropylene, polyethylene, or polyester.

These reinforcement come in different shapes; such as strips, grids, sheet, fibers, and rods.

2.3.2.3 Reinforcement Extensibility

There are two classes of extensibility:

- Inextensible: reinforcement deformation at failure is much less than the soil deformation. Steel strip and bar mat reinforcements are an example on the inextensible type.
- Extensible: reinforcemenat deformation at failure is comparable to or greater than the soil deformation. Geogrid, geotextile, and woven steel wire mesh reinforcements are an example on the extensible type.

An inextensible metallic reinforcement makes the structure brittle and the extensible geosynthetic increases the ductility of the reinforced soil structure.

2.3.3 Facing Elements

In front of the backfill, there are the facing elements which control the aesthetics of the MSE wall (see Figure 2.5) since they are the only visible part of the completed structures. Facing also protect the soil and reinforcing elements from weathering effects, in addition, the facing provides protection against backfill erosion, and provides, in some cases drainage paths. A wide range of finishes and colors can be provided.

Figure 2.5 Facing element of MSE walls (http://www.reinforcedearth.com)

The facing which are currently used include plate and metal sheets, precast concrete elements, concrete block, welded wire mesh, rubber tires, timber, and shotcrete and as illustrated below:

a) Segmental precast concrete panels: the typical nominal panel dimensions of precast concrete panels are 1.5 m high, 1.5 or 3 m wide, and minimum thickness of 140 mm. They are of a cruciform, square, rectangular, diamond, or hexagonal geometry. Temperature and tensile reinforcement are required but will vary with the size of the panel. Vertically adjacent units are usually connected with shear pins (see Figure 2.6).

b) Dry cast modular block wall (MBW) units: these are relatively small, squat concrete units that have been specially designed and manufactured for retaining wall applications. The mass of these units ranges from 15 to 50 kg , with units of 35 to 50 kg routinely used for highway projects. Unit heights typically range from 100 to 200 mm for the various manufacturers. Exposed face length usually varies from 200 to 450 mm. Nominal width typically ranges between 200 and 600 mm. MBW facings are available in a variety of shapes and textures.

Units may be manufactured solid or with cores. Full height cores are filled with aggregate during erection. Units are normally dry-stacked (i.e. without mortar) and in a running bond configuration. Vertically adjacent units may be connected with shear pins, lips, or keys (see Figure 2.7).

c) Welded Wire Mesh (WWM): it can be bent up at the front of the wall to form the wall face. This type is used for example in Reinforced Earth wire faced retaining wall systems (see Figure 2.8).

d) Gabion Facing: These polymer or steel – wire baskets are filled with stone, can be used as facing with MSE wall with reinforcing elements consisting of welded bar-mats, welded wire mesh geogrids, geotextiles or the double-twisted woven mesh placed between the gabion baskets (see Figure 2.9).

e) Post-construction Facing : for wrapped faced walls, the facing – whether it was geotextile, geogrid, or wire mesh – can be attached after construction of the wall by shotcreting, guniting, cast-in-place concrete or by attaching prefabricated facing panels made of concrete, wood, or other materials. This type of facing adds cost but the significant settlement is anticipated (see Figure 2.10).

f) Geosynthetic Facing: variouse types of geosynthetic reinforcements can looped around at the facing to form the exposed face of the retaining wall. These faces are susceptible to ultraviolet light degradation, vandalism, and damage due to fire.
Geogrid are also warpped around the backfill in a similar manner to welded wire mesh. The geogrids need protection against ultraviolet light and vandalism; therfore, an asphalt or concrete coating is usually applied (see Figure 2.11). Vegetation can also grow through the grid structure and can provide both ultraviolet light protection for the geogrid and a pleasing appearance.

g) Timber facing: this type of facing are made of railroad ties or of other large elements of treated timber, geogrid either held by friction, or attached by batten strips when placed bettween the treated timber elements .

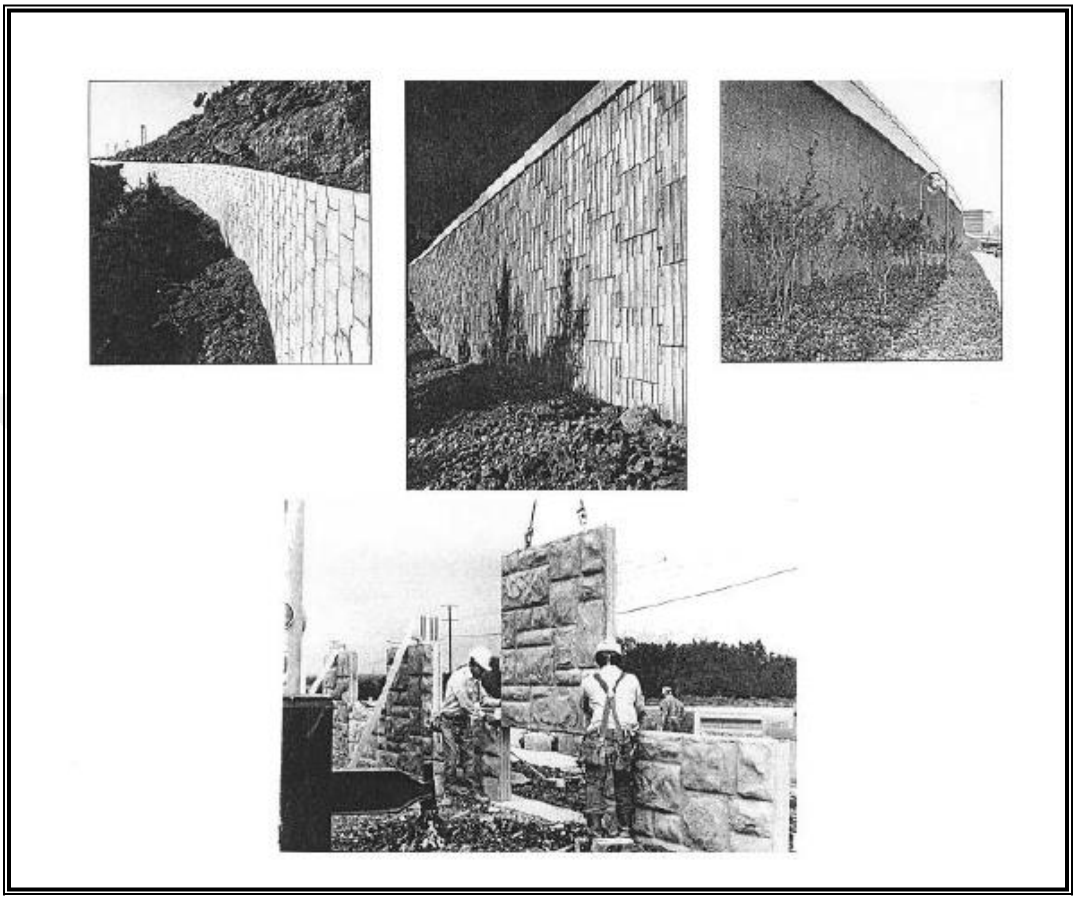


Figure 2.6 Example MSE wall facing treatments (Berg et al., 2009)

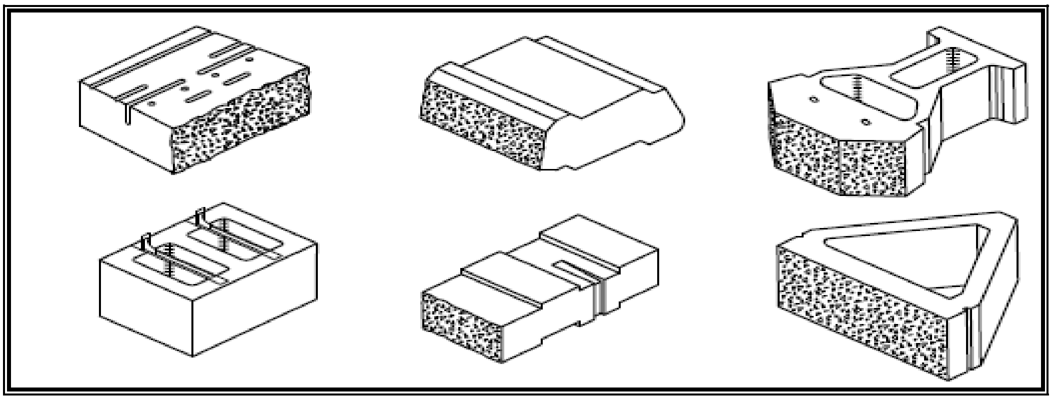


Figure 2.7 Examples of commercially available MBW units (Berg et al., 2009)

Figure 2.8 Welded Wire Mesh (http://www. atlanticcivil.com. au)

Figure 2. 9 Gabion Facing (http://www. kaengineers. com)

Figure 2.10 Post construction Facing (http:// www.tensarcorp.com)

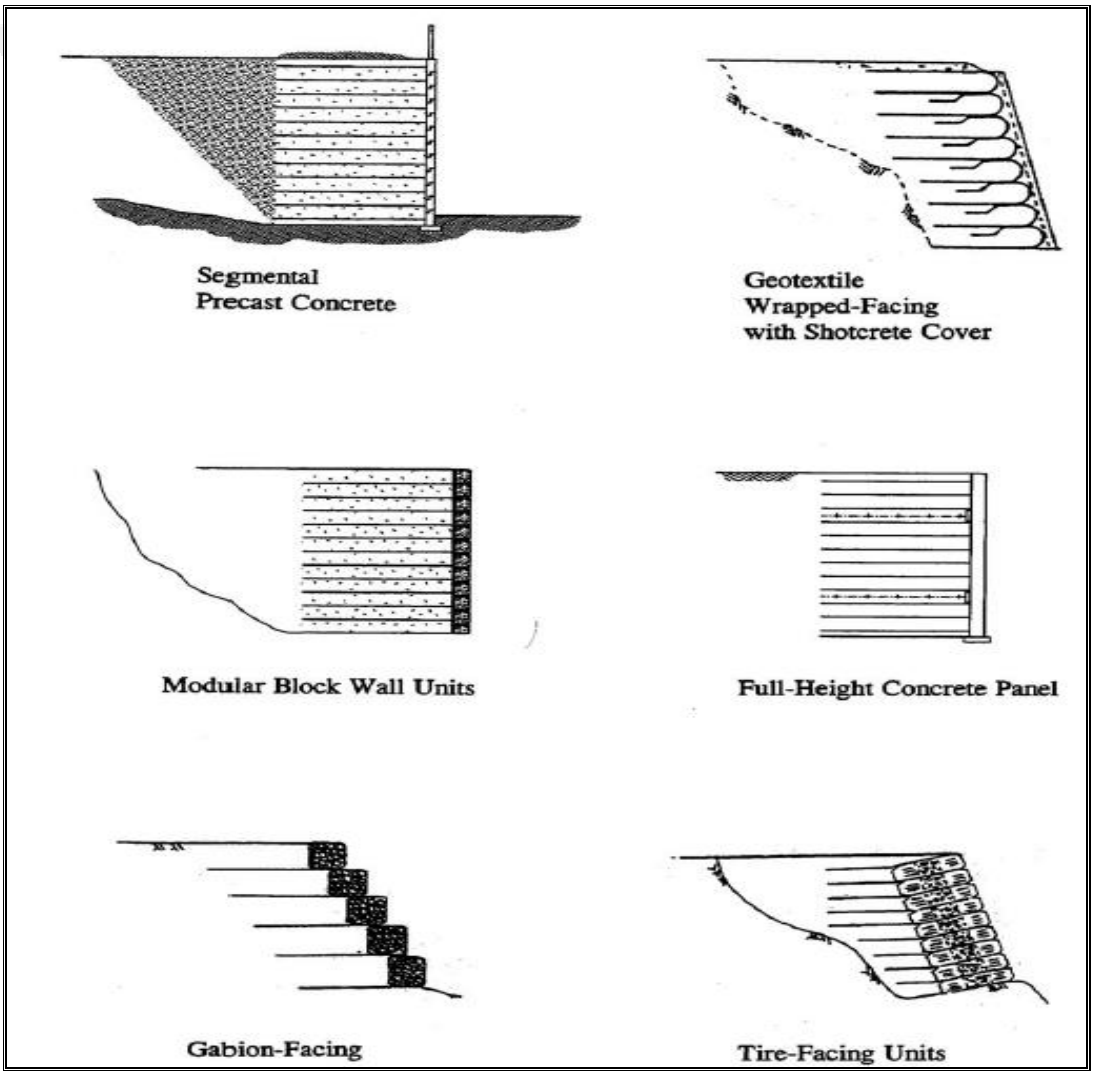


Figure 2.11 Types of reinforced soil wall facing (Berg et al., 2009)

2. 4 Design Life

The designing of a service life for MSE wall should be bsased on consideration of the potential longterm effects of material deterioration, stray currents, seepage, and other potentially deleterious envirnmental factors on each of the material components comprising the MSE wall.

Generally and for most applications retaining walls can be divided into two types :

- permanent retaining walls and it should be designed for a minimum service life of 75 years.
- Retaning wall for temporary applications and it should be designed for a service life of 36 months or less.

A greater level of safety and longer service life (i.e., 100 years) may be appropriate for buildings, walls which support bridge abutments, critical utilities, or other facilities for which the consequences of poor performance or failure would be severe (Elis et al., 2001).

2.5 Modes of failure

Analysis of failure mechanisms consider the stability of an equivalent gravity structure comprising the facing units, geosynthetic reinforcement and reinforced soil fill. Several possible failure modes are checked in reinforced soil walls depending on type of the structure itself and the field conditions, they are checked in the design of reinforced soil structures are mentioned below:

- External stability:

(a) Vertical and horizontal deformations resulting into unacceptable differential settlement.

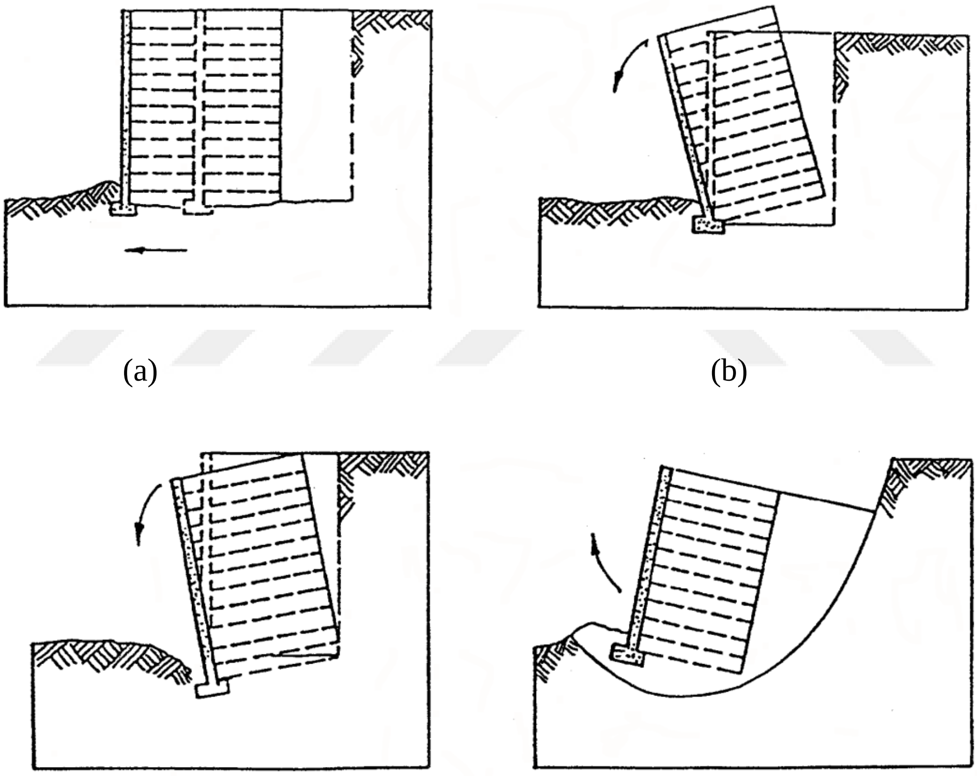
(b) Lateral sliding of reinforced soil.

(c) Overturning failure due to rotation about toe of the wall.

(d) Bearing capacity failure (punching) of the foundation soil under the reinforced soil.

(e) Overall collapse of the reinforced wall or embankment or nailed slope (Figure 2.12).

- Internal stability: analysis addresses reinforcement rupture failure and pull-out failure of reinforcement which depend on interaction with reinforced fill (Figure 2.13).
- compound stability: identify possible compound failure modes which initiate outside the reinforced zone and exit through the reinforcement and facing.



(a) (b) (c) (d)

Figure 2.12 Potential external failure mechanisms for a MSE wall : (a) sliding, (b) overturning, (c) bearing capacity, (d) deep seated stability (eccentricity) (Elise et al., 2001)

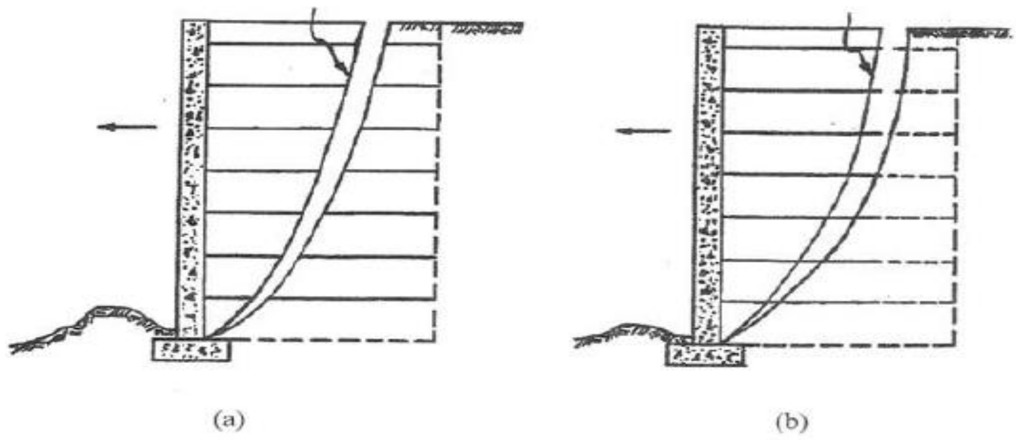


Figure 2.13 MSE Wall Internal Failure Mechanism: (a) Tension failure (b) pullout failure (SCDOT Geotechnical Design Manual, 2010)

2.6 Relative cost of MSE wall

Site specific costs of a soil-reinforced structure are a function of many factors, including requirements of the cut-fill, type and size of wall/slope, type of soil in-situ, available backfill materials, type of facing finish, permanent or temporary application.

It has been found that MSE walls with precast concrete facings are usually less expensive than reinforced concrete retaining walls for heights greater than about 3 m and average foundation conditions. Modular block walls are competitive with concrete walls at heights of less than 4.5 m.

In general, the use of MSE walls results in savings on the order of 25 to 50 percent and possibly more in comparison with a conventional reinforced concrete retaining structure, especially when the latter is supported on a deep foundation system (poor foundation condition) (Elis et al., 2001). The mean costs of publicly financed walls for four different types are shown in Figure (2.14). Based on area of wall facing, it is seen that MSE walls with geosynthetic reinforcement are the least expensive of all wall types.

Even further, the classical gravity type of walls are more than twice as expensive as another type (Koerner and Koerner, 2011).

Savings in cost is obtained by elimination of the deep foundations (usually possible), because reinforced soil structures can accommodate total and differential settlements. Other cost saving features include speed of construction and ease of construction.

The actual cost of a specific MSE wall structure will depend on the cost of each of its principal components (Elis et al., 2001). For segmental precast concrete wall, typical relative costs are:

- Erection of panels and contractors profit - 20 to 30 percent of total cost.
- Reinforcing materials - 20 to 30 percent of total cost.
- Facing system - 25 to 30 percent of total cost.
- Backfill materials including placement - 35 to 40 percent of total cost, where the fill is a granular fill from an off site borrow source.

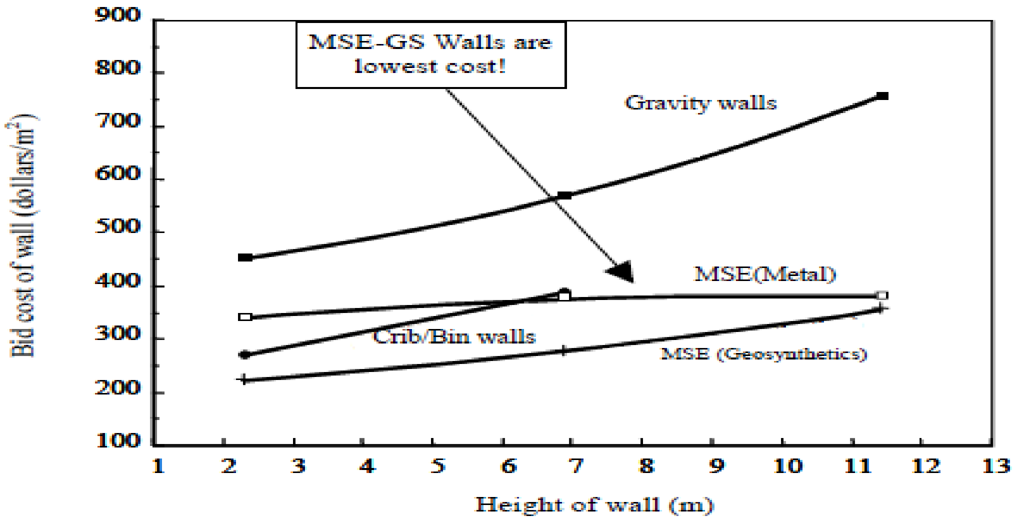


Figure 2.14 Cost comparison for retaining walls (Koerner and Koerner, 2011)

2.7 Mechanics of soil reinforced

A reinforced soil mass is somewhat analogous to reinforced concrete in that the mechanical properties of the mass are improved by reinforcement placed parallel to the principal strain direction to compensate for soil's lack of tensile resistance.

The composite material has the following characteristics:

- Stress transfer between the soil and reinforcement continuously takes place along the reinforcement.
- The distribution of the reinforcements throughout the soil mass with a degree of regularity (not be localized).

Several experimental and theoretical investigations have been performed since the invention of Reinforced Earth wall to understand the concepts and mechanism of reinforced soil structure and interaction among its basic components, reinforcing elements, backfill soil and facing. In literature, three concepts have been proposed to explain the mechanical behavior of a Geosynthatices Reinforced Soil (GRS) mass: (1) the concept of enhanced confining pressure, (2) the concept of enhanced material properties, and (3) the concept of reduced normal strains.

2.7.1 Concept of Apparent Cohesion

In this concept, a reinforced soil increase the major principle stress at failure from σ_1 to σ_{1R} (with an apparent cohesion C'_R) due to the presence of the reinforcement (Wu et al., 2013), as shown by the Mohr stress diagram in Figure (2.15). The apparent cohesion C'_R can be determined if a series of triaxial tests on unreinforced and reinforced soil elements were conducted. The φ value for unreinforced and reinforced sand were about the same as long as slippage at the soil reinforcement interface did not occur.

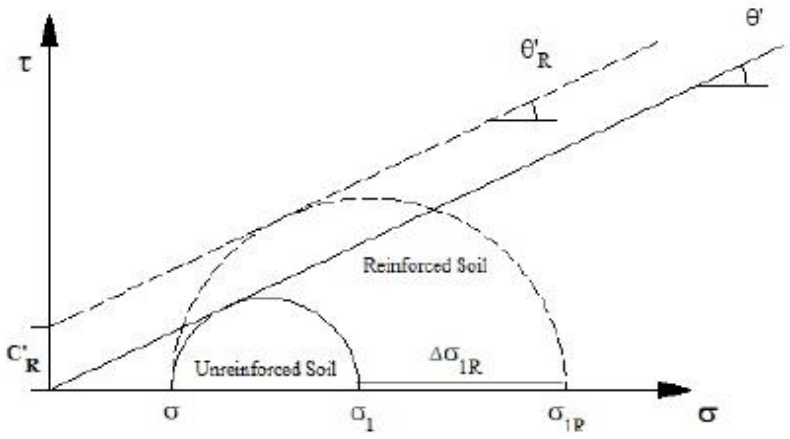


Figure 2.15 Illustration Concept of apparent cohesion due to the presence of reinforcement (Wu et al., 2013)

2.7.2 Concept of Apparent Confining Pressure

In this concept, the axial strength increased from σ_1 to σ_{1R} in a reinforced soil (with an increase of confining pressure, $\Delta\ \sigma_{3R}$), as shown in Figure (2.16), due to the tensile inclusion. The value of $\Delta\ \sigma_{3R}$ can also be determined from a series of triaxial tests by assuming that φ will remain the same (Wu et al., 2013).

The concept of apparent confining pressure the apparent cohesion can be determined from the strength data for the unreinforced soil.

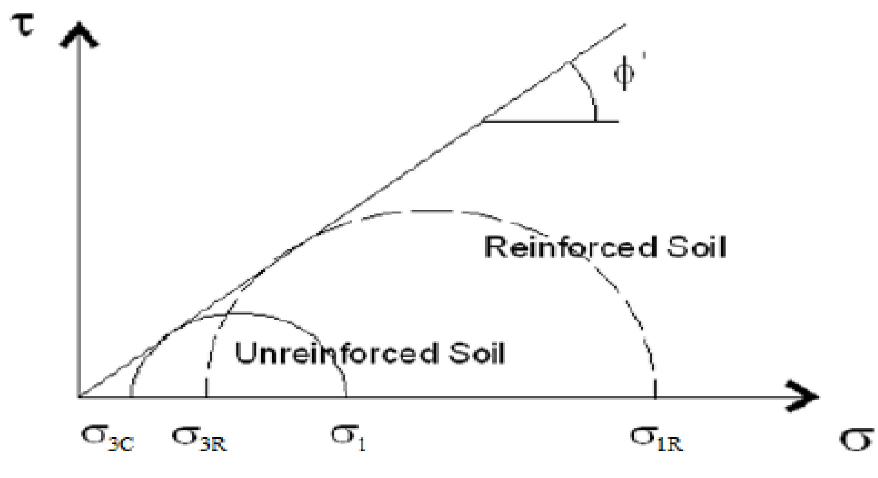


Figure 2.16 Illustration Concept of apparent confining pressure due to the presence of reinforcement (Wu et al., 2013)

2.8 Stress Transfer Mechanisms

Stresses are transferred between the reinforcement and the soil by friction (Figure 2.17 a) and/or passive resistance (Figure 2.17 b) depending on reinforcement geometry:

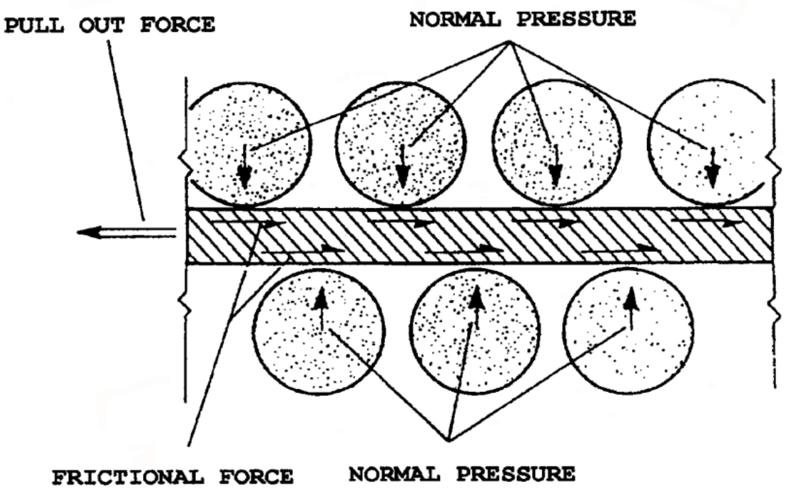
- Friction**:** the reinforcement elements should be aligned where friction is important in the direction of soil reinforcement relative movement. Generally, friction develops at locations where there is a relative shear displacement and corresponding shear stress between soil and reinforcement surface. Examples of such reinforcing elements are steel strips, longitudinal bars in grids, geotextile and some geogrid layers are an examples of such type of reinforcement element.

- Passive resistance**:** whean the bearing type stresses developed on "transverse" reinforcement surfaces normal to the direction of soil reinforcement relative movement. Generally, For rigid geogrids, bar mat, and wire mesh reinforcements Passive resistance is considered to be the primary interaction, also there is some passive resistance provided by the transverse ridges on "ribbed" strip reinforcement .

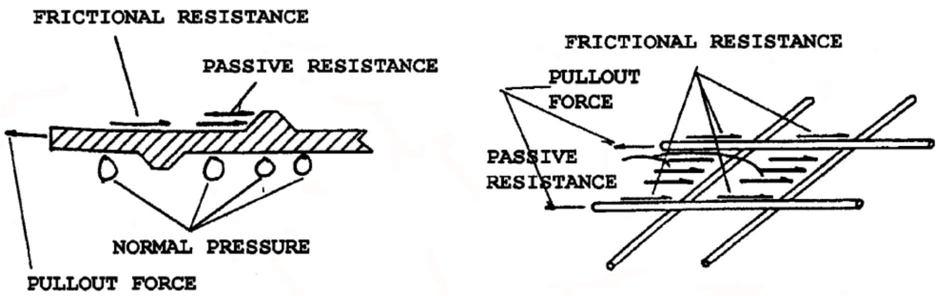
The contribution of each one from transfer mechanism which mension above for a particular reinforcement will depend on:

- Skin friction (the roughness of the surface).
- Effective normal stress.
- The dimension of grid opening.
- Thickness of the transverse members.
- Reinforcement elongation characteristics.

Also the characteristics of the soils including grain size, grain size distribution, particle shape, density, water content, cohesion, and stiffness have equally important for the development of the interaction.



(a)



(b)

Figure 2.17 Stress transfer mechanisms for soil reinforcement: (a) frictional stress transfere between soil & reinforcement surface, (b) soil passive resistance on reinforcement surface (Elise et al., 2001)

2.9 Mode of Reinforcement Action

The most important function of reinforcements is to restrain the deformation of the soil. In so doing, stresses are transferred from the soil to the reinforcement. These stresses are resisted by the reinforcement tension and/or shear and bending (Berg et al., 2009).

- Tension: is the most common mode of action of tensile reinforcements. Generally all "longitudinal" reinforcing elements that cross shear plan are subjected to high tensile stresses which usually developed in flexible reinforcements.

- Shear and Bending: this mode of action can withstand by "Transverse" reinforcing elements which have some rigidity.

2.10 Lateral Movements of the wall facing

The most lateral deformations of MSE wall face usually occur during construction processe. Post construction movements, however, may take place due to post construction surcharge loads, settlement of wall fill, or due to the foundation soil long-term settlement.

The magnitude of lateral displacement depends on the techniques of fill placement, the effect of the compaction, reinforcement length and extensibility, connection details between reinforcement-to-facing, and wall facing details.

A deformation response analysis allows for an evaluation of the anticipated performance of the structure with respect to horizontal (and vertical) displacement (Berg et al., 2009). Using numerical modeling will be warranted and give more accurate calculation for bridge abutments which consider critical structures.

The results of horizontal deformation analysises are most difficult, in many cases they are done only approximately and it may impact the choice of facing, facing connections, or backfilling sequences.

2.11 Vertical Movement and Bearing Pads

Bearing pads are placed in horizontal joints of segmental precast concrete panels in order to allow the panel and the reinforcement to move down with the reinforced fill as it is placed and settles, mitigate downdrag stress, and provide flexibility for differential foundation settlements (Berg et al., 2009).

Generally, the internal settlement within the reinforced fill is practically immediate beside some minor movement which occure due to elastic compression happened in granular materials after construction process.
The total amount of the movement is the combination of the external differential movement and the internal movement.
Normally for well graded granular fill the internal movement is negligible, and when using sand type fill and/or marginal fill containing an appreciable amount of fines, the internal movement can be significant which leads to concrete panel cracking and/or downdrag on connections resulting in bending of connections and/or out of panel movement can occur.

2.12 Finite Elements Analysis

Finit element method is one of the most powerful numerical techniques ever devised for solving differential equations of initial and boundary value problems.
In order to analyzing an element by the finit element method, the following parameters hould to define:

- The domain.
- The boundry condition.
- The physical properties.
- The initial condition.

After define this data and if the analysis is done carfully, it can be said that the process to do this analysis is very methodical, and it will give us satisfactory results. Finit Element Aanalysis of problem is so systematic, it can be divided into logical steps that can be implemented on adigital computer and can be used to solve different types of problems just by changing the input data in the computer program.
PLAXIS 2D program based on the finite element method and intended for 2-Dimensional and 3-Dimensional geotechnical analysis of deformation and stability of soil structures, as well as groundwater and heat flow, in geo-engineering applications such as excavation, foundations, embankments and tunnels.

2.12.1 General Philosophy of FEM

Finite element method is the representation of a body or a structure by ingathering of subdivisions called finite elements, which assumed to be inter-connected at points called nodes. This method is a numerical procedure for analysing structures and continua. FEM is a powerful tool used to analysis of complicated to simpled geometries.

2.12.2 Modelling the components of MSE Wall

The processes of the incorporation of MSE wall parameters (mechanism of soil-reinforcement- facing interaction) in the FEM are greatly influenced by the construction method, compaction, propping of facing during construction thus, making it difficult to model the problem.

- Soil and Rock: under the load Soil & Rock tend to behave in highly non-linear way. The famouse model of mohr- coulomb which is elastic perfectly – plastic model can be considered as a first order approximation of real soil behaviour, this model requires five basic input parameter namely Young modulus, Cohesion, Poisson’s ratio, Friction angle, and a Dilatancy angle.

- Reinforcement: is generally modelled by linear bar element (the objects geogrid are generally used to model soil reinforcement) capable of taking only axial tensile forces with no bending moment. Behaviour of extensible geosynthetic materials is generally nonlinear. The only material property of geogrid is an elastic normal (axial) stiffness EA .

- Facing: MSE Wall facing is slender structure with significant bending stiffness (or flextural rigidity). The objects plate are structural subject used to model facing, the most important parameters are the flextural rigidity (bending stifness) EI, and the axial stifness EA.

2.13 Typical Current Design Methods

In order to determine the geometric and reinforcement requirements to prevent internal and external failure in MSE wall, a number of design methods have been deriven, all of these methods are either empirical in nature or based on limit equilibrium analysis. The main purpose of these methodes is to compute the factor of safety against several modes of failure.

2.13.1 Force Equilibrium Methods

Jewell Method (1987)**:** This method describes a link between soil stresses in a reinforced soil mass in which a constant mobilized angle of friction is assumed with the resulting displacements (velocity fields). There are two parameters for plane-strain plastic deformation of soil: the plane-strain angle of friction φ_{ps}, and the angle of dilation ψ.

The planes on which the maximum shearing resistance φ_{ps} is mobilized are inclined at (45+ **Error!**φ_{ps}) to the direction of major principal stress, The directions along which there is no linear extension strain in the soil are called the "velocity characteristics" and are inclined at (45 + **Error!**ψ) to the direction of major principal stress (Figure 2.18). In this method, the reinforced soil structure is divided into 3 zones based on the reinforcement force as shown in Figure (2.19). The boundary between zone 1 and 2 is at an angle (45+ **Error!**ψ) to the horizontal, and the boundary between zone 2 and 3 is at an angle φ_{ds} .

Large reinforcement forces are required in zone 1 to maintain stability across a series of critically inclined planes. In zone 2, the required reinforcement forces reduce progressively. Jewell method has been found to give the closest agreement with FE. It is only applicable to reinforced soil walls where there is little facing rigidity, such as a wrapped-faced GRS wall. In this method several design charts are provided (Figure 2.20).

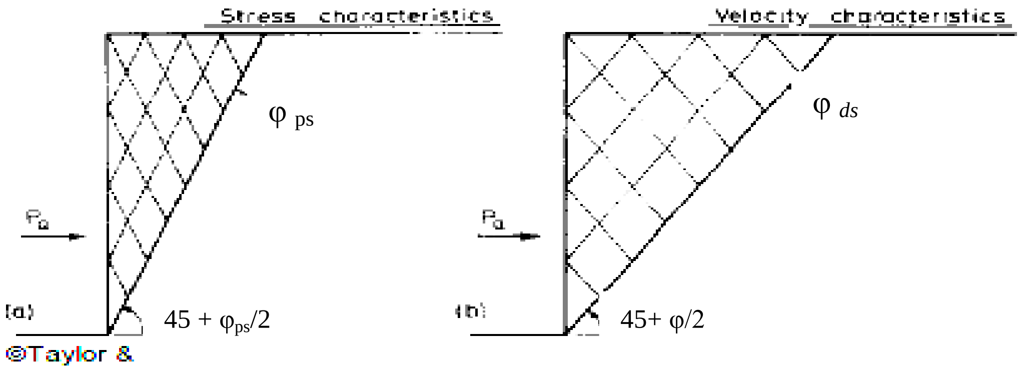


Figure 2.18 Illustration Major zones of reinforcement forces in a GRS wall and the force distribution along reinforcement with ideal length (Wu et al., 2013)

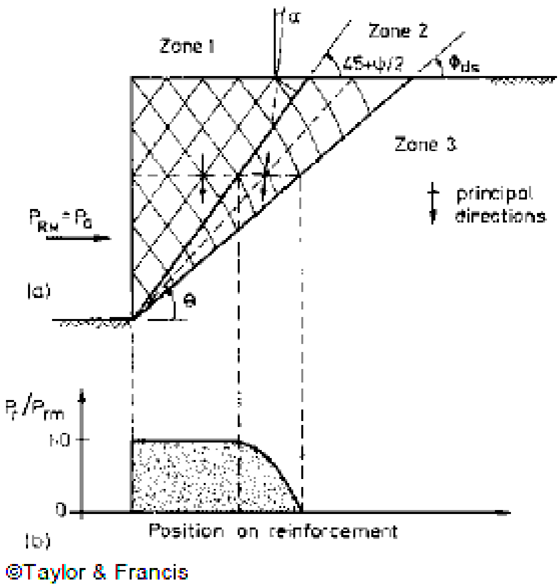


Figure 2.19 Illustration Major zones of reinforcement forces in a GRS wall and the force distribution along reinforcement with ideal length (Wu et al., 2013)

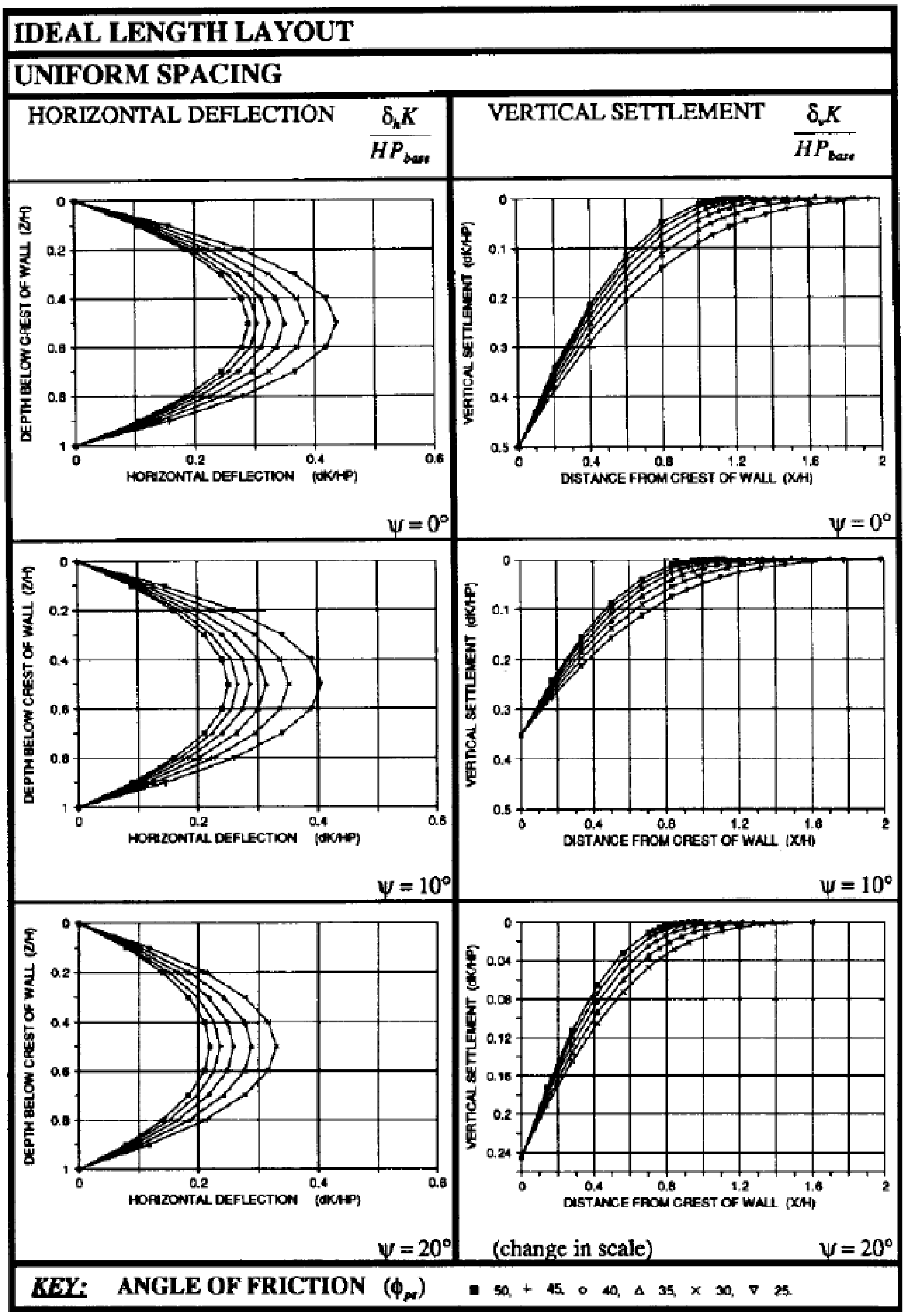


Figure 2.20 Illustration Charts for estimating lateral displacement of GRS walls with the ideal length layout(Wu et al., 2013)

2.13.2 Slope Stability Methods

Many basic methods have been derived from the conventional slope stability studies; Fellenius or Bishop methods or the Wedges methods represent the most widely used methods. There are three noticeable differences among these methods as follow:

(a) Failure surface shape.

(b) Reinforcement force distribution.

(c) The means by which a surcharge is considered.

- Typical slope stability methods are as follows:

1. Fellenius Method

Developed by Wolmar Fellenius as a result of slope failures in sensitive clays in Sweden, it is First method of slices to be widely accepted; also produces the lowest factor of safety.

- Assumptions for Fellenius Method:

- Compressional force of stress are not significant included Forces .
- Weight of the slice, including weight of water.
- Resisting Shear forces at base of slice, both those from the cohesion of the soil and those from effective stress.
- Resisting moments are generated by the shear strength of the soil at the failure surface.

- Governing Equation for Fellenius Method

$$F_s = \frac{Force\ resisting\ sliding}{Force\ inducing\ sliding} = \frac{\sum[cb + Wcos\alpha . tan\phi]}{\sum Wsin\ \alpha} \quad \text{Eq (2.1)}$$

Where:

W- The weight of sliced blocks.

b - The length of sliding plane in sliced block.

ϕ - The angle of internal friction of sliding surface.

c - The cohesion of sliding surface.

α - Inclination of sliding surface with horizontal.

2- Bishop's method

The Modified (or Simplified) Bishop's Method proposed by Alan W. Bishop of Imperial College is a method for calculating the stability of slopes. It is an extension of the Method of Slices. By making some simplifying assumptions, the problem becomes statically determinate and suitable for hand calculations.

The method has been shown to produce factor of safety values within a few percent of the "correct" values. In this methof Factor of Safty is give as follows:

$$F = \frac{\sum[\frac{c' + ((W/b) - u)\tan\phi'}{\psi}]}{\sum[(W/b)\sin\alpha]} \qquad \text{Eq (2.2)}$$

Where :

$$\psi = \cos\alpha + \frac{\sin\alpha\tan\phi}{F} \qquad \text{Eq (2.3)}$$

c'- is the effective cohesion.

Φ' - is the effective internal angle of internal friction.

b - is the width of each slice, assuming that all slices have the same width.

W - is the weight of each slice.

u - is the water pressure at the base of each slice.

2.14 Drainge control for MSE walls

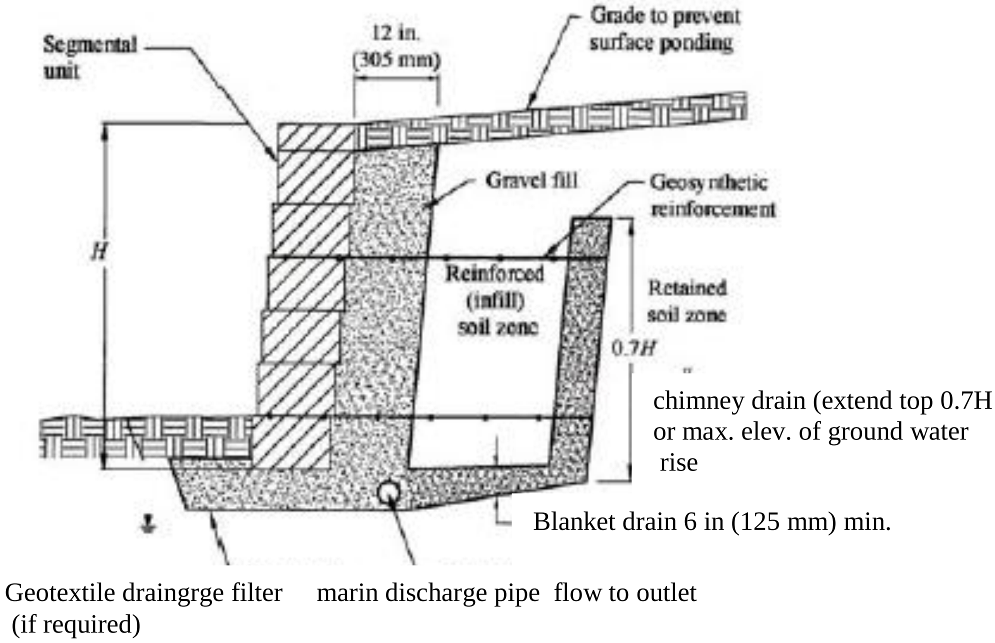
In the usuall design of MSE wall system are a ssumed to contain “free drainge” components, which means that the water will easily dischared through the reinforced soil mass. This is expected but only if sand and gravels (free – draining) are used in the reinforced soil zone.

If silt or clay (poorly draining) soils are used, they must be used with proper drainage components. Of course, if sands and/or gravels are used throughout the reinforced soil system then such drainage controls are essentially not necessary. Four situations of specific drainage control must be considered in the design of MSE walls when using silt and/or clay soil backfills in the reinforced soil zone.

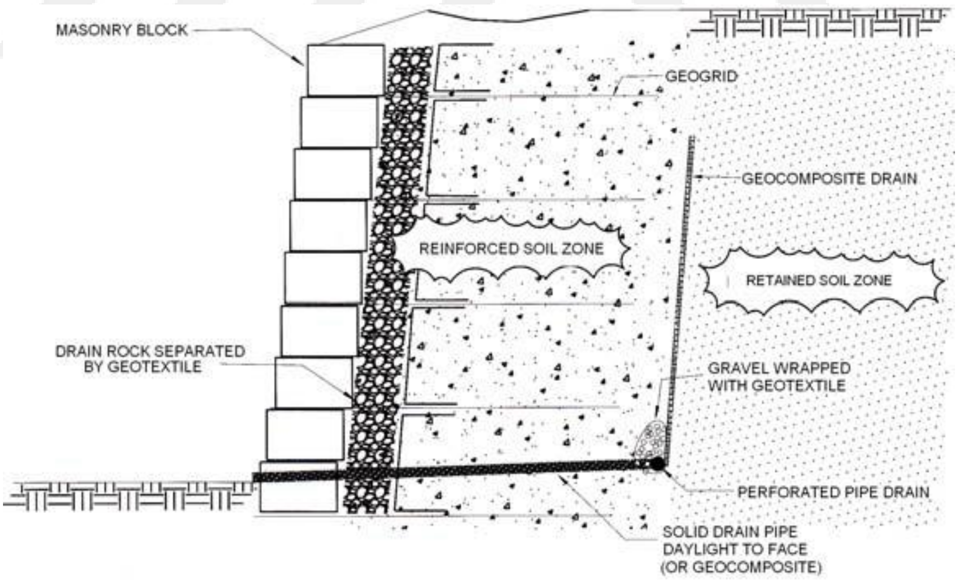
2.14.1 Retained Soil Drainage

Groundwater drainage from the retained soil zone can be large which is particularly a concern in cut-sections (Figure 2. 21), and if hambered by a low permeability backfill soil will cause the mobilization of hydrostatic pressure. In this case a back drain must be used between the retained soil and reinforced zone (Figure 2.22 a, b, c) Note that due to the difficulty in constructing vertical layers of soil, it is recommended to forms a vertical continuation of the base.

Figure 2.21 Groundwater exiting from retained soil zone in cut-situation
(Koerner and Koerner, 2012)


Segmental unit
12 in.
(305 mm)
Grade to prevent surface ponding
H
Gravel fill
Geosy nthetic reinforcement
Reinforced (infill) soil zone
Retained soil zone
0.7H
chimney drain (extend top 0.7H
or max. elev. of ground water
rise
Blanket drain 6 in (125 mm) min.
Geotextile draingrge filter
(if required)
marin discharge pipe flow to outlet

(a)


MASONRY BLOCK
GEOGRID
GEOCOMPOSITE DRAIN
REINFORCED SOIL ZONE
RETAINED SOIL ZONE
DRAIN ROCK SEPARATED BY GEOTEXTILE
GRAVEL WRAPPED WITH GEOTEXTILE
PERFORATED PIPE DRAIN
SOLID DRAIN PIPE
DAYLIGHT TO FACE
(OR GEOCOMPOSITE)

(b)

(c)

Figure 2.22 Various approaches to providing back drainage behind MSE walls:
(a) Back drain using sand, (b) Back drain using drainage geocomposite, (c) Use of continuous and intermittent geocomposite back drains when using fine grained soils in the reinforced soil zone (Koerner and Koerner, 2012)

2.14.2 Drainage from Paved Surfaces and Adjacent Structure

The familir reason for constructing a wall is to gain horizontal space along the upper surface this space generally required for parking, storage area, roadways, and buildings and homes.

By so doing, The accumulated flow (rainwater and snowmelt) which is coming from these surfaces should collected in a catch basin, inlet, or manhole located within the reinforced soil zone in order to prevent it from flowing over the top of the wall. Whenever, the reinforced zone consists of poor drainage soil (silts and/or clays) it will be very dangerous to bring the accumulated water into the reinforced zone .

Furthermore, if there is no very good compaction control, such design should be prevented.

Settlement of the drainage system,

will increase by increasing the outward deformation of the wall itself, and this is behaviour usually leads to leakage and may be cause pipe breakage as shown in the Figure (2.23).

Figure 2.23 internal drainage failures (Koerner and Koerner, 2012)

The solution to this situation is shown in Figure (2.24). Here is the drainge flows should directed away from the face of the wall to the end of the reinforced soil zone . At this location, the inlet and pipe transmission system is constructed. Thus the reinforcement is not interrupted in any way. Furthermore, if leakage occurs at this location, it can be accumulated and transmitted into the back drain and eventually out of the system via the base drain (Koerner and Koerner, 2012) .

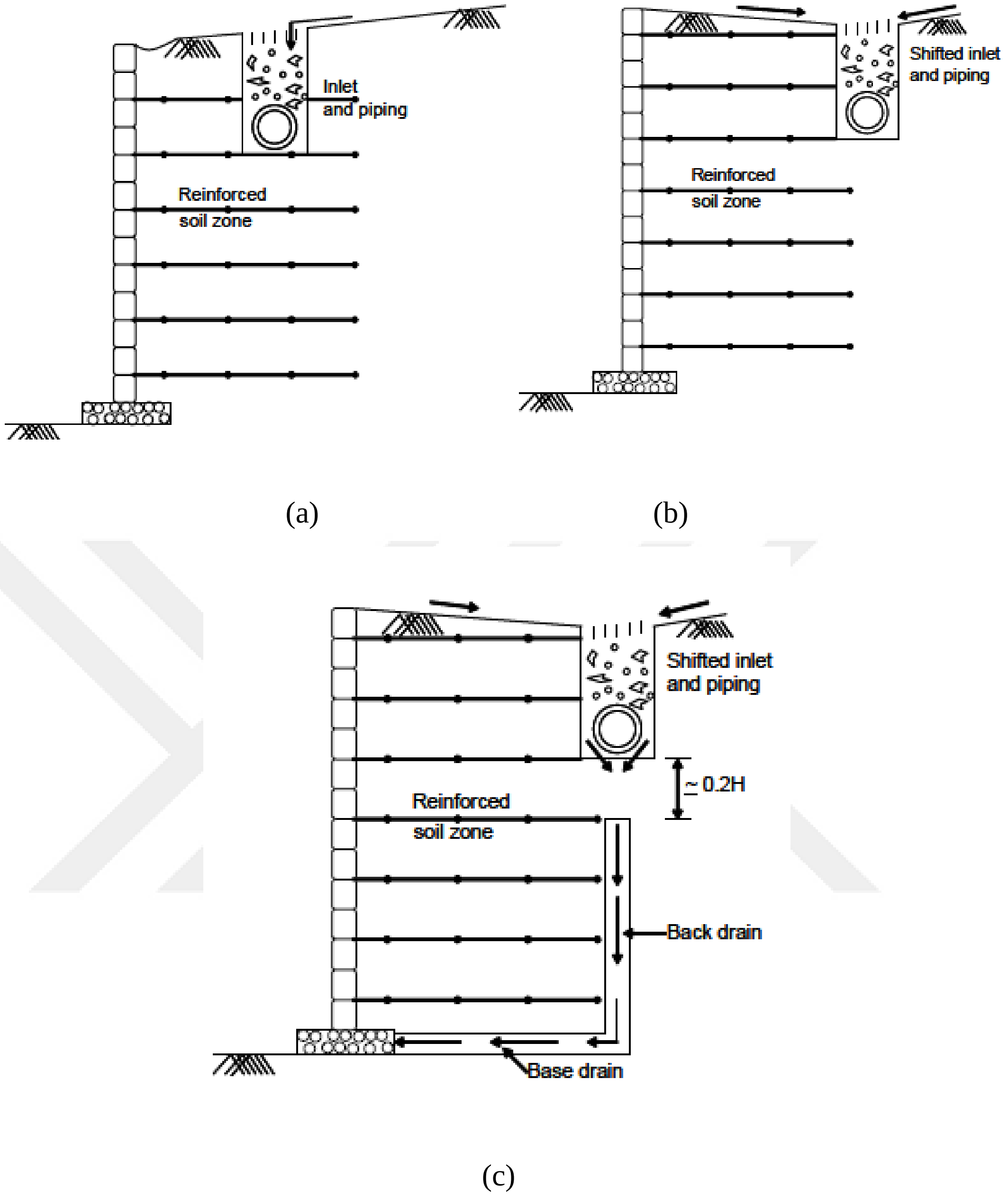


Figure 2.24 Recommended backgrading from wall face and shifting of internal drainage systems from within to behind the reinforced soil zone : (a) Customary internal drainage for surface water within reinforced soil zone, (b) Recommended external drainage for surface water behind reinforced soil zone, (c) Recommended external drainage for surface water coupled with back/base drain (Koerner and Koerner, 2012)

2.14.3 Waterproofing Backfilled Surface

The water and snowmelt which accumulate on the ground surface when using low permeability soils in the reinforced zone, it often infiltrates into the backfill soil forming hydrostatic pressure against the wall facing causing deformation or may be led to actual collapse.

In this case a geomembrane covering used to cover the surface as shown in Figure (2.25).

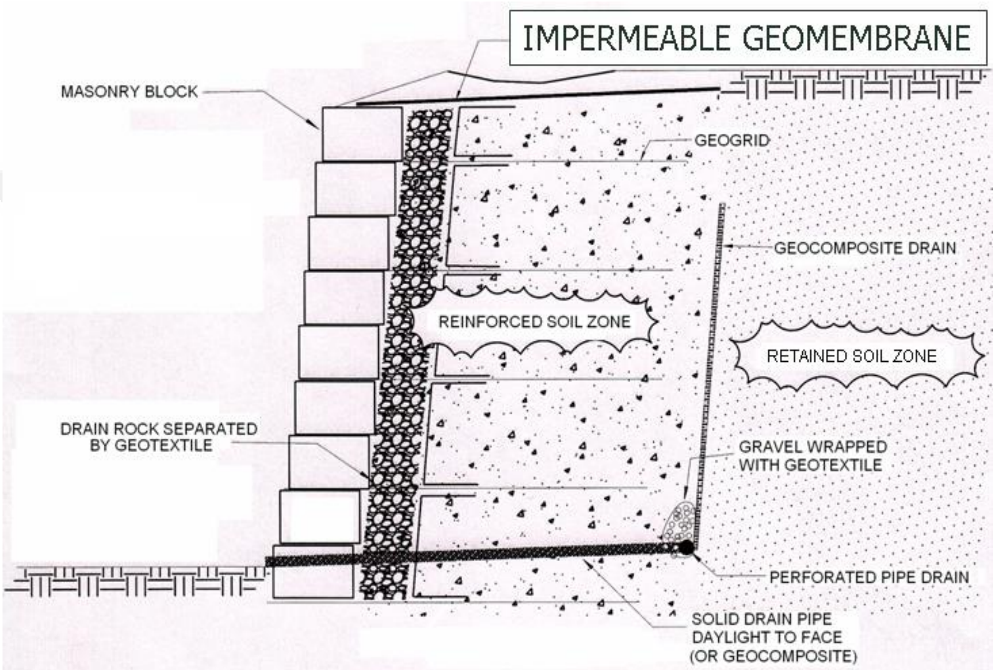


Figure 2.25 Use of a geomembrane waterproofing layer above the reinforced soil zone (Koerner and Koerner, 2012)

2.14.4 Tension Crack Sealing

When dealing with the silt and or clay backfilled soil, tension cracks usually occur at the end of the reinforcement (Figure 2.26). This occurs primarily due to volume decrease of the reinforced soil mass, but also due to the outward deformation of the wall facing (Koerner and Koerner, 2012).

When these surface tension cracks fill with water, which lead to forming hydrostatic pressure against the reinforced soil mass.

With increasing of the wall movement, a set of masonry blocks start to falls off of its supporting layer. The rows of blocks still fails until the wall face collapses and majority of the reinforced soil mass remains behind, as illustrate in Figure (2.27). The solution of this type of external drainage issue is to select a good quality of geomembrane waterproofing (high extensibility, flexibility, and durability), extend it beyond the reinforcement soil zon and onto retained soil zone.

Figure 2.26 Tension cracks occurring exactly at the end of the wall reinforcement (Koerner and Koerner, 2012)

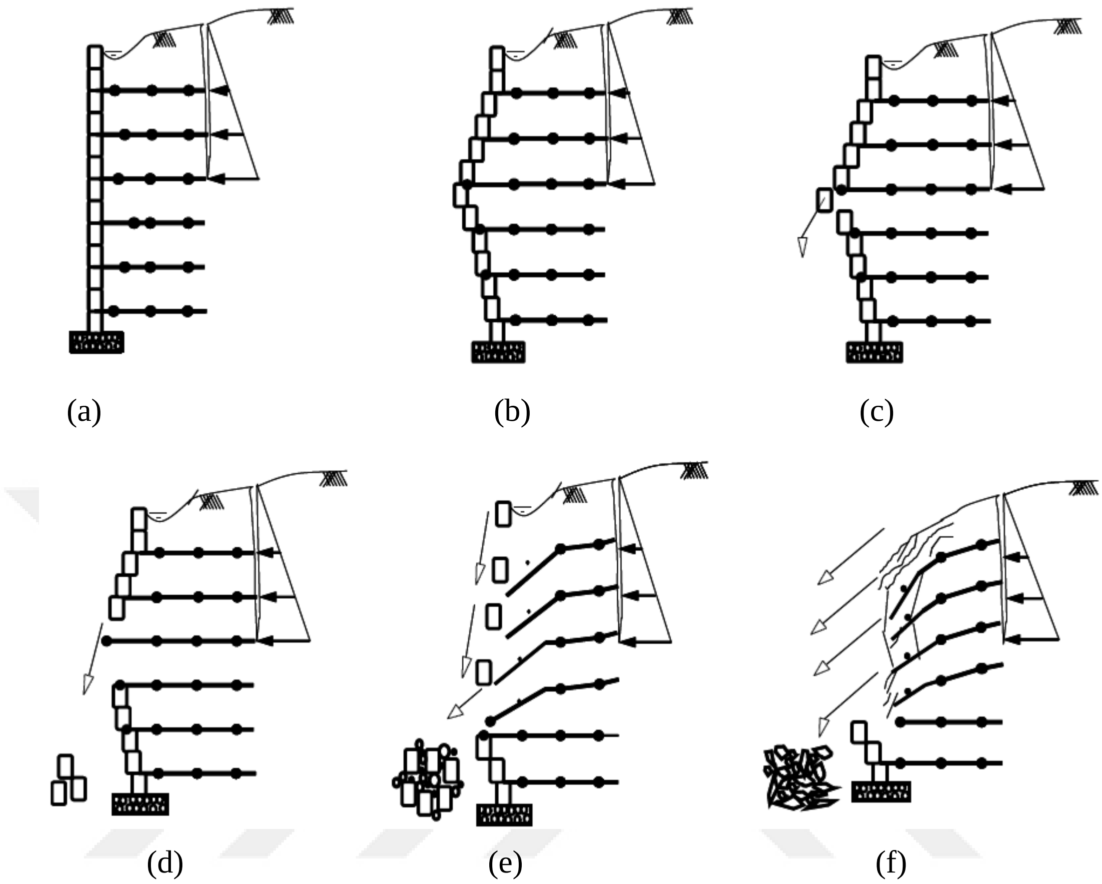


Figure 2.27 Modular block wall collapse progression due to hydrostatic pressure in tension cracks: (a) Crack forms, water enters and pressure is mobilized, (b) Wall deforms; pressure continues, (c) Deformations continiouse single block dislodges and drops to toe of wall, (d) Overlying blocks drop accordingly, (e) blocks progressively drop along with gravel and some backfill soil, (f) after the wall facing collapses; majority of the MSE mass remains behind (Koerner and Koerner, 2012)

CHAPTER III

NUMERICAL ANALYSIS

3.1 Overview of plaxis software

The two-dimensional finite element program PLAXIS version 8.0 is a finite element software program developed in the Netherlands for analysis of deformation in geo-structures and geotechnical engineering problems.

It is equipped with features to deal with various aspects of geotechnical structures and construction processes using excellent theoretically computational procedures.

Geotechnical applications require advanced constitutive models for the simulation of the non linear, time dependent and anisotropic behaviour of soils and/or rock, also Special procedures are required to deal with hydrostatic and non hydrostatic pore pressures in the soil because the soil is multi phase material. Although the modelling of the soil itself is an important issue. Many projects involve the modelling of structures and the interaction between the structures and the soil.

PLAXIS 2D software has been optimised to accurately simulate this highly non-linear behaviour.

Typical PLAXIS 2D applications include: assessing street level displacements during the tunnel construction, consolidation analysis of embankments, soil displacements around an excavation pit, dam stability during different water levels, stability of retaining walls and much more.

3.2 General modelling aspects by plaxis software

With PLAXIS 2D the geometry of the model defined as the representation of the physical problem and it is consist points, lines, and clusters which used to define soil layers, structural elements and loads.

The geometry can be easily defined in the soil and structures modes, after which independent solid models can automatically be intersected and meshed. The staged construction mode allows for simulation of construction and excavation processes by activating and deactivating soil clusters and structural objects.

The calculation kernel enables a realistic simulation of the non linear, time dependent and anisotropic behaviour of soils and/or rock. Since soil is a multi phase material, special procedures allow for calculations dealing with hydrostatic and non hydrostatic pore pressures in the soil. The output consists of a full suite of visualization tools to check the details of the 2D underground soil-structure model. The analysis prosses started in input program, and it is carried out in the sequence indicated below.

3.2.1 Composing a geometry model

The geometry is the representation of the physical problem. Principle, first by using the geometry line option which have several functions (or properties). The user can define graphical input of geometry contour, construction stage and the physical boundaries of the geometry**.** Plates are structural objects used to model slender structures in the ground with a significant flexural rigidity or normal stiffness. Plates can be used to simulate the walls, shells or linings extending in z-direction. Geogrids are slender structures with their normal stiffness generally used to model reinforcement. Interfaces used to model the interaction between the reinforcement and the soil, interfaces are used as intermediate between smooth and fully rough, then boundary conditions and then loading.

3.2.1.1 Soil properties

The behaviour of Soil and rock under load can be descibed as a highly non-linear stress-strain behaviour, which can be modeled at several levels of sophistication .

Table 3.1 **:** Units

Type	Unit
Length	m
Force	kN
Time	day

Table 3.2 : Model Dimensions

	Min.	Max.
X	0	21
Y	0	11

Table 3.3 **:** The Model

Model	Plain strain
Element	15- node

The first order approximation of real soil behaviour is the well-known model of Mohr-Coulomb. There are five basic input parameters which are required in Mohr-coulomb model. Namely a Young’s modulus E, a Poisson’s ratio ν, a cohesion c, a friction angle ϕ, and a dilatancy angle ϕ.

The material properties and model parameters for soil clusters are entered in material data sets. The material properties of interfaces which are related to the soil properties are entered in the same data sets as the soil properties .

PLAXIS 2D can handle cohesionless sands (c = 0), but some options may not perform well.To avoid complications, the value of cohesion will assumed to be equal to 1 kN/m^2, this value is usually used during analysis.

In the case of full drainge due to high permeability and also if dry soil are used , no excess pore pressure are generated. The behaviour of the soil will be described as drained behaviour which also can be used to simulate long term soil behaviour without the need to model the precise history of consolidation and undrained loading.

Undrained behaviour used for full development of excess pore pressure. In the case of low permeability soil and/or high rate of loading the flow of pore water can be neglected.

Table 3.4 Soil data parameters

Mohr-Coulomb		Foundation soil	(1) Loose sand	(2) Dense sand	(3) Clayey soil
Type of the soil		Drained	Drained	Drained	Drained
γ_{unsat}	[kN/m³]	22	16	17	20.4
γ_{sat}	[kN/m³]	24	20	20	20.4
kx	[m/day]	1	1	1	1.1E-5
ky	[m/day]	1	1	1	1.1E-5
E_{ref}	[kN/m²]	60000	10000	35000	2.2E+5
ν	[-]	0.25	0.3	0.35	0.3
c_{ref}	[kN/m²]	1	1	1	25
ϕ	[°]	45	33	40	35
ψ	[°]	15	3	10	0
$R_{inter.}$	[-]	0.65	0.67	0.67	0.67

3.2.1.2 MSE Wall Facing

Plates are structural objects used to model slender structures in the ground with a significant flextural rigidity (or bending stiffness) and a normal stiffness. The input of stiffness parameters is completed by Poisson's ratio.

plate can be used to simulate the influence of walls extending in Z- direction. The vertical component of the MSE facing units were modeled using "plate" (beam) and it is able to sustain axial forces.

A footing is also provided at the base of the facing wall by using "plate" element. It is used for alignment purposes only and served no structural purpose. A concrete footing protection block was cast at the toe of the wall at the early stages of construction to prevent the base of the wall from significantly pushing out (Row and Skinner, 2001).

The block walls and the footing, both are made of concrete. The input parameters for plates are flextural rigidity (EI), Normal stiffness (EA), and element thickness.

Table 3.5 Plate data sets parameters

Identification	EA [kN/m]	EI [kNm²/m]	W [kN/m/m]	ν [-]
Diaphragm wall	5.00E+06	2.6041E+04	3.75	0
Footing	5.00E+06	8.50E+03	10	0

3.2.1.4 Geogrid

Geogrid are Structural elements often combined together to simulate the mechanical behaviour of real engineering structures and it is used to model soil reinforcements.

Geogrids are slender structures with a normal stiffness but with no bending stiffness, it sustain tensile forces and no compression force.
The only material property of geogrid is an elastic normal (axial) stiffness EA, which can be specified in the material data base.

In the calculation phases the geogrids can be activated or de-activated by using staged construction as loading input.

Table 3.6 Geogrid data sets parameters

Identification	EA [kN/m]	ν [-]
Geogrid	1100	0

3.2.1.5 Interface Element

Each interface has assigned to it is “virtual thickness” which is an imaginary dimension used to define the material properties of interface. The higher the virtual thickness led to generation more elastic deformation.

The virtual thickness should be small because interface element are supposed to generate very little elastic deformation. A typical application of interface would be to model the interaction between the sheet pile and the soil, which is intermediate between smooth and fully rough. The roghness of the interaction is modelled by choosing a suitable value for the strength reduction factor in the interface (R_{inter}).

Generally the effect of interface is a reduction of contact friction, thus enabling a more realistic modelling of the mechanical behaviour than a perfectly “glued” contact type, which would be what one obtains without introducing any interface. Therefore, interface elements allow relative displacements between structure and subsoil.

A typical value of R_{inter} = 0.67 is used for the fill soil.

3.2.1.6 Boundary Condition

On selecting standard fixities from loads menu PLAXIS 2D automatically impose a set of general boundary conditions to the geometry model.

PLAXIS 2D offers the “standard fixities” option In order to prevent horizontal displacements on the left and right side of the mesh and horizontal and vertical displacements at the bottom.

Figure (3.1) represent the modeling configuration used for MSE wall simulation and analysis. The model indicates geometry with all dimensiones, boundary (standard fixities) conditions, clusters, and structural objects (facing, foundatin, and reinforcement).

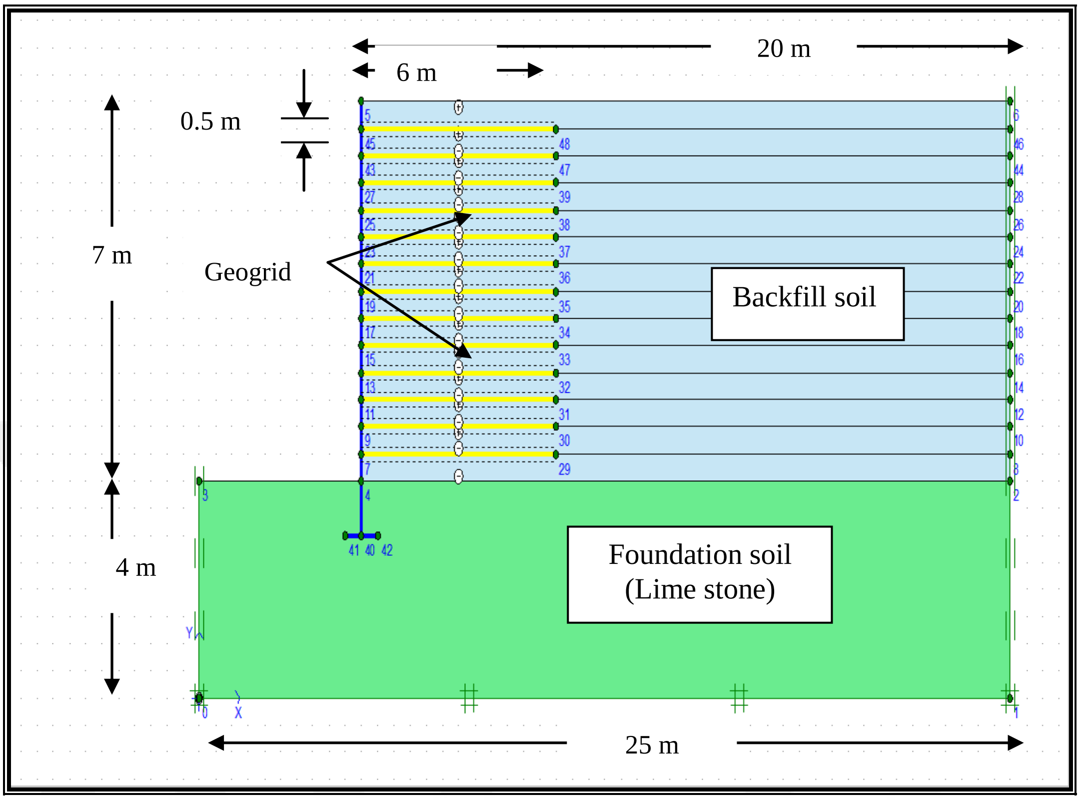


Figure 3. 1 Geometry Model with all structural elements

3.2.2 Creating and assigning data sets

Enter model parameters which include soil properties and material properties of structure as data sets, then sorted in a material database. The database sets of properties are assigned to the soil clusters or to the corresponding structural objects in the geometry model.

In modelling soil behaviour PLAXIS 2D supports various models to simulate the behaviour of soil such as the linear elastic model, Mohr-Coulomb model, jointed rock model, hardening soil model, soft soil model, soft soil creep model and other user define models.

3.2.3 Generating a finite element mesh

The material properties should be assigned to all structural objects and clusters after completing the model geometry, then performing the finit element calculations by dividing the geometry into finit element. A composition of finite elements is called a mesh. The basic type of element in amesh is 15-node tringular element or 6-node tringular element (Figure 3.2).

The 15 – node triangle is very accurate element that has produced high quality stress results for difficulte problems, but at the same time it led to slow calculation and high memory consumption.

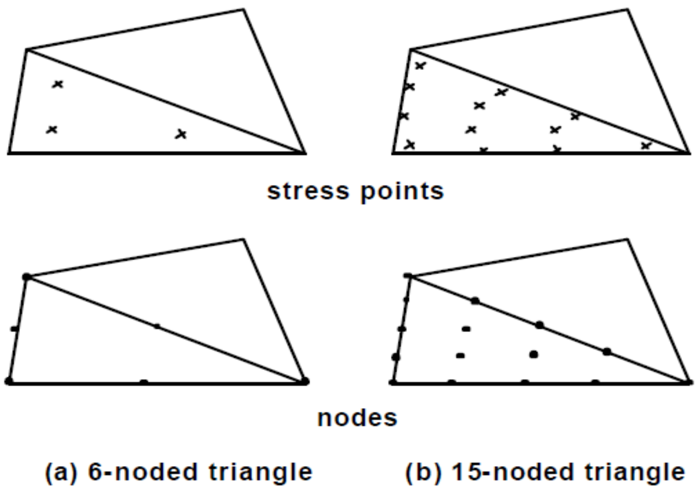


Figure 3.2 position of nodes and stress point in the soil elements
(http:// learnplaxis. blogspot.com.tr)

Meshing is very important in obtaining a realistic result. In PLAIS 2D version 8 software the user should to choose one from five types of meshing: very coarse, coarse, medium, very fine and fine.

A too coarse meshing is unfavorite choice because it is fails to capture the subtle changes in the stresses generated in different parts of the medium, especially at the stress concentrations points. On the other hand, a too fine mesh is uneconomic because it is consume very long time to complete calculation process. Hence, a trade-off is required to obtain an approximately accurate solution in a reasonable time.

PLAXIS 2D version 8.0 allows simulation of a fully automatic mesh generation for finite element analyses.

The two vertical boundaries were free to move vertically and were considered to be fixed in the horizontal boundary direction. The foundation soil is not considered in this analysis because it was assumed to be stiff soil. Therefore, the bottom boundary has been modeled as fixed boundary. Mesh generation are illustrated in Figure (3.3).

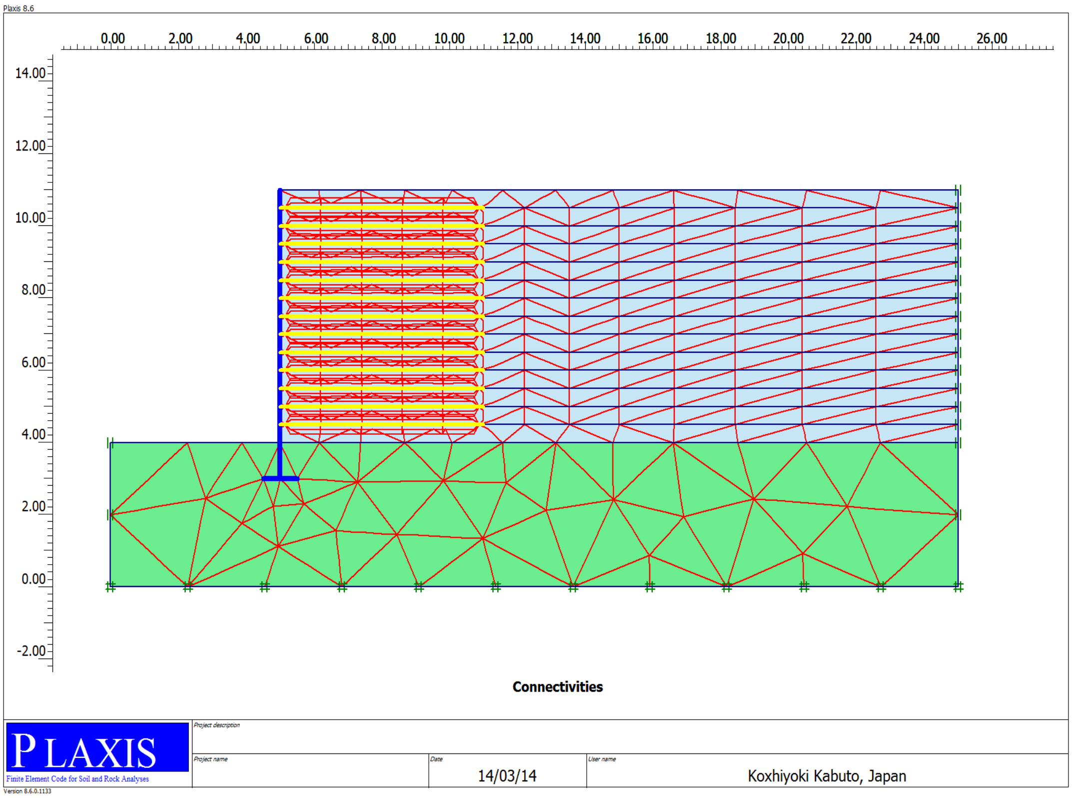


Figure 3.3 Generated Mesh at initial condition

3.2.4 Generation initial Pore Water Pressure

The water pressure is generated on the basis of phreatic level. The geometry for all cases of backfill soil does not involve water pressure, which means the presence of the water table was not observed during soil analysis.

Therfore, a phreatic level is automatically placed at the bottom of the geometry. The resultant generated pore pressure distribution is shown in Figure (3.4).

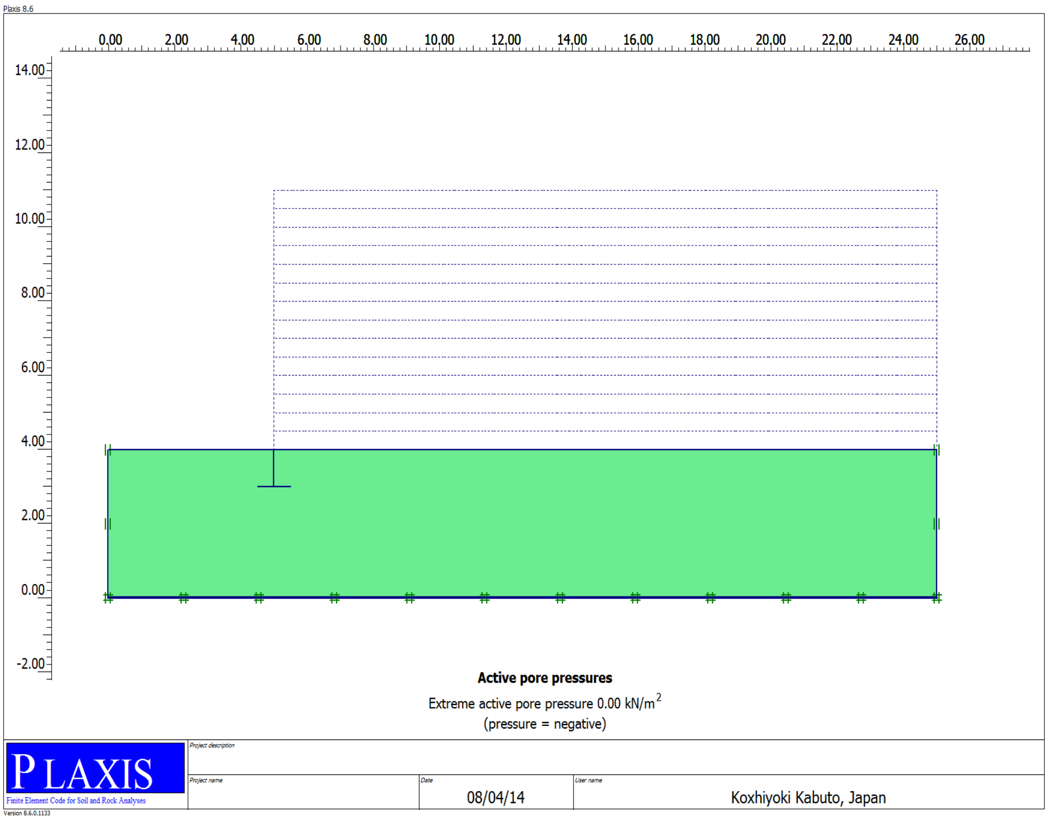


Figure 3.4 Active Pore Pressure

3.2.5 Generating initial conditions

Once the geometry has been created and finite element mesh has been generated, the initial stress state and the initial configuration must be specified. This is done by the initial conditions part of the input program. Finite element model used in this analysis is plain strain model.

Displacements and strains in z-direction are assumed to be zero. However, normal stresses in z direction are fully taken into account. The 15-node triangle, used in this study, is a very accurate element that has produced high quality stress results for difficult problems.

The initial stress forming in soil body is influenced by the history of the soil formation and by the weight of the material. This stress in characterized by initial vertical effective stress. The initial horizontal effective stress is related to initial vertical effective stress by coffitient of lateral earth pressure.

In PLAXIS 2D, the effective initial stress analysis is done by K_0 procedure. The result after the development of initial effective stress is as illustrated in Figure (3.5).

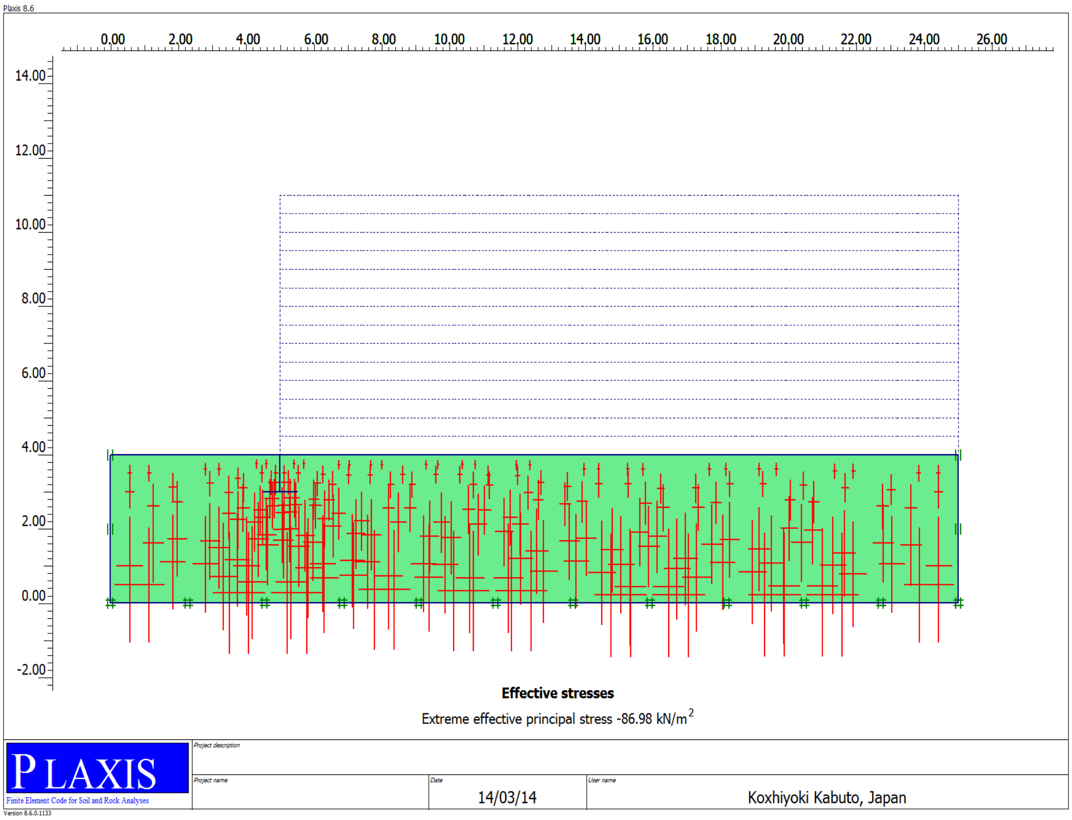


Figure 3.5 Effective Stress Distribution

CHAPTER IV

RESULTS AND DISCUSSION

4.1 Numerical Analysis for MSE walls

Finite element analysis was carried out using commercial software PLAXIS 2D version 8.0. Seven different parametras are studied to investigate their effecte on Horizontal displacements of the wall face and Axial force development on geogid layers. The results are compared and reported in this chapter.

4.1.1 Deformed Mesh

Figure (4.1) shows deformed mesh. The performance of the wall in the software is basically depends upon the mesh data in which the project has been generating. The figure clearly shows the outward displacement of the facing wall and the bend shape of geogrids which is scaled to ensure that the deformation are visible.

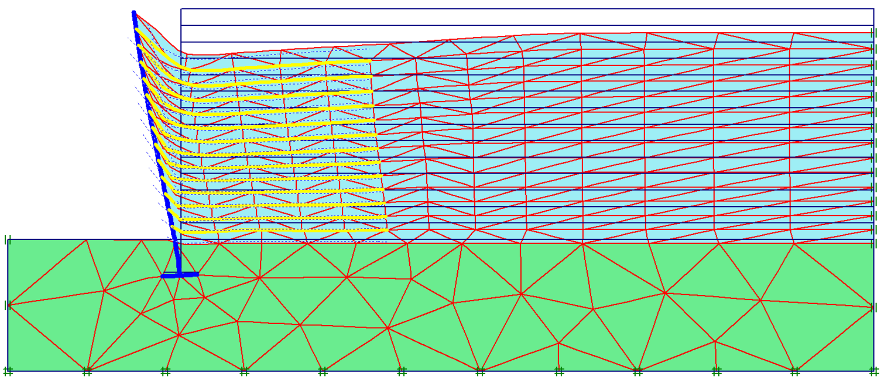


Figure 4.1. Deformed Mesh

4.1.2 Displacement

Figure (4.2) shows the total displacement of all nodes as arrows, which is an indication of their relative magnitude and direction. The direction of displacement is indicated by red arrows and the length of each arrow represents the magnitude of the displacement at the corresponding point.

Figure (4.3) shows contour lines of the total displacement which is lablled. Each color in contour plot corresponds to limited range of displacement which is presented at the right of the figure by index. Red color represent the highest displacement zone, when there is decrease in the displacement, the color of the legend changes gradually from red to blue. The blue colored zone represents the zero displacement area.

Figure (4.4) show color shading of the total displacement. An index is presented with the displacement values at the color boundaries.

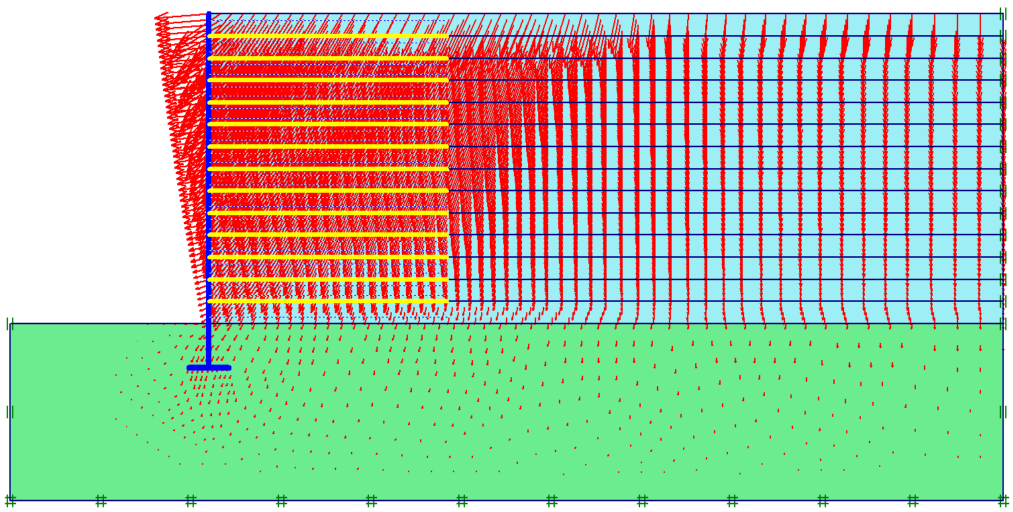


Figure 4.2 Total displacement as Arrows

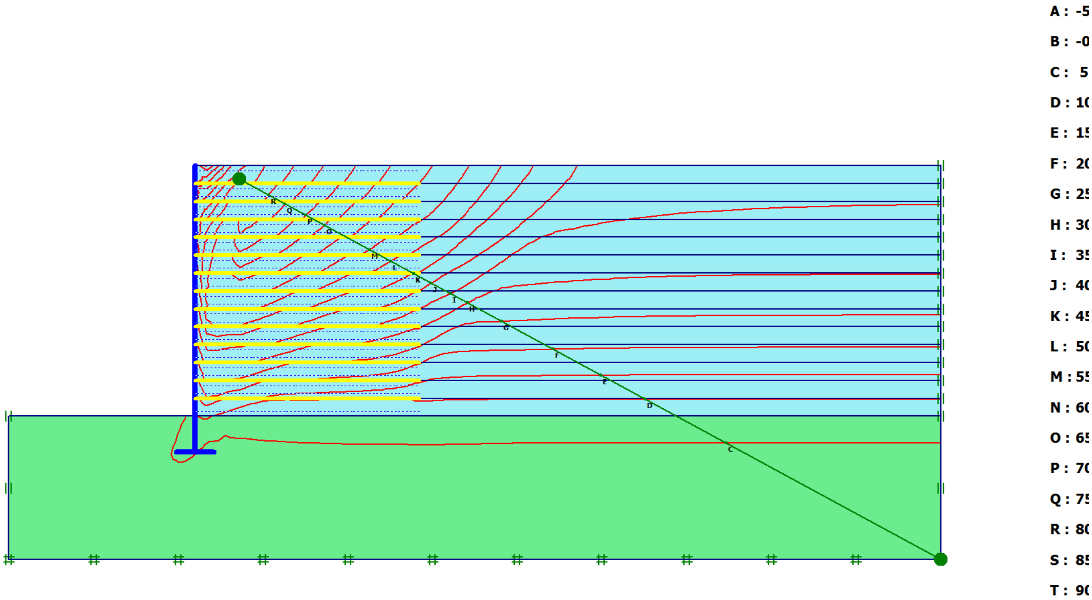


Figure 4.3 Total displacement as contour lines

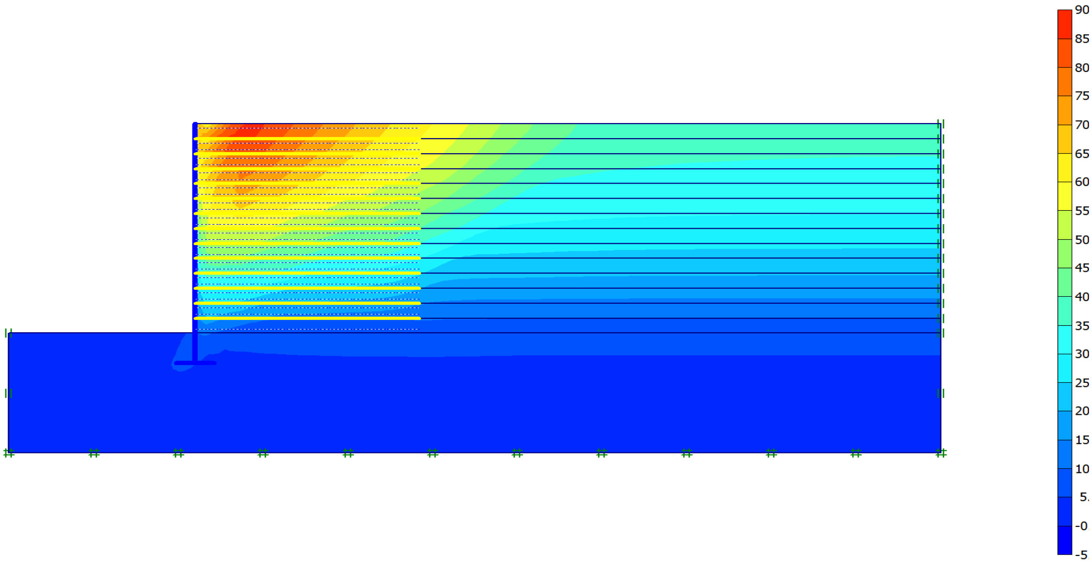


Figure 4.4 Total displacement as color shading

4.1.3 stresses on the soil

Figure (4.5) show the total effective principal stresses. The stress is increases with the depth of wall.

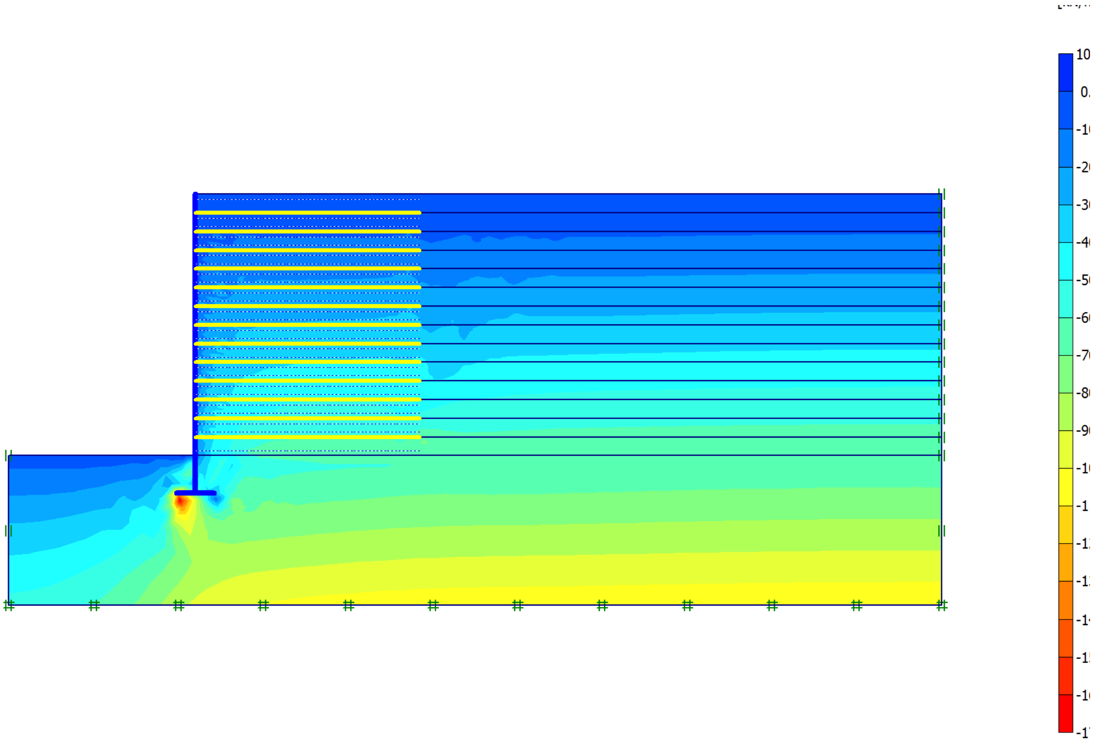


Figure 4.5 Effective mean stresses

4.1.4 Movement of the Facing wall

Figure (4.6) presents the displacement and forces acting on the facing wall. The displacement, which is specifies the change in position of a point in reference to a previous position. In simple terms, it's the difference between the initial position and the final position of an object.

Figure 4.6 (a) shows the total displacement along the facing wall. The maximum total displacement occurs at the top point of the facing wall. As the depth of facing wall increases, the value of total displacement decreases.

Figure 4.6 (b) & (c) present the vertical and horizontal movement of wall respectively.

The maximum horizontal displacement is found at the top point of the wall and as depth increases, the displacement reduces. The movement is very small and the direction of movement is opposite at the bottom portion of wall, which is pushed down under the natural soil.

Figure 4.6 (d) indicate the axial force diagram along the facing wall.

The maximum value of axial force occurs at the lowest point of the wall just above the foundation soil.

Figure 4.6 (e) & (f) indicate the shear force and bending moment diagram along the facing wall.

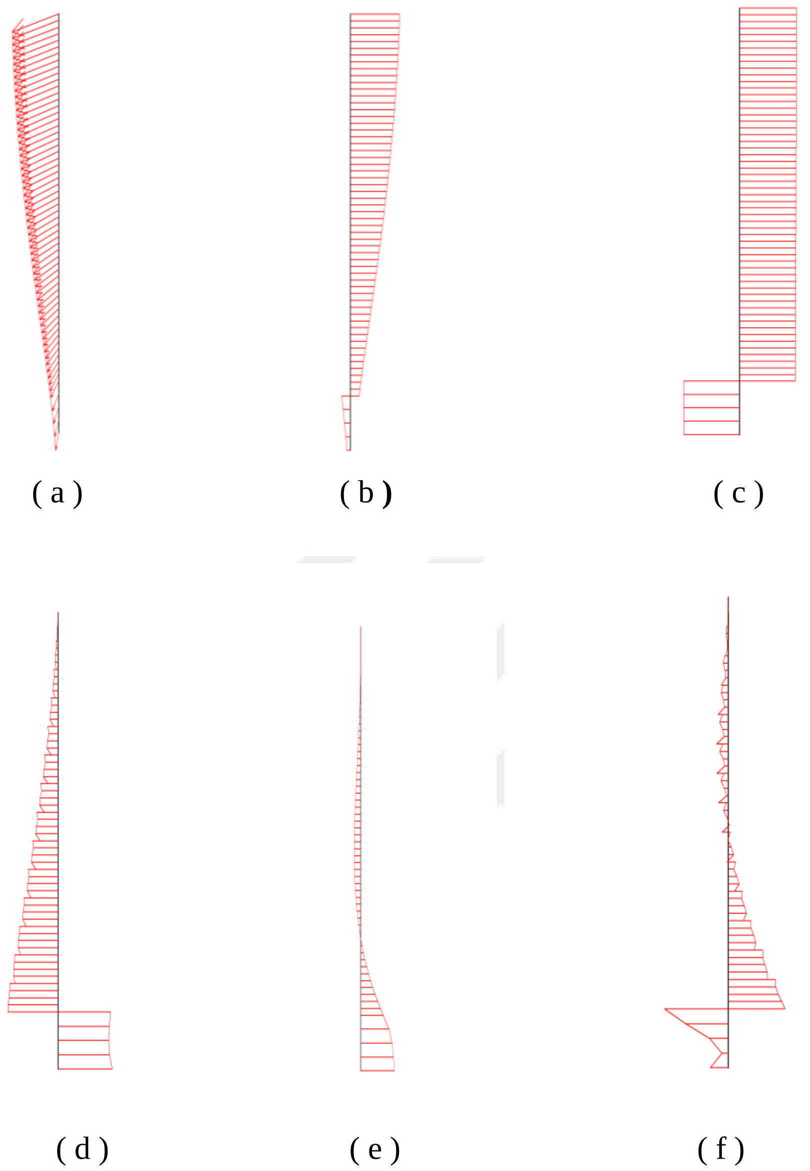


Figure (4.6)

(a) Total displacements (b) Horizontal displacements (c) Vertical displacements
(d) Axial force (e) Bending moments (f) Shear force

4.1.5 Analysis of Geogrid Behavior

Geogrids are placed in horizontal layers to unify the mass of the composite MSE wall structure to increase the resistance of the wall to the destabilizing forces generated by the retained soils and surcharge loads.

To achieve a composite MSE wall structure, geogrids must possess adequate tensile strength, be placed in sufficient layers, and develop sufficient connection and anchorage capacity to hold the composite MSE structure together.

Figure (4.7) represent the axial force in geogrid. Figures (4.8), (4.9), (4.10) represent the Total, Horizontal, and Vertical displacement in geogrid. The displacements generally increase toward the top of the wall and it decreasing with increasing the depth below the top of the wall.

Due to high confining pressure behind rigid facing, the location of the overall reaction force becomes closer to the facing.

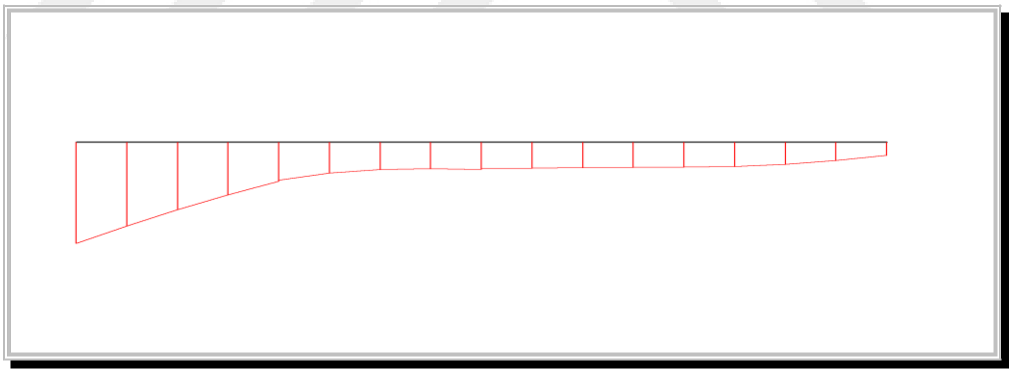
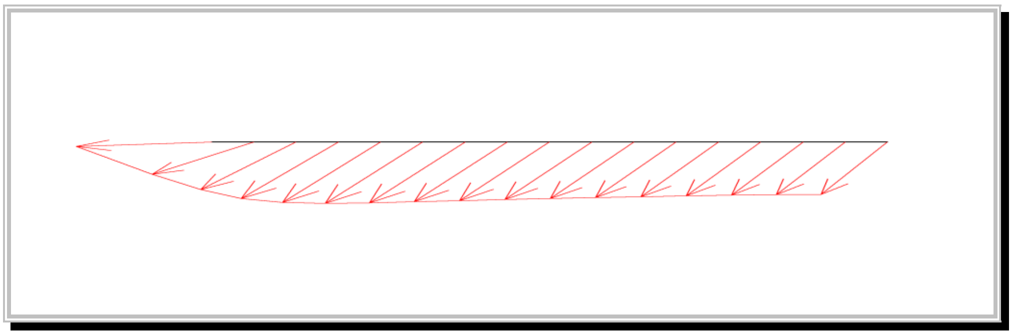


Figure 4.7 Axial force in geogrid



Figurre 4.8 Total displacement in geogrid

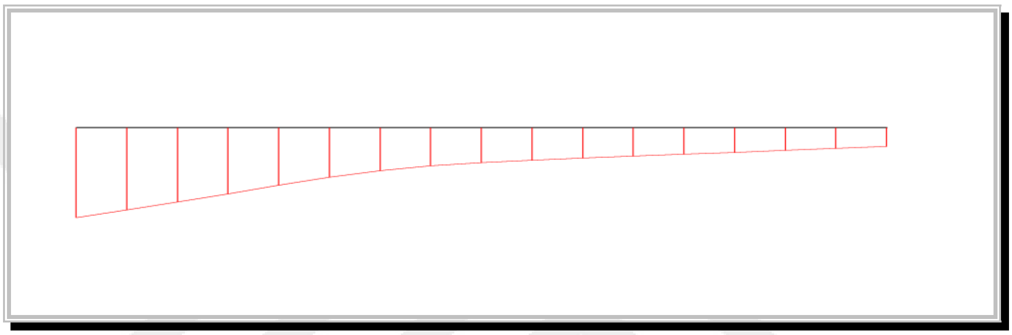


Figure 4.9   Horizontal displacement in geogrid

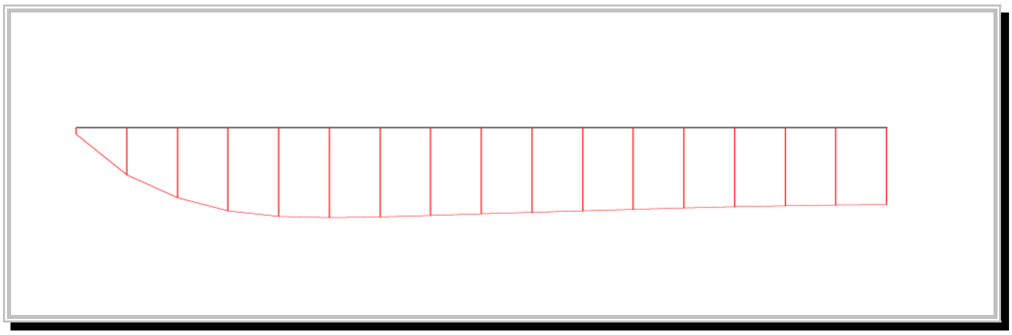


Figure 4.10 Vertical displacement in geogrid

4.2 Parametric studies

The most common parameters which effect on the behaviour of MSE wall are studied in this current research and they are presented in Table 4.1.

Different properties of geogrid (length, spacing, and strength) and different position of traffic surcharge load in the reinforced zone behind the facing of MSE wall. Different value of the wall hight and the facing thickness, all of these parameters analysis with three types of backfill soil loose sandy soil, dense sandy soil, and clayey soil. An adequate amount of graphs and charts are provided to represent the effect of various parameters on the final extreme horizontal displacement of the MSE wall and the axial force in geogrid.

The resultes are compared under different conditions. These values are taken as they are the typical values that are generally used to design MSE wall.

Table 4.1 Presentation of different Parameters and their typical values used in this study

NO.	PARAMETERS	UNITS	VALUES USED IN THE STUDY	NOTES
1	MSE Wall height (H)	m	4, 7, 10	_
2	Geogrid length (L/H) ratio	_	0.3, 0.5, 0.7, 1, 1.3	This analysis was conducted for three wall height H = 4, 7, 10 m
3	Geogrid strength (EA)	kN/m	1000, 3000, 6000	This analysis was conducted for three wall height H = 4, 7, 10 m
4	Geogrid vertical spacing (Sv)	m	0.3, 0.5, 0.7	_
5	Face element thickness (D)	m	0.25, 0. 30, 0.35, 0.4	_
6	Traffic surcharge load	kN/m^2	20	_
7	Angle of internal friction (φ)	[°]	30, 33, 40, 42	_

4.2.1 Effect of Wall Height (H) on Horizontal Displacement & Axial Force in Geogrid

Design Parameters:

The height of the wall (H) are 4, 7, and 10 m, geogrid stifness (EA) = 1100 kN/m, vertical spacing between geogrid layers (Sv) = 0.5 m, face element thickness (D) = 0.25 m, Reinforcement length to height (L/H) ratio = 0.7.

This analysis was carried out for three different type of backfill soil, in order to see the effect of wall height on horizontal displacement and axial force in the reinforcement layers.

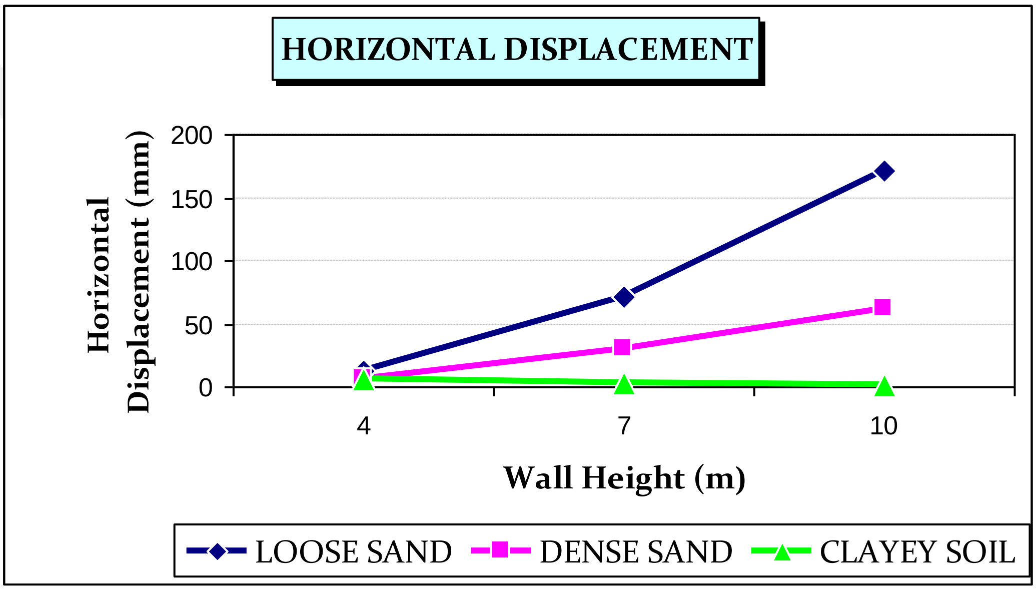


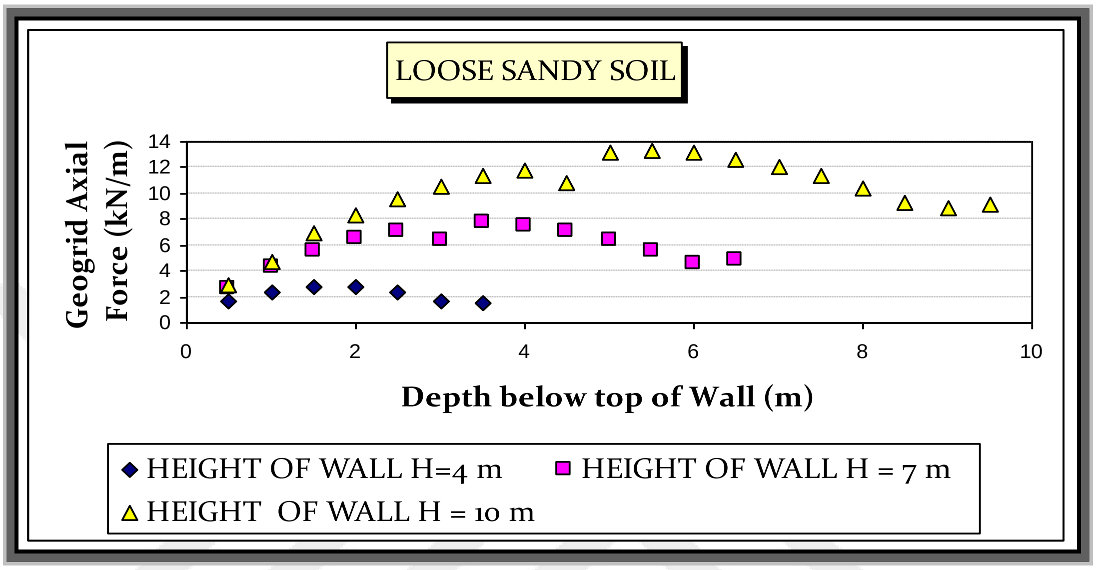
Figure 4.11 Effect of wall height and backfill soil type on horizontal displacement

Figure (4.11) shows the effect of wall height and back fill soil type on horizontal displacement of MSE wall. As can be seen from the Figure, horizontal displacement increase with increasing the height of the wall for loose and dense sandy soil.

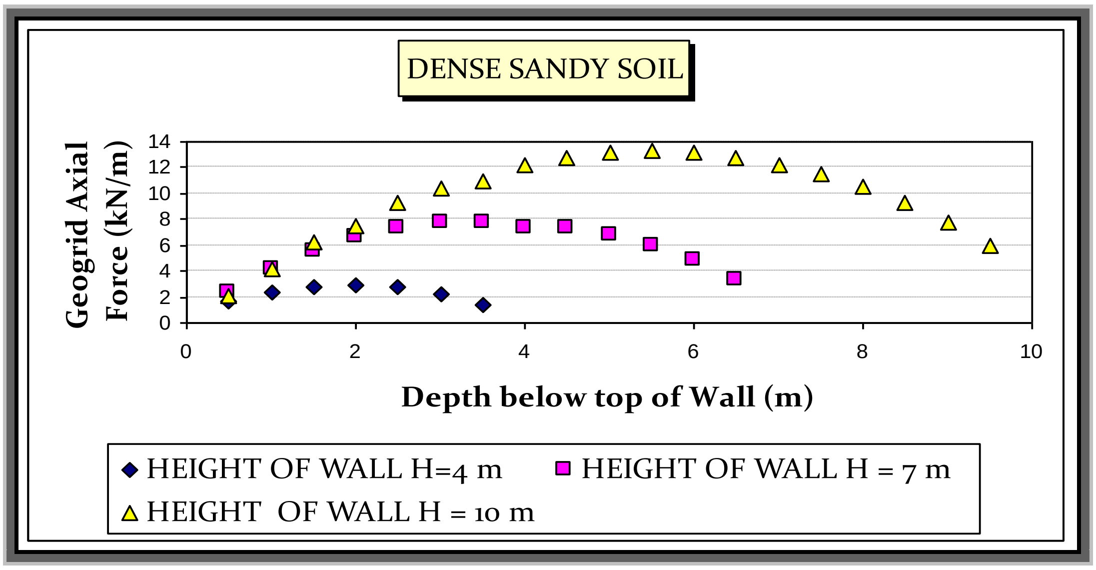
In granular backfill soil increasing the value of friction angel will reduce the displacement; because stronger backfill soil will give more friction between the geogrid and soil interface, which will reduce the movement of wall. Internal friction angel (φ) between soil and geogrid depends on soil friction angel. So, when φ increases, (φ_{inter}) also increases; which results in decreased displacement of the

MSE wall. Reduction in displacement is around 58% when the percent change in internal firiction angle is 21%.

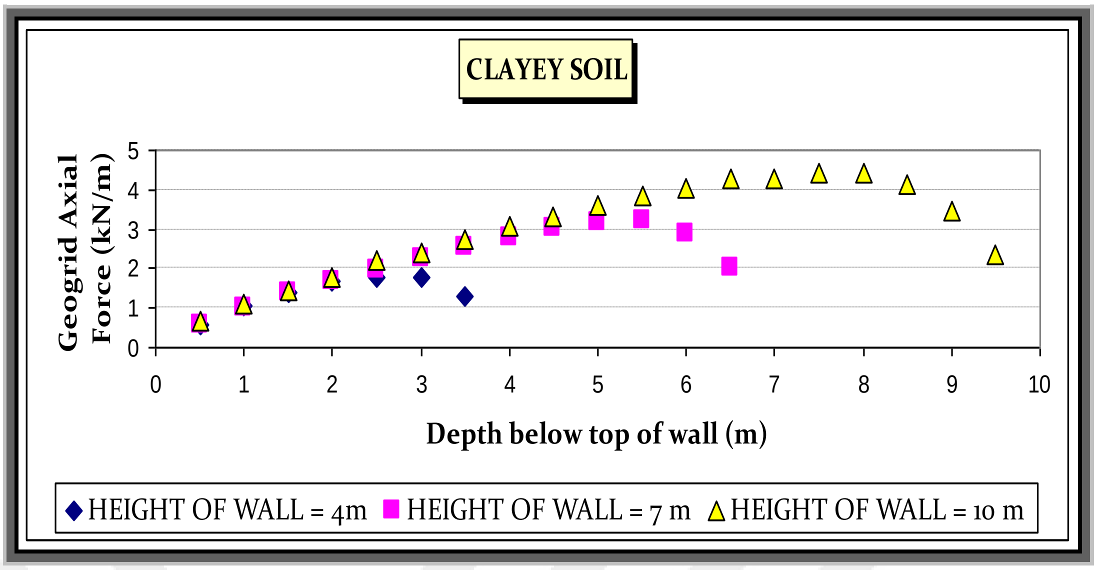
It is interesting to note that change in the height of the the wall has no effect on the face displacement of the wall for calyey soil. This may be due to effect of cohesion of the calyey soil that hold soil together without reinforcement to a cerain height.



(a)



(b)



(c)

Figure 4.12 Effect of wall height on geogrid axial force in: (a) loose sandy soil, (b) Dense sandy soil, (d) clayey soil

Figures 4.12 (a), (b), (c) show the change in the axial force on geogrid layers for different height and soil types. Generally, the axial force developed in geogrid layers increase with increasing the height of the wall.

For garanular backfill soil, maximum force was developed at the mid hight of the wall independent of the internal friction angle. For clayey backfill soil the position of the reinforcement layer on which maximum tensile force occurs is located at an elevation nearby 25 percent of the total height (measured from the base of the wall).

4.2.2 Effect of Reinforcement Length to Height (L/H) on Horizontal displacement & Axial force in Geogrid

Design Parameters:

This analysis was conducted on a MSE wall with height (H) = 4, 7, and 10 m, geogrid stifness (EA) = 1100 kN/m, vertical spacing between geogrid layers (Sv) = 0.5 dm, face element thickness (D) = 0.25 m, reinforcement length to height (L/H) ratio = 0.3, 0.5, 0.7, 1, and 1.3.

This analysis was carried out for three different type of backfill soil, in order to see the effect of (L/H) ratio on horizontal displacement and axial force in the reinforcement layers.

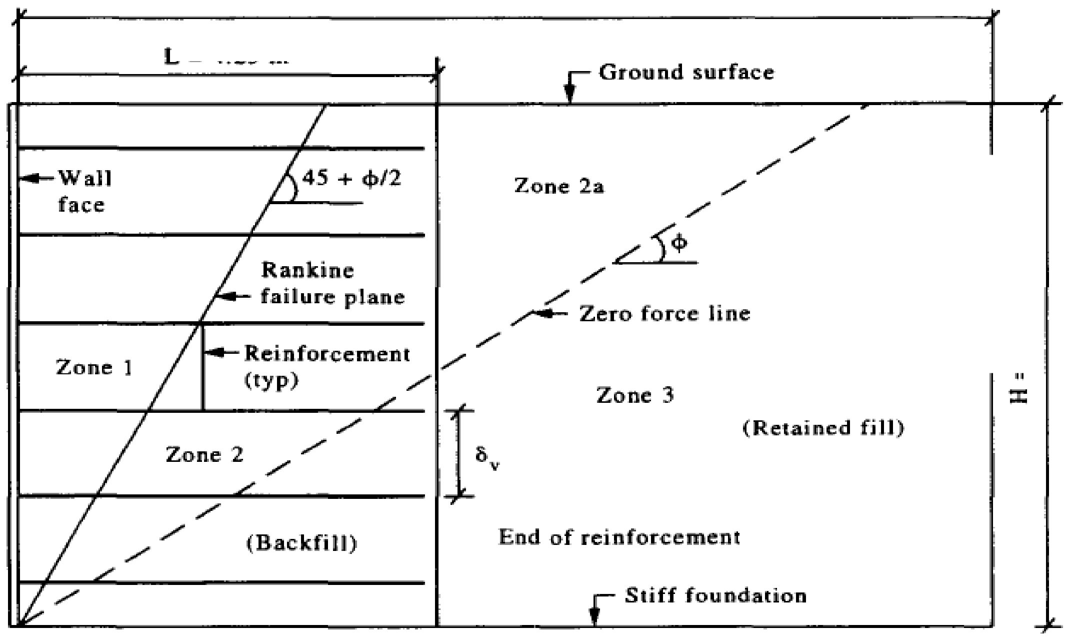
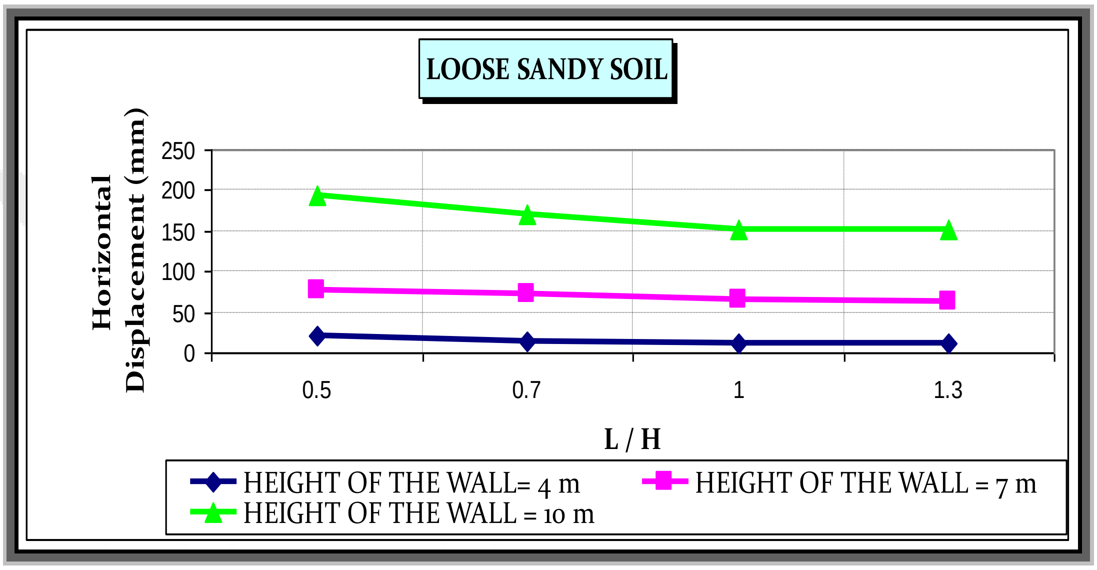
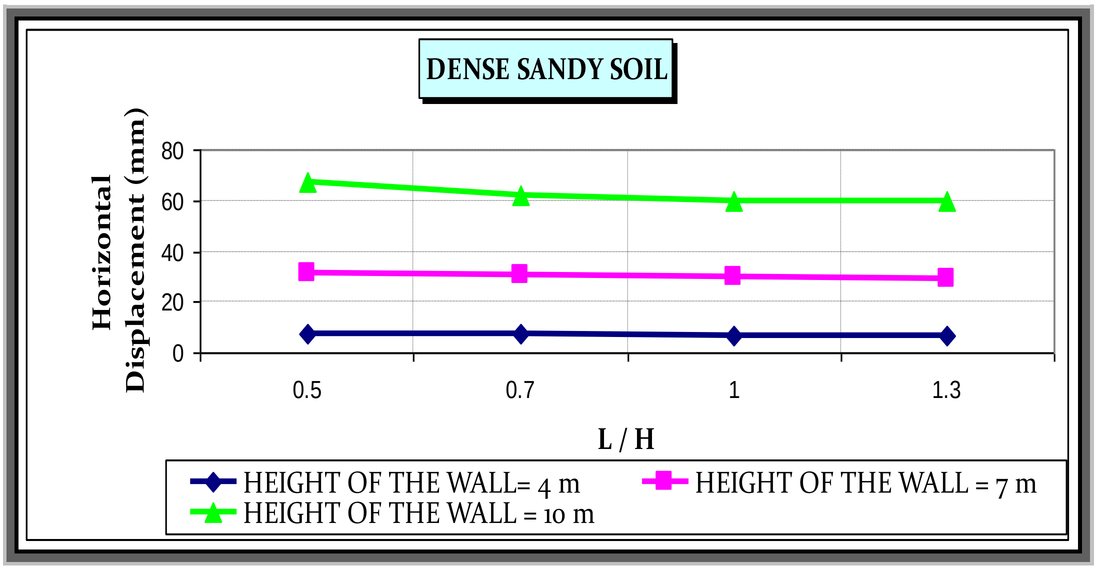


Figure 4 . 1 3   Rankine active failure zone

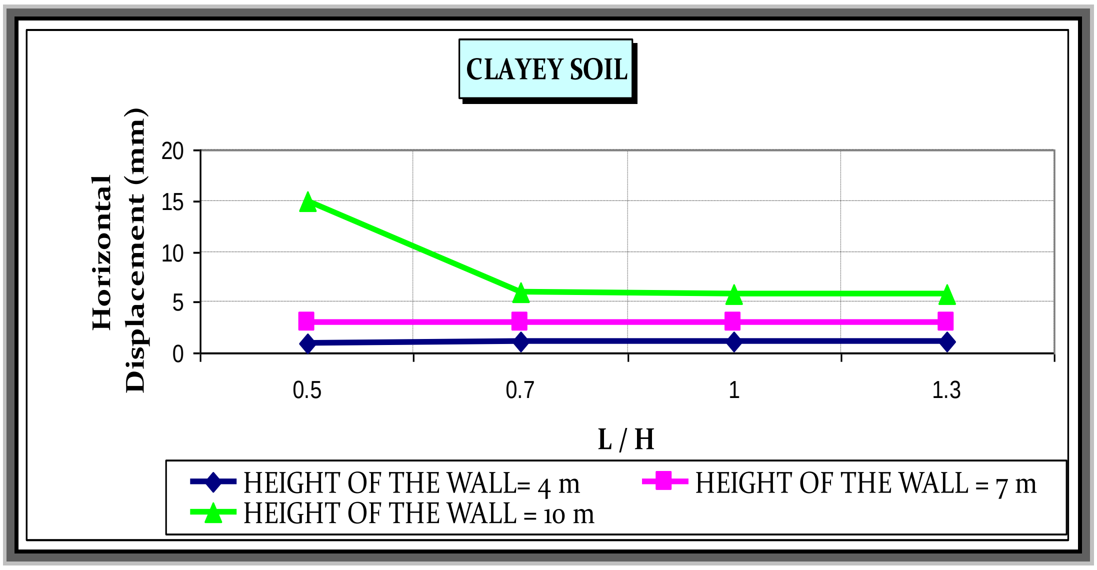
The effective length of the geogrids along which frictional resistance is developed may be conservatively taken as the length that extends beyond the limits of the Rankine active failure zone (Figure 4.13), which makes an angel of (45 - Φ/2) with the vertical. A reduction in the horizontal displacement occurred when the length of the reinforcement extended beyond the Rankine failure plane. So when geogrid length increases (the length beyond the failure line increases), it will gives more frictional resistance with the backfill soil.



(a)



(b)



(c)

Figure 4.14 Effect of reinforcement length on horizontal displacement in (a) Loose sandy soil, (b) Dense sandy soil, (c) Clayey soil

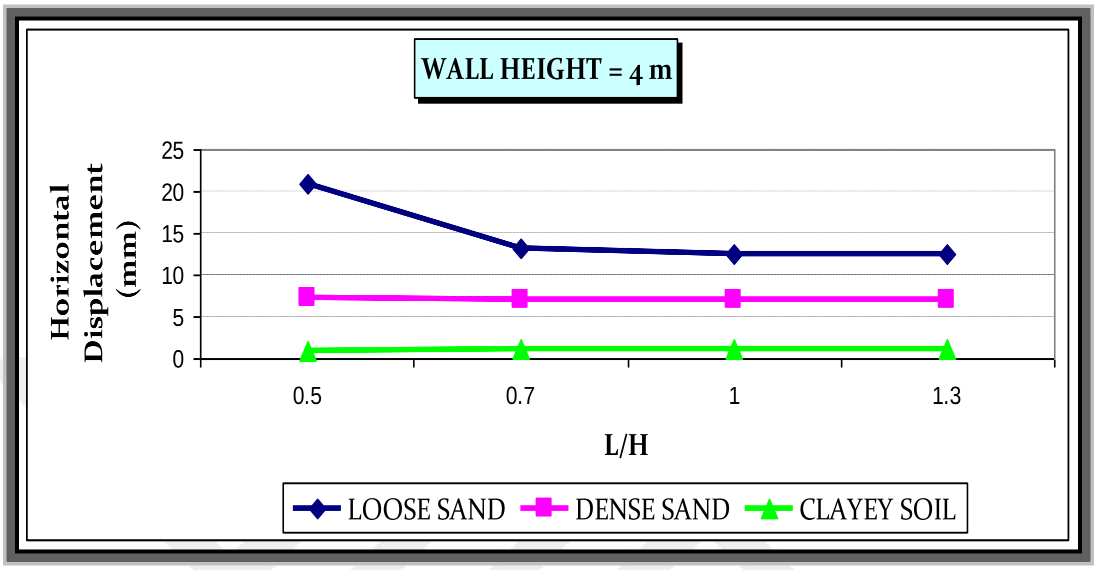
Generally, the results show the maximum wall deformation increases as the reinforcement length is reduced from 0.7 to 0.5.

In case of loose sand, a significant reduction in horizontal movement occurred for an increase in L/H ratio from 0.5 to 0.7 for H = 7 m & 10 m. When L/H increased from 0.7 to 1 avery small decrease observed in wall deformation especially for H= 4 m &7 m. There is no observed change in horizontal displacements when L/H > 1, as show in Figure 4.14 (a).

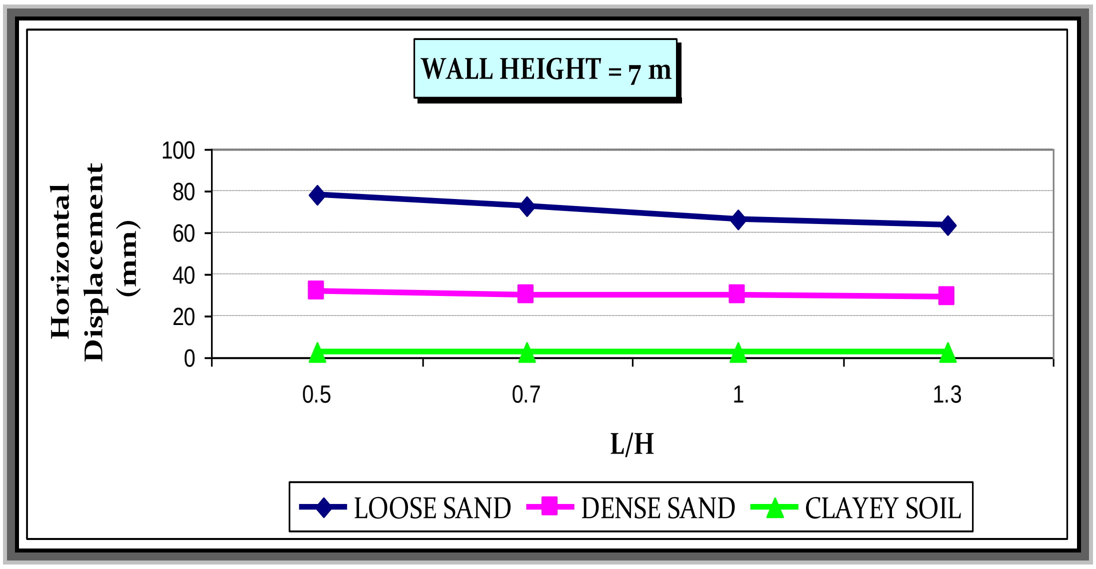
In case of dense sand, for wall height H = 10 m, the horizontal displacement decrease when L/H change from 0.5 to 0.7, while there is small change in horizontal displacements when the L/H > 0.7. Increasing L/H ratio have no significant effect on horizontal displacement in H= 4 & 7 m, the displacements neither increase nor decrease and this shows that the soil has reached its state of equilibrium as shown in Figure 4.14 (b).

In clayey soil there is no effect of geogrids length on displacements as shown in Figure 4.14 (c). The results of FEM analysis show a good agreement with the results of the studying conducted by Kibria et al., (2014); Mahmood (2009).

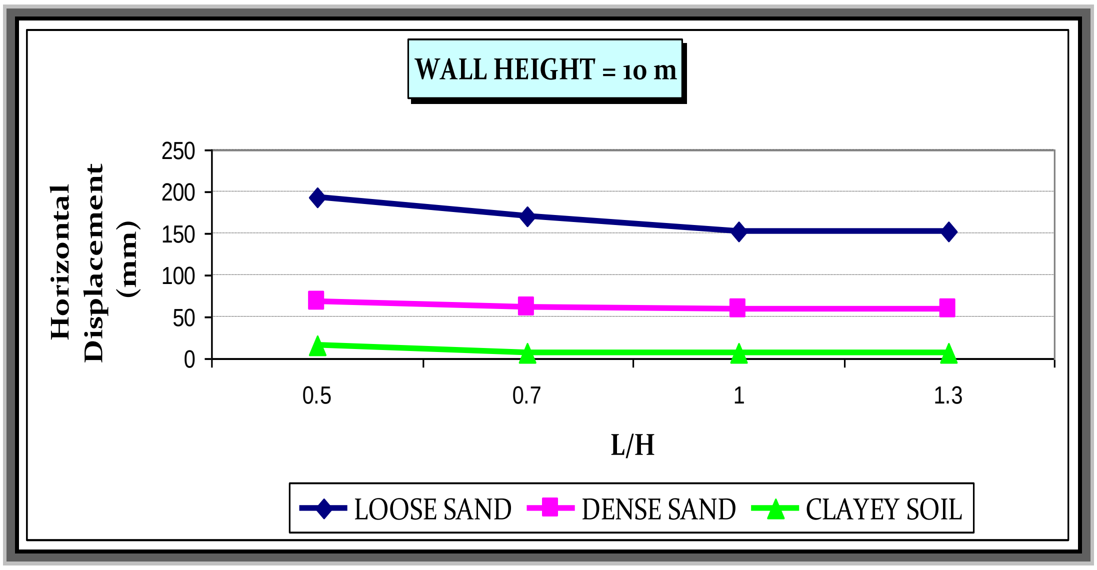
The analysis performed by Kibria et al., (2014) on MSE wall with height equall to 4, 8, and 12 m, the ratio of reinforcement length to wall height 0.3–1.0. The results showed that a significant reduction in horizontal movement occurred for an increase in L/H ratio from 0.5 to 0.7.



(a)



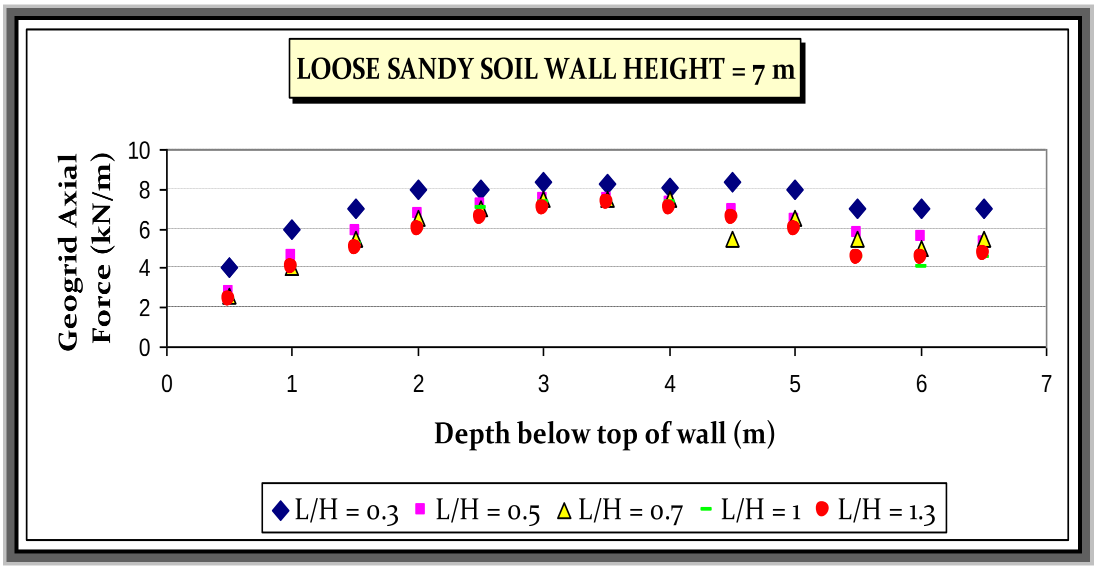
(b)



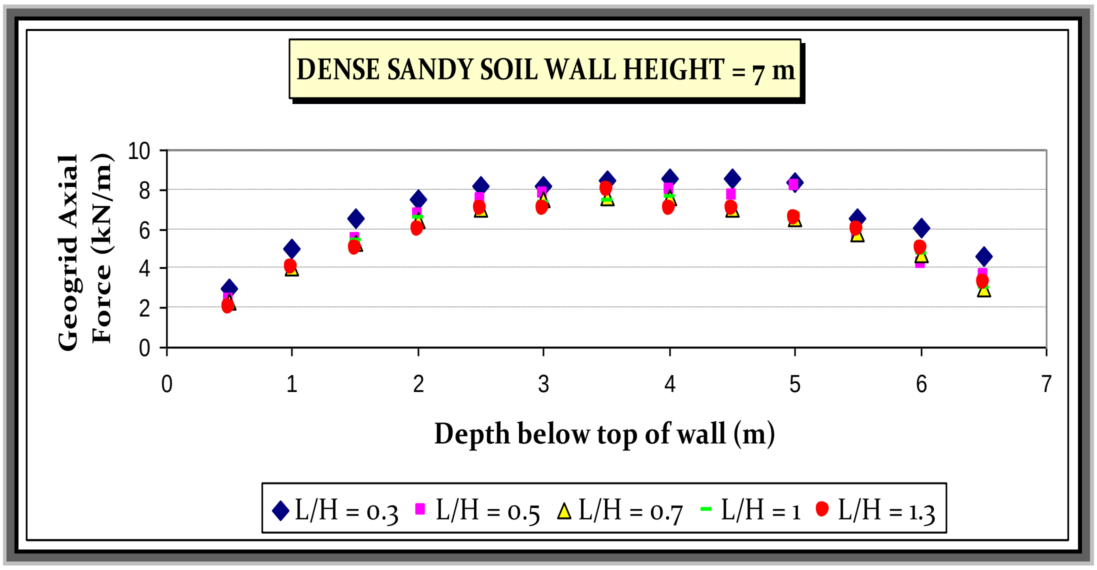
(c)

Figure 4.15 Effect of reinforcement length and type of reinfrced soil on horizontal displacement for (a) H= 4 m, (b) H= 7 m, (c) H= 10 m

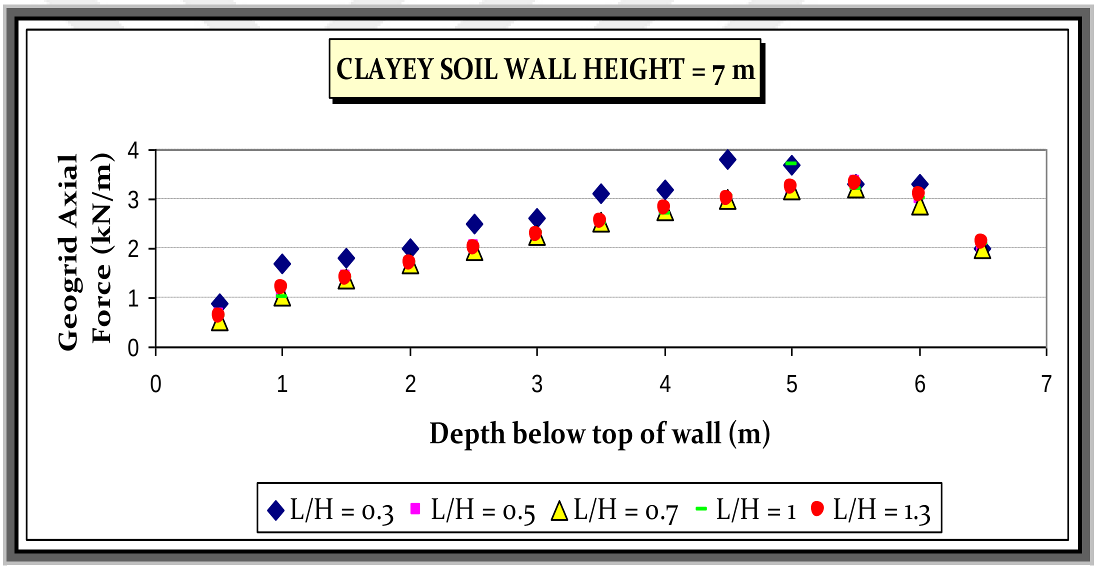
As shown in Figures 4.15 (a), (b), (c) geogrid length shows significant effect in case of loose sand even in case of H = 4 m, which means that deformation increase with decrese length of geogride and angle of internal friction for backfill soil in reinforcement zone. Generally, The choice of 70% of height for minimum reinforcement length is one of design specification and the designer's preference. There is considerable evidence that walls experience greater deformation with shorter reinforcement "L/H" ratios (L/H< 0.7).



(a)



(b)



(c)

Figure 4.16 Effect of reinforcement length on geogrid axial force for (a) Loose sandy soil, (b) Dense sandy soil, (c) Clayey soil

As shown in the Figures 4.16 (a), (b), (c) when L/H is greater than about 0.7, there is no variation in the geogrid axial force and when the ratio L/H is decreased below 0.7, there is a slight increase the geogride axial force. The same results was observed in the study conducted by Ho and Rowe (1996).

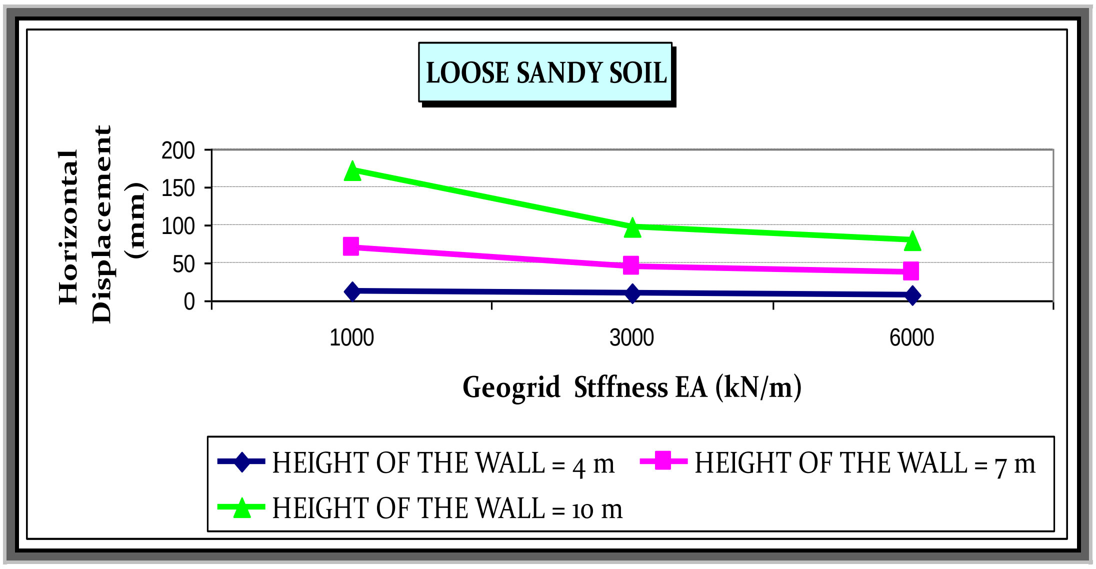
According to Ho and Rowe (1996), when the ratio L/H is decreased below 0.7, there is a substantial increase in all the forces required for equilibrium. The lateral thrust pushes the reinforced soil block from behind and produces a moment about the toe of the wall, inducing higher vertical and horizontal stresses towards the front of the wall. The vertical stress may increase beyond the theoretical overburden value (i.e. $\sigma_v = \gamma$ h) despite the presence of the resistance from facing/soil interface friction.

4.2.3 Effect of Reinforcement Stiffness (EA) on Horizontal Displacement & Axial Force in Geogrid

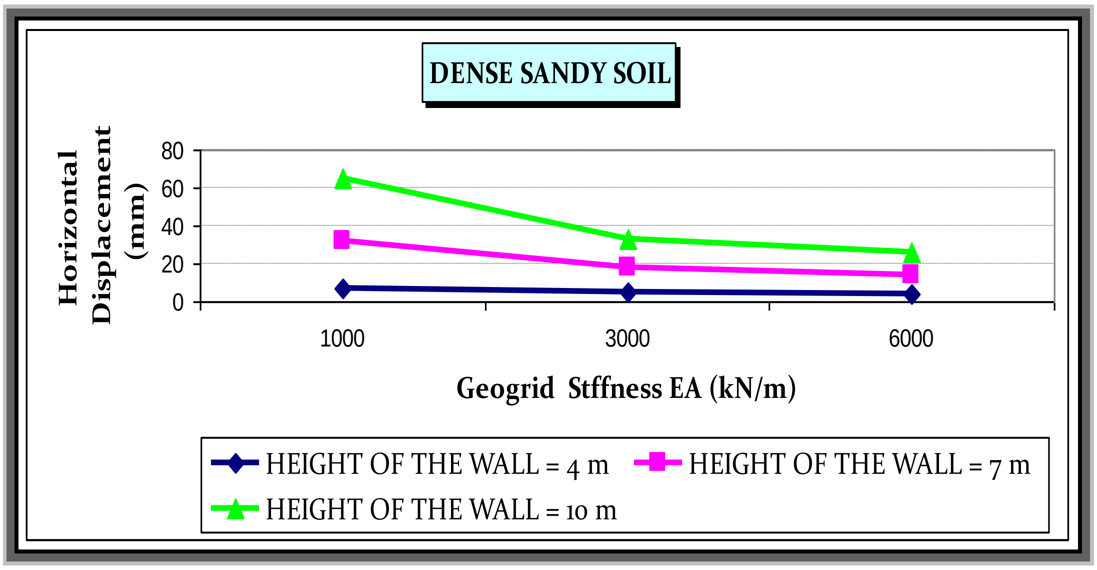
Design Parameters:

This analysis was conducted on MSE wall with height (H) = 4, 7, and 10 m, geogrid stiffness (EA) = 1000, 3000, and 6000 kN/m, vertical spacing between geogrid layers (Sv) = 0.5 m, face element thickness (D) = 0.25 m, reinforcement length to height (L/H) ratio = 0.7.

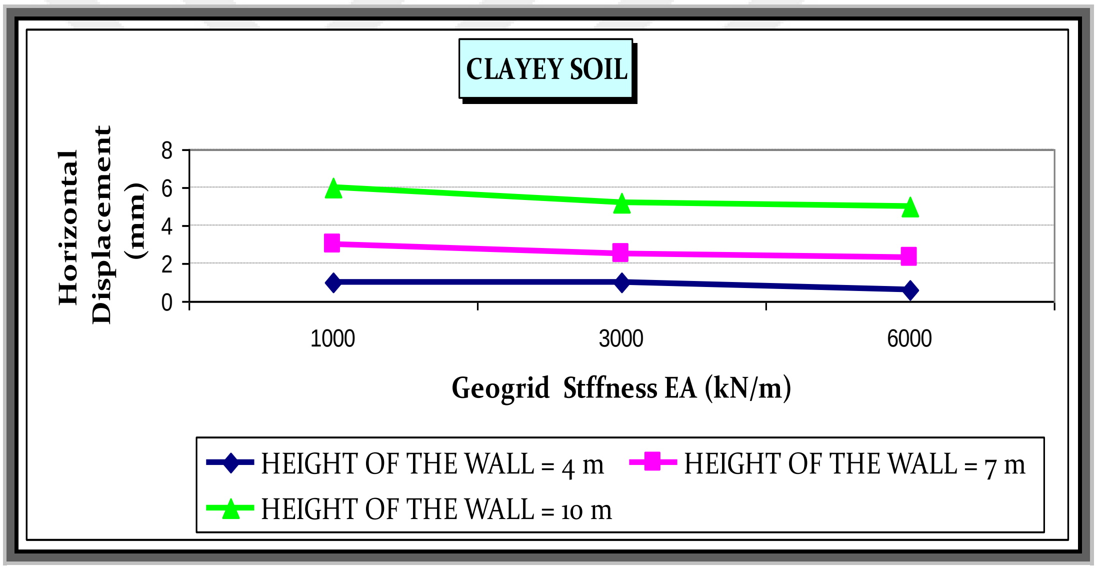
This analysis was carried out for three different type of backfill soil , in order to see the effect of geogrid stiffness on horizontal displacement and the axial force in the reinforcement layers.



(a)



(b)



(c)

Figure 4.17 Effect of geogride stiffness and wall height on horizontal displacement in: (a) Loose sandy soil, (b) Dense sandy soil, (c) Clayey soil

The variations in horizontal displacement with reinforcement stiffness were identified for three MSE wall height H = 4, 7, and 10 m .

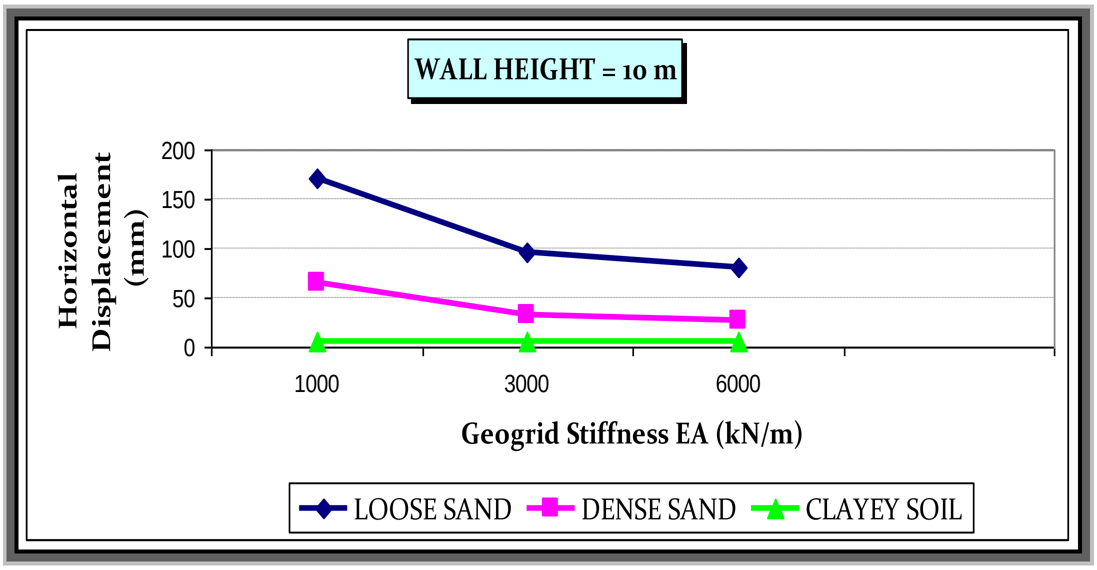
The illustrated results shows that horizontal displacement decreased with an increase in the reinforcement stiffness for all reinforcement wall height.

The results observed from figures 4.17 (a), (b), (c) are Similar to the results which have been reported by Kibria et al., (2014); Oyebile (2011).

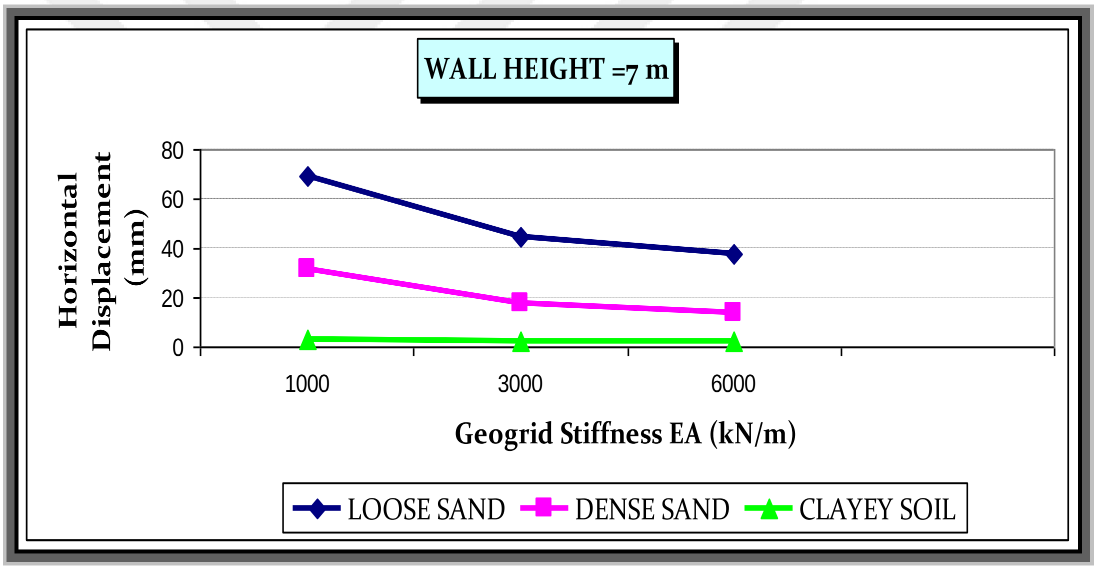
The reinforced soil wall studied by Oyebile (2011), provides support to the lower side of the Egnatia motor park. Awide range of geogrid stiffness used in this study (range between 1.00E+02 to 1.00E+15 kN/m). A previous study conducted by this author indicated that the horizontal displacements changes immediately uponfurther reduction in the stiffness of the geogrids even though the rate of change is smaller compared to the preceding rate of change in the stiffness of the geogrids. This result shows a wide range of values of geog`rids stiffness for steady and stable displacements of the reinforced wall.

As shown in Figures 4.17 (a), (b) it was observed that the effect of stiffness was not significant at a wall height of 4 m, but there is a substantial decrease in horizontal displacement in H= 7 m & 10 m. The range of movement decreased from 172 to 55mm in case of loose sand, and from 65mm to 18 mm for case of dense sand, for an increase of stiffness from 1000 to 30000 kN/m, at H = 10 m. Horizontal deformation of the wall increased significantly at a stiffness lower than 3000 kN/m.

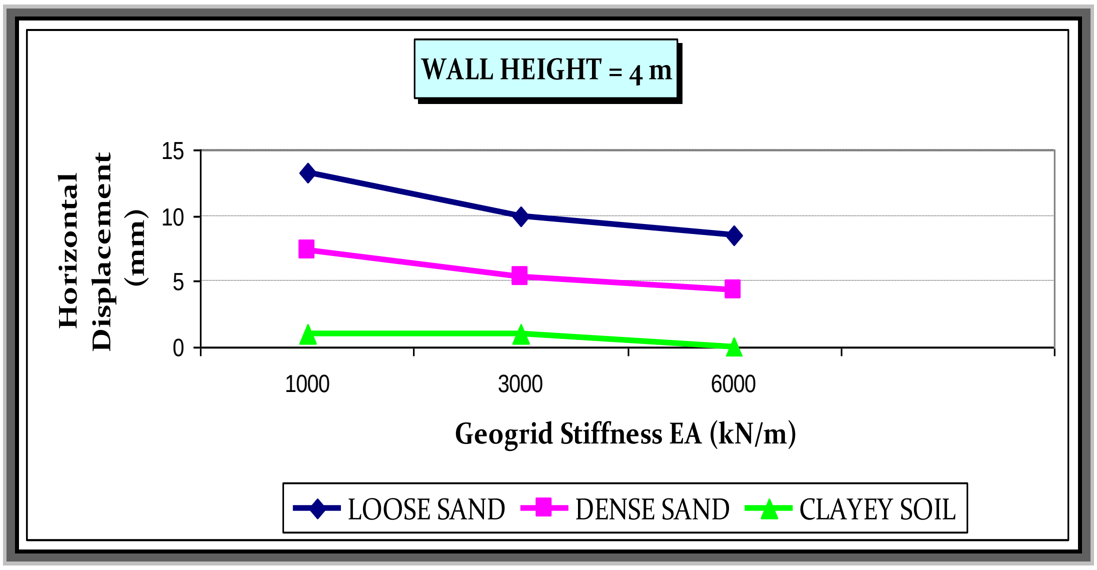
In clayey backfill soil Figure 4.17 (c), a substantial change in horizontal displacement had not occurred at stiffness greater than 3000 kN/m. the effect of stiffness was not significant at a wall height of 4 m.
Cohesive soil always allow less displacement. But Cohesionless soil with the same strength shows relatively large displacement.


WALL HEIGHT = 10 m
Horizontal Displacement (mm)
200
150
100
50
0
1000
3000
6000
Geogrid Stiffness EA (kN/m)
LOOSE SAND
DENSE SAND
CLAYEY SOIL

(a)


WALL HEIGHT =7 m
Horizontal Displacement (mm)
80
60
40
20
0
1000
3000
6000
Geogrid Stiffness EA (kN/m)
LOOSE SAND
DENSE SAND
CLAYEY SOIL

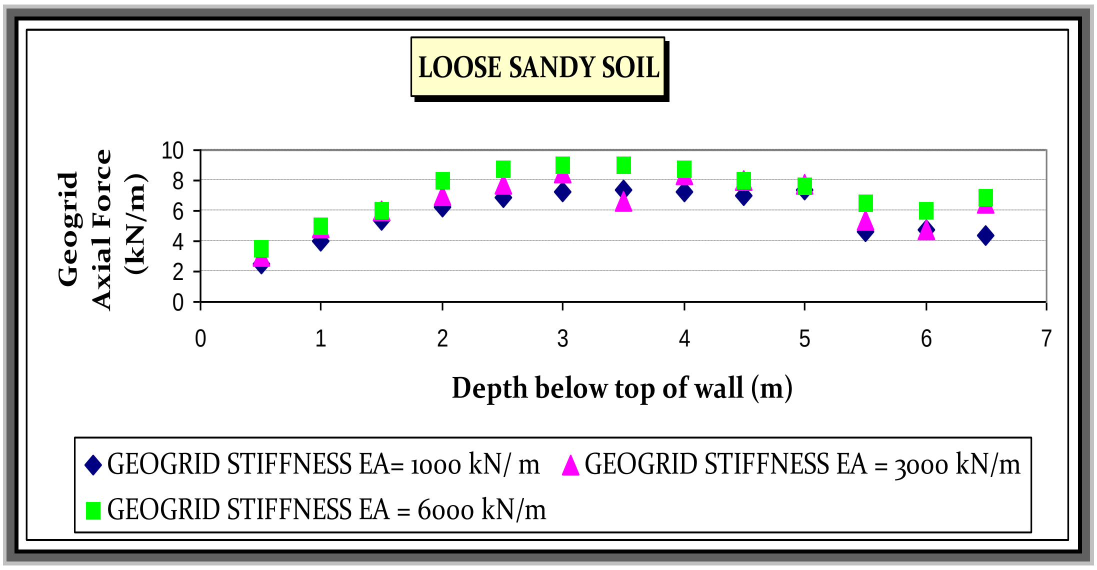
(b)



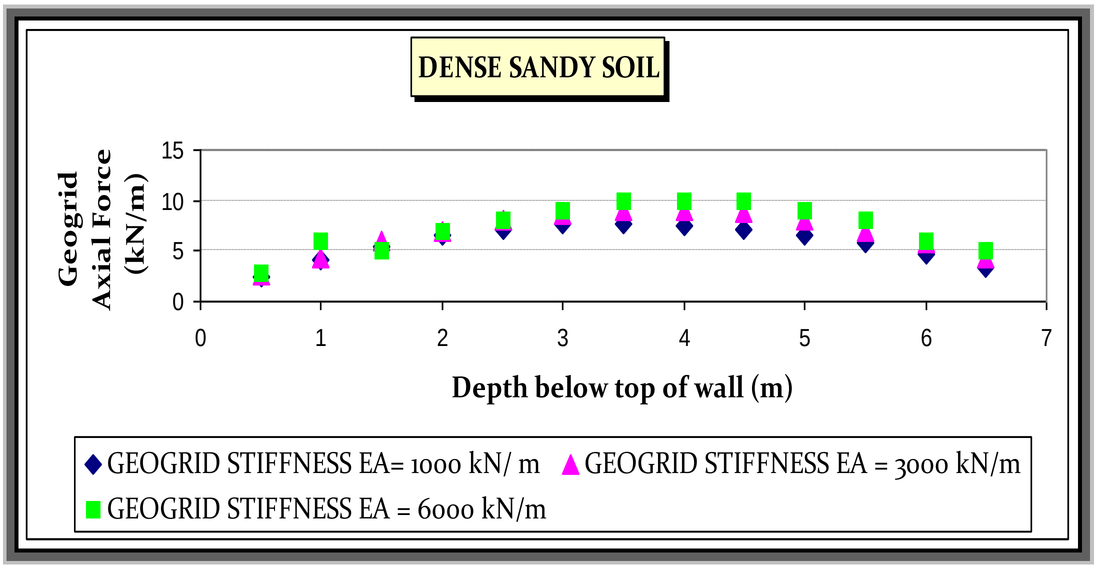
(c)

Figure 4.18 Effect of geogride stiffness and backfill soil type on horizontal displacement for: (a) H= 10 m, (b) H= 7 m, (c) H= 4 m

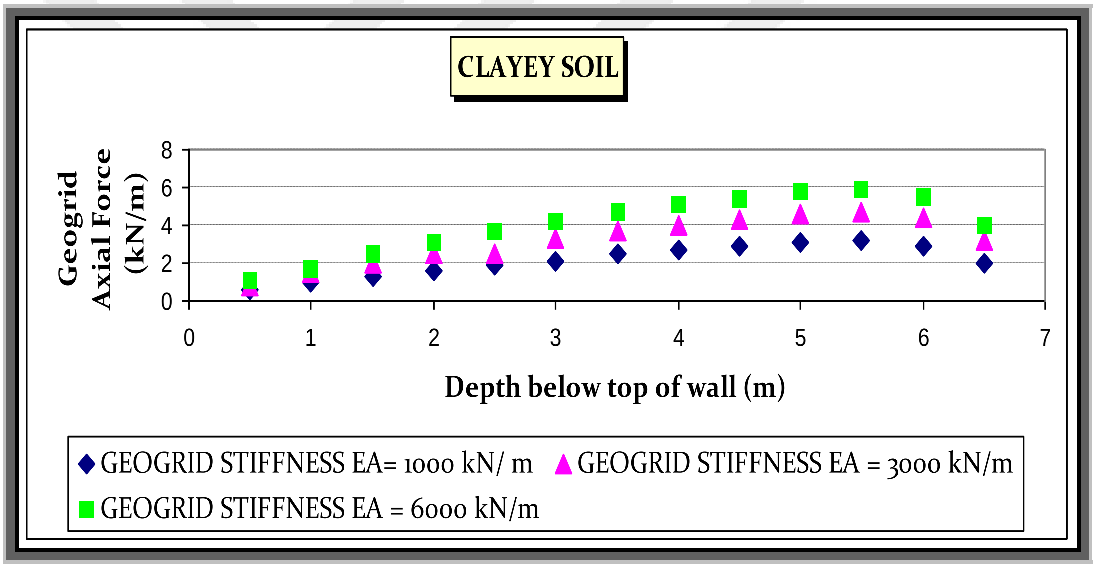
From Figures 4.18 (a), (b), (c), it can be shown that the horizontal deformation of wall face element increase with increasing MSE wall height, horizontal deformation decrease with increasing geogrid stiffness, and finally horizontal deformation increase with decreasing angle of internal friction for backfill reinforcement soil.



(a)



(b)



(c)

Figure 4.19 Effect of geogride stiffness on geogrid axial force in: (a) Loose sandy soil, (b) Dense sandy soil, (c) Clayey soil

Effect of geogrid strength on axial force mobilized in reinforcement layers shown in the Figures 4.19 (a), (b), respectively. These Figures show that; the geogrid strength has no significant effect on axial force developed. The pattern of the reinforcement axial force are the same for a different computed case by PLAXIS in the study conducted by Mahmood (2009).

This author study the effect of geogrid strength on axial force developed at upper & lower most geogrid in MSE wall with height 4 m; without water and with water. The results indicated that the geogrid strength has no significant effect on axial force developed.

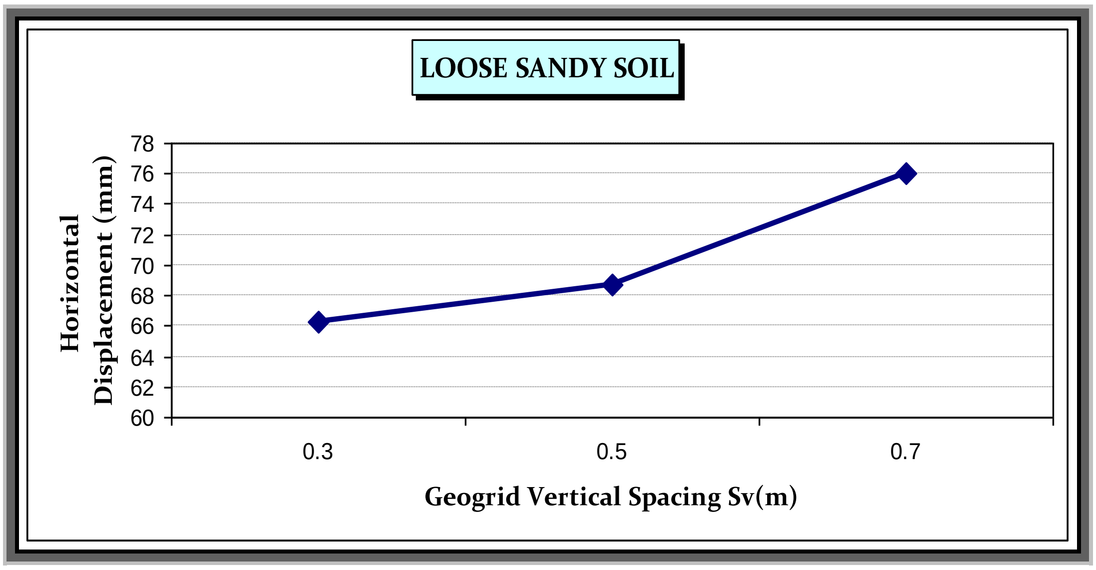
A reinforced soil system provides resisting force to maintain equilibrium condition in the MSE wall.When geogrid strength increases, the pull out stress of geogrid is also increased. So, it requires more force to displace the geogrids from their original position. Tensile failure of the reinforcement at any level leads to progressive collapse of the wall. Again, when geogrid strength is inadequate, then geogrid breaking may occur, which will increase the displacement. Slip at soil reinforcement interface may occur, while geogrid strength is insufficient. Slip at the soil reinforcement leads to redistribution of stresses and progressive deformation of the wall.

4.2.4 Effect of Reinforcement Vertical Spacing (Sv) on Horizontal displacement & Axial force in Geogrid

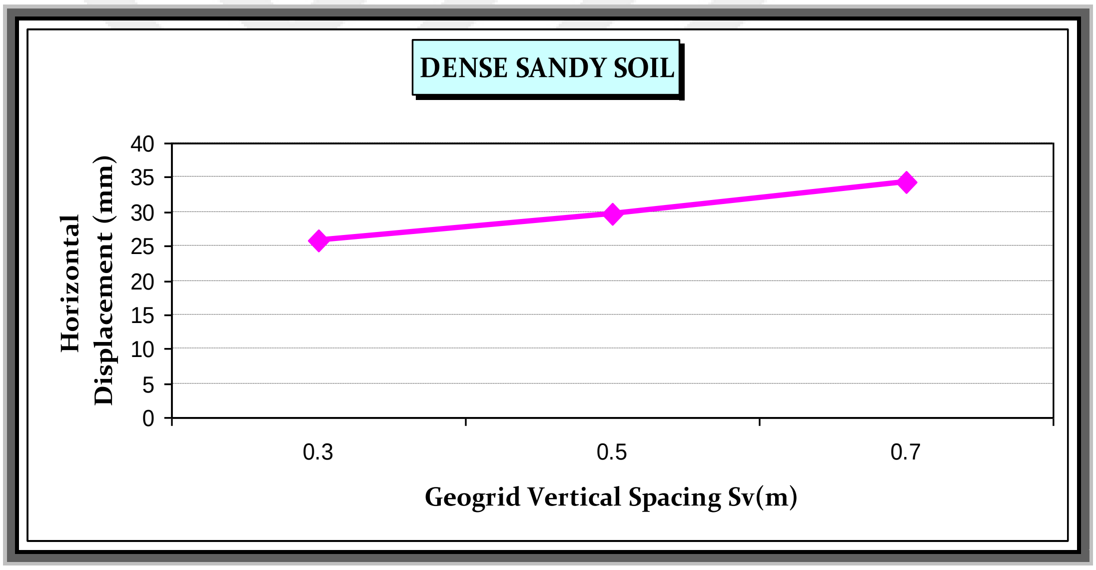
Design Parameters:

This analysis was conducted on MSE wall with height (H) = 7 m, geogrid stifness (EA) = 1100 kN/m, vertical spacing between geogrid layers (Sv) = 0.3, 0.5, and 0.7 m, face element thickness (D) = 0.25 m, reinforcement length to height (L/H) ratio = 0.85.

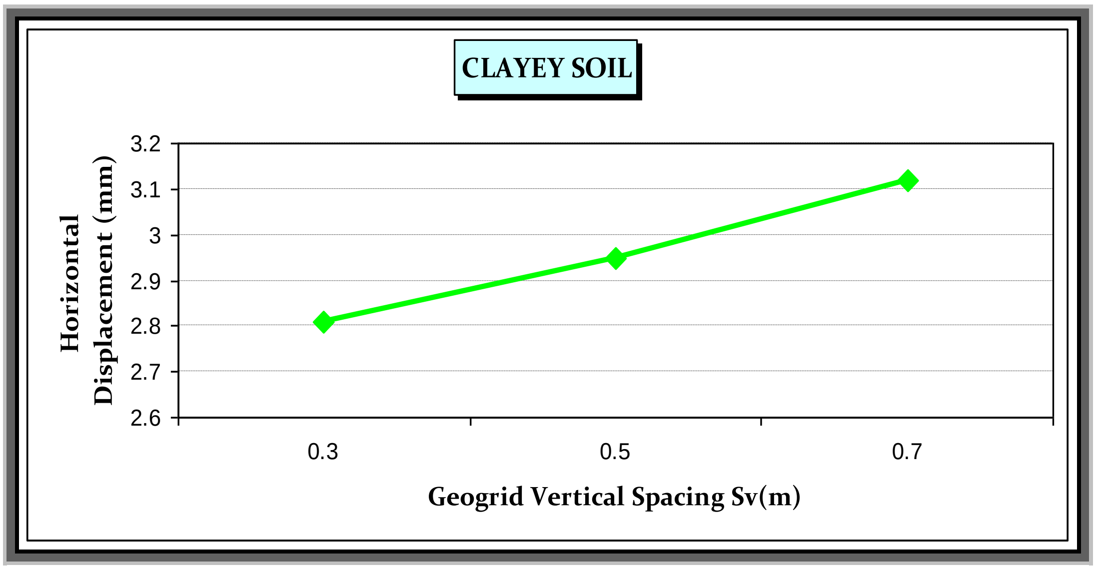
This analysis was carried out for three different type of backfill soil, in order to see the effect of (Sv) on horizontal displacement and axial force in the reinforcement layers.


LOOSE SANDY SOIL
Horizontal Displacement (mm)
78
76
74
72
70
68
66
64
62
60
0.3
0.5
0.7
Geogrid Vertical Spacing Sv(m)

(a)


DENSE SANDY SOIL
Horizontal Displacement (mm)
40
35
30
25
20
15
10
5
0
0.3
0.5
0.7
Geogrid Vertical Spacing Sv(m)

(b)



(c)

Figure 4.20 Effect of vertical spacing on horizontal displacement in: (a) Loose sandy soil, (b) Dense sandy soil, (c) Clayey soil

Figures 4.20 (a), (b) shows that horizontal displacements increase rapidly with increase in verticall spacing of geogrids. The range of movement increased from 66 mm to 76 mm in case of loose sand, and from 26 mm to 34 mm for case of dense sand, for an increase of geogrid vertical spacing from 0.3 m to 0.7 m at H = 7 m. This meanse that displacements increase by nearly 15% for loose sand and 31% for dense sand. This behaviour reported herein agrees well with other case studies by Ho and Row (1996); Oyegbile (2011).

Figure 4.20 (c), shows that the value of horizontal displacement increases steadily with an increase in spacing of geogrids. This observed behaviour is due to cohesive property of clayey soil.

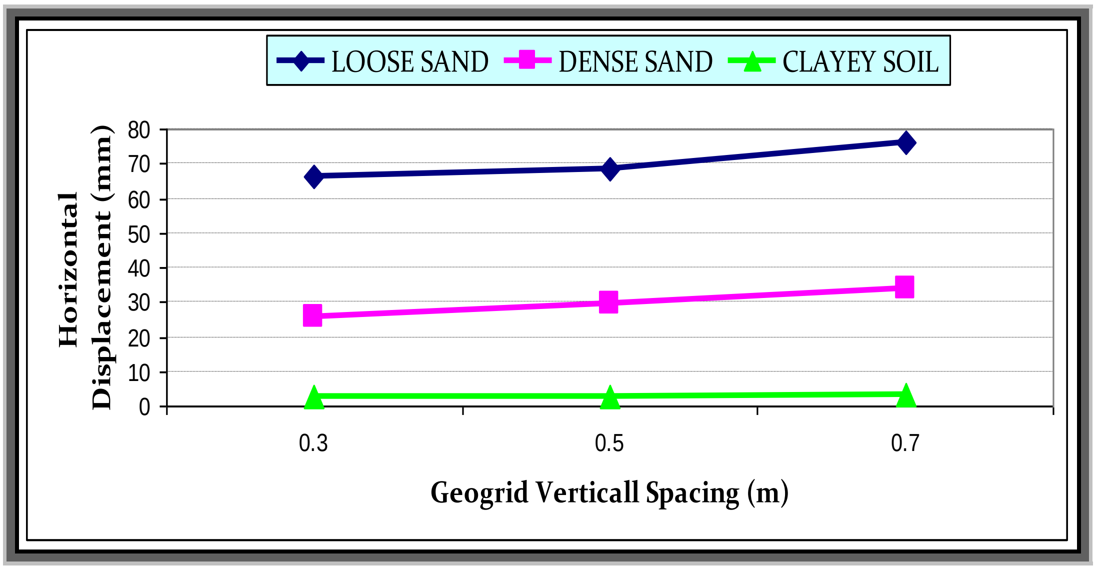
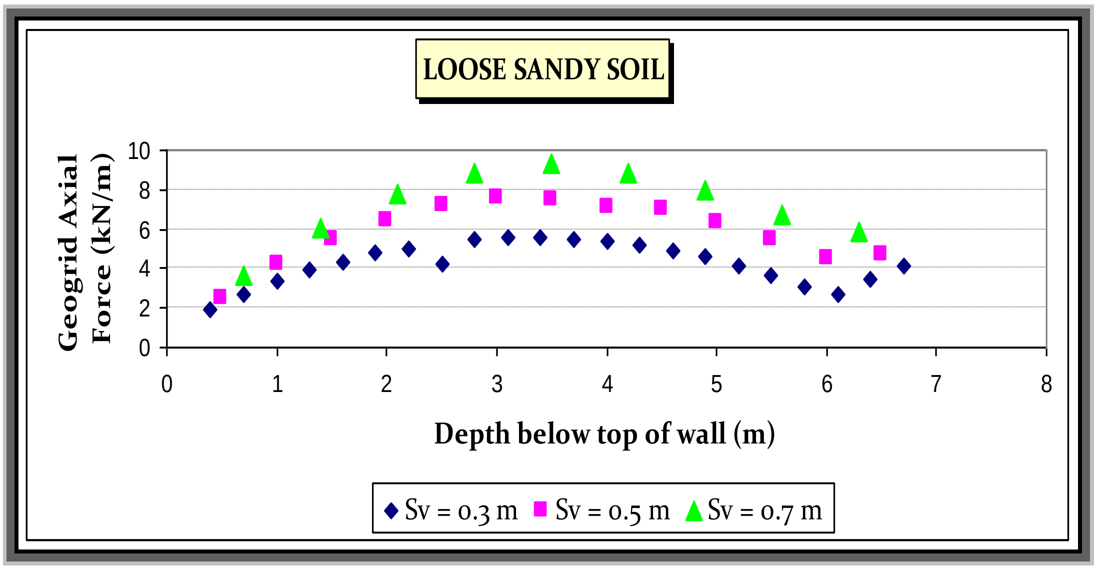
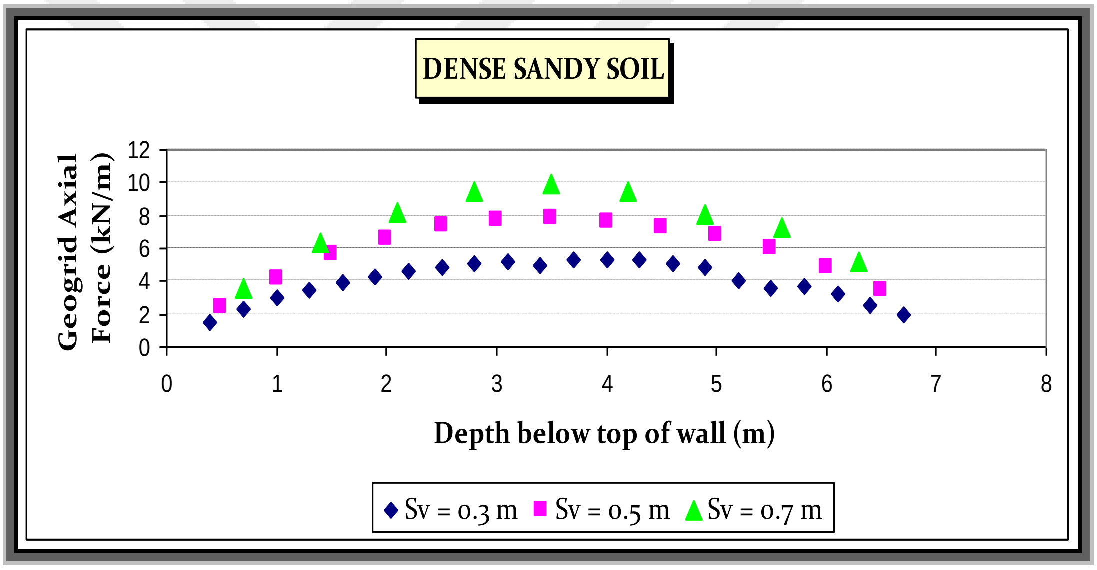


Figure 4.21 Effect of vertical spacing and backfill soil type on horizontal displacement

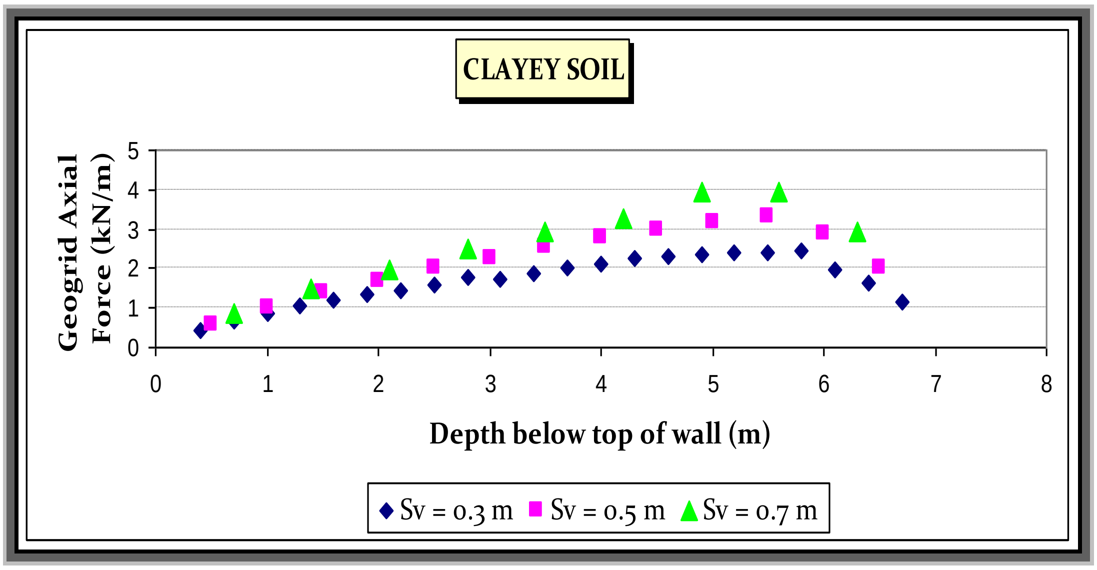
Cohesive soil always allow less displacement as shown in Figure (4.21). But cohesionless soil with the same vertical spacing shows relatively large displacement.


LOOSE SANDY SOIL
Geogrid Axial Force (kN/m)
10
8
6
4
2
0
0
1
2
3
4
5
6
7
8
Depth below top of wall (m)
Sv = 0.3 m
Sv = 0.5 m
Sv = 0.7 m

(a)


DENSE SANDY SOIL
Geogrid Axial Force (kN/m)
12
10
8
6
4
2
0
0
1
2
3
4
5
6
7
8
Depth below top of wall (m)
Sv = 0.3 m
Sv = 0.5 m
Sv = 0.7 m

(b)



(c)

Figure 4.22 Effect of vertical spacing on geogrid axial force in: (a) Loose sandy soil, (b) Dense sandy soil, (c) Clayey soil

Effect of Geogrid vertical spacing on axial force developed at the geogrid layers shown in the Figures 4.22 (a), (b), (c), respectively. These figures show that, the axial force increasing with the increasing of geogrid vertical spacing (Sv) which have significant effect on force developed in geogrid layers embedded in the different types of backfill soil used in this analysis.

The horizontal stress acting on the facing and the connection loads can be reduced by using more layers of reinforcement. Reinforcement spaced too far apart leads to failure of the soil as if it were not reinforced at all. Thus, a close spacing is necessary to activate the reinforcement strength for the structure to be effective. closer reinforcement spacing increases internal stability, which means less facing deformation and less axial force developed.

In order for reinforcement strength to be used, it must be mobilized. As the load is carried by the backfill between the reinforcements, the soil, which is much weaker than the geosynthetic; starts to slide against the reinforcement causing friction to develop, and mobilizing the reinforcement tensile strength.

4.2.5 Effect of Face Element Thickness (D) on Horizontal Displacement & Axial Force in Geogrid

Design Parameters:

The hight of the wall (H) = 7 m, geogrid stifness (EA) = 1100 kN/m, vertical spacing between geogrid layers (Sv) = 0.5 m, face element thickness (D) = 0.25, 0.3, 0.35, and 0.4 m, reinforcement length to height (L/H) ratio = 0.85.

This analysis was carried out for three different type of backfill soil, in order to see the effect of face element thickness on horizontal displacement and axial force in the reinforcement layers.

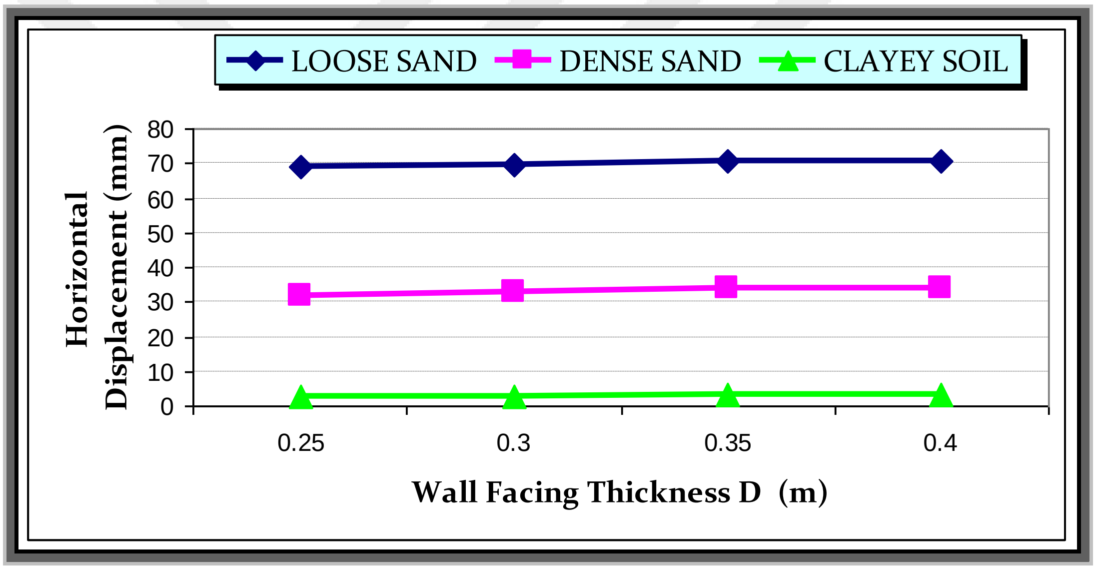
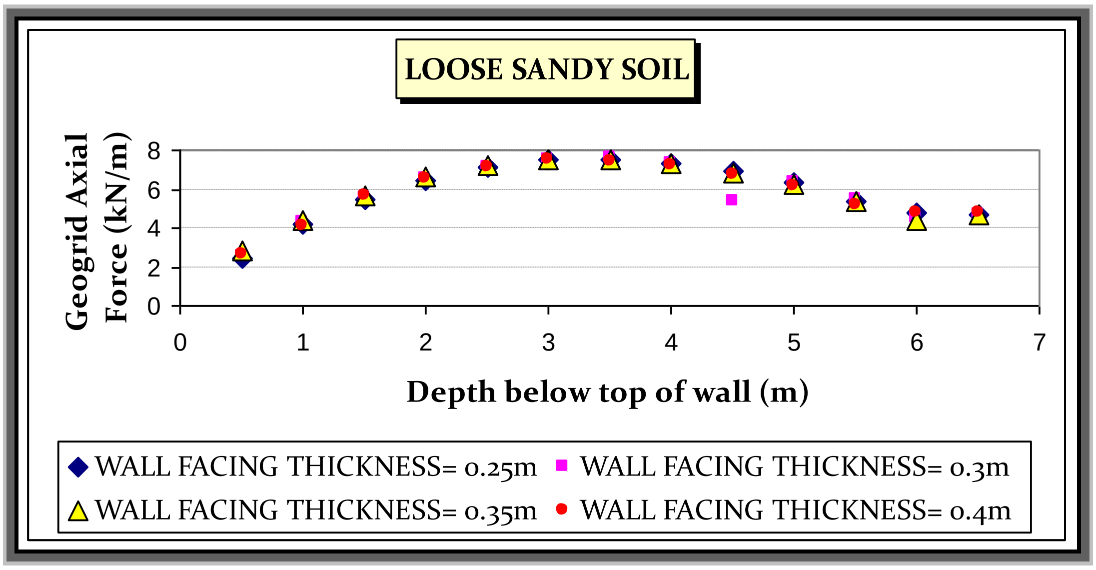
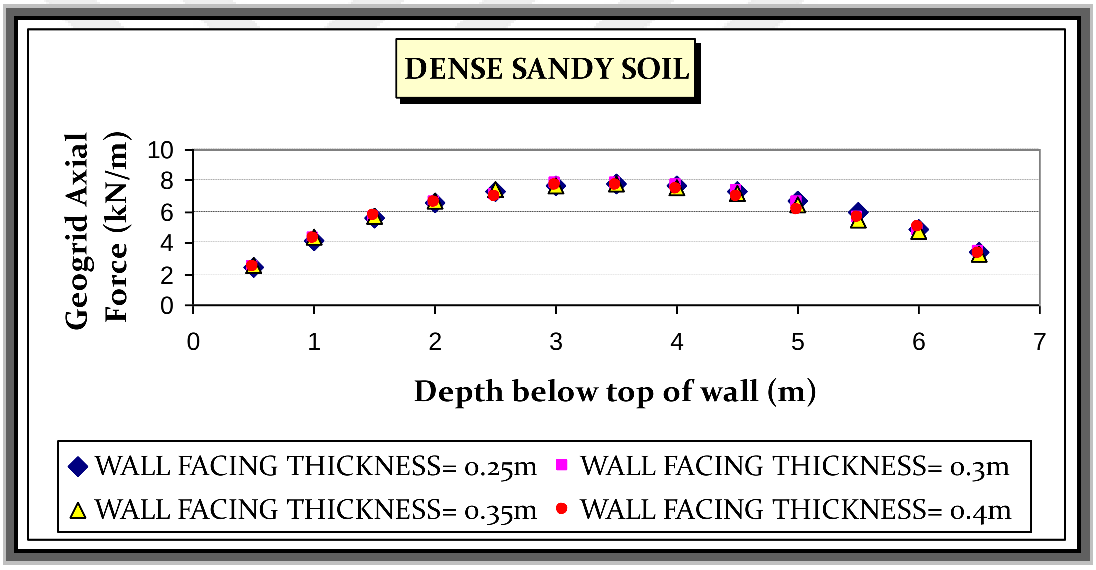


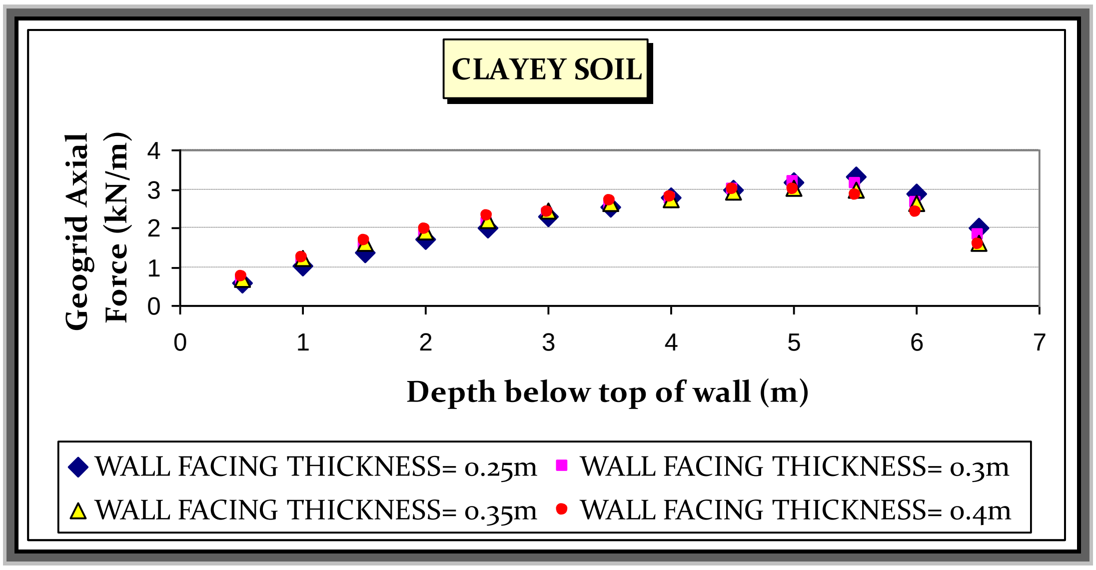
Figure 4.23 Effect of wall thickness and backfill soil type on horizontal displacement


LOOSE SANDY SOIL
Geogrid Axial Force (kN/m)
0
2
4
6
8
0
1
2
3
4
5
6
7
Depth below top of wall (m)
WALL FACING THICKNESS= 0.25m
WALL FACING THICKNESS= 0.3m
WALL FACING THICKNESS= 0.35m
WALL FACING THICKNESS= 0.4m

(a)


DENSE SANDY SOIL
Geogrid Axial Force (kN/m)
0
2
4
6
8
10
0
1
2
3
4
5
6
7
Depth below top of wall (m)
WALL FACING THICKNESS= 0.25m
WALL FACING THICKNESS= 0.3m
WALL FACING THICKNESS= 0.35m
WALL FACING THICKNESS= 0.4m

(b)



(c)

Figure 4. 24 Effect of wall thickness and backfill soil type on horizontal displacement

From Figure (4.23), it can be seen that increasing the wall thickness has no effect on extreme total displacement, nor axial force developed in the geogrid layers.

The facing plays a minor structural role in the stability of the structure. The facing element protects the soil and reinforcing elements from weathering effects and used to keep the backfill soil from flowing, Since the facing is the visible part of the structure. It also controls the aesthetics of the reinforced earth wall.

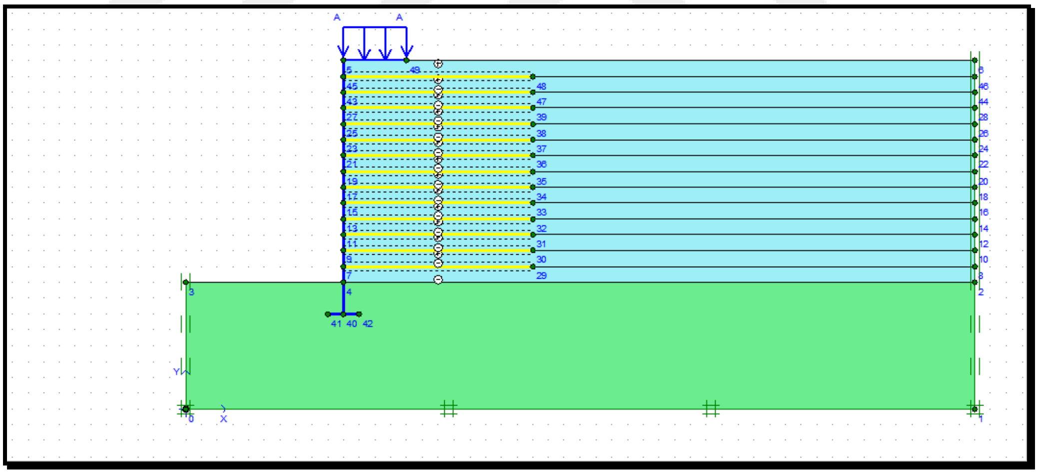
The influence of facing panel thickness on the axial force is not significant as illustrated in Figures 4.24 (a), (b), (c).

4.2.6 Effect of Traffic Surcharge Load Position on Horizontal Displacement & Axial Force in Geogrid

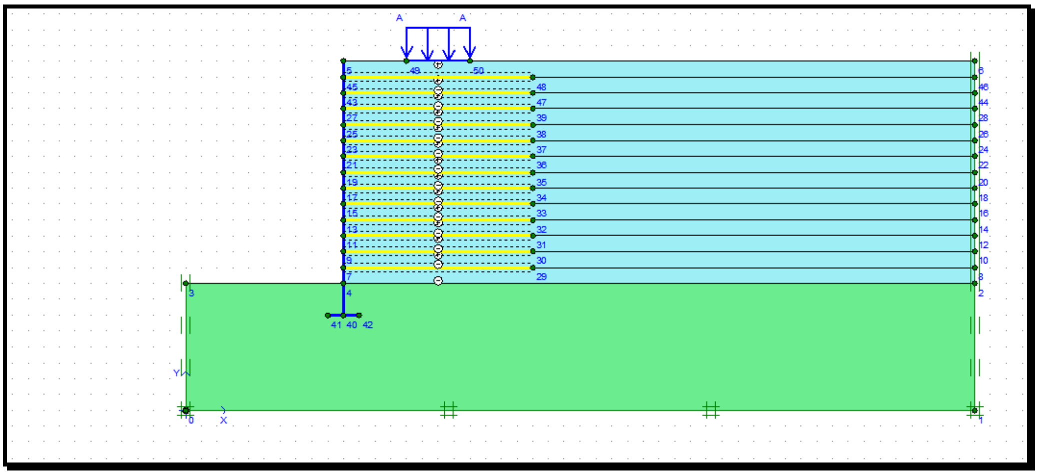
Design Parameters:

This analysis was conducted on MSE wall with height (H) = 7 m, geogrid stifness (EA) = 1000 kN/m, vertical spacing between geogrid layers (Sv) = 0.5 m, face element thickness (D) = 0.25 m, reinforcement length to height (L/H) ratio = 0.7, with shifting surcharge traffic load equal to 20 kN/m^2 as illustrated in Figure 4.25 (a), (b), (c).

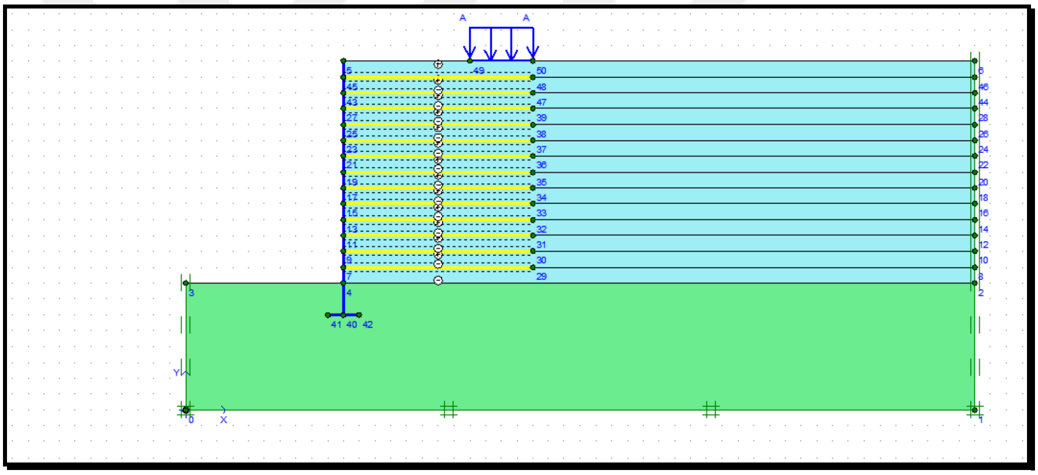
This analysis was carried out for three different type of backfill soil to see the effect of traffic surcharge load on horizontal displacement, and maximum axial reinforcement force.



(a)



(b)



(c)

Figure 4.25 position of traffic surcharge load: (a) position 1, (b) position 2, (c) position 3

Generally MSE walls may be subjected to a variety of loads, including concentrated vertical surcharge loads. In this case, an external uniform vehicular traffic live load as illustrated in Figure 4.25 (a), (b), (c), applied to the reinforced soil mass behind the wall facing .The effect of a surcharge load on the wall's stability depends upon its magnitude and its location from the face of the wall. The closer the location of the surcharge load to the wall, (i.e., back of the reinforced zone) the worse the impact on wall stability White (2010). That is mean when the distance between the load and the wall face increase the axial load and horizontal displacement will decrease.

According to White (2010), If the distance between surcharge load and the wall is greater than a certain distance, then its effect is insignificant. This distance is known as the extent of active zone (L_P). L_P can be calculated as shown in the following equation:

$L_p = B + H \tan(45^{\circ}$ - **Error!**φ_f) Eq (4.1)

Where:

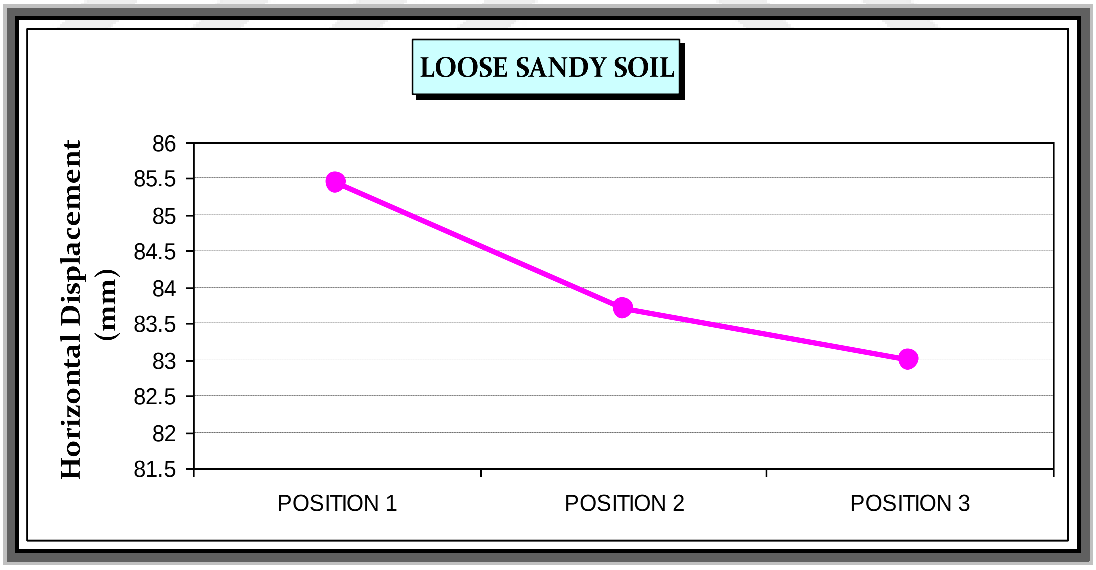
L_P - Extent of active zone.

B - Width of reinforced soil mass .

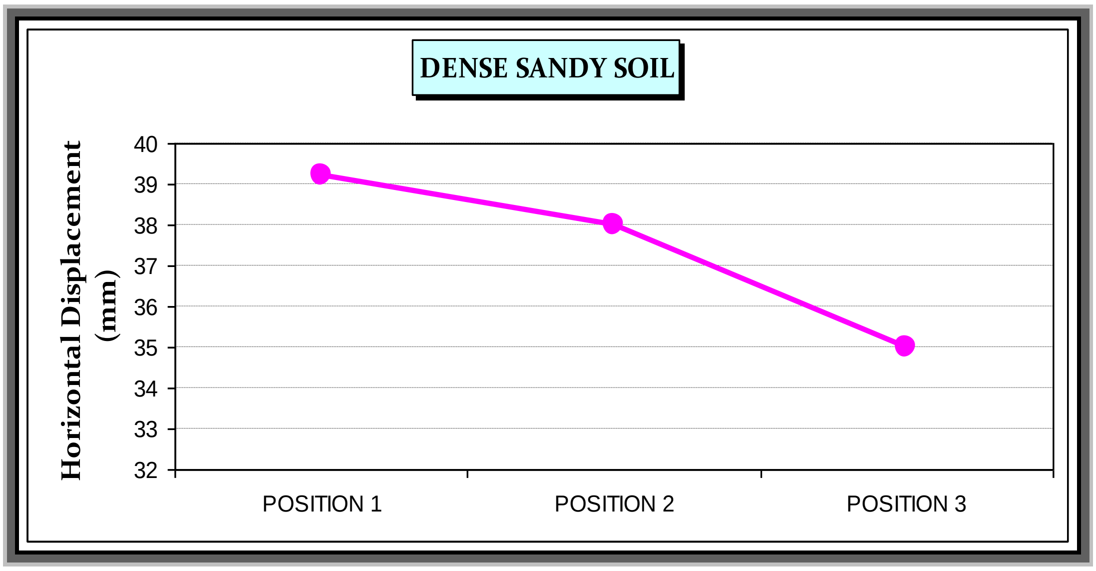
In general, if the reinforced soil mass is reasonably stable, then surcharge loads located beyond the distance equal to the design height of wall (H) would not significantly impact the wall stability.

H - Design wall height .

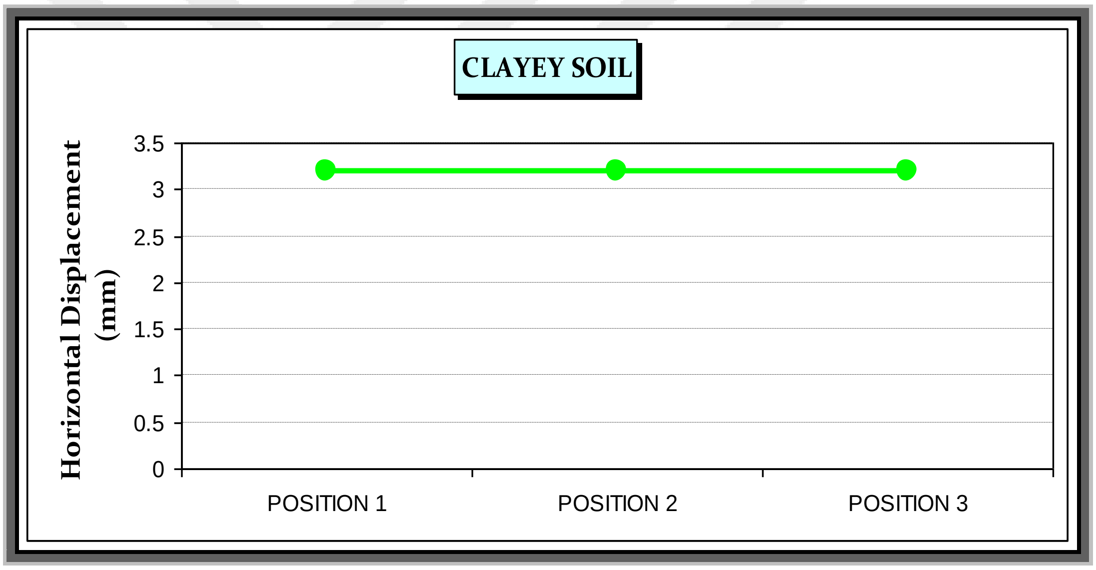
Thus, L_P can also be roughly taken equal to H, the design wall height ($L_P \approx H$).



(a)



(b)



(c)

Figure 4.26 Effect of traffic surcharge load position on horizontal displacement in : (a) Loose sandy soil, (b) Dense sandy soil, (c) Clayey soil

The results shows that in granular soil the horizontal displacement of the wall facing decrease with increase the distance between traffic surcharge load and the wall face as illustrated in Figures 4.26 (a) and (b). The influence of traffic surcharge load position on the horizontal displacement is not significant in clayey soil as illustrated in Figure 4.26 (c).

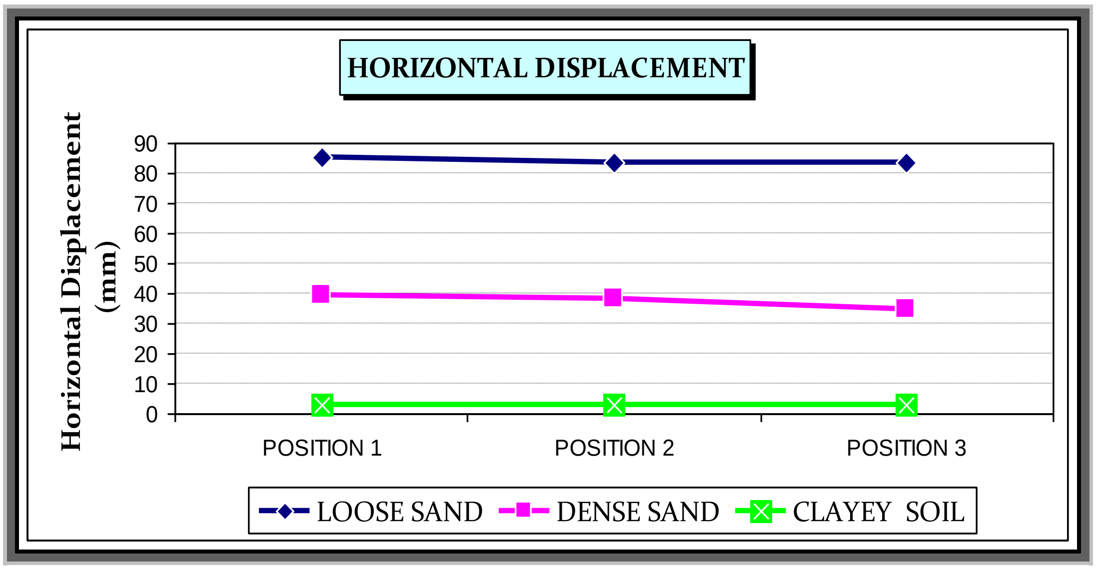
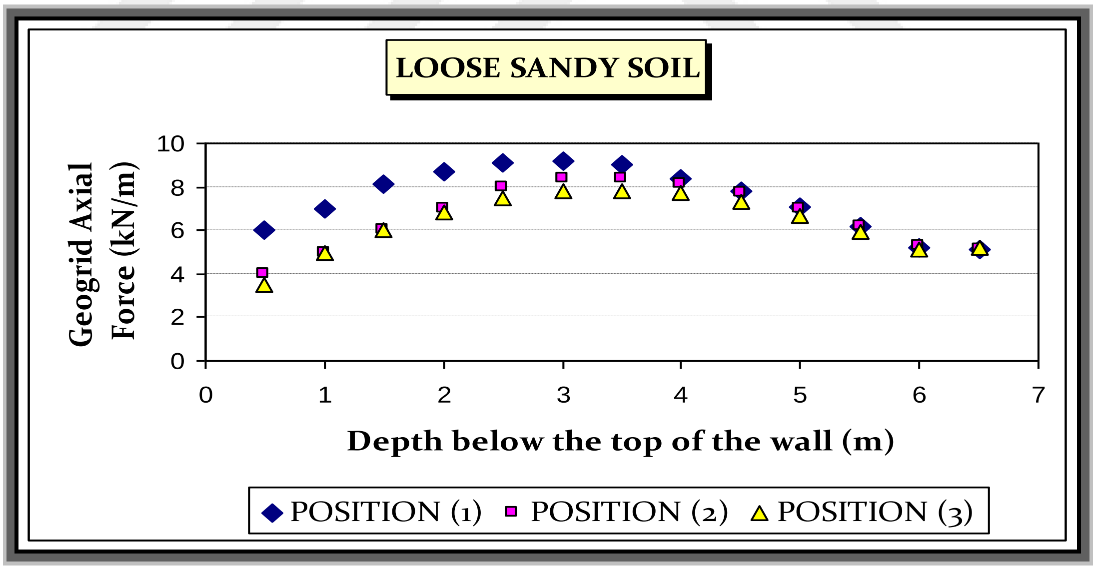
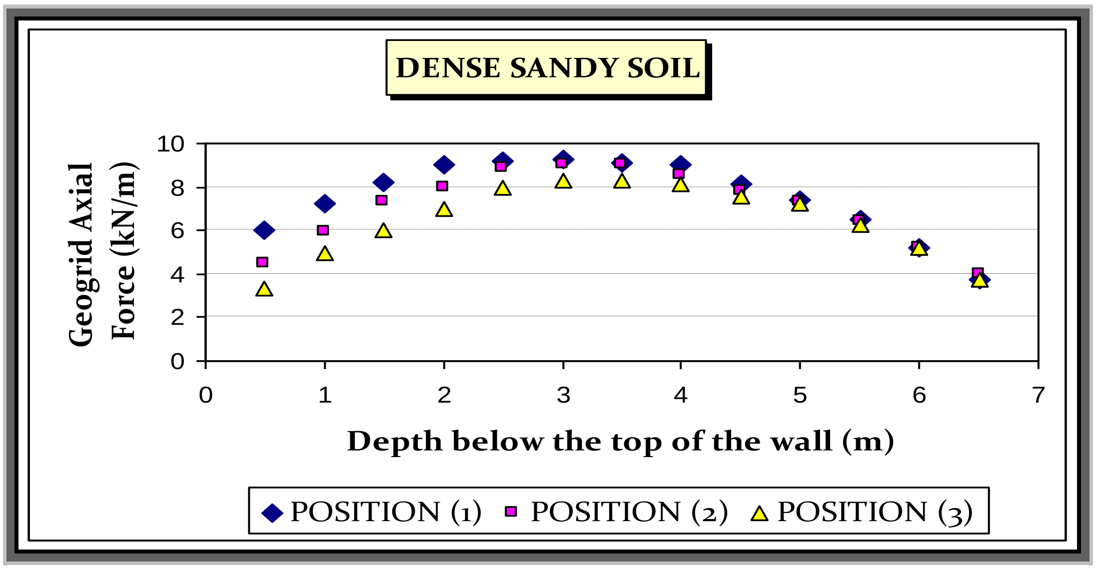


Figure 4.27 Effect of the surcharge load and reinforced backfill soil type on horizontal displacement.

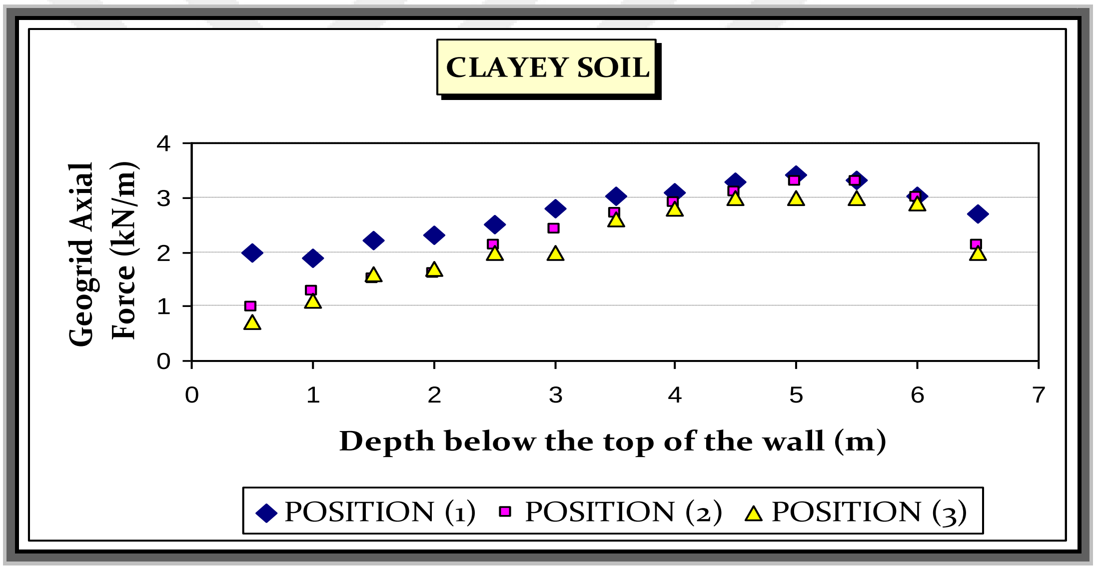
Figure (4.27) shows that the greatest wall deformation happened in the case of loose sand backfill reinforcement soil.



(a)



(b)



(c)

Figure 4.28 Effect traffic asurcharge load position on geogrid axial force in : (a) Loose sandy soil, (b) Dense sandy soil, (c) Clayey soil

The axial force mobilized in the reinforcement layers rather reduce with increase the distance between traffic surcharge load and the wall as illustrated in Figures 4.28 (a), (b), (c).

4.2.7 Effect of Reinforcement Soil Friction Angle (φ) on Horizontal Displacement & Axial Force in Geogrid

Design Parameters:

This analysis was conducted on MSE wall with height (H) = 7 m, geogrid stifness (EA) = 1100 kN/m, vertical spacing between geogrid layers (Sv) = 0.5 m, face element thickness (D) = 0.25 m, reinforcement length to height (L/H) ratio = 0.85.

This analysis was carried out for loose sand reinforced soil, with friction angle which rainging from 30° to 42° to see the effect of angle of internal friction on horizontal displacement, and maximum axial reinforcement force developed in geogrid layers.

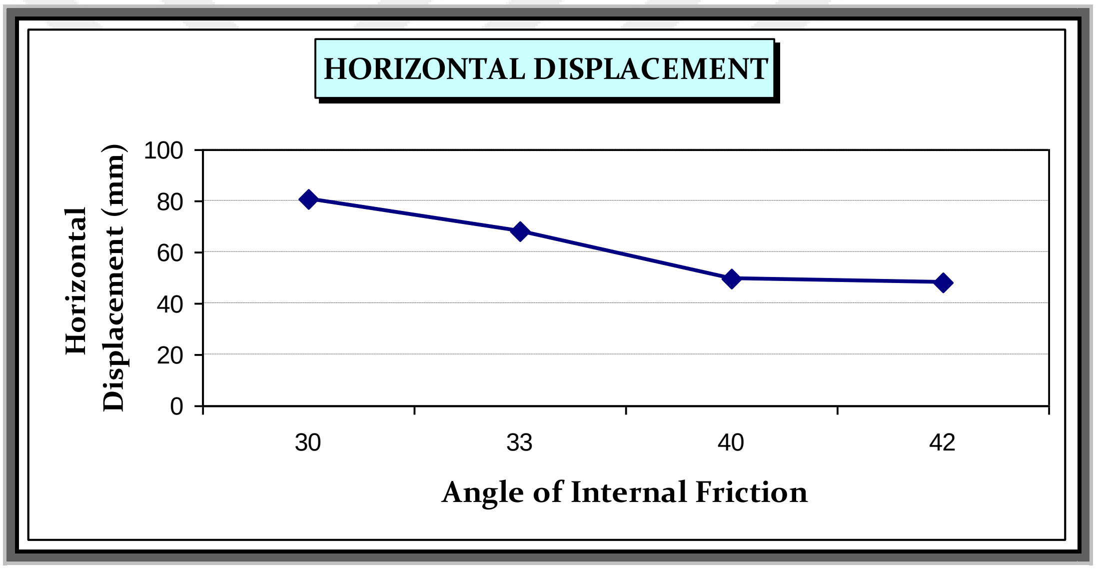


Figure 4.29 Effect of soil reinforcement angle on horizontal displacement

The results show that the wall deformations are significantly influenced by reinforced soil friction angle. The variations in horizontal displacement for the increase in reinforced-fill friction angle from 30° to 42° are presented in Figure (4.29). It was observed that the horizontal movement reduce with the increase in reinforced-fill friction angle. However, movement reduced from 81 mm to 48 mm for the 7 m wall height, an increase in friction angle from 30° to 42° caused a 69% reduction in displacement.

When the angle of internal friction greater than 40° there is very small change in horizontal displacement.

According to Kibria (2014), the interface friction between soil and reinforcement increases with an increase in reinforced-fill friction angle. An equilibrium condition in the reinforced mass occurs at a smaller force when the reinforced-fill friction angle is heigh. This might cause a reduction in displacement with an increase in reinforced-fill friction angle.

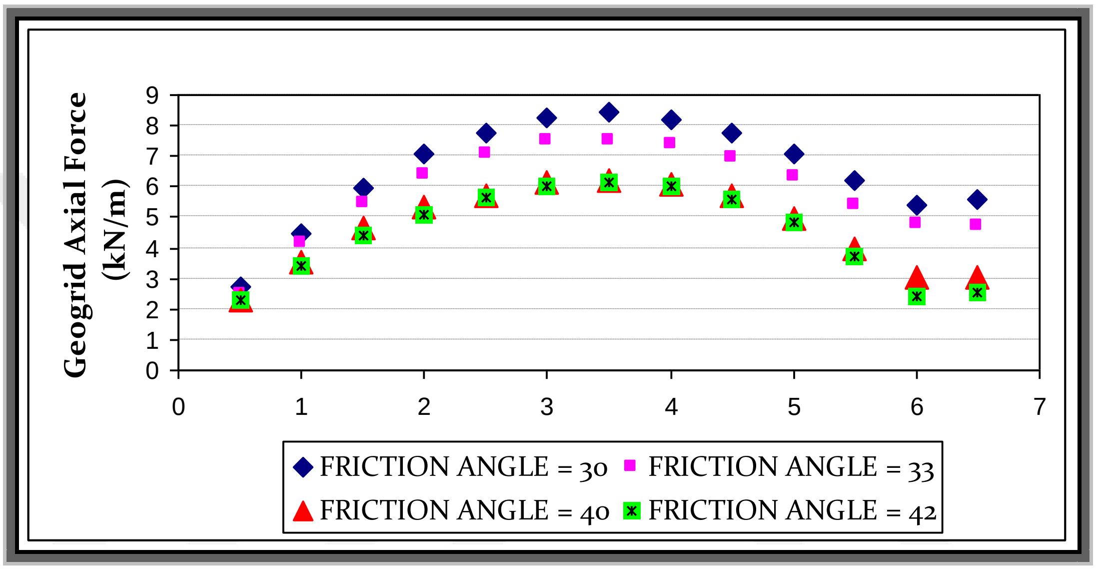


Figure 4.30 Effect of soil reinforcement angle on geogrid axial force

The maximum reinforcement force increases as the reinforced soil friction angle decrease as shown in Figure (4.30). A ccording to Biligin and Kim (2010); Mahmood (2009) the horizontal displacement of the wall face element and axial force in geogrid layers reduced with increasing reinforced soil friction angle. The results obtained in the Figurs (4.29 & 4.30) were in good agreement with the study conducted by the authors mentioned above .

The reinforcement force increasing approximately 45% when friction angle changes from 42° to 30° (Figure 4.30).

CHAPTER V

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Following the results obtained from static analyses carried out on MSE wall presented in this research work which examined under different geometric parameters such as reinforcement length, reinforcement stiffness, vertical spacing between reinforcement layers, wall facing thickness, the position of imposed surcharge load, types of reinforced soil, and wall height. The effect of all these parameters on the forces developed in the reinforcement and wall deformation is examined, the following concluding remarks can be made:

1) The behaviour of MSE wall are influenced significantly with these parameters backfllsoil type, surcharge load positions, reinforcement stiffness, the vertical spacing between reinforcement layers, and reinforcement length.

2) Increasing stiffness of reinforcement elements can result in decreased deformations and it will end up with smaller displacement; however, these effects are limited to up to certain values after which they fail to show any significant effects. So geogrid with higher strength is recommended to use.

3) Increasing reinforcement stiffness in case of dense sand and clayey soil reduce the deformaton which required minimum length to reached its state of equilibrium comparsion with loose sand which require longer length to reduce face deformation.

4) Increasing length of reinforcement elements can result in decreased deformations and it will end up with smaller displacement; however, these effects are limited to

up to certain values after which they fail to show any significant effects.

5) The shape of relative distribution of tensile force along the wall height is independent of the length of reinforcement layers, their tensile stiffness, their number, thickness of face element, and also independent of the friction angle of backfill soil in reinforced zone.

6) In garanular backfill soil, the reinforcement layer on which maximum tensile force occurs is located at the mid height of the wall. This position is independent of the length of reinforcement layers, reinforcement stiffness, number of reinforcement layers, thickness of face element, and also independent of the friction angle of backfill soil in reinforced zone variables.

7) Loose sandy soil showed higher degrees of instability when it is used as backfill materials in reinforced zone.

8) Dense sandy soil and clayey soil are more stable and suitable as a backfill materials in reinforced zone. Generally, clayey soil shows less wall movements than cohesionless soil. Howeve; it must be used with proper drainage components.

9) There exists a wide range of values of geogrids stiffness and when the stiffness of the geogrid is increased beyond certain limit, MSE wall behave like brittle material which is undesirable behaviour. The failure of reinforced soil retaining wall will show brittle failure mode.

10) Increasing vertical spacing between reinforcement layers will increase wall deformation for granular backfill soil, but there is no significant effect in cohesion soil. the optimum value in this analysis is equal to 0.5 m.

5.2 Recommendations

The following are the possible ways of improving this research work:

1) By studying wrapped faced walls and make comparison with MSE walls with modular block wall unit in order to investigate and compare the general stability of these two walls.

2) By studying the slope reinforcement wall and and make comparison with verticall MSE walls in order to investigate and compare the general stability of these two walls .

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Anderson, P. L., Gladstone, R. A., Sankey, J. E. (2012). State of the Practice of MSE Wall Design for Highway Structures. Geotechnical Engineering state of the Art and practice, Geocongress, Oakland, California, United States, March 25-29, pp. 443-463.

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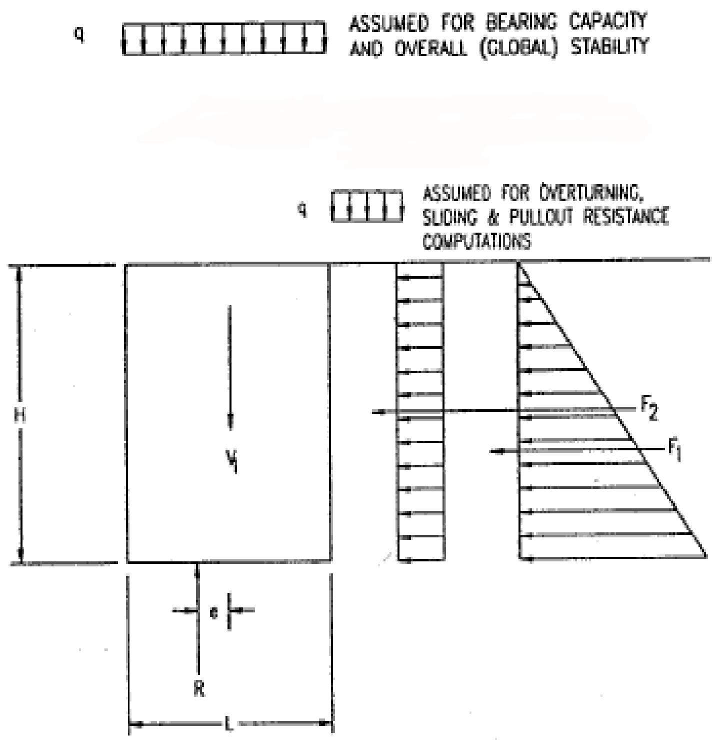
APPENDICES

APPENDIX A

HAND CALCULATION EXAMPLE

Horizontal Backslope with traffic surcharge – Static analysis Geogrid with 100% coverage. The following example is a modular block unit faced, geogrid soil reinforcement wall system.



$H = 4$ m	$L = 3.5$ m
$V_1 = \gamma_r HL$	e = Eccentricity
$F_1 = ½\, \gamma_b\, H^2\, K_a.$	q = Traffic Surcharge = 20 kN/m^2
$F_2 = qHK_a$	R = Resultant of Vertical Forces
	$(V_1 + qL)$

SOIL PROPERTIES

- REINFORCED SOILS

$\gamma r = 19.6\ kN/m^3$ $\varphi_r = 34$ c = 0 kPa

$K_a = \tan^2(45 - \varphi/2)$ (Section 4.3B FHWA - Demo 82)

$= \tan^2(45-34/2) = 0.28 = K_a$

- RETAINED BACKFILL SOILS

$\gamma_b = 19.6\ kN/m^3$ $\varphi_b = 30$ c = 0 kPa

- FOUNDATION SOILS

$\gamma_f = 19.6\ kN/m^3$ $\varphi_f = 30$ c = 0 kPa

$Ka = \tan^2(45 - \varphi/2) = \tan^2(45-30/2)$ (Section 4.2d FHWA - Demo 82)

$= K_a = 0.33$

EXTERNAL STABILITY

H = 4 m

B = L = 3.5 m (assumed L > 0.7 H)

- LOADS

$V_1 = \gamma_r HL = (19.6)(4)(3.5)$ $= 274.4$ kN/m

$V_2 = qL = (20)(3.5)$ $= 70$ kN/m

$R = \Sigma V = V_1 + V_2 = 274.4 + 70$ $= 344.4$ kN/m

$F_1 = ½\ \gamma_b H^2 K_a = (0.5)(19.6)(4^2)(0.33)$ $= 51.75$ kN/m

$F_2 = q\ H\ K_a = (20)(4)(0.33)$ $= 26.4$ kN/m

- MOMENTS

M = Overturning Moment $= F_1 (H/3) + F_2 (H/2)$

$= (51.75)(4/3) + (26.4)(4/2)$ $= 121.8$ kN m/m $= M_o$

M_{RO} = Resisting Moment = V_1 (L/2) = (274.4)(3.5/2)

= 480.2 kN m/m

M_{RBP} = Resisting Moment in Applied Bearing Pressure Calculation

= V_1 (L/2) + V_2 (L/2) = 274.4(3.5/2) + 70(3.5/2)

= 602.7 kN m/m

$FS_{sliding} = \frac{\Sigma P_R}{\Sigma p_d}$ (Section 4.2e of FHWA - Demo 82) = $\frac{V_1 \tan \varphi}{(F_l + F_2)}$,

where φ is the lesser of φ_r and φ_f

$= \frac{274.4 \tan 30}{(51.75+26.4)}$ =2.03 > 1.5

$FS_{overturning} = \frac{M_{RO}}{M_O} = \frac{480.2}{121.8}$ = 3.94 > 2.0

- MAXIMUM APPLIED BEARING PRESSURE

L/6 = 3.5/6 = 0.58 m

$e = \frac{L}{2} - \frac{(M_{RBP} - M_O)}{V_1 + V_2}$

$= \frac{3.5}{2} - \frac{(602.7 - 121.8)}{274.4 + 70}$ = 0.35 m < 1.25 m

L' = L - 2e = 3.5 - 2(0.35) = 2.8 = L'

σv = Max. Applied Bearing Pressure = $\frac{V_1 + q_L}{L - 2e} = \frac{V_1 + V_2}{L'}$ (AASHTO 97)

$= \frac{274.4 + 70}{2.8}$ = 123 kN/m^2

q_{ult} = Ult. Bearing Capacity of Fndn. Soil = $C_f N_c + 0.5 (L - 2e) \gamma_f N_\gamma$

(Section 4.2f of FHWA - Demo 82)

C_f = Cohesion = 0 kN/m^2

N_c = Dimensionless Bearing Capacity Coefficient

$q_{ult} = 0.5\ L'\ \gamma_f\ N_\gamma = (0.5)\ (2.8)\ (19.6)(22.4)$ $= 614.7\ kN/m^2$

$FS_{\text{bearing capacity}} = q_{ult} / \sigma_v = \dfrac{614.7}{123}$ $= 5 > 2.5$

FSsliding AT BASE OF FIRST GRID (Bottom of the wall)

F1 @ FIRST GRID = $\frac{1}{2}\ \gamma b(\ d17\ )^2\ K_a$

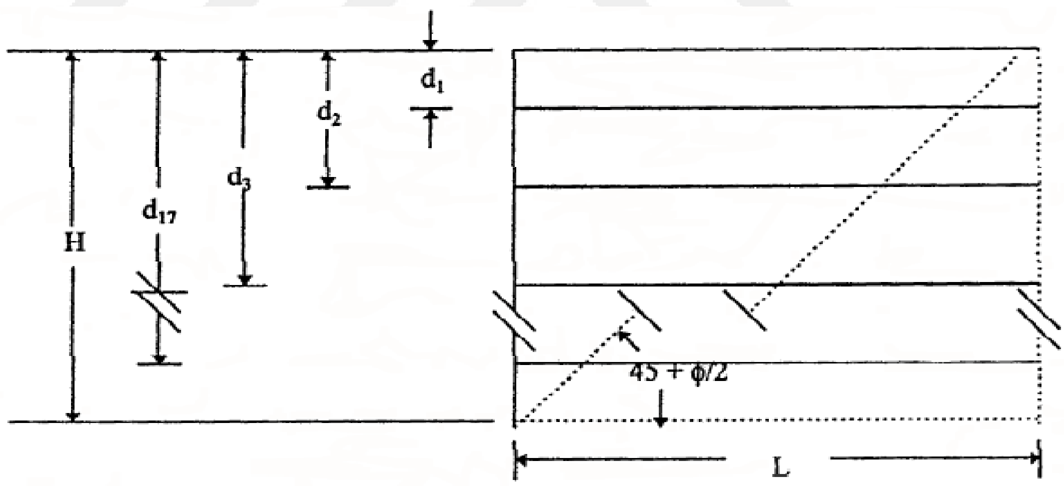
$= (\frac{1}{2})\ (19.6)\ (3.8)^2\ (0.33)$ $= 46.7\ kN/m$

F2 @ FIRST GRID = $q\ d17\ K_a = (20)\ (3.8)\ (0.33)$ $= 25\ kN/m$

$FS_{sliding} = \dfrac{\gamma_r\ d17\ L\ \tan\varphi_r\ C_i}{(Fl + F2)} = \dfrac{(19.6)\ (3.8)\ (3.5)\ (\tan 34)\ (0.8)}{(51.75+26.4)}$

$FS_{sliding} = 1.8 > 1.5$
(at first grid)

- **INTERNAL STABILITY**



$d_1 = 0.5$ m

$d_2 = 1$ m

$d_3 = 1.5$ m

$d_4 = 2$ m

$d_5 = 2.5$ m

d_6 = 3m

d_7 = 3.5m

d_8 = 4m

$V_1 = d1 + ½ (d_2 - d_1) = 0.5 + ½ (1-0.5)$ $V_1 = 0.75$ m

$V_2 = ½ (d_2 - d_1) + ½ (d_3 - d_2) = ½ (1- 0.5) + ½ (1.5 - 1)$ $V_2 = 0.5$ m

$V_3 = ½ (d_3 - d_2) + ½ (d_4 - d_3) = ½ (1.5 - 1) + ½ (2 – 1.5)$ $V_3 = 0.5$ m

$V_4 = ½ (d_4 - d_3) + ½ (d_5 - d_4) = ½ (2- 1.5) + ½ (2.5 - 2)$ $V_4 = 0.5$ m

$V_5 = ½ (d_5 - d_4) + ½ (d6 - d_5) = ½ (2.5 - 2) + ½ (3- 2.5)$ $V_5 = 0.5$ m

$V_6 = ½ (d_6 - d_5) + ½ (d_7 - d_6) = ½ (3- 2.5) + ½ (3.5 - 3)$ $V_6 = 0.5$ m

$V_7 = ½ (d_7 - d_6) + (H-d_7) = ½ (3.5 - 3) + (4- 3.5)$ $V_7 = 0.75$ m

- TENSION CALCULATION AT EACH REINFORCEMENT LEVEL $T_{(MAX)}$

$T_{MAX} = \sigma H\, S_V = \sigma H\, V_i$ (Section 4.3B - FHWA - Demo 82)

$\sigma H = K_{AR} (\gamma_R\, d_i + q)$ (Section 4.3B - FHWA - Demo 82)

Note: The geogrid strengths shown below were obtained using the following equation:

Allowable Strength = (Ultimate Strength x Rc) / ($FS_{uncertainties}$ x F_{SID} x F_{SD} x Creep Reduction Factor) where: $FS_{uncertainties}$ = 1.5, F_{SID} varies from 1.1 to 1.2 depending on geogrid type, and F_{SD} = 1.1. The Creep Reduction Factor = 3.10. R_c is the percent coverage ratio. (100% coverage was assumed for this Example).

LAYER 1 $TMAX_1$ = (0.28)[(19.6)(0.5) + 20] (0.75) = 6.26 kN/m = $TMAX_1$

LAYER 2 $TMAX_2$ = (0.28)[(19.6)(1) + 20] (0.5) = 5.54 kN/m = $TMAX_2$

LAYER 3 $TMAX_3$ = (0.28)[(19.6)(1.5)+20](0.5) = 6.92 kN/m = $TMAX_3$

LAYER 4 $TMAX_4$ = (0.28)[(19.6)(2)+20](0.5) = 8.3l kN/m = $TMAX_4$

LAYER 5 $TMAX_5$ = (0.28)[(19.6)(2.5)+20](0.5) = 9.66 kN/m = $TMAX_5$

LAYER 6 $TMAX_6$ = (0.28)[(19.6)(3) + 20 (0.5) = 11.03 kN/m = $TMAX_6$

LAYER 7 $TMAX_7$ = (0.28)[(19.6)(3.5)+20](0.75) = 18.61 kN/m = $TMAX_7$

- PULLOUT CALCULATIONS AT EACH LAYER

$T_{max} < (1 / F_{SPO})(F)(\gamma)(z)(L_e)(C)(R_c)(\alpha)$

Where $F_{SPO} = 1.5$ $F = \tan \varphi$ Ci

R_c = % coverage of reinforcement (may vary from 100% to 71%). R_c assumed to be 100% for this example.

C_i = interaction coefficient determined from pullout testing for a particular reinforcement type.

$$L_e \geq \frac{1.5 T_{MAX}}{C \tan\theta C_i \gamma z R_c \alpha}$$

C = 2 for geogrids $C_i = 0.8$

γ = unit weight of soil.

z = depth below top of wall.

L_e = length of reinforcement in resistance zone.

α = scale effect correction factor ($\alpha = 1.0$ determined in laboratory tests performed on the geogrids used in this Example).

Layer 1	$L_e > 0.89 > 1$ m	use $L_e = 1$ m
Layer 2	$L_e > 0.39 > 1$ m	use $L_e = 1$ m
Layer 3	$L_e > 0.33 > 1$ m	use $L_e = 1$ m
Layer 4	Le > 0.3 > 1 m	use $L_e = 1$ m
Layer 5	$L_e > 0.27 > 1$ m	use $L_e = 1$ m
Layer 6	$L_e > 0.26 > 1$ m	use $L_e = 1$ m
Layer 7	$L_e > 0.38 > 1$ m	use $L_e = 1$ m

- CALCULATE LA / LAYER

LA = (H - d_i) tan (45 - φ/2) for geogrids (tan (45 - φ/2)) = 0.532

LA_1 = (4 - 0.5) (0.532) = 1.862 m

LA_2 = (4 - 1) (0.532) = 1.596 m

LA_3 = (4 - 1.5) (0.532) = 1.33 m

LA_4 = (4 - 2) (0.532) = 1.064 m

$LA_5 = (4 - 2.5)(0.532) = 0.798$ m

$LA_6 = (4 - 3)(0.532) = 0.532$ m

$LA_7 = (4 – 3.5)(0.532) = 0.266$ m

- CALCULATE LT AND COMPARE TO DESIGN LENGTH

(Geogrid lengths of 3.5 m control from external seismic stability analysis).

Layer 1 $LT_1 = 1.862 + 1 = 2.862 <3.5$ ˆ use 3.5 m

Layer 2 $LT_1 = 1.596 + 1 = 2.596 < 3.5$ ˆ use 3.5 m

Layer 3 $LT_1 = 1.33 + 1 = 2.33 < 3.5$ ˆ use 3.5 m

Layer 4 $LT_1 = 1.064 + 1 = 2.064 < 3.5$ ˆ use 3.5 m

Layer 5 $LT_1 = 0.798 + 1 = 1.798 < 3.5$ ˆ use 3.5 m

Layer 6 $LT_1 = 0.532 + 1 = 1.532 < 3.5$ ˆ use 3.5 m

Layer 7 LT1 = 0.266 + 1 = 1.266 < .5 ˆuse 3.5m

APPENDIX B

SIZE AND SPECIFICATION OF GEOGRIDS

MACCAFERRI

TECHNICAL DATA SHEET

Rev: 01, Issue Date 03.26.2009

PARALINK™ 1100

Product Description

ParaLink™ 1100 manufactured from high tenacity, multifilament polyester yarns placed in tension, then co-extruded with polyethylene to form polymeric strips. The polymeric strips are laid flat in the machine direction and a secondary member is laid and welded across the full width in the cross direction. The process generates a stable and strong geogrid. While polyester is the load bearing element maintaining minimal deformation, whilst the polyethylene sheathing maintains both the integrity of the product and encases the yarns protecting them from aggressive environments (such as high/low pH) and harsh installation conditions. **ParaLink™ 1100** is ideal for applications where reinforcement of soils is essential such as MSE walls, embankments over soft soil, steepened slopes, lagoon closures, load transfer platforms, basal foundations and any other geotechnical application in which soils require enhancement. **ParaLink™ 1100** has been tested internally and independently in accordance to published standards and will conform to the property values listed below. All values are Minimum Average Roll Values (MARV[1]) unless noted.

PROPERTY	TEST METHOD	M.A.R.V.
		Metric
Mechanical		
Tensile Strength (ultimate) Elongation @ Ultimate strength Creep Reduced Strength[2] Long Term Design Strength (LTDS)[3]	ASTM D6637 ASTM D6638	1100.0 kN/m 12% 797.1 kN/m 669.2 kN/m
Polymeric		
Carboxyl End Group (CEG Max.) Molecular Weight (# average)	GRI-GG7 GRI-GG8	< 30 mmol/kg >25000 M_w
Physical		
Grid aperture size (MD) Grid aperture size (XMD) Mass/Unit Area	 ASTM D5261	450mm 225mm 1087g/m²
Roll presentation[4][5]:		
Roll Dimension Roll Area Estimated Roll Weight	Width Length	4.50m 50m 225m² 234kg

Notes:

1. Minimum average roll values (MARV) are calculated as typical minus two standard deviations. Statistically, it yields a 97.7% degree of confidence that any samples taken from quality assurance testing will exceed the value reported.
2. Creep Reduce strength calculated as: $T_{creep} = T_{ult} / RF_{Creep}$ for 120 years design life @ 20 °C
3. LTDS calculated as: $LTDS = T_{ult} / (RF_{Creep} \times RF_{Installation\ Damage} \times RF_{Durability})$ in Coarse sand (SW) for 120 years design life @ 20 °C
4. The property values listed above are effective: April 1st 2008.
5. Width and length values per roll are **MINIMUM**. Roll area is **estimated** and rounded up to the closest square metre.
6. Maccaferri Inc. can engineer specific solutions in any of our products; please contact us if you may need a specific solution for your project.

Maccaferri Canada Ltd. reserves the right to amend product specifications without notice and specifiers are requested to check as to the validity of the specifications they are using.

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Calgary, AB tel: 403-244-6556
Vancouver, BC tel: 604-683-4824

MACCAFERRI

TECHNICAL DATA SHEET

Rev: 01, Issue Date 03.26.2009

PARALINK™ 1250

Product Description

ParaLink™ 1250 manufactured from high tenacity, multifilament polyester yarns placed in tension, then co-extruded with polyethylene to form polymeric strips. The polymeric strips are laid flat in the machine direction and a secondary member is laid and welded across the full width in the cross direction. The process generates a stable and strong geogrid. While polyester is the load bearing element maintaining minimal deformation, whilst the polyethylene sheathing maintains both the integrity of the product and encases the yarns protecting them from aggressive environments (such as high/low pH) and harsh installation conditions. **ParaLink™ 1250** is ideal for applications where reinforcement of soils is essential such as MSE walls, embankments over soft soil, steepened slopes, lagoon closures, load transfer platforms, basal foundations and any other geotechnical application in which soils require enhancement. **ParaLink™ 1250** has been tested internally and independently in accordance to published standards and will conform to the property values listed below. All values are Minimum Average Roll Values (MARV[1]) unless noted.

PROPERTY	TEST METHOD	M.A.R.V.
		Metric
Mechanical		
Tensile Strength (ultimate)	ASTM D6637	1250.0 kN/m
Elongation @ Ultimate strength	ASTM D6638	12%
Creep Reduced Strength[2]		905.8 kN/m
Long Term Design Strength (LTDS)[3]		854.1 kN/m
Polymeric		
Carboxyl End Group (CEG Max.)	GRI-GG7	< 30 mmol/kg
Molecular Weight (# average)	GRI-GG8	>25000 M_w
Physical		
Grid aperture size (MD)		450mm
Grid aperture size (XMD)	ASTM D5261	225mm
Mass/Unit Area		1094g/m²
Roll presentation[4][5]:		
Roll Dimension	Width	4.50m
	Length	50m
Roll Area		225m²
Estimated Roll Weight		246kg

Notes:

1. Minimum average roll values (MARV) are calculated as typical minus two standard deviations. Statistically, it yields a 97.7% degree of confidence that any samples taken from quality assurance testing will exceed the value reported.
2. Creep Reduce strength calculated as: $T_{creep} = T_{ult} / RF_{Creep}$ for 120 years design life @ 20 °C
3. LTDS calculated as: $LTDS = T_{ult} / (RF_{Creep} \times RF_{Installation\ Damage} \times RF_{Durability})$ in Coarse sand (SW) for 120 years design life @ 20 °C
4. The property values listed above are effective: April 1st 2008.
5. Width and length values per roll are **MINIMUM**. Roll area is **estimated** and rounded up to the closest square metre.
6. Maccaferri Inc. can engineer specific solutions in any of our products; please contact us if you may need a specific solution for your project.

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TECHNICAL DATA SHEET

Rev: 01, Issue Date 03.26.2009

PARALINK™ 1000

Product Description

ParaLink™ 1000 manufactured from high tenacity, multifilament polyester yarns placed in tension, then co-extruded with polyethylene to form polymeric strips. The polymeric strips are laid flat in the machine direction and a secondary member is laid and welded across the full width in the cross direction. The process generates a stable and strong geogrid. While polyester is the load bearing element maintaining minimal deformation, whilst the polyethylene sheathing maintains both the integrity of the product and encases the yarns protecting them from aggressive environments (such as high/low pH) and harsh installation conditions. **ParaLink™ 1000** is ideal for applications where reinforcement of soils is essential such as MSE walls, embankments over soft soil, steepened slopes, lagoon closures, load transfer platforms, basal foundations and any other geotechnical application in which soils require enhancement. **ParaLink™ 1000** has been tested internally and independently in accordance to published standards and will conform to the property values listed below. All values are Minimum Average Roll Values (MARV[1]) unless noted.

PROPERTY	TEST METHOD	M.A.R.V.
		Metric
Mechanical		
Tensile Strength (ultimate)	ASTM D6637	1000.0 kN/m
Elongation @ Ultimate strength	ASTM D6638	12%
Creep Reduced Strength[2]		724.6 kN/m
Long Term Design Strength (LTDS)[3]		683.3 kN/m
Polymeric		
Carboxyl End Group (CEG Max.)	GRI-GG7	< 30 mmol/kg
Molecular Weight (# average)	GRI-GG8	>25000 M_w
Physical		
Grid aperture size (MD)		450 mm
Grid aperture size (XMD)	ASTM D5261	225 mm
Mass/Unit Area		988 g/m²
Roll presentation[4][5]:		
Roll Dimension	Width	4.50m
	Length	50m
Roll Area		225m²
Estimated Roll Weight		222kg

Notes:

1. Minimum average roll values (MARV) are calculated as typical minus two standard deviations. Statistically, it yields a 97.7% degree of confidence that any samples taken from quality assurance testing will exceed the value reported.
2. Creep Reduce strength calculated as: $T_{creep} = T_{ult} / RF_{Creep}$ for 120 years design life @ 20 °C
3. LTDS calculated as: $LTDS = T_{ult} / (RF_{Creep} \times RF_{Installation\ Damage} \times RF_{Durability})$ in Coarse sand (SW) for 120 years design life @ 20 °C
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