A

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"ANALYSIS OF REINFORCED EARTH WALL"

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SUBMITTED BY

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CERTIFICATE

This is to certify that the project entitled **"Analysis Of Reinforced Earth Wall"** being submitted by me, is a bonafide record of my own work carried by me under the guidance of Dr. A.K.Gupta (Asst. Professor) and Mr. Manish Gupta (Senior Research Officer) in partial fulfillment of requirements for the award of the Degree of Master of Engineering with specialization in Structural Engineering, Delhi University, Delhi.

The matter embodied in this project has not been submitted for the award of any other degree.

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ABSTRACT

In the present work displacement mechanism and behavior of reinforced soils are studied under different soils and loading conditions. A comparative study of initial load-settlement relationship, distribution of axial and shear forces in the reinforcement and stress distribution in reinforced soil structures is presented.

Finite element analysis is carried out using commercial software PLAXIS version 8 for this problem with different soil conditions. The results are compared and reported in this dissertation.

Behaviors of reinforced soil structures under different conditions are investigated using PLAXIS version 8. Effect of the soil reinforcement is shown through the improvements in the load-deformation relations, the reinforcement axial force, stress distribution and displacement in the soil mass. The effect of spacing of geogrids is explained through displacements variation. Wall displacements and strain distribution along geogrid layers are observed. Effectiveness of the reinforced soil wall is also evaluated using a geogrid material.

Substantial improvement in the response of the soil structure due to the soil reinforcement is demonstrated through the model test results. Through analysis of reinforced earth model, a new concept on the positioning of reinforcements has been recommended depending on the reinforcement type and soil.

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CHAPTER-1 INTRODUCTION

1.1 General

Reinforced earth is composite material formed by interaction between frictional soil and reinforcing strips. Reinforced earth is an economical means of improving the mechanical properties of basic material i.e. soil, by reinforcing it with another material; steel The reinforcing strips resist stresses produced within the soil mass; stresses are transferred via friction.

Over the past 20 years, designers of railway systems worldwide have found Reinforced Earth structures ideally suited for support of track bed, bridge and trestle abutments, earth retention structures adjacent to right of ways, and for deflector walls to protect bridge piers from impact in the event of a derailment. In addition to versatility, speed of construction and economy, Reinforced Earth structures require very little space, and may be built entirely from the backfill side of the retaining wall. This allows construction of structures right up to an adjacent railroad right-of-way. Reinforced Earth is a strong and versatile construction material created by the frictional interaction of granular soil and steel reinforcing strips.

The past three decades have shown great achievements in the advancement of reinforced soil system using stiff metal to flexible extensible geosynthetic materials as reinforcing elements. Many reinforced soil structures have been performing well and are considered safe and convenient in construction. Parallel to the advancement in the construction technology, in these years a lot of efforts has been devoted to find a suitable method/procedure for the analysis and design (e.g. Vidal, 1966; Schlosser and Long, 1972; Haussman, 1976; Chapius, 1978; Yang, 1972; ASCE, 1978; Jarret et al., 1987, Tatsuoka, 1992; Yamanouchi et al., 1978; Ochiai, 1992). Many assumptions have been postulated and many solution procedures have been proposed about the mechanism of different components comprising these systems (e.g. reinforcement force, soil- reinforcement- facing interaction) and the mechanism is still not well understood. The commonly accepted analysis and design method is still lagging.

In the analysis of most soil engineering problems, specially reinforced soil structure, stability and deformation are considered both critical and independent but they are always dealt separately. In this dissertation, these two aspects of the behavior of reinforced soil structures are studied introducing some mechanisms to model the spacing of reinforcement and and its lenth in different soil conditions.. The length along reinforcing element is assumed to be constant by imposing a constrained condition of no-length change.. Further, the difference between soil anchors and soil reinforcement is distinguished through the axial/shear forces developed along the soil reinforcement and soil anchors. In the former, the axial force can be controlled externally (out side the soil/anchor system) while in the latter (i.e. soil reinforcement) case, the axial force is not externally controllable, rather develops internally due to soil-reinforcement interaction depending on the confining pressure. In this context, the conventional methods of the reinforced soil structures that require the tensile force distribution along reinforcements be prescribed, as an initial condition cannot be accepted, at least, from the theoretical point of view.

Thus, the deformation behavior of reinforced soil structure under a different loading stage is studied by modelling a reinforced soil structure in finite element method FEM based PLAXIS version 8 software. The modelling is demonstrated through some typical soil engineering deformation problems and the results reveal that the reinforcement is much effective in reducing the lateral deformation in addition to vertical settlements of reinforced soil mass.

1.2 SCOPES AND OBJECTIVES OF PRESENT STUDY

In the present work an attempt has been made to study the behaviour of reinforced soil mass under different loading conditions. The studied were carried out by using the FEM based software PLAXIS version 8 to create different models and analyse the behaviour of different components of renforced soil structure. A comparative study has also been made on the load-deformation relations using the reinforcement axial force, spacing of geogrid and displacements in the soil mass.

2.1 GENERAL

Strength of the natural fill soil in earth structures is improved by various techniques, e.g., mechanical processes, chemical process, inserting a strong material into the soil mass (sand compaction piles, bamboo strip, straw, etc.) and the interesting one is natural plant roots. Besides these natural and traditional techniques, the important development of Reinforced Earth, and the concept of reinforced soil as construction material, introduced by its inventor French architect

H. Vidal, in the sixties, have introduced the modern form of soil reinforcement technique (Schlosser and Delage, 1987). This technique has been used in various structures, e.g. slopes and embankment, retaining walls, foundations, dams and others. Mitchell (1981) noted that no other soil improvement techniques have been so intensively studied and having advanced application in the past several years, as has soil reinforcement.

The concept of soil reinforcement is based on the existence of strong soil-reinforcement interaction like roots, due to their tensile strength and frictional or adhesion properties reinforce the soil. Many hypotheses have been postulated, in the past *25* years, about the load transfer between the soil and reinforcement and their interaction. A lot of research has been carried out to find suitable method for the analysis and design of reinforced soil structures.

2.2 HISTORY AND DEVELOPMENTS ON REINFORCING SYSTEMS

Development of Reinforced Earth

Henri Vidal (1963) invented the Reinforced Earth and much of the current development can be attributed to his pioneering work. Vidal introduced the basic mechanism underlying reinforced soil behaviors in his first paper published in 1966. Reinforced Earth is a composite construction material in which the strength of fill is enhanced by the addition of strong inextensible as well as extensible reinforcing materials. The basic mechanism of Reinforced Earth involves the generation of frictional interaction between soil and reinforcements (Schlosser and Delage, 1987).

York Method:

Jones (1973) developed the York method, which is similar b the Reinforced Earth technique except two minor differences, regarding facing units and sliding mechanism of reinforcements. The York method is the first reinforced soil wall totally built with plastic material (Schlosser and Delage, 1987). According to Jones (1978), differential settlements can easily be accommodated in the sliding mechanism.

GRS-R W System:

Geosynthetic-reinforced soil retaining wall (GRS-RW) system, developed in Japan, is a hybrid wall system of mechanically reinforced earth wall with a cast-in-place full-height rigid facing . Some advantages of GRS-RW system are small lateral deformation due to full height continuous rigid facing, and excavation may not be required because of short reinforcements. This system can be used in sites e.g. bridge abutment or laterally loaded walls.

Miscellaneous:

There are several other reinforcing systems developed by many manufacturers used for particular purpose and suitable for typical site conditions. Tervoile, Websol system, Cellular Confining system, Genesis Highway Wall System consisting of Tensar structural geogrids, Con-wall system, etc. are interesting systems to be noted here.

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2.3 TYPES OF REINFORCING MATERIALS

In the literature, there mainly two groups of reinforcements, extensible and inextensible, are discussed with respect to the stress-strain response of soil mass. Stress-strain characteristics of typical inextensible and extensible reinforcing materials are illustrated in Fig. 2.1. McGown et al. (1978) originally defined inextensible and extensible reinforcements and Bonaparte et al. (1987) extended as follows:

(a) **Inextensible reinforcement** is reinforcement used in such a way that the tensile strain in the reinforcement is significantly less than the horizontal extension required to develop an active plastic state in the soil. An “absolutely” inextensible reinforcement is so stiff that equilibrium is achieved at virtually zero horizontal extension (K0 conditions prevail)

(b) **Extensible reinforcement** is reinforcement used in such a way that the tensile strain in the reinforcement is equal to or larger than the horizontal extension required developing an active plastic state in the soil. An “absolutely” extensible reinforcement has such a low modulus that virtually no tensile forces are introduced to the soil mass at the strain required to develop an active plastic state (Ka conditions theoretically prevail)

Bonaparte et al. (1987) considered steel reinforcement as an inextensible reinforcement and geosynthetic reinforcing materials as extensible reinforcements, for almost all practical applications. Thus, an inextensible metallic reinforcement makes the structure brittle and the extensible geosynthetic increases the ductility of the reinforced soil structure (Fig. 2.2).

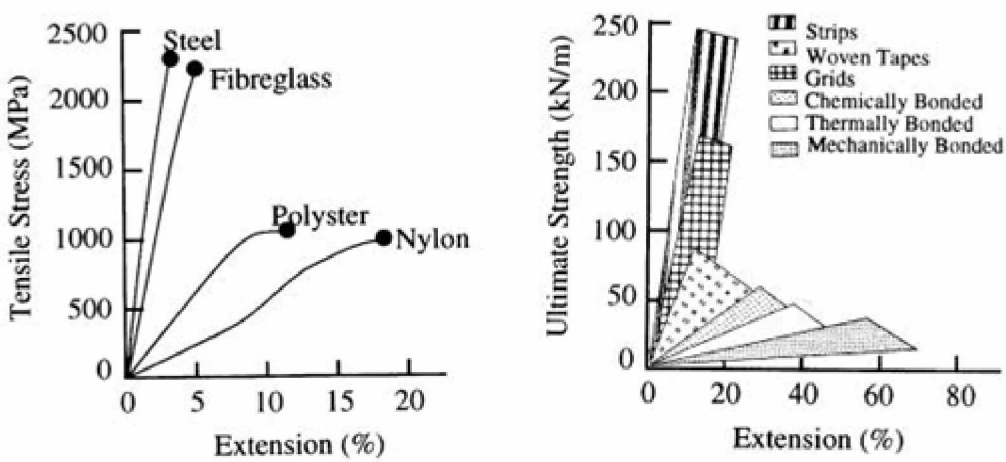
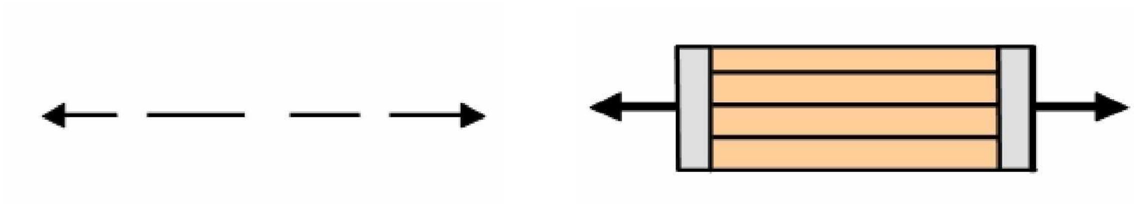


Figure 2.1 Stress-strain characteristics of typical reinforcing materials
(McGown, A., K.Z. Andrawes, M.M. Al-Hasani (1978))

2.3.1 Inextensible reinforcements

Steel Bars fiber glass reinforcements:

The choices on the reinforcing material vary from inextensible reinforcements like steel, fiberglass to extensible polyester resins. Galvanized steel has been used in wide variety of environments over very long periods, thus, its corrosion mechanism and the rate of corrosion have been known for long time. Similarly, polyester coated fiberglass, stainless steel and aluminum are also used. The corrosion rate of these metals is faster than galvanized steel. Despite these drawbacks, the steel and fiberglass reinforcing materials have also gained popularity specially when the construction requires less post construction deformation such as in the case of bridge abutments, railway embankments, etc. The advantage of steel and fiberglass is due to their unique combination of elasticity, ductility/stiffness and favorable economics. Bonaparte et al.(1987) states that the tensile stiffness of steel reinforcements is stiff enough to keep the state of soil stress close to the at-rest (K0) condition.



*(a)*Series failure (inextensible reinforcements)(b) Parallel Failure (extensible reinforcements)

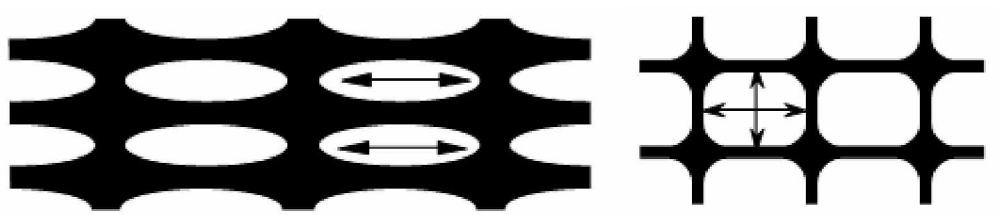
Figure 2.2 Analogy of reinforced soil fail mechanisms

(Bonaparte, R. and G.R. Schmertmann (1987))

2.3.2 Extensible Reinforcements

Geosynthetic and related products.

Major geosynthetic materials currently used as reinforcements in soil structures are geogrid sheet (Fig.2.3), woven and non-woven geotextile sheet, coated fiber strips, rigid plastic strips, composites and three-dimensional honeycomb type products. Geosynthetic materials have large ranges of deformation modulus and tensile strengths compared to metals (Fig.2.2). Geosynthetic materials also exhibit creep behavior. Bonaparte et al.(1987) has grouped geosynthetic reinforcements as extensible reinforcements, thus, the state of soil stress is far from at-rest (K0) condition.



(a) Uniaxial geogrid (b) Biaxial geogrid

Figure 2.3 Typical geogrids used as soil reinforcement mechanisms.

(Jones, C.J.F.P. (1994))

2.3.3 Miscellaneous

There are several other types of reinforcing materials used for particular purposes. Small inclusions (fibers, small plates) or continuous filaments (e.g. Texsol) are some typical reinforcing materials. Sometime natural materials (e.g. bamboo, jute) are also used as reinforcing material. In UK and USA, redundant car tires have been used as reinforcement.

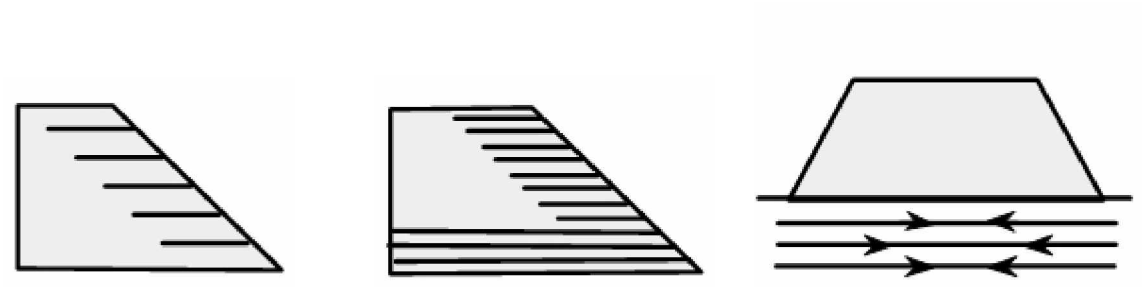
2.4 APPLICATIONS OF REINFORCED SOILS

More common applications of reinforced soil are in the form of retaining walls. Reinforced soil structures can be grouped into three classes (Ingold, 1982), (a) Embankment and retaining walls, (b) Foundations / sub-soil reinforcements and (c) In-situ reinforcement *oil nailing)*- existing slopes and excavations.

2.4.1 **Embankments/ Retaining Walls**

Several reinforcing systems with varieties of reinforcing materials and facings have been successfully used to construct many reinforced embankment and retaining walls

A primary role of reinforcement in an embankment or a retaining wall is to support the outward earth pressure (lateral thrust) in the fill while maintaining the full bearing capacity in the foundation. The reinforcement provided at the embankment base prevents lateral displacements of the embankment and foundations soils, subsequently the bearing capacity of the soft soil and stability of embankments are increased significantly. Purpose of these reinforcements is to perform as (i) superficial slope reinforcement and edge stiffening; (ii) main body reinforcement; (iii) reinforcement at the base of the retaining walls. Reinforcement in the main body is essentially the major application of reinforcement in reinforced embankment or retaining wall structures.



(a) Superficial reinforcement (b) Body reinforcement (c) Foundation reinforcement

Figure *2.4* Embankment reinforcing modes. (Ref: Ingold, T.S. (1982))

2.4.2 Subsoil Reinforcement Beneath Foundations

In the soil beneath the reinforced soil foundation two distinct zones are formed (e.g., Binquet - Lee, 1975), and John, 1987) as shown in Fig. 2.5. In the first zone, the wedge of soil directly beneath the structure is forced vertically downwards (punching failure) whilst outside the footing, there are symmetrical zones which have both lateral and upward movements, the function of an effective reinforcement being to hold these two zones together. Binquet-Lee (1975), Oka et al. (1992), Takemura et al.(1 992) and other researchers reported that the maximum bearing capacity ratio occurs at a depth ratio 0.8 to 1.0.

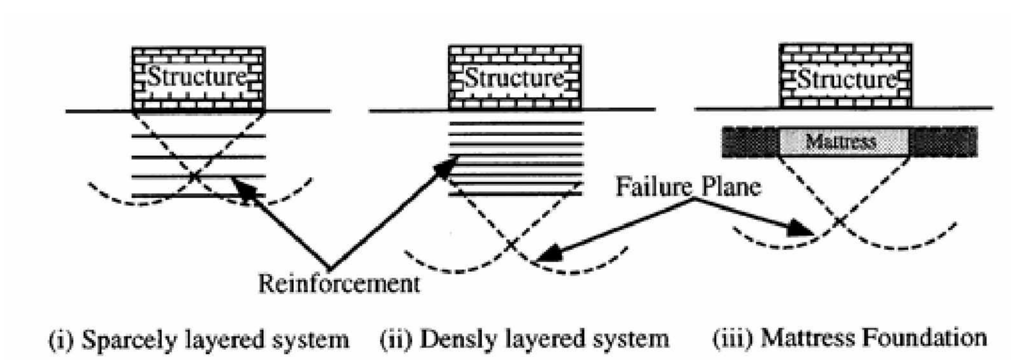


Figure 2.5 Effect of sub-soil reinforcements. (Ref: Ingold, T.S. (1982))

2.4.3 In-situ Reinforcement *(soil nailing): slope stability/excavation*

Soil nailing is an in-situ soil reinforcement technique, which has been used during the last two decades. Soil nailing is being used at present to stabilize natural slopes, cuts or excavation, walls in stiff clays, granular soils (with some suction) and also soft rocks. The purpose of this technique is essentially to limit the decompression and the opening of pre-existing discontinuities by restraining the deformations. They are usually steel rods 20-30 mm in diameter that are inserted into the soil either by simple driving or by grouting in predrilled borehole (Fig.2.6). Soil nailed slopes behave like a reinforced soil wall although there are some major differences between these two techniques, e.g.,

i Construction method: Soil nailed slopes have top-downwards construction method whereas reinforced soil walls are constructed from the bottom upwards

ii. Shear and bending stresses may develop in soil nails depending on the stiffness of the nails relative to soil, while this is not generally observed in soil reinforcements.

iii. Soil nailing is applied to existing soil slopes and may therefore involve more cohesive soils than the selected fills used for reinforced soil walls.

iv. Soil reinforcement sheets or strips are usually laid horizontally, whereas soil nails are usually driven at an inclined angle.

Schlosser (1982) observed that the active failure zone for nailed slopes was similar to, but larger than, that of a reinforced soil wall. In both cases, the active failure zone is smaller than the standard Coulomb active wedge assumed with the other retaining structures. He suggested that this difference in behavior is attributable to the inclination of the soils nails.

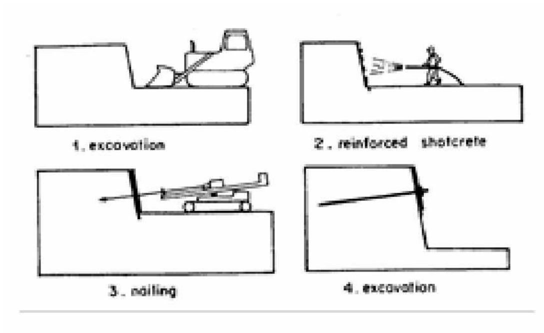


Figure 2.6 Typical in-situ soil-reinforcing techniques.
(Schlosser, F.(1990))

2.5 CONCEPTS AND MECHANISM OF REINFORCED SOIL

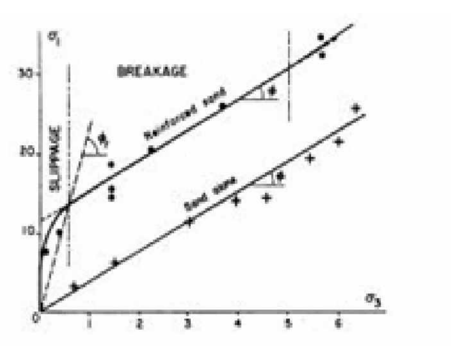
Several experimental and theoretical investigations have been performed since the invention of Reinforced Earth wall (Vidal, 1963) to understand the concepts and mechanism of reinforced soil structure and interaction among its basic components, generally, reinforcing elements, backfill soil and facing. H. Vidal, the pioneer of Reinforced Earth system seems to be the first person to propose a general and realistic concept of reinforcing a soil.

Anisotropic Cohesion Concept

Schlosser and Long (1972) indicated that the reinforced soil has higher shear strength than unreinforced plain samples (Fig.2.7). Hausemann (1976) independently postulated a more unified anisotropic cohesion theory. They have shown that two failure modes can develop in such reinforced sand samples: (a) failure by slippage of the reinforcement at low confining pressure leading to a curved yield line passing through the origin and (b) failure by reinforcement breakage at higher confining pressure leading to a straight failure line which proves that the reinforced sand behaves as a cohesive material having the same frictional angle as the original sand and an anisotropic pseudo-cohesion due to reinforcements as shown in Fig. 2.8. This pseudo-cohesion is very rapidly mobilized at low axial deformations.

Enhanced Cohesion Concept

Chapius (1972) and Yang (1972) independently presented *enhanced cot fining pressure concept* on the mechanism of reinforcing a soil mass. This concept is based on the assumption that the horizontal and vertical planes are no longer principal stress planes due to the shear stresses induced between the soil and reinforcements. Mohr's circle of stress is shifted due to reinforcing of the soil mass (Fig. 2.8b) while failure envelope remained same for both reinforced and unreinforced samples. Such effect is called enhanced confining pressure effect.



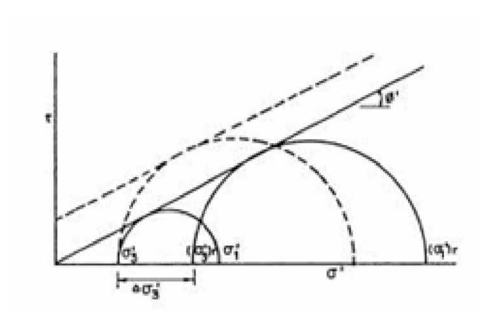


Figure 2.7 Reinforced and unreinforced samples in triaxial tests

Figure 2.8 Anisotropic Cohesion and Enhanced Cohesion Concepts

Ref: Schlosser, F. (1990): Mechanically Stabilized Earth Retaining Structures

2.6 BEHAVIOR OF REINFORCED SOIL STRUCTURES

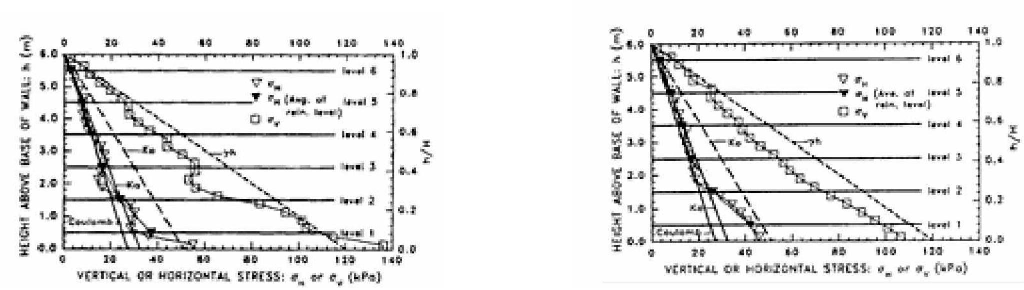
In the analysis and design of reinforced soil structure, stability and deformation are considered both critical and independent concerns for a soil structure and they are always dealt separately. Past research reveals that major work was concentrated on stability analysis compared to the deformation problems. In deformation analysis, serviceability with respect to excessive differential settlement and horizontal deformation of the slope face are considered important. The stability analysis of reinforced soil structures is divided into internal and external stability analyses (Gourc, 1992; Rowe and Ho, 1992) as will be illustrated in later sub-sections.

Rowe and Ho (1993) suggested that the overall behavior of a reinforced soil structure may be considered known if one understands:

(a) State of stress within the reinforced soil mass.
(b) State of strain in both the soil and the reinforcement.
(c) Axial force distribution in the reinforcement.
(d) Horizontal soil pressure acting at the back of the reinforced soil mass and the vertical soil pressure at the base.
(e) Vertical soil stress on each reinforcement layer.
(f) Horizontal soil pressure acting at the face.
(g) Horizontal and vertical forces transferred to the wall face.
(h) Horizontal deformation of the reinforced soil mass
(i) Effect of varying the design parameters (i.e. reinforcement stiffness, soil properties, reinforcement spacing , surcharge condition, construction procedures, etc.) on the response of the system.

2.6.1 Vertical and Horizontal Soil Stress Distribution:

Several types of vertical stress distribution patterns are assumed in the analysis and design of reinforced soil mass. Uniform, trapezoidal, Meyerhof distributions and 2:1 stress dispersion method are typical examples. Maximum stress is attained within the reinforced zone. Close to the far end of reinforced zone the vertical soil stress reaches a minimum. Further away into the unreinforced retained fill, the vertical soil stress attains the minimal value. The vertical soil stress close to the facing depends on the facing rigidity (Tatsuoka, 1993). Rigid facing decreases the vertical soil stress close to the facing due to load transfer from the soil to the facing. Such effect of the facing leads to higher reinforcement force and requires higher bearing capacity in the design of foundations. Horizontal soil stress primarily depends on the number of reinforcement layer, the stiffness and the creep of the reinforcement and the degree of yielding of the wall face as shown in Fig.2.9. Relative deformation of the wall face and soil with the reinforcement results increased transfer of horizontal stress to reinforcement rather than to facing. The horizontal soil stress increases as the number of reinforcement layers is increased. Rowe and Ho (1993) noted that there are no literatures giving any real observed information on the horizontal soil stress distribution further back into the reinforced soil.



(a) at wall face (b) at back of reinforced soil block

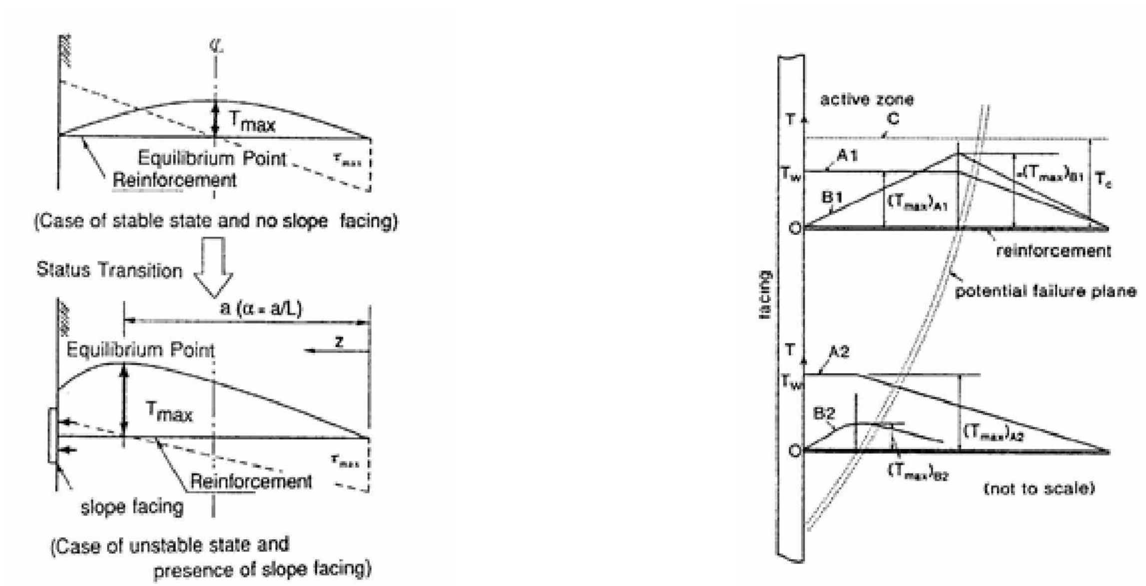
Figure 2.9 Vertical and Horizontal soil stress distributions from numerical analysis (Ho and Rowe. 1992)

2.6.2 Force in Reinforcement

The magnitude of reinforcement force primarily depends on the shear strength mobilized in the backfill, the horizontal soil strain, the stiffness of the reinforced system, and the creep of reinforcements. Maximum tensile force close to toe is usually observed less than predicted by the Rankine active condition . Fannin (1991), Jewel (1987) and Ho-Rowe (1992) indicated that the maximum force in reinforcement becomes more uniform with decreasing reinforcement stiffness and lower near the bottom due to the influence of foundation.

Variation in soil properties and construction methods results shifting of the position of maximum tensile forces away from the failure plane. It also depends on the length and stiffness of reinforcements. Jewell (1987) stated that the locus of maximum tensile force will always be inclined to $45+\phi/2$ to the horizontal if the soil-reinforcement interface is sufficiently bonded, otherwise, the locus will move towards the facing. The maximum tensile force shifts towards the facing in the case of short reinforcements.

Force distribution in a reinforcement layer. The force distribution in a reinforcement layer is most influenced by the construction method, the existence of facing, the lateral restraint of facing during construction and the facing reinforcement connections. There are two general type axial force distributions as shown in Fig.2. 10(a)&(b).



(a) Muramatsu et al. (1992)   (b) Tatsuoka (1992)

Figure 2.10 General tensile force distribution patterns along a reinforcement.

Type A: This pattern is observed when lateral deformation of the wall face is restrained till the end of construction, e.g., ideal pull-out test. In this situation the maximum tensile force is induced at the back of the facing and remains more or less constant up to the potential failure plane and decreases to zero close to inner end of the reinforcement. When perfect lateral restraining of facing during construction is not possible, the tensile force in the reinforcement at the back of facing may be much smaller than its maximum value attained near the potential failure surface.

Type B: The parabolic tensile force distribution is observed when facing provides little or no lateral restraint against deformation e.g. wrapped back facing, slope face without any facing. The maximum force in the reinforcement is assumed to occur at the potential failure plane as shown in Fig. 2.10(b).

2.6.3 Horizontal Displacement

Magnitude of horizontal movement depends on the interaction between various components of reinforced soil structure and construction methods. Higher reinforcement density and stiffness reduce the strain in the soil, and larger shear strength of fill results in less force in the reinforcement, being required to maintain equilibrium and hence less deformation. The soil movement behind the reinforced zone depends on the strain level of the unreinforced zone above the stable slope.

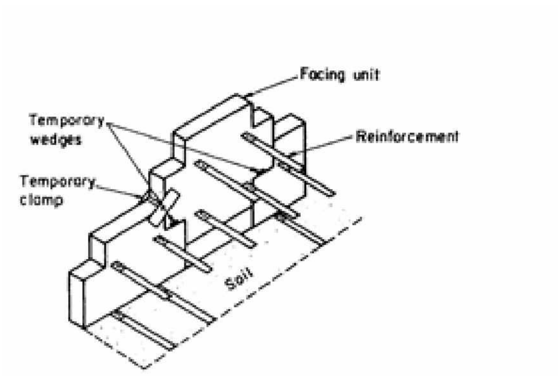
2.6.4 Role of Facing rigidity:

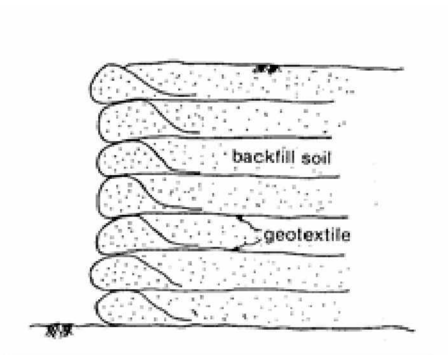
Currently, facing material ranges from rigid full- faced concrete facing to flexible wrapped around geosynthetic facing as shown in Fig. 2. 11(a-f). Most of the soil reinforced stabilization techniques assume that facing does not play a significant structural role; they are rather used for aesthetic reason . However, Tatsuoka (1993) has demonstrated the roles of the facing in improving the stability of reinforced soil structures based on extensive literature review. Horizontal movement of the wall face and subsequent earth pressure development within the reinforced zone as well as the reinforcement force are significantly affected by the facing rigidity.

Tatsuoka (1993) has classified various types of facing according to the degree of facing rigidity. The facing rigidity increases the stability of wall in the following three ways:

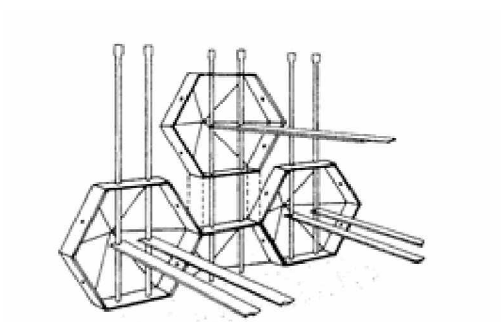
1. Rigid facings (Types D and E) support the combination of earth pressure and tensile force in reinforcement.

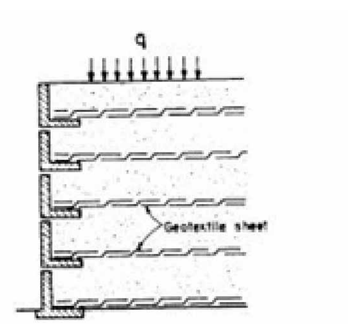
2. Weight of backfill is partly transmitted to the facing through the frictional force on the back face.
3. Due to high confining pressure behind rigid facing, the location of the overall reaction force becomes closer to the facing.



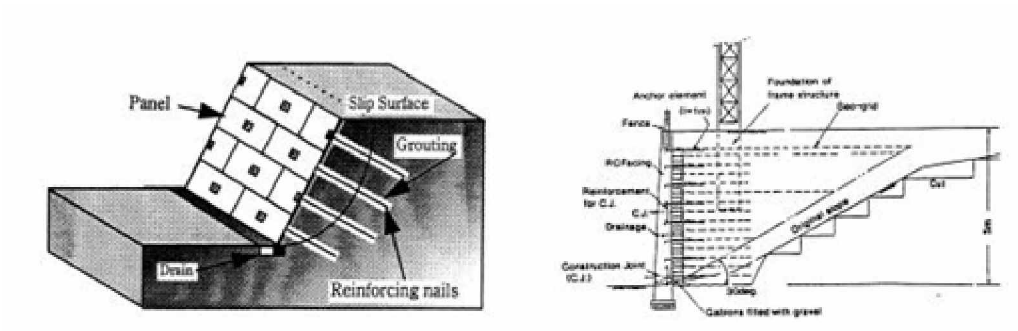


(a) Concrete Panel facing *(Reinforced Earth system)* (b) Wrapped around facing





(c) York wall facing (d) L-shaped concrete facing



(e) Reinforced Concrete Panel (f) Full Height Rigid
Reinforced Concrete

Figure 2.11 Currently used typical facings in reinforced soil structures. *(Jones*, 1992)

Tatsuoka et al. (1989) studied the effect of facing rigidity in a set of GRS-RWs model tests having facing Types A, D. The test result reveals that the location of failure surface moved from an intermediate elevation to the bottom of the facing depending on the facing rigidity. The ratio of earth pressure on the back of the facing to q remained almost constant with the facing rigidity.

Similarly, the tensile force just behind the facing is greatly influenced by the facing rigidity . Location of Tmax (Fig. 2.10) approaches back of the facing with increasing facing rigidity. Thus, the contribution of the facing rigidity on the stability of the reinforced soil structure was clearly demonstrated and similar conclusions are also reported by several other researchers (e.g., Juran-Schlosser, 1978, Bolton-Pang, 1982, and Koga et al., 1992).

2.7 **TYPICAL CURRENT DESIGN METHODS**

For the analysis and design of reinforced soil structures numerous approaches have been developed. All methods are either empirical in nature or based on limit equilibrium analysis. These methods don’t consider either the stress- deformation characteristics of the structure or the interactions between the wall components e.g. the soil, the reinforcement, the facing and the foundation. Their main purpose is to compute the factor of safety against several modes of failure. In general, the design methods use the allowable strengths (corresponding to each components) which are significantly lower than the ultimate strengths and further partial safety factors are applied to account for the uncertainties in the behavior of the reinforcement and soil/reinforcement interaction mechanism. As a consequence, these methods are lagging in adequately describing the real behavior of the reinforced soil structures. Hence, their application typically introduces an extra level of conservatism. Rimoldi (1988) based on eight case histories reported that current design methods are conservative.

Most of the current design methods can be divided into two main categories. The first category use simple force equilibrium analysis where the horizontal forces developed in the reinforcement balance the destabilizing horizontal force from the soil. The forces considered in these methods are:

a. the vertical soil stress,

b. the horizontal soil stress,

c. the stress in the reinforcement and

d. the horizontal resistance to pull-out of the reinforcement behind the potential failure plane.

Two independent factors of safety, for reinforcement rupture and pullout resistance, are calculated for each layer of reinforcement. The methods in the second category evaluate the force and or moment equilibrium on an assumed failure surface similar to conventional slope stability analysis but with the inclusion of the balancing force/moment developed in the reinforcement.

2.7.1 Force Equilibrium Methods

Some of the widely used force equilibrium methods for the design of numerous reinforced soil structures are as follows:

1. Jewell method (1987)- This method was proposed and applied first to predict the performance of Royal Military College trial wall in 1987. In this method, the reinforced soil structure is divided into 3 zones based on the reinforcement force as shown in Fig.2. 12

Zone-1: The zone between the wall face and the most critical surface where the reinforcement force required to maintain equilibrium is constant (i.e. between the surface and wall face). Thus, the most critical surface was defined as a surface through the toe that requires the greatest total reinforcement force to maintain equilibrium on this surface. The surface in vertical wall case is inclined at an angle $\Phi=(45+/2)$ to the horizontal as shown in Fig.2.12

Zone-2: This zone is confined between the aforesaid most critical surface and the locus of zero required force as shown in Fig.2. 12. A surface beyond which no additional stresses are required from the reinforcement to maintain equilibrium is called the locus of zero required force. Ideally beyond this zone the reinforcement can be truncated and equilibrium can be maintained by soil itself Such length of the reinforcement is called the ideal reinforcement length.

Zone-3: The zone beyond the locus of zero required force is in equilibrium without requiring any reinforcements.

Jewell (1987) proposed uniform spacing and ideal spacing pattern for reinforcement spacing. He further explained a truncated length concept and consequences of the truncation in the design. He also provided several design charts.

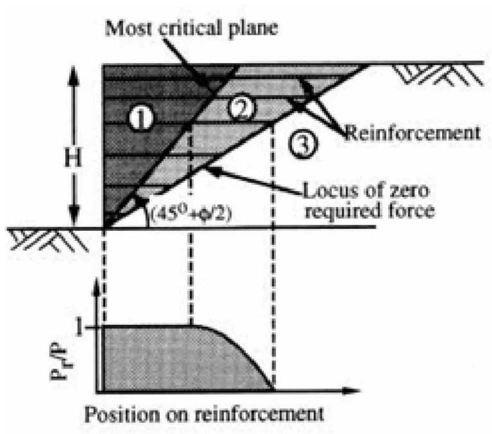


Figure 2.12 Reinforcement layout and force distribution for ideal length case. (Jewell, R.A. (1988))

2. Bonaparte et al. method (1987) - In this design method, the extensible and inextensible reinforcements are clearly distinguished. Then, the influence of reinforcement extensions is evaluated by defining hyperbolic relations between KEh. Detailed explanation about the method may be referred to Bonaparte et al.(1987).

3. Tie back design method (1978)- Tie back method was originally developed by the U.K. Department of Transport (1978) and is based upon limit equilibrium methods. It is independent of the reinforcement material and is used with both inextensible and extensible reinforcement and with anchors.

2.7.2 Slope Stability Methods

Many basic methods have been derived from the conventional slope stability studies; the most widely used (Rowe and Ho, 1992; Smith, 1992) being the Fellenius or Bishop methods or the Wedges methods. There are three noticeable differences among these methods as follow: a. the shape of the failure surface b. the distribution of force in the reinforcement and c. the means by which a surcharge is considered. Typical slope stability methods are as follows:

Fellenius Method:

In this method, it is assumed that for each slice the resultant of the interslice forces is zero. Taga et al.(1992) have summarized all the possible combination of various forces based on the Fellenius (simplified) method used in the analysis and design of reinforced soil structures where the basic computational formula used is as follows *:*

Sliding & Safety Factor,

$$\mathbf{F}_S = \frac{\textit{Force resisting sliding}}{\textit{Force inducing sliding}} = \frac{\sum \left[\, cb + W \cos\alpha \tan\phi \,\right]}{\sum W \sin \alpha}$$

where, W: weight of sliced blocks

b: length of sliding plane in sliced block

ϕ: Internal friction angle of sliding surface

c: cohesion of sliding surface

a: inclination of sliding surface with horizontal.

There are two reinforcement effects of the tensile force generated in the reinforcements in the sliding surface (see Fig. 2.13).

(1) Anchoring effect, $T \cos\alpha$

(2) Confining effect, $T \sin\alpha \cdot \tan \phi$

Regarding the confining effect (2), involves the equation, Eq.(2.1), and regarding the anchoring effect, two possible conditions arise, it may be considered as a resisting force (numerator) and as a sliding forced (denominator). Sometime, both effects are considered simultaneously together depending on the problem. Thus following five combinations can be derived by coupling these two effects with the Eq.(2. 1).

Formula (a)

$$Fs = \frac{\sum \left[cb + W \cos\alpha \tan\phi + T \cos\alpha \right]}{\sum W \sin\alpha}$$

Formula (b)

$$Fs = \frac{\sum \left[cb + W \cos\alpha \tan\phi \right]}{\sum (W \sin\alpha - T \cos\alpha)}$$

Formula (c)

$$Fs = \frac{\sum \left[cb + W \cos\alpha \tan\phi + T \sin\alpha \tan\phi \right]}{\sum W \sin\alpha}$$

Formula (d)

$$Fs = \frac{\sum \left[cb + W \cos\alpha \tan\phi + T \cos\alpha + T \sin\alpha \tan\phi \right]}{\sum W \sin\alpha}$$

Formula (e)

$$Fs = \frac{\sum \left[cb + W \cos\alpha \tan\phi + T \sin\alpha \tan\phi \right]}{\sum (W \sin\alpha - T \cos\alpha)}$$

Bishop Method.

In this method, it assumed that the resultant forces on the sides of the slices are horizontal. Thus, moment equilibrium is checked in this method as follows (refer Fig. 2.13):

$F_s = (M_R + \Delta M_R)/M_D$

where M_D = sliding moment, M_R =resisting moment of soil,

ΔM_R =resisting moment of geogrid, ΔM_R =R T_i, R =radius of slip circle, and T_i =sum of tensile strengths of geogrid. A typical formula for computing the factor of safety based on Bishop's Method is:

$$F_s = \frac{\sum \left[cb + (WP + - ub + T \sin\gamma)\tan\phi \right]}{\sum \left[W \sin\alpha + P \sin\alpha - T\cos(\alpha + \gamma) \right]}$$

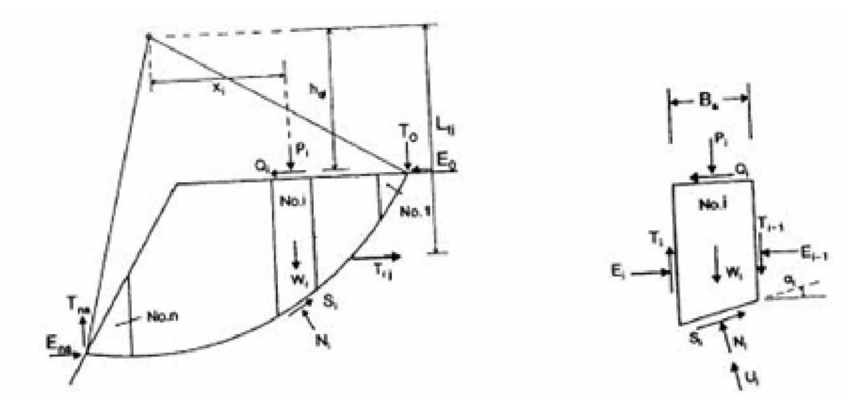


Figure 2.13 Bishop's Simplified Method of analyzing reinforced soil structures (Alan Mc Gown, Khen Yeo and Andrawes)

Trial Wedges Method.

Slip surfaces in the trail wedge method can be assumed to be two straight-line slips caused by the horizontal earth pressure, similar to the experimental data.

$$F_s = \frac{\sum Ti}{P_H}$$

In this equation,

P_H = horizontal earth pressure and

$\sum Ti$ = sum of tensile strengths of the geogrid.

Total horizontal earth pressure components of the two straight- line slips, divided into two areas, Zone-1 and Zone-2, as shown in Fig. 2.14, can be obtained based on the concept of force polygons. It can be determined that the embankment is stable tien the external force of restraining wall acting is larger than P_H.

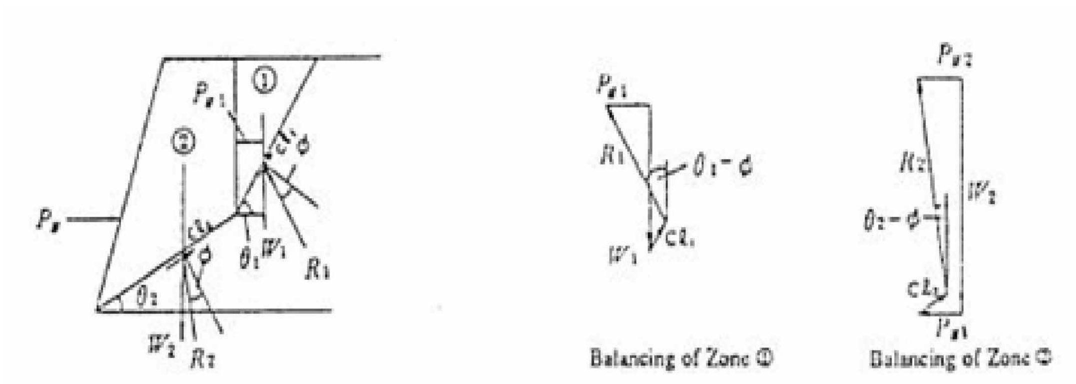


Figure 2.14 Trial wedge method of analyzing reinforced soil structures (*Taga* et al., 1992)

2.7.3 Failure Modes

Sometimes several possible failure modes are checked in reinforced soil walls depending on type of the structure itself and the field conditions. Generally, five independent types of failure (i.e. limit) modes are suggested sufficient enough for most of the geotechnical design problems (Bolton, 1989). These failure modes are grouped into two (external and internal) stability criteria. Typical failure modes that are checked (Jones, 1993) in the design of reinforced soil structures are as 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.

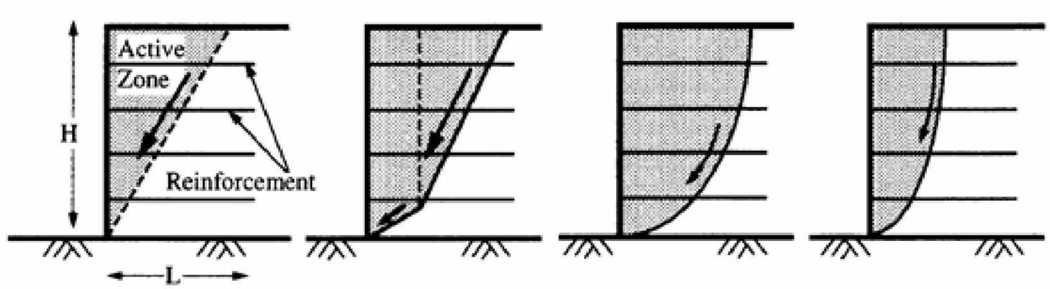
d. Bearing capacity failure (punching) of the foundation soil under the reinforced soil.

e. Overall collapse of the reinforced wall or embankment or nailed slope.

Internal Stability

a. Rupture failure of reinforcement

b. Pull-out failure of reinforcement



(a) Straight wedge (b) Two-part wedge (c) Circular arc (d) Logarithmic spiral

Figure 2.15 - Common shapes for potential failure surfaces for

2.8 FINITE ELEMENT ANALYSIS

Finite element method (FEM) is vigorous well known method of numerically solving boundary value problems which can accommodate highly non- linear stress- strain relations of materials including even creep, any geometrical configuration with complex boundaries, construction sequence, etc. FEM has been used as the standard tool for the design and analysis (e.g. prediction of safety factor and settlement analysis) of many geotechnical structures. Similarly, it is becoming a design and analysis tool for the reinforced soil structures. These features of FEM can be achieved only when material parameters, constitutive equations and boundaries are appropriately defined or modeled.

2.8.1 GENERAL

Finite element method is the representation of a body or a structure by an assemblage of subdivisions called finite elements, these elements are considered interconnected at points which are called nodes. This method is a numerical procedure for analysing structures and continua. FEM is a powerful tool in structural analysis of simple to complicated geometries.

2.8.2 STEPS IN FEM

Following steps are followed in finite element method

1. Divide the structure or continuum in finite elements.
2. Formulate the properties of each element. In stress analysis, this is nodal loads associated with all element deformation states that are allowed.
3. Assemble the elements to obtain the finite element model of the structure.
4. Apply the known loads: nodal force or/and moments in stress analysis.
5. Impose boundary conditions.

6. Calculate the displacement vector.

7. Calculate strain, and finally calculate stress from strain.

2.8.3 Modeling of Components: ***soil, reinforcement and facing***

The incorporation of mechanism of soil-reinforcement-facing interaction in the FEM are greatly influenced by the construction method, compaction, propping of facing during construction and its release later including the boundary conditions (loading on top, etc.), thus, making it difficult to model the problem.

Soil: most researchers as pointed out by Gourc, 1992, have adopted non-linear elastic or elasto- plastic models. The initial deformation is sometime calculated using linear elastic constitutive models and failure load is calculated using limiting equilibrium methods employing appropriate constitutive models e.g. Mises or Mohr- Coulomb, Drucker-Prager etc.

Reinforcement:Reinforcement is generally modeled by linear bar element capable of taking only axial tensile forces. Behavior of extensible geosynthetic materials is generally nonlinear. Sometime metallic reinforcements are also modeled as continuous beam element and the bending moment is calculated in addition to the axial force.

2.8.4 Modeling of Soil Reinforcement Interface

Several authors have proposed various types of interface elements to model the interface behavior. Most of the interface elements, originally developed in rock mechanics, are used in the analysis of reinforced soils. Interface elements can be classified (Gens et al.,1989) into the following categories:

a. Standard finite elements of small thickness

b. Quasi-continuum elements possessing a weakness plane in the direction of the interface.

c. Linkage elements in which only the connections between opposite nodes are considered

d. Interface elements in which relative displacement between opposite nodes are the primary deformation variables. They can have finite or zero thickness.

Several differences exist among these methods and the main argument concerns the physical existence of shearing band of soil around reinforcement. FEM methods are based on continuity of soils except the contact plane between soils and reinforcing materials. Goodman element (1968) is the original interface element introduced in the geotechnical contact problems. This type of interface element is extensively used in the reinforced soil problems. A typical interface element is illustrated in Fig.2.16 below.

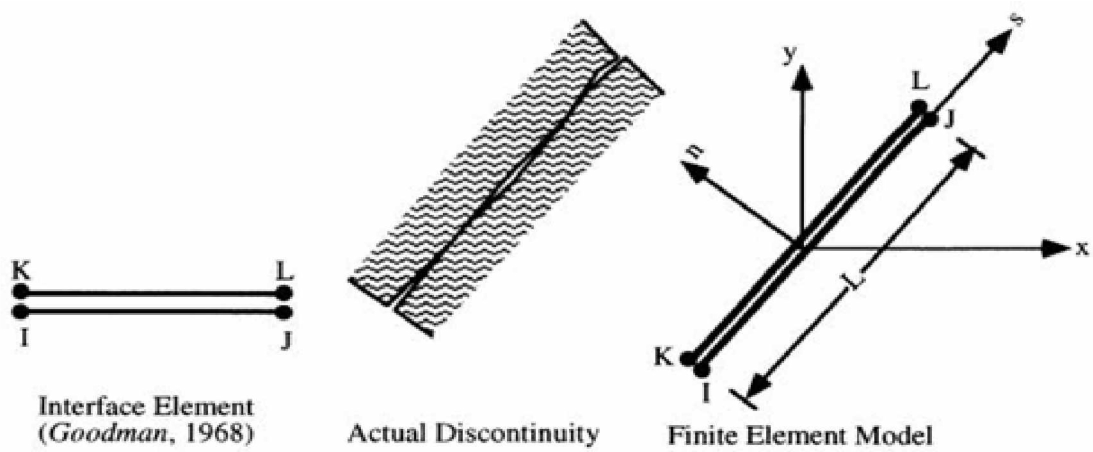


Figure 2.16 A typical interface element used in the modeling of the soil-reinforcement interfaces *(Goodman, 1968)*

3.1 OVERVIEW OF PLAXIS SOFTWARE

3.1.1General

PLAXIS version 8 is the finite element package for the 2 – dimensional analyses of deformation and stability on geotechnical engineering. PLAXIS is equipped with the features to deal with various aspects of the complex geo- technical structure. Real situation may be modeled either by the plain strain or as axisymmetric model. In a plane strain analysis the calculated forces from prescribed displacements represent from per unit length on the out of plain direction (z- direction). On axisymmetric analysis, the calculated forces are those that act on the boundary subtending an angle of 1 radian. In order to obtain the forces corresponding to the complete problem therefore, these forces should be multiplied by 2π.

3.1.2 Input program

To carry out finite element analysis using PLAXIS, the user has to create a finite element modal and specify the material properties and boundary condition. This is done in the input program to set up a finite element model , the user must create a two dimensional geometry model composed of points , lines and other components in the x-y plane . The generation of an element level is automatically performed by the PLAXIS mesh generator based on the input of the geometry model. User may also customize the finite element mesh in water pressure and initial stresses to the initial stage.

3.1.3 Preparing mode using PLAXIS tools

In principle, first draw the geometry contour, and then add the soil layers, then structural objects, then construction layers, then boundary conditions and then loadings. Using the geometry line option, the user may draw points and lines in the draw area. 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. Geogrid are slender structures with a normal stiffness are generally used to model reinforcements. To model the interaction between the sheets pile wall and the soil, the interfaces are used which is intermediate between smooth and fully rough.

3.1.4 Modeling of soil behavior

In PLAXIS, soil properties and material properties of structure are stored in material data sets from the database sets are assigned to the soil clusters or to the corresponding structural objects in the geometry model. PLAXIS supports various model to simulate the behavior of soil and other continua such as linear elastic model, Mohr-coulomb model, jointed rock model, hardening soil model, soft model, soft soil creep model and user defined models. 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 initial conditions part of the input program.

3.1.5 Calculations

After this, the actual finite element calculations must be executed. Therefore it is necessary to define which types of calculations are to be performed and which type of loadings or construction stages are to be activated during the calculations. PLAXIS allows for different types of finite element calculations in the engineering practice, a project are divided into calculation phases. Examples of calculation phases are the activation of a particular loading at a certain time, the simulation of a construction stage, the introduction of a consolidation period, the calculation of safety factors etc.

3.1.6 Output program

The main output quantities of a finite element calculation are the displacement at the nodes and the stresses at the stress points. In addition, when a finite element model involves structural elements, structural forces are calculated in these elements. Extensive ranges of facilities exist within PLAXIS to display the results of a finite analysis. The curves program can be used to draw load-displacement curves, stress-strain curves and stress or strain paths of pre – selected points in the geometry. These curves visualize and this gives an insight into the global and local behavior of the soil. When subsequently clicking on the output button the result of all construction phases are displayed on separate windows in the output program. In this way results of phases can be obtained.

3.2. GENERAL INFORMATION OF MODEL

Table [1] Units

Type	Unit
Length	m
Force	kN
Time	day

Table [2A] Model dimensions

	min.	max.
X	0.000	25.000
Y	0.000	11.000

Table [2B] Model

Model	Plane strain
Element	15-Noded

Table [3] Numbers, type of elements, integrations

Type	Type of element	Type of integration	Total no.
Soil	15-noded	12-point Gauss	246
Plate	5-node line	4-point Gauss	10
Geogrid	5-node line	4-point Newton-Cotes	24
Interface	5-node line	4-point Newton-Cotes	61

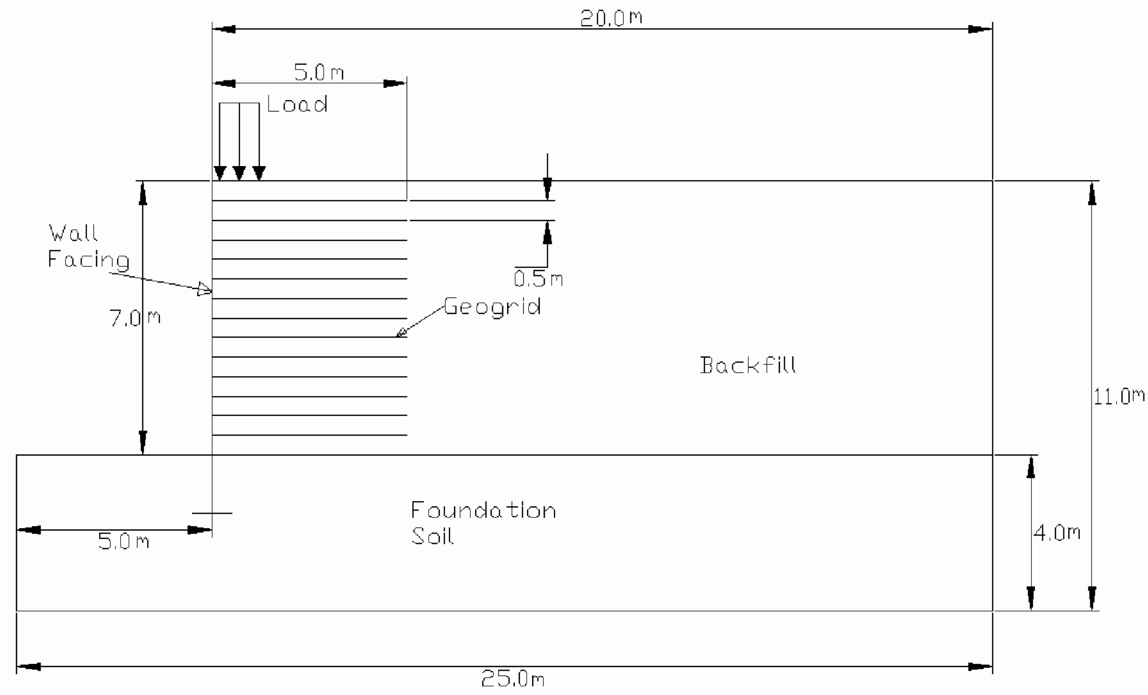
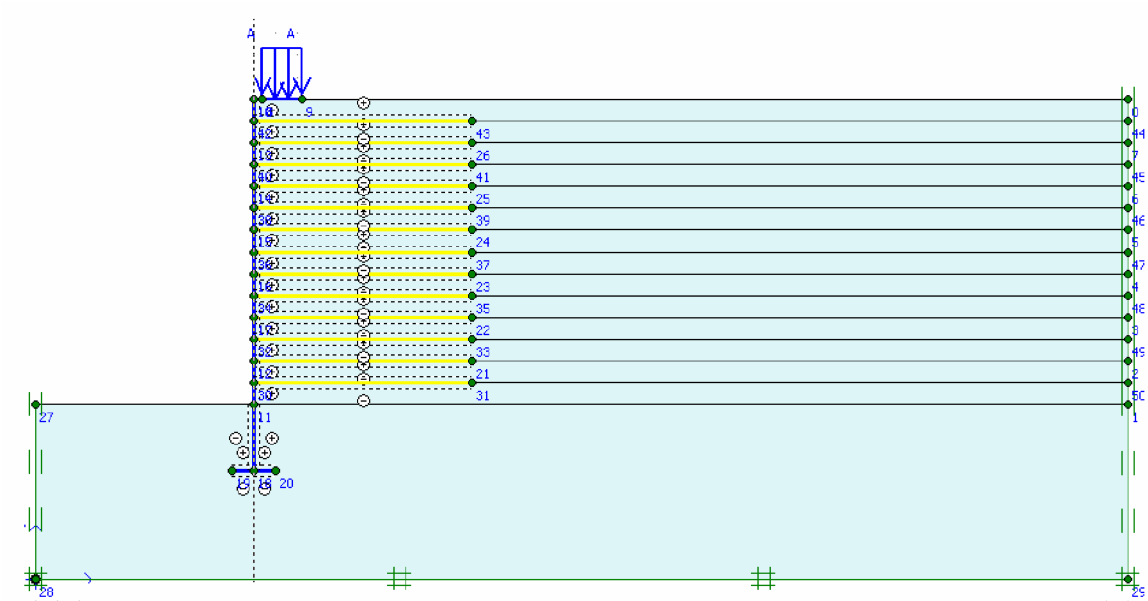


Fig 3.1 Line diagram of model



Model-1 : Reinforced earth wall system with loose sand used as a backfill material

Table[4]properties of soil

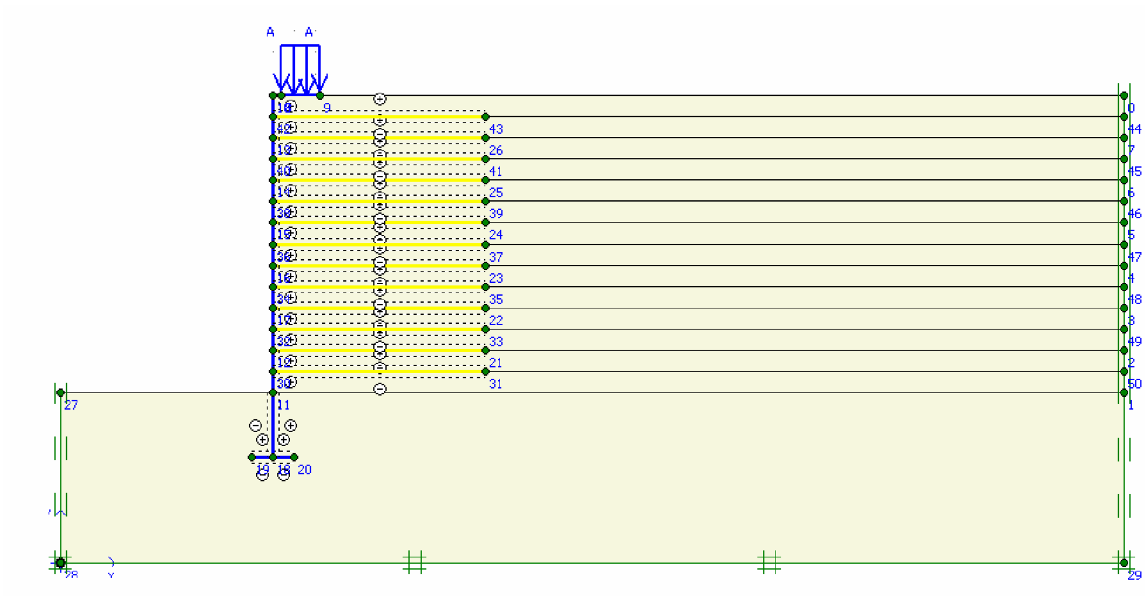
Mohr-Coulomb		Loose sand
Type		Drained
γ	[kN/m³]	16.5
k_x	[m/day]	1
k_y	[m/day]	1
E	[kN/m²]	20000
ν	[-]	0.25
c	[kN/m²]	0
φ	[°]	34
ψ	[°]	0
$R_{inter.}$	[-]	0.67
Interface permeability		Neutral

Table [5] Beam data sets parameters

No	Identification	EA [kN/m]	EI [kNm²/m]	ν [-]	Mp [kNm/m]
1	Diaphragm wall	7.5E6	1E6	0.00	1E15
2	Footing	5E6	8500.00	0.00	1E15

Table [6] Geogrid data sets parameters

No	Identification	EA [kN/m]	ν [-]
1	Geogrid	1500	0.00



Model-2 : Reinforced earth wall system with dense sand used as a backfill material

Table [7] Properies of soil

Mohr-Coulomb		Dense sand
Type		Drained
γ	[kN/m³]	18.0
$\mathbf{k_x}$	[m/day]	1
$\mathbf{k_y}$	[m/day]	1
E	[kN/m²]	65000
ν	[-]	0.3
c	[kN/m²]	0
φ	[°]	400
ψ	[°]	10
$\mathbf{R_{inter.}}$	[-]	0.8
Interface permeability		Neutral

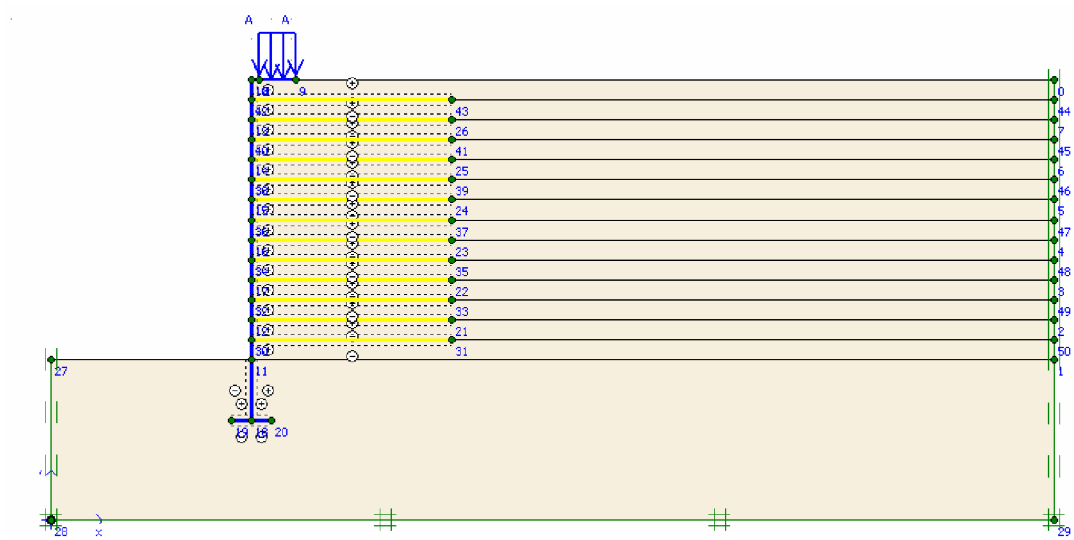
Table [8] Beam data sets parameters

No	**Identification**	**EA** [kN/m]	**EI** [kNm²/m]	ν [-]	**Mp** [kNm/m]
1	Diaphragm wall	7.5E6	1E6	0.00	1E15
2	Footing	5E6	8500.00	0.00	1E15

Table [9] Geogrid data sets parameters

No	**Identification**	**EA** [kN/m]	ν [-]
1	Geogrid	1500	0.00

Model-3



Model-1 : Reinforced earth wall system with silty sand used as a backfill material

Table [10] Properies of soil

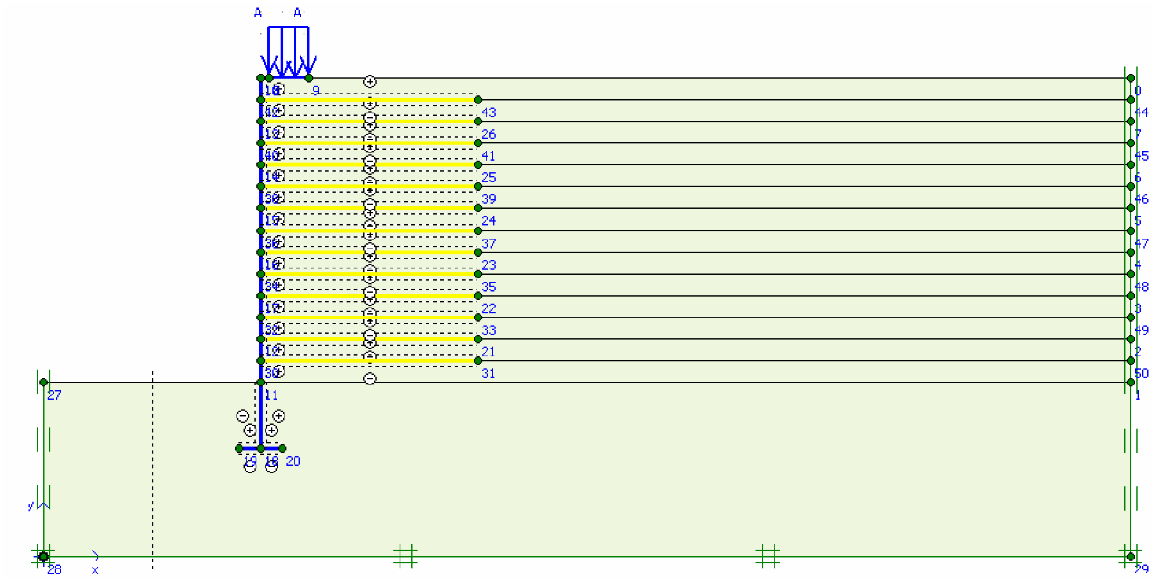
Mohr-Coulomb		Silty sand
Type		Drained
γ	[kN/m³]	17.0
k_x	[m/day]	1
k_y	[m/day]	1
E_{ref}	[kN/m²]	15000
ν	[-]	0.35
c	[kN/m²]	0
φ	[°]	32
ψ	[°]	4
$R_{inter.}$	[-]	0.8
Interface permeability		Neutral

Table [11] Beam data sets parameters

No	Identification	EA [kN/m]	EI [kNm²/m]	ν [-]	Mp [kNm/m]
1	Diaphragm wall	7.5E6	1E6	0.00	1E15
2	Footing	5E6	8500.00	0.00	1E15

Table [12] Geogrid data sets parameters

No	Identification	EA [kN/m]	ν [-]
1	Geogrid	1500	0.00



Model-4 : Reinforced earth wall system with clayey sand used as a backfill material

Table [13] Properies of soil

Mohr-Coulomb		**Clayey sand**
Type		drained
γ	[kN/m³]	18.9
$\mathbf{k_x}$	[m/day]	1
$\mathbf{k_y}$	[m/day]	1
$\mathbf{E_{ref}}$	[kN/m²]	40000
ν	[-]	0.3
c	[kN/m²]	10
φ	[°]	30
ψ	[°]	2
$\mathbf{R_{inter.}}$	[-]	0.85
Interface permeability		Neutral

Table [14] Beam data sets parameters

No	**Identification**	**EA** [kN/m]	**EI** [kNm²/m]	ν [-]	**Mp** [kNm/m]
1	Diaphragm wall	7.5E6	1E6	0.00	1E15
2	Footing	5E6	8500.00	0.00	1E15

Table [15] Geogrid data sets parameters

No	**Identification**	**EA** [kN/m]	ν [-]
1	Geogrid	1500	0.00

CHAPTER 4. RESULTS AND DISCUSSION

Finite element analysis is carried out using commercial software PLAXIS version 8 for the four type of problems mentioned in the previous chapter. The results are compared and reported in this chapter. Behaviors of reinforced soil structures under different conditions are investigated using PLAXIS version 8. Effect of the soil reinforcement is shown through the relationships between load and deformation. The effect of spacing of reinforcement on the soil is explained through the displacements developed. the reinforcement stiffness and displacement in the soil mass Effectiveness of the geogrid length and axial stiffness is also noticed.

4.1) Load- displacement variation of reinforced soil wall system for loose sand case.

Loads in KN	Total displacement in mm	Horizontal displacement in mm	Vertical displacement in mm
10	99.31	80.09	80.02
20	106.75	86.22	85.32
30	114.31	91.63	90.67
40	120.32	96.91	94.91
50	125.96	102.01	99.07
60	136.17	107.11	107.92
70	144.91	113.91	116.6
80	157.81	119.92	122.91
90	159.9	124.11	130.14
100	168.21	129.84	134.28

Table-16: Displacements under different loads for loose sand

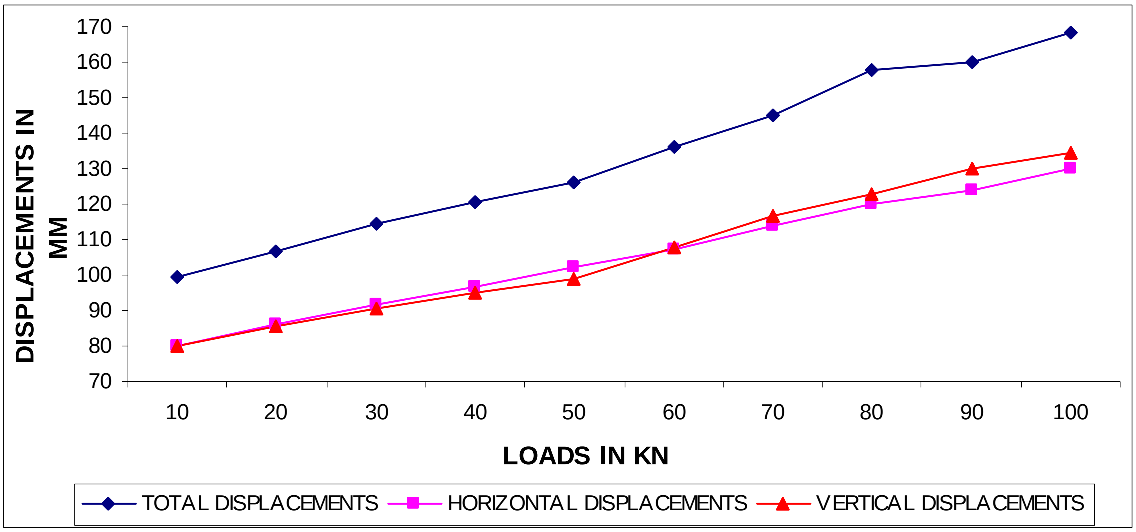


Figure-G1 Load displacement relationship for loose sand

- Figure shows that with the increase of load there is steady increase of total deflections. The variation of total displacement in loose sand case is around 100mm to 170 mm there is also a steady increase in horizontal and vertical displacements in loose sand case. Both horizontal and vertical displacements are almost of same magnitude under different loads. Initially horizontal displacement is more but at the end vertical displacement surpasses horizontal displacement.

4.2) Load- displacement variation of reinforced soil wall system for dense sand case

Load in KN	Total displacement in mm	Horizontal displacement in mm	Vertical displacement in mm
10	27.31	23.81	22.03
20	29.44	25.91	23.69
30	31.56	27.32	25.81
40	33.7	29.48	27.31
50	35.81	31.68	28.32
60	37.91	33.81	30.19
70	40.03	36.92	32.52
80	43.15	39.01	34.33
90	46.24	40.24	36.66
100	48.09	42.41	38.15

Table-17: Displacements under different loads for dense sand

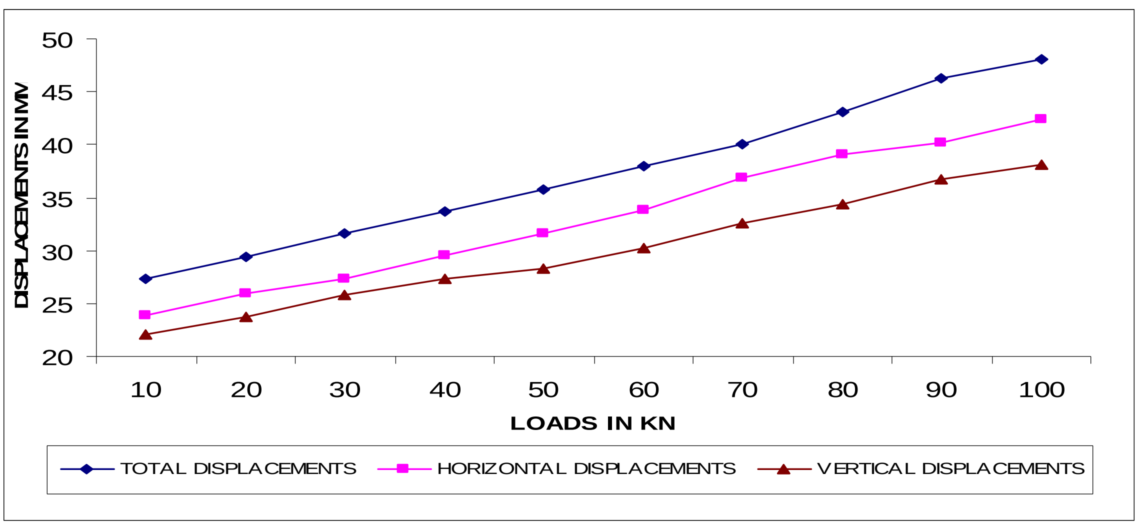


Figure-G2: Load displacement relationship for loose sand

Figure shows that with the increase of load there is very less increase of total, horizontal and vertical deflections. The variation in 3 displacements is not much in dense sand case. There is not much increase of displacements seen with increase of load.

4.3) Load- displacement variation of reinforced soil wall system for silty sand case

load in KN	Total displacement in mm	horizontal displacement in mm	Vertical displacement in mm
10	86.66	69.21	69.17
20	90.71	72.32	71.93
30	94.82	76.58	74.21
40	98.81	80.8	77.03
50	102.93	82.91	80.68
60	105.12	86.21	84.79
70	110.41	90.34	89.91
80	116.68	94.58	93.98
90	122.7	97.62	97.58
100	127.97	100.58	101.87

Table-18: Displacements under different loads for silty sand

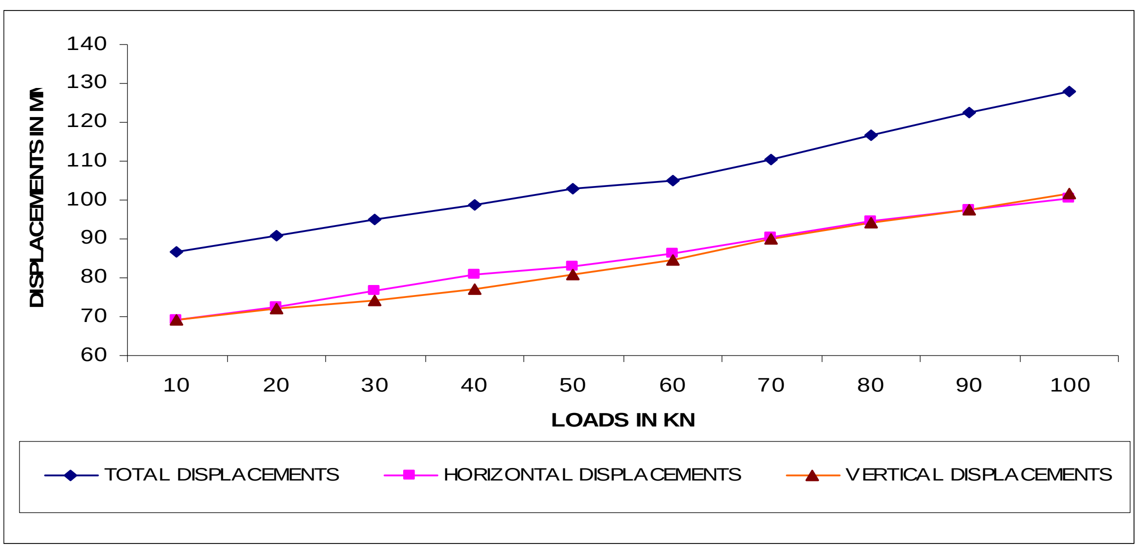


Figure-G3: Load displacement relationship for silty sand

- Figure shows that with the increase of load there is steady increase in 3 deflections. The nature of variation of displacements in silty sand case is same as that of loose sand case. Both horizontal and vertical displacements are almost of same magnitude under different loads.

4.4) Load- displacement variation of reinforced soil wall system for clayey sand case

Load in KN	Total displacement in mm	Horizontal displacement in mm	Vertical displacement in mm
10	25.45	13.46	24.63
20	26.69	14.58	25.42
30	27.87	15.69	26.24
40	28.91	17.74	26.9
50	29.53	18.67	27.73
60	31.23	21.32	30.12
70	34.38	23.65	32.63
80	38.41	26.71	34.97
90	42.68	28.93	37.64
100	45.44	30.88	40.7

Table-19: Displacements under different loads for clayey sand

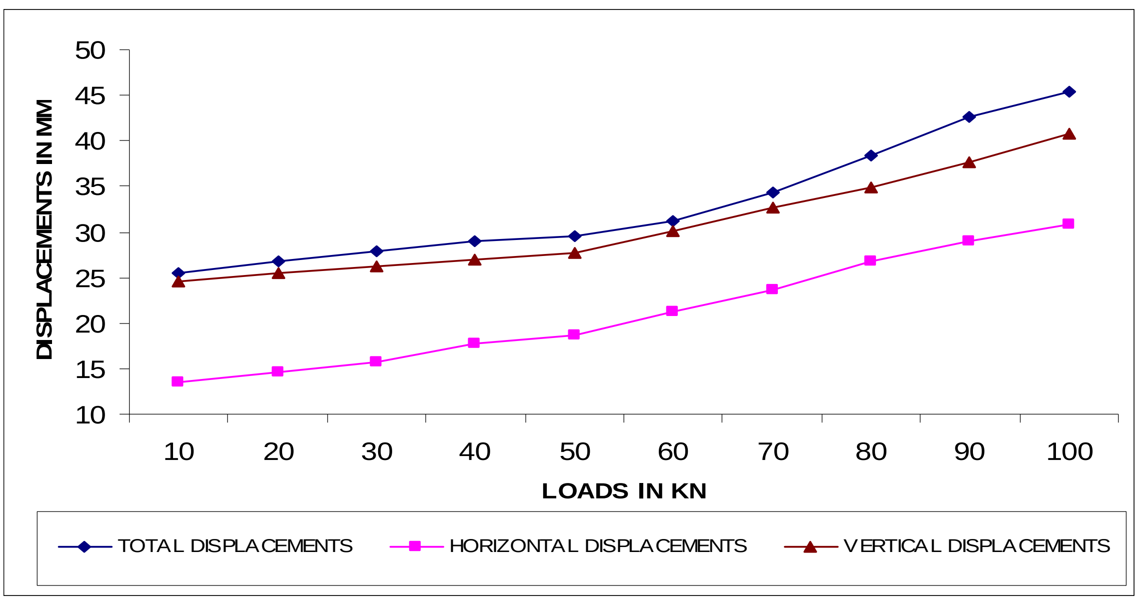


Figure-G4: Load displacement relationship for clayey sand

- Figure shows that in clayey sand case minimum horizontal displacements is observed. at initial loads total and vertical displacements are almost same but difference can be seen in later loads

4.5) Spacing-displacement variation of reinforced soil wall system for loose sand case

Spacing in M	Total displacement in mm	Horizontal displacement in mm	Vertical displacement in mm
0.3	109.75	73.21	91.67
0.5	168.21	129.84	134.28
0.8	210.85	176.35	180.67

Table-20: Displacements under different spacing for loose sand

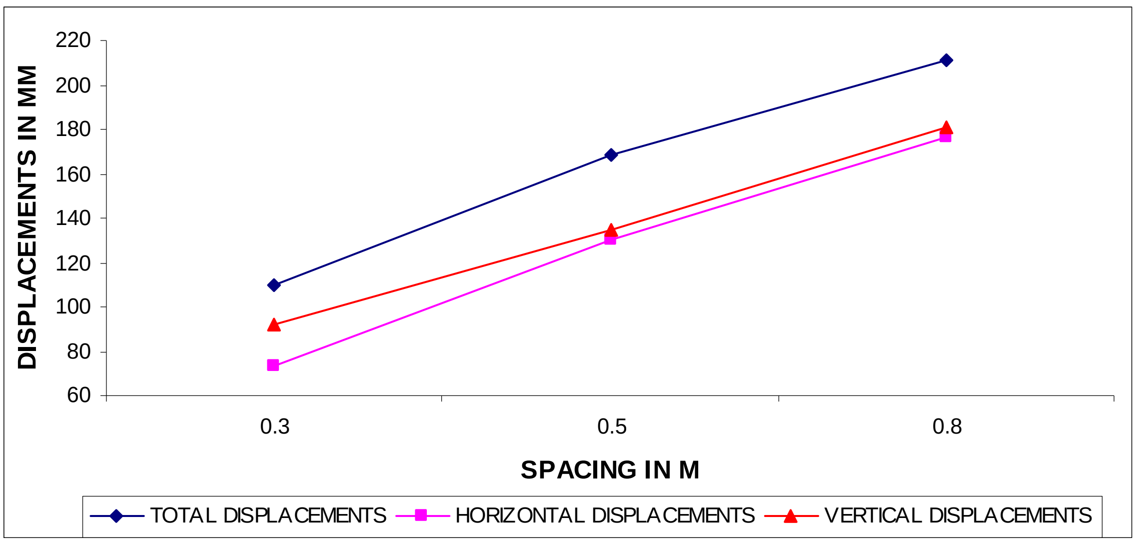


Figure-G5: Spacing displacement relationship for loose sand

- Figure shows that displacements increases by nearly 100% if we increases the spacing of geogrid from 0.3 m to 0.8 m. it is also seen that the reinforced soil body collapses if we incraese the spacing beyond 0.8 m

4.6) Spacing-displacement variation of reinforced soil wall system for dense sand case

Spacing in m	Total displacement in mm	Horizontal displacement in mm	Vertical displacement in mm
0.3	28.8	20.02	24.81
0.5	48.09	42.41	38.15
0.8	60.32	54.32	47.51

Table-21: Displacements under different spacing for dense sand

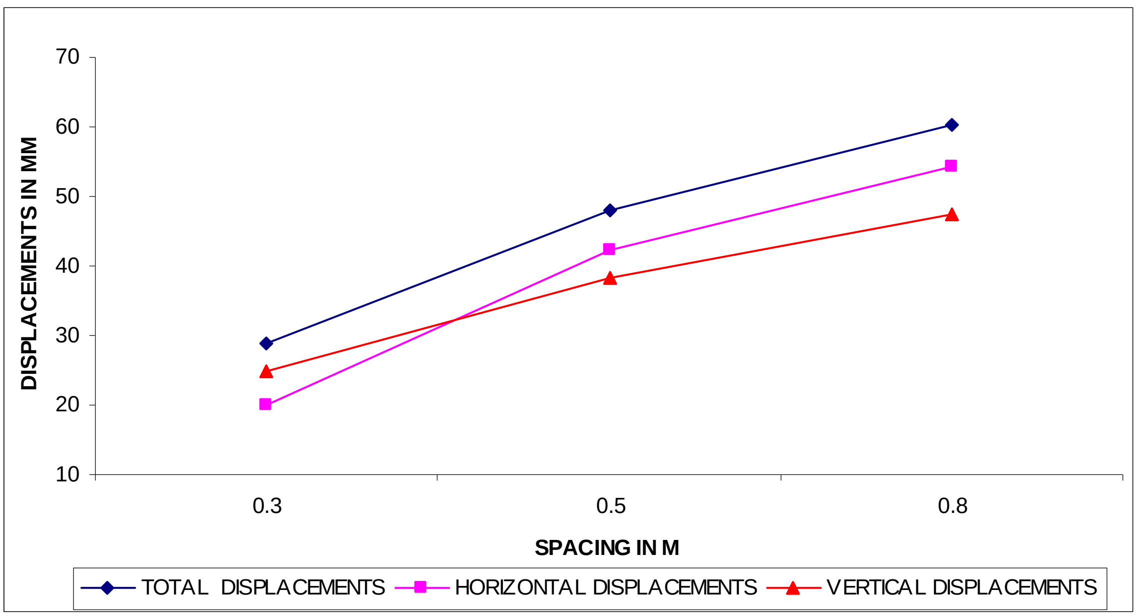


Figure-G6: Spacing displacement relationship for dense sand

- Figure shows there is a gradual increase in displacements with the increase in spacing of geogrid from 0.3 to 0.8 m. at 0.3 m spacing vertical displacement is more but at 0.5 m spacing horizontal displacement exceeds vertical displacements. it is also seen that the reinforced soil body collapses if we increase the spacing beyond 0.8 m

4.7) Spacing-displacement variation of reinforced soil wall system for silty sand case.

Spacing in m	Total displacement in mm	Horizontal displacement in mm	Vertical displacement in mm
0.3	100.54	70.33	83.21
0.5	127.97	100.58	101.87
0.8	158.89	134.01	120.03
1	199.9	156.97	138.69
1.2	217.42	187.1	156.54

Table-22: Displacements under different spacing for silty sand

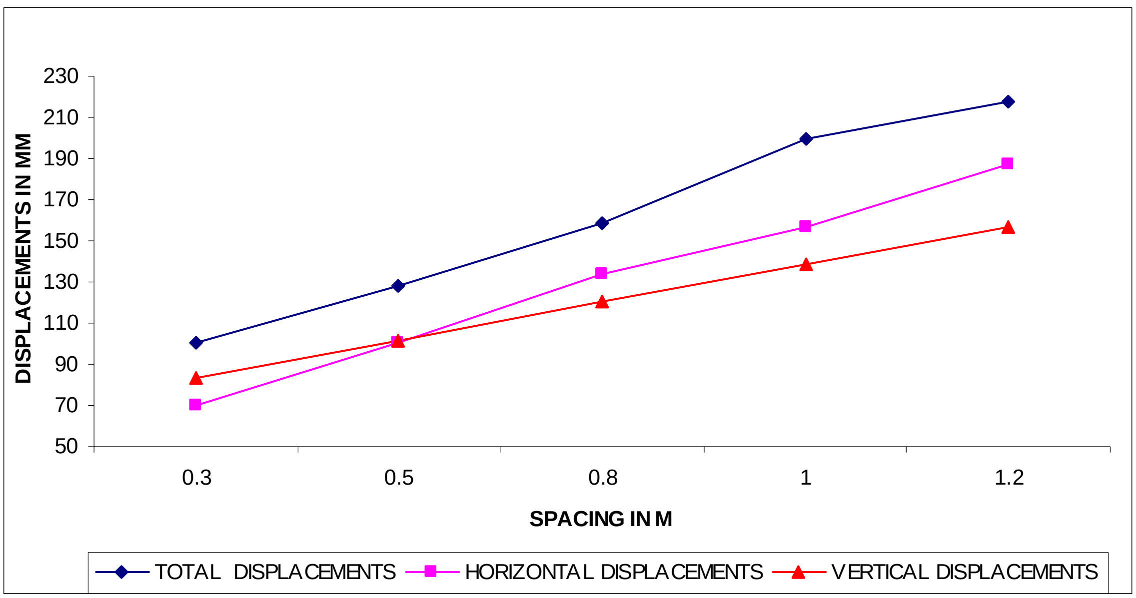


Figure-G7: Spacing displacement relationship for silty sand

- Figure shows that the displacements increases rapidly with increase in spacing of geogrid at 0.3 m spacing vertical displacement is more but after 0.5 m spacing horizontal displacement exceeds vertical displacements. The variation of displacements is more than 100%.

4.8) Spacing-displacement variation of reinforced soil wall system for clayey sand case.

Spacing in m	Total displacement in mm	Horizontal displacement in mm	Vertical displacement in mm
0.3	37.5	20.84	34.64
0.5	45.44	30.38	40.7
0.8	53.98	41.36	48.01
1	62.71	50.49	53.84
1.2	72.75	64.59	60.58
1.5	82.84	77.65	67.55
2	136.86	121.88	103.35

Table-23: Displacements under different spacing for clayey sand

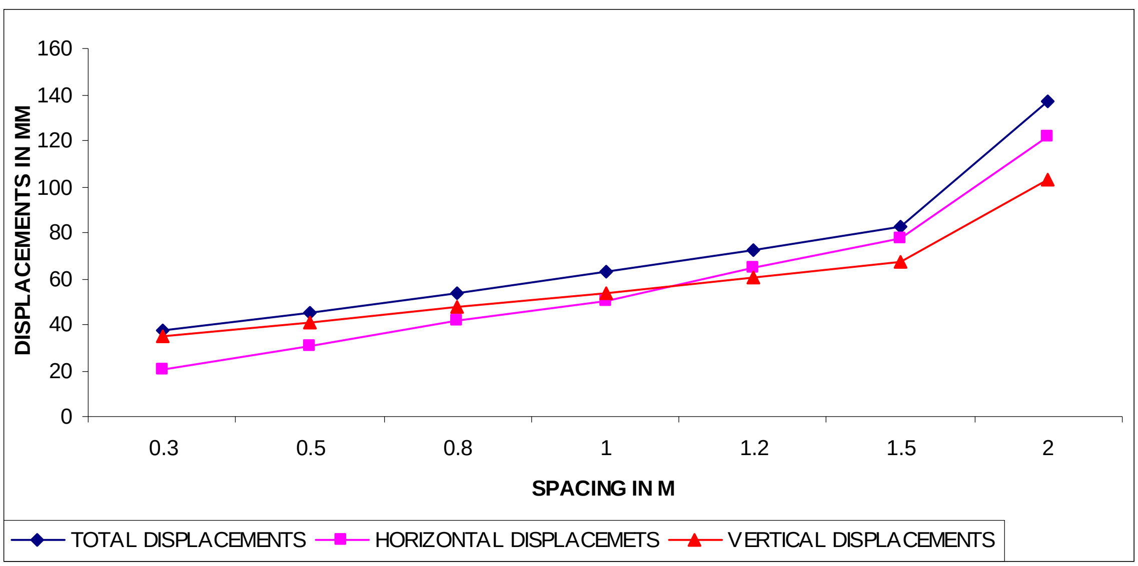


Figure-G8: Spacing displacement relationship for clayey sand

- Figure shows that there is steady increase in 3 displacements upto 1.5 m of spacing of geogrids due to cohesive property of clayey sand but after 1.5 m excessive displacements can be seen.

4.9) Comparison of displacements for different soil cases under same spacing of geogrid.

S.no.	Type of soil	Total displacements in mm	Horizontal displacements in mm	Vertical displacements in mm
1	loose sand	168.21	129.84	134.28
2	Dense sand	48.09	42.41	38.15
3	Clayey sand	127.97	100.58	101.87
4	Silty sand	45.44	30.38	40.7

Table-24: Displacements for different soil cases under same spacing of geogrid.

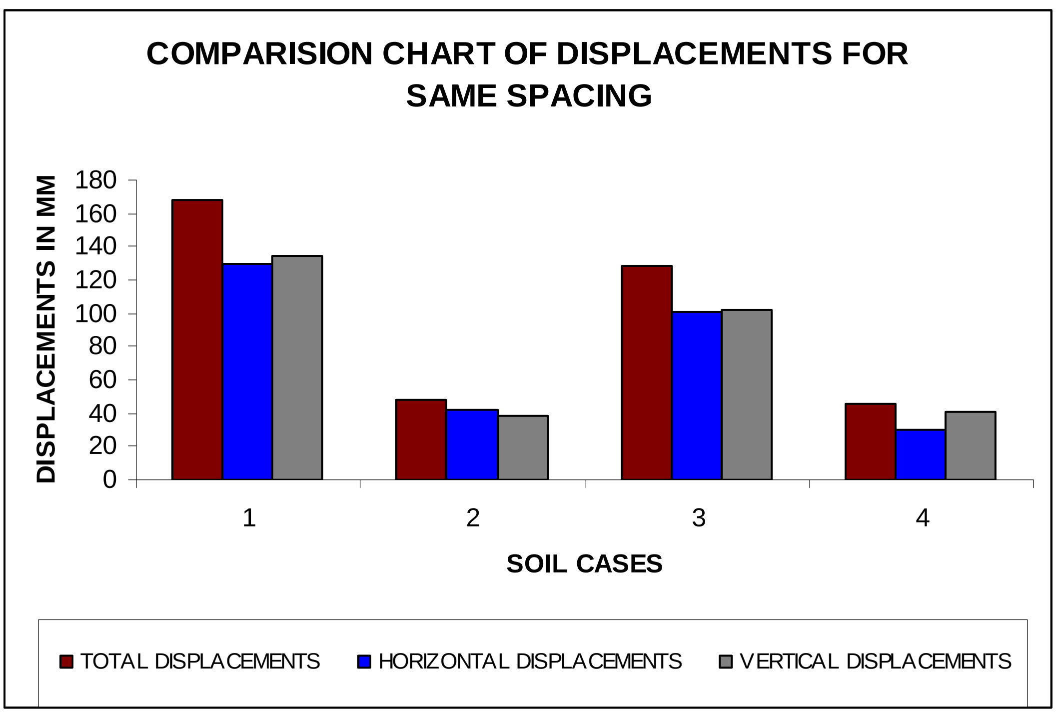


Figure-G9

4.10) Variation of displacements with the length of geogrid for loose sand case

Length of geogrid in m	Total displacements in mm	Horizontal displacements in mm	Vertical displacements in mm
5	101.93	88.93	82.23
6	100.76	84.02	81.85
7	99.14	81.85	81.37
8	98.6	79.32	81.02

Table-25 Displacements variation with the length of geogrid for loose sand case

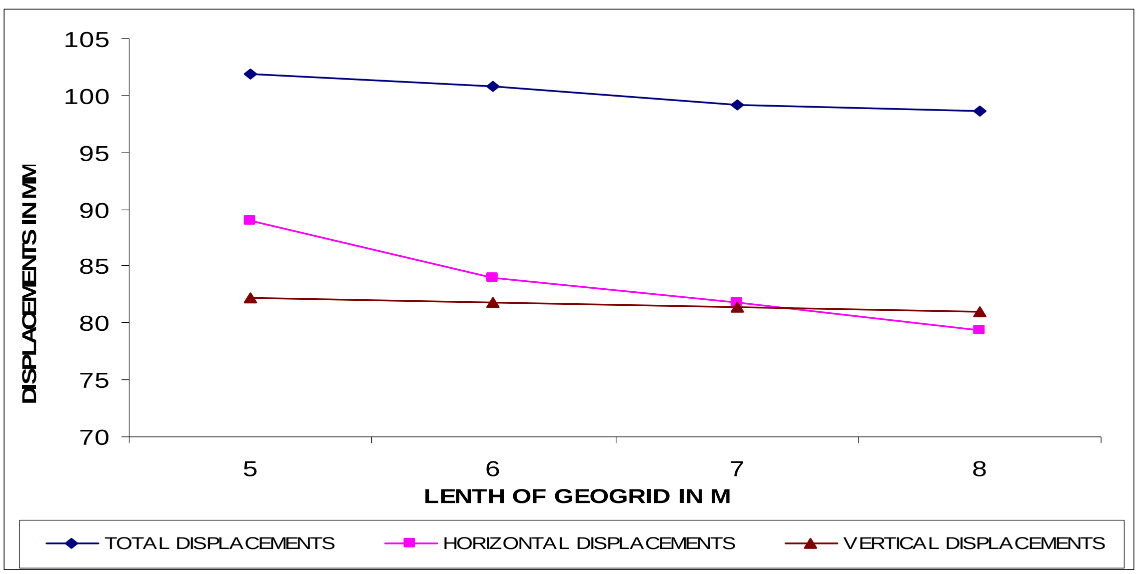


Figure-G10: Displacements variation with the length of geogrid for loose sand case

- Figure shows that with the increase of length of geogrid all 3 displacements decreases. Initially horizontal displacement is more than vertical displacement but at later stages horizontal displacement is minimum. There is very minimum or no displacement in vertical direction as loading conditions is same for different cases.

4.11) Variation of displacements with the length of geogrid for dense sand case

Length of geogrid in m	Total displacements in mm	Horizontal displacements in mm	Vertical displacements in mm
5	35.1	24.14	18.25
6	34.53	23.28	18.09
7	32.15	22.7	17.85
8	31.66	21.98	17.69

Table-26 Displacements variation with the length of geogrid for dense sand case.

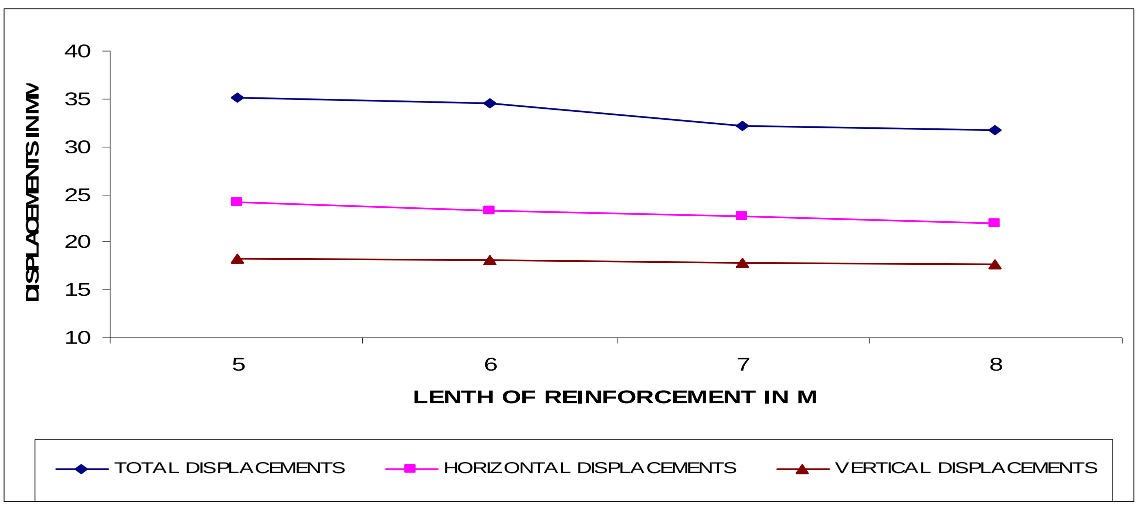


Figure-G11: Displacements variation with the length of geogrid for dense sand case.

Ø Figure shows that in dense sand case very minimum effect of geogrid length can be observed in displacements. All 3 displacements show very less deviation with increase in length.

4.12) Variation of displacements with the length of geogrid for silty sand case

Length of geogrid in m	Total displacements in mm	Horizontal displacements in mm	Vertical displacements in mm
5	110.93	100.35	83.09
6	101.99	90.34	79.27
7	99.68	87.22	78.77
8	98.77	85.66	78.56

Table-27 Displacements variation with the length of geogrid for silty sand case

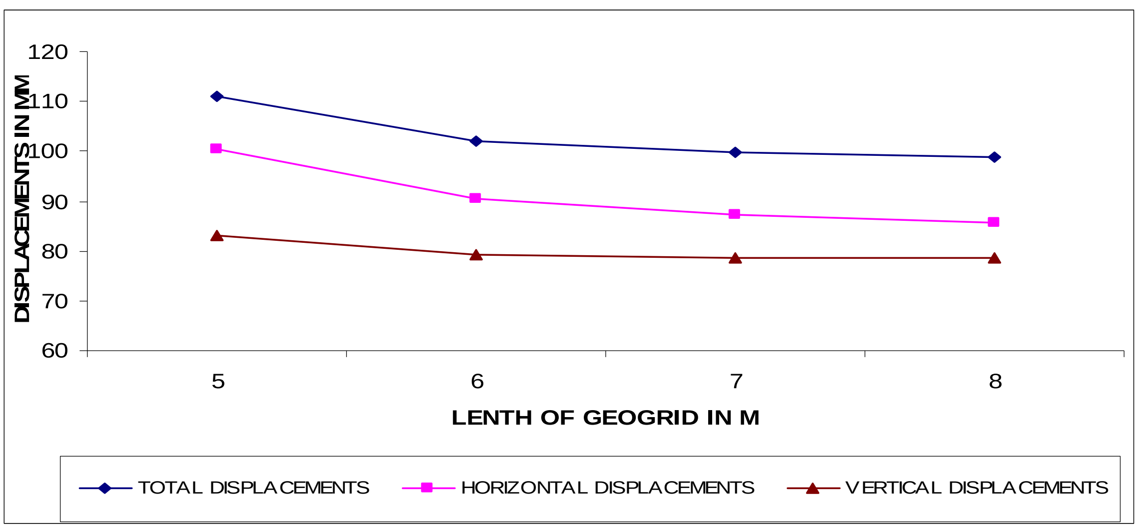


Figure-G12: Displacements variation with the length of geogrid for silty sand case

- Figure shows that a considerable decrease in displacements if geogrid length is increases from 5m to 6m and thereafter displacements are very less.

4.13) Variation of displacements with the length of geogrid for clayey sand case

Length of geogrid in m	Total displacements in mm	Horizontal displacements in mm	Vertical displacements in mm
5	25.46	17.5	24.49
6	25.27	16.95	24.44
7	25.18	16.87	24.33
8	25.11	16.81	24.29

Table-28: Displacements variation with the length of geogrid for clayey sand case.

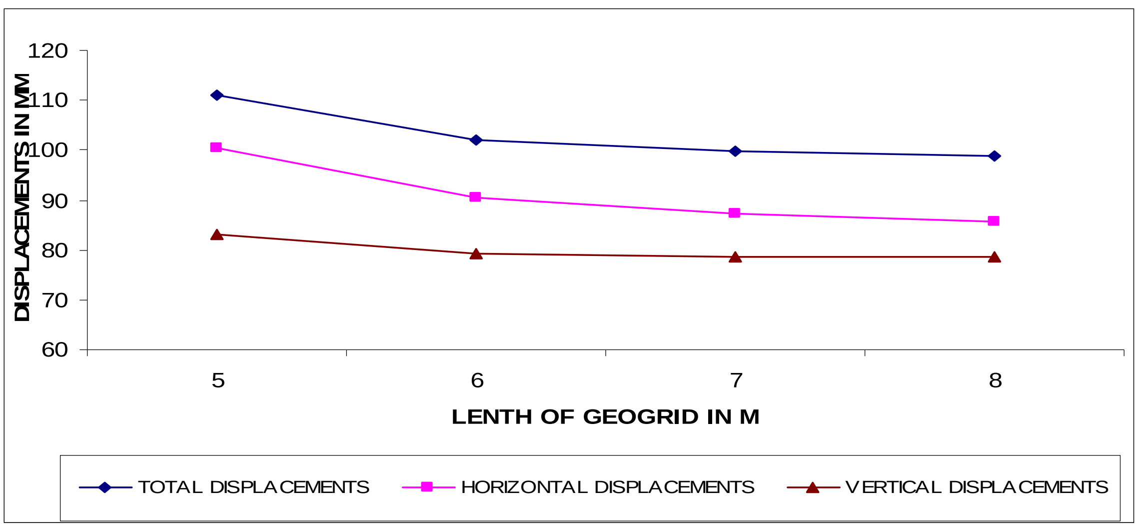


Figure-G13: Displacements variation with the length of geogrid for clayey sand case.

Ø Figure shows that there is no effect of geogrid length on displacements in case of clayey sand as the clayey sand is cohesive I nature so minimum length of reinforcement should be provided.

4.14) Variation of displacements with the change in axial stiffness of geogrid for loose sand case

Axial stiffness of geogrid in KN/m	Total displacements in mm	Horizontal displacements in mm	Vertical displacements in mm
1000	92.19	74.13	76.28
1500	78.21	58.01	66.99
2000	71.42	51.44	63.21
2500	65.68	42.49	59.4

Table-29: Displacements variation with the change in axial stiffness of geogrid for loose sand case.

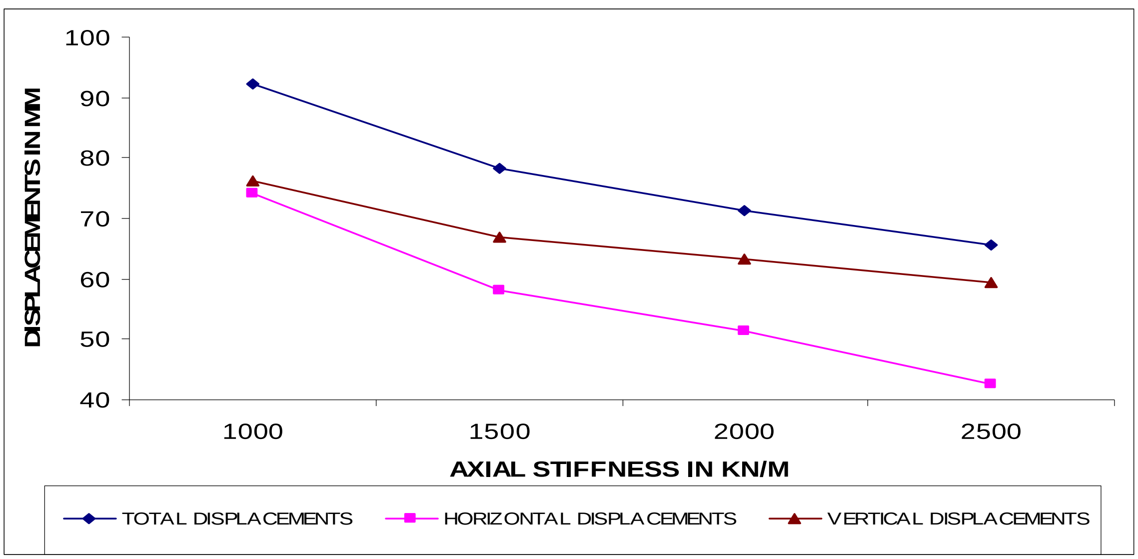


Figure-G14: Displacements variation with the change in axial stiffness of geogrid for loose sand case

- Figure shows that axial stiffness of geogrid plays a significant role in reducing the displacements. almost 50 reduction in horizontal displacements is seen with the increase in axial stiffness in loose sand condition

4.15) Variation of displacements with the change in axial stiffness of geogrid for dense sand case

Axial stiffness of geogrid in KN/m	Total displacements in mm	Horizontal displacements in mm	Vertical displacements in mm
1000	25.25	21.72	20.81
1500	21.17	16.67	18.27
2000	18.95	14.32	17.22
2500	17.7	12.08	16.05

Table-30: Displacements variation with the change in axial stiffness of geogrid for dense sand case.

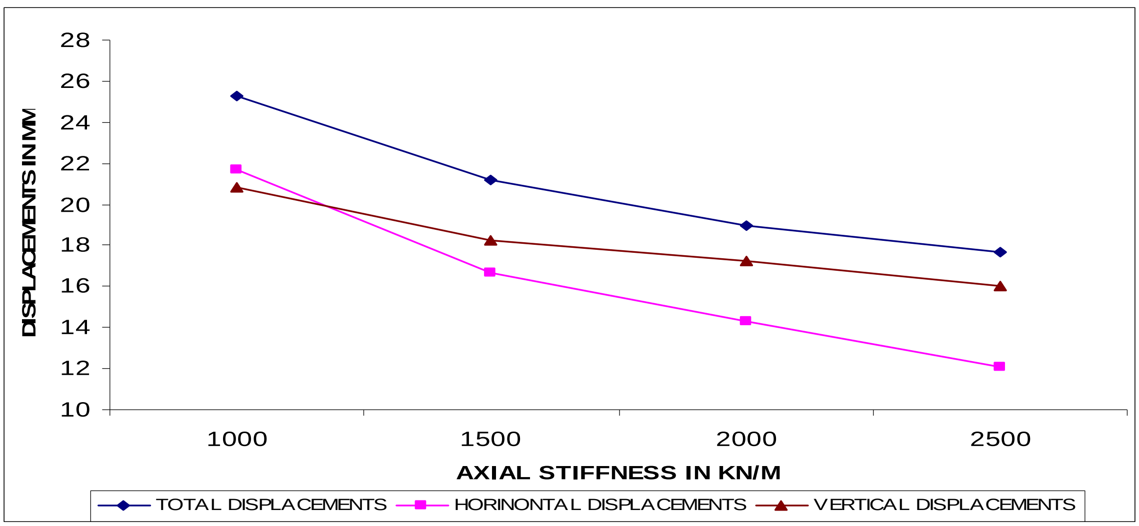


Figure-G15: Displacements variation with the change in axial stiffness of geogrid for dense sand case.

- Figure shows that in dense sand case there is not much reduction in total and vertical displacements but horizontal displacements are reduced by 50%.

4.16) Variation of displacements with the change in axial stiffness of geogrid for silty sand case

Axial stiffness of geogrid in KN/m	Total displacements in mm	Horizontal displacements in mm	Vertical displacements in mm
1000	97.72	83.24	75.91
1500	82.58	66.32	66.66
2000	76.94	58.91	63.47
2500	70.62	52.27	60.08

Table-31 Displacements variation with the change in axial stiffness of geogrid for silty case

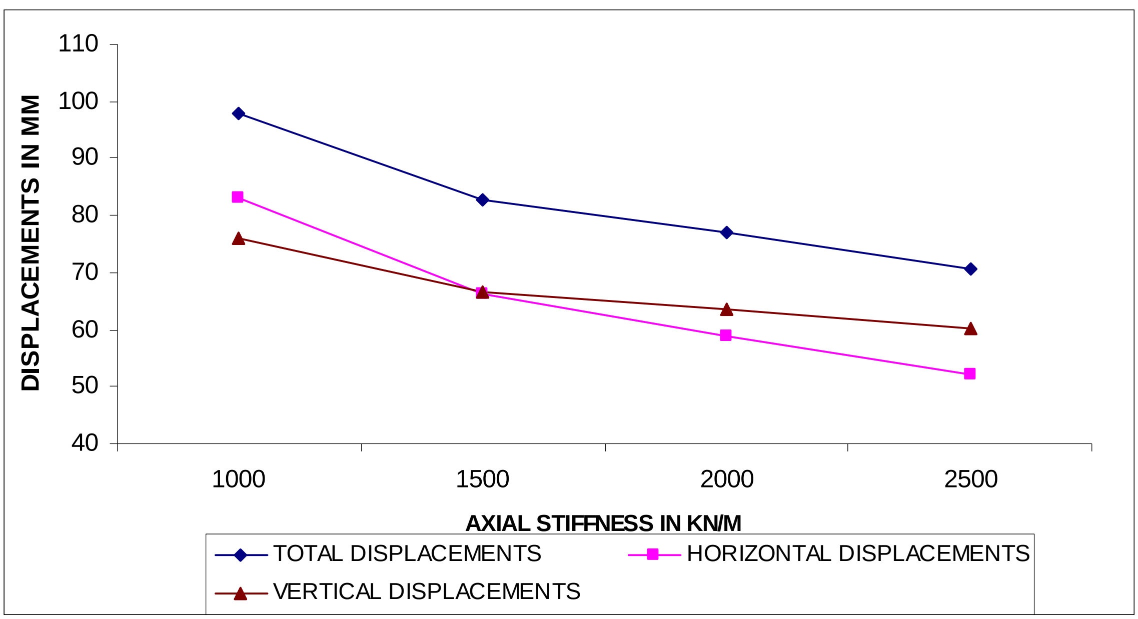


Figure-G16: Displacements variation with the change in axial stiffness of geogrid for silty sand case

Ø Figure shows that at initial increase in stiffness there is a sharp decrease in displacements especially horizontal displacement, but if stiffness of geogrid is further increased steady decrease in displacements is observed.

4.17) Variation of displacements with the change in axial stiffness of geogrid for clayey sand case.

Axial stiffness of geogrid in KN/m	Total displacements in mm	Horizontal displacements in mm	Vertical displacements in mm
1000	25.28	14.47	24.48
1500	24.65	12.65	24
2000	24.35	11.39	23.86
2500	24.12	10.67	23.58

Table-32: Displacements variation with the change in axial stiffness of geogrid for clayey sand case.

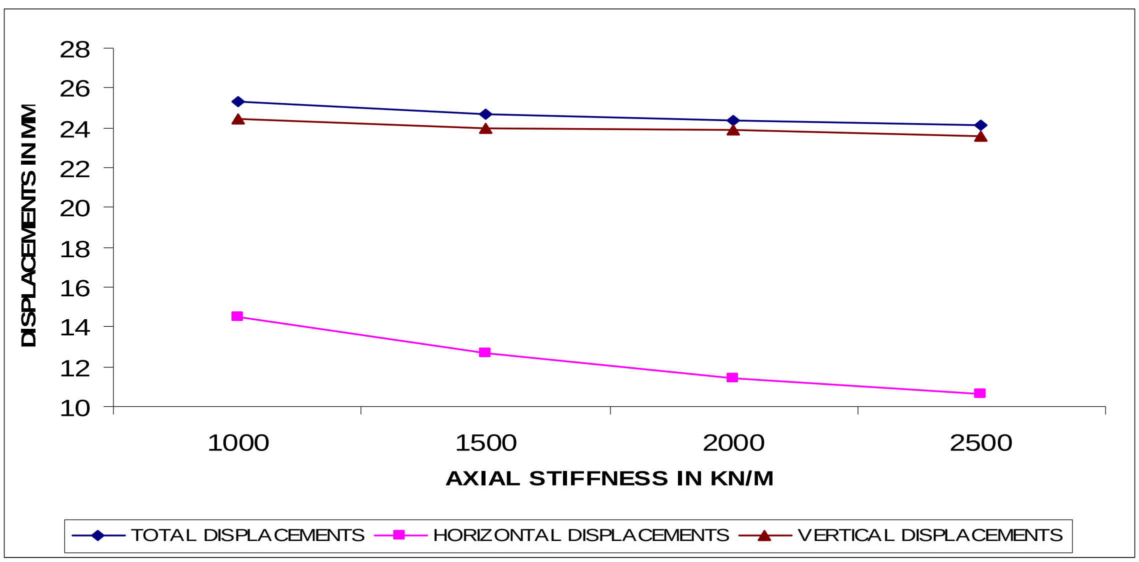


Figure-G17: Displacements variation with the change in axial stiffness of geogrid for clayey sand case.

- Figure shows that all 3 displacements are not much affected by increasing the axial stiffness of the geogrid so axial stiffness does not play significant role in case of clayey sand.

5 CONCLUSION

On the basis of finite element analysis and modelling done on reinforced earth wall with different backfill properties, the following point can be concluded:

(1) With the increase in load the variation of total displacement in case of loose sand is between 100 mm to 170 mm and there is also a steady increase in horizontal and vertical displacements in this case.

(2) Both horizontal and vertical displacements are almost of same magnitude under different loads for loose sand. Initially, horizontal displacements are more but at the end vertical displacements surpass horizontal displacements.

(3) The variation in the three displacements is not much in dense sand case. There is not much increase of displacements seen with increase of load

(4) In case of Clayey sand the displacements observed are minimum.

(5) Displacements increases by nearly 100% if the spacing of geogrid increases from 0.3 m to 0.8 m.for dense sand.

(6) In both loose sand and dense sand cases the reinforced soil body collapses if we increase the spacing beyond 0.8 m

(7) The variation of displacements is more than 100% in case of clayey sand.

(8) The effect of spacing of geogrids on displacements is least seen in case of clayey sand.

(10) With the increase of length of geogrid all the three displacements decrease. In case of dense sand effects of geogrid lengths can be observed in displacements.

(11) A considerable decrease in displacements can be seen if geogrid length is increased from 5m to 6m and thereafter displacements are very small.

(12) Axial stiffness of geogrid plays a significant role in reducing the displacements.

(13) Horizontal displacements are reduced by 50% and more if we increase the axial siffness of geogrid.

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APPENDIX-A

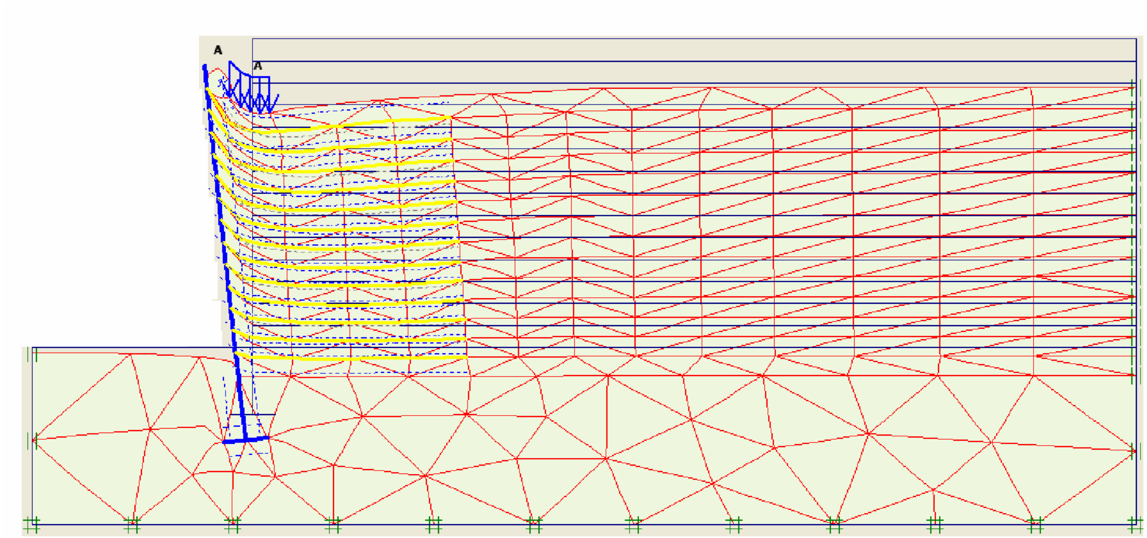

FIG A1: DEFORMED MESH OF MODEL

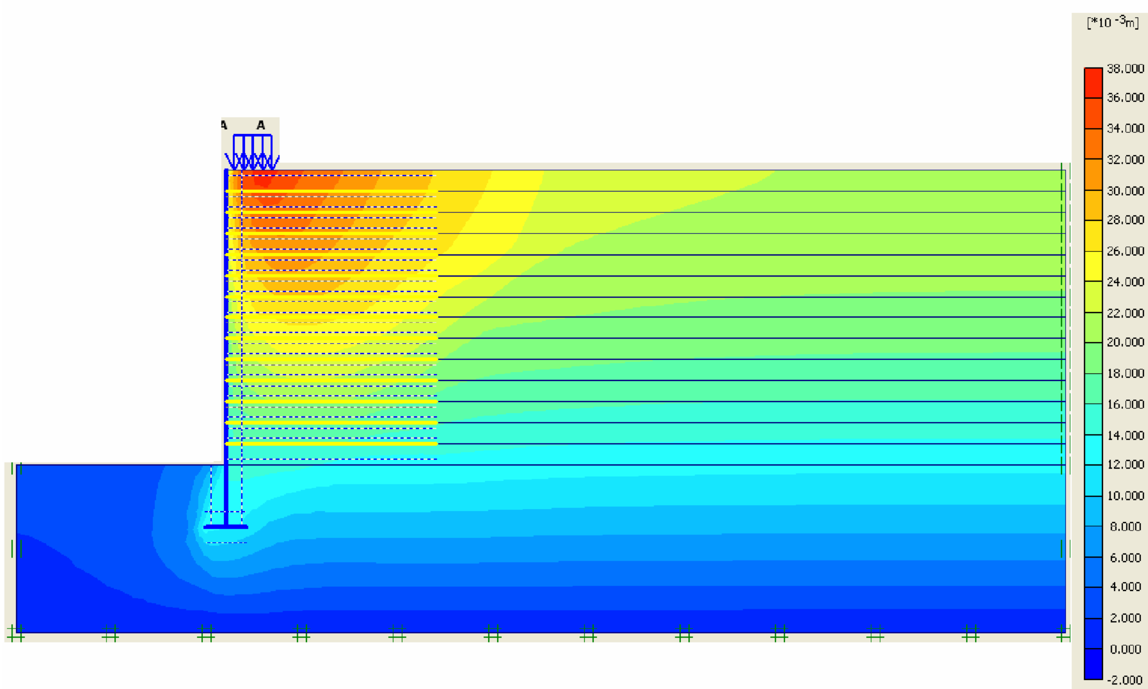

FIG A2: TOTAL DISPLACEMENTS OF MODEL

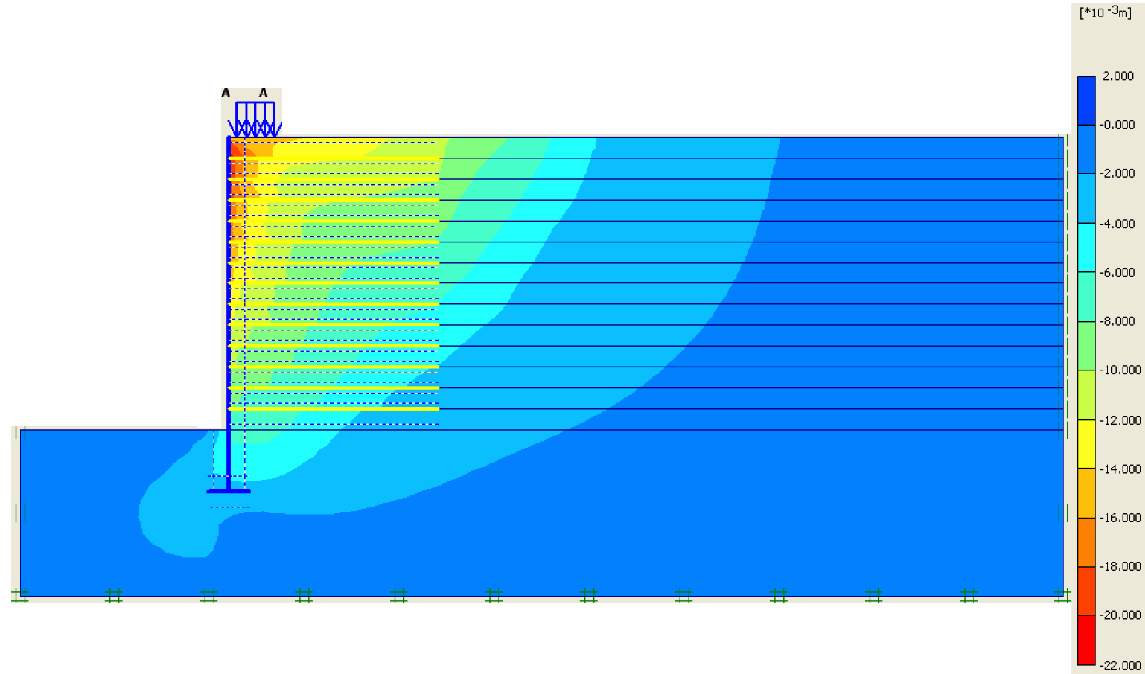

FIGURE A3: HORIZONTAL DISPLACEMENTS OF MODEL

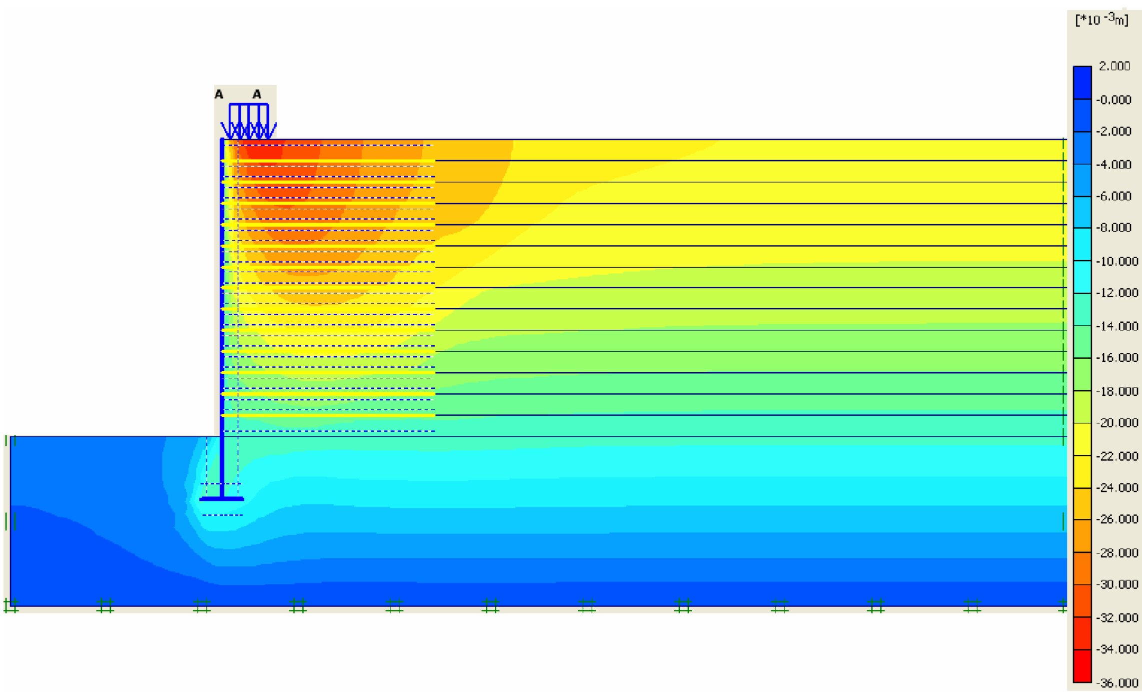

FIGURE A4: VERTICAL DISPLACEMENTS OF MODEL

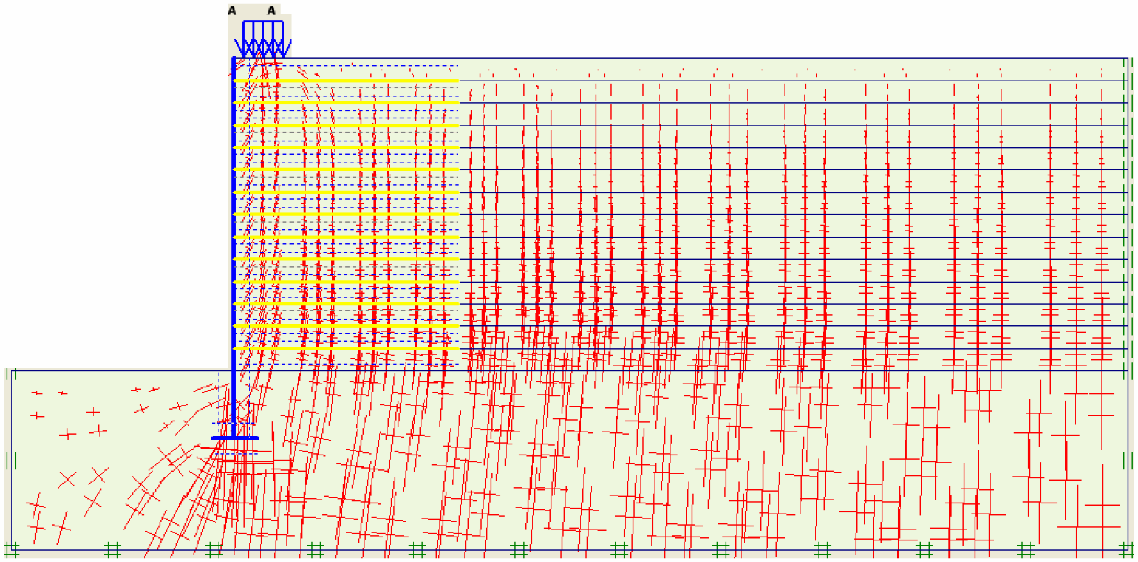

FIGURE A5: EFFECTIVE STRESS CONTOUR

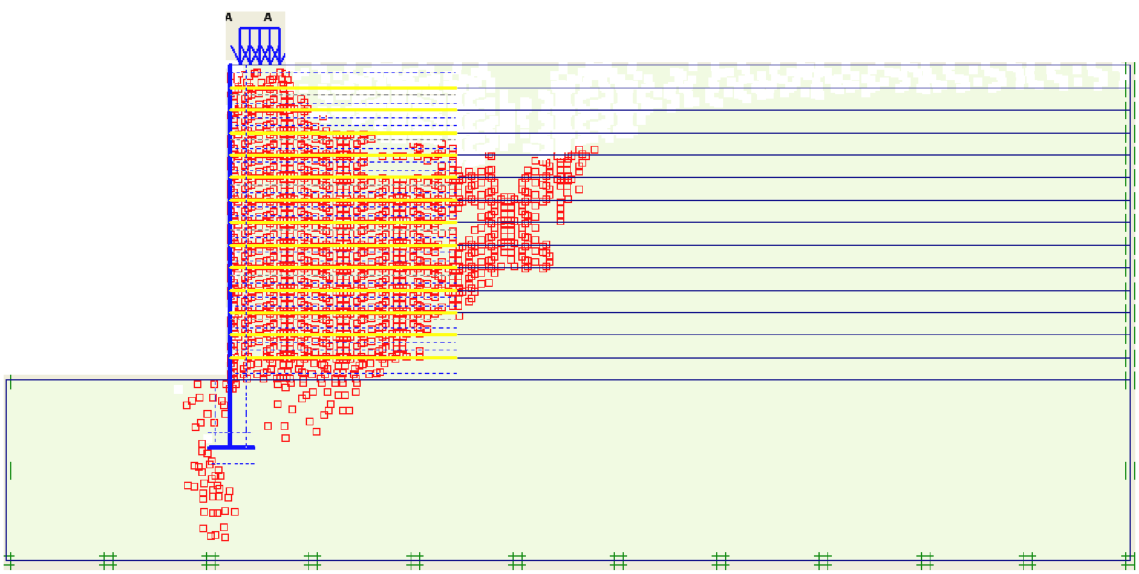

FIGURE A6: PLASTIC POINTS CONTOUR