DESIGN AND NUMERICAL MODELING OF REINFORCED EARTH RETAINING STRUCTURES

by

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B.S., in C.E., Istanbul Technical University, 2005

Submitted to the Institute for Graduate Studies in
Science and Engineering in partial fulfillment of
The requirements for the degree of
Master of Science

Graduate Program in Civil Engineering
Boğaziçi University

2007

ACKNOWLEDGEMENTS

I would like to express my appreciation to my advisor Dr. Ayşe Edinçliler for giving me the opportunity working on such an interesting project. Every progress of this work would not have been possible without her guidance and support. I also would like to express my gratitude to all the faculty members in the Geotechnical Department for making the master's program a pleasant experience.

I will give a special thanks to my beloved family for always standing by my side, and their endless support. I could not be the one who I am right now without my mother's sincere love and my father's infinite wisdom.

I owe heartfelt gratitude to my Department Manager Ali İhsan Çakır and Güçlü Coşkungür at Enka Teknik Company for their unchanging friendship, guidance and kind help during the last year. Their sincere support has always encouraged and motivated me.

I am indebted to my friends, Gizem Karlılar and Volkan Ayhan, for their generous help and friendship during the last two years. Without them, my life in Bogaziçi would have never been the same.

Countless others with a variety of affiliations have helped by providing data, answering questions, and overall cooperation and support. To all who have contributed in one way or another, I thank you.

ABSTRACT

The Reinforced Earth theory was created by French H. Vidal in 1966 and has showed up a progressive development till today. With cost and time savings that the technique brings with, Reinforced Earth Retaining Structures have became one of the most preferred solutions for many engineering projects.

In this study, design and behavior principles of Reinforced Earth Retaining Structures under static and dynamic loading cases have been investigated. Design principles are selected in accordance with relevant regulations of Turkey, France and USA Highway Administrations. Finite Element Model of Reinforced Earth Retaining Wall has been realized with respect to codes of U.S Department of Transportation.

A parametric study of static and dynamic response analysis of reinforced earth retaining structures was performed using a finite element analysis with commercial computer software, Plaxis. It is aimed to observe the influence of reinforcement type, reinforcement spacing, backfill material and reinforced slope on induced permanent displacements and stability of system. In order to compare and observe the influences of variable parameters; one natural slope model and three different reinforced earth retaining wall models were examined by Plaxis.

After completion of Plaxis analyses, results obtained by Plaxis have been compared with the results obtained by hand calculations in order to check accuracy of analyses and observe differences in analysis methods. The influence of applied models to the system has been evaluated and interpreted.

ÖZET

1966'da Fransız bilim adamı H. Vidal tarafından gündeme getirilen donatılı zemin tekniği, ortaya atıldıkları tarih itibari ile üzerinde yapılan çalışmalar ışığında günümüze kadar büyük bir ilerleme ile gelmiştir. Teknik, beraberinde getirdiği düşük maliyetler ve iş süresi ile birçok projeye mühendislik çözümü olarak sunulmaktadır.

Bu çalışmada donatılı zemin istinat yapılarının statik ve dinamik yüklemeler altında davranış ve tasarım ilkeleri incelenmektedir. Tasarım aşamasında Türkiye, Fransa ve ABD karayolları idarelerinin ilgili şartname ve yönetmelikleri kullanılmıştır. Çalışma içerisinde ABD karayolları şartnamesi temel alınarak model bir donatılı zemin istinat duvarı tasarlanmış, aynı model dinamik ve statik yüklemeler altında Plaxis Sonlu Elemanlar Programı ile analiz edilmiştir.

Donatılı zemin istinat yapılarının davranış prensiplerini analiz etmek için Plaxis sonlu elemanlar programı kullanılarak parametrik bir çalışma gerçekleştirilmiştir. Bu çalışmanın amacı; donatı tipinin, aralığının, dolgu malzemesi tipinin ve donatılandırılmış bir şevin, sistem stabilitesine etkisini incelemektir.

Çalışmanın son bölümünden ise elle ve Plaxis Sonlu Elamanlar Programı ile yapılan analizlerin sonuçları karşılaştırılarak, uygulanan modellerin sisteme etkisi üzerinde yorumlarda ve değerlendirmelerde bulunulmaktadır.

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LIST OF SYMBOLS/ABBREVIATIONS

A_0	Maximum Ground Acceleration
A_m	Maximum Acceleration on Wall
a	Horizontal distance from W to toe of footing
a_{max}	Maximum horizontal acceleration at ground surface (pga)
A_p	Anchor pull force (sheet pile wall)
c	Cohesion based on total stress analysis
c'	Cohesion based on effective stress analysis
c_a	Adhesion between bottom of footing and underlying soil
d	Resultant location of retaining wall forces
d_1	Depth from ground surface to groundwater table
d_2	Depth from groundwater table to bottom of sheet pile wall
D	Depth of retaining wall footing
E	Lateral distance from Pv to toe of retaining wall
F,	FS Factor of safety
FS_L	Factor of safety against liquefaction
g	Acceleration of gravity
H	Height of retaining wall
k_A	Active earth pressure coefficient
k_{AE}	Combined active plus earthquake coefficient of pressure
k_h	Seismic coefficient, also known as pseudostatic coefficient
k_0	Coefficient of earth pressure at rest
k_p	Passive earth pressure coefficient
k_v	Vertical pseudostatic coefficient
L	Length of active wedge at top of retaining wall
m	Total mass of active wedge
M_{max}	Maximum moment in sheet pile wall
N	Sum of wall weights W plus, if applicable, Pv
P_A	Active earth pressure resultant force
P_E	Pseudostatic horizontal force acting on retaining wall
P_{ER}	Pseudostatic horizontal force acting on restrained retaining wall

P_F	Sum of sliding resistance forces
P_H	Horizontal component of active earth pressure resultant force
P_L	Lateral force due to liquefied soil
P_p	Passive resultant force
P_R	Static force acting upon restrained retaining wall
P_v	Vertical component of active earth pressure resultant force
P_1	Active earth pressure resultant force
P_2	Resultant force due to uniform surcharge
Q	Uniform vertical surcharge pressure acting on wall backfill
R	Resultant of retaining wall forces
s_u	Undrained shear strength of soil
W	Total weight of active wedge
β	Slope inclination behind the retaining wall
δ, ϕ_{cv}	Friction angle between bottom of wall footing and underlying soil
δ, ϕ_w	Friction angle between back face of wall and soil backfill
ϕ	Friction angle based on total stress analysis
ϕ'	Friction angle based on effective stress analysis
γ_b	Buoyant unit weight of soil
γ_{sat}	Saturated unit weight of soil
γ_t	Total unit weight of the soil
θ	Back face inclination of retaining wall
σ_{avg}	Average bearing pressure of retaining wall foundation
σ_{mom}	That portion of bearing pressure due to eccentricity of N
ψ	Equal to $\tan^{-1}(a_{max}/g)$
ASTM	American Society of Testing Materials
FEM	Finite Element Method
HDPE	High Density Polyethylene
USEPA	United States Environmental Protection Agency

1. INTRODUCTION

The sloped soil masses are forced to move downward because of their self weights and in some cases because of other additional forces. These other forces can be listed as; masses on slopes, forces formed by movements of water and ground water and seismic forces.

In case of unsuitable conditions for application of sufficient slopes, construction of retaining structures are needed for preventing soil from sliding and slope stability.

The retaining structures which's main purpose is to provide required slope stability can be constructed by various ways. In general it is possible to classify the retaining structures as follows;

- Gravity Retaining Structures
- Semi-Gravity Retaining Structures
- Flexible Retaining Structures

The retaining structure type which is the subject of this study belongs to Flexible Retaining Structures category. The concept of reinforced earth was first mentioned by H. Vidal in 1966; and regarding to its easiness of application, short installation time, multi functionality and economy, it was accepted as a very common construction method for retaining structures.

Reinforced earth concept has widely been used in retaining structures, foundation soil improvement applications, pier constructions, water structures and road construction areas. The shortness of time requirement in construction phase, less space requiring, using of environment friendly facing members and its cost effectiveness are some reasons of application of reinforced earthing systems in retaining structures. And most recently the manufacturing of geotextile materials in our country also helps the decreasing of costs.

This study includes the design principles of reinforced earth retaining structures as to static and dynamic loads. The designs criteria are mostly based on the USA highway regulations but in some necessary cases TSE and French standards also used. The study also includes modeling of a reinforced earth retaining structure according to USA highway standards.

A parametric study of static and dynamic analysis of Reinforced Earth Retaining Structures was performed using a finite element analysis with a commercial software; Plaxis. The influence of the wall dimensions, stretching rigidity of reinforcement and backfill material on static and dynamic response of Reinforced Earth Retaining Structure has been investigated.

2. RETAINING WALLS

2.1. Retaining Wall Concept

Retaining walls those that rely on their weight for the stability of the wall are called as "Gravity walls" and those that mobilize earth pressures in the ground to provide resistance are called as "Embedded walls". Within each category there are a variety of wall types. The decision to select a particular retaining wall system for a specific project requires a determination of both technical feasibility and comparative economy. With respect to economy, the factors that should be considered are (Lambe et al., 1990):

- Soil and groundwater conditions
- Height and ground topography
- Availability and cost of select backfill material
- Construction constraints (space, access, equipment, specialist techniques available)
- Foundation conditions (i.e. would a deep or shallow foundation be appropriate for a cast-in-place concrete retaining wall)
- Need for temporary excavation support systems
- Environment – appearance and impact during construction
- Ground movements and their affects on adjacent structures
- Underground obstructions and services
- Complicated horizontal and vertical alignment changes
- Design life and maintenance requirements
- Aesthetics
- Cost

There are four principal modes of failure that need to be analysed for any gravity wall. These are;

- Sliding
- Overturning

- Bearing capacity
- Total Collapse of the soil and wall

In addition, it is generally necessary to check that the wall deformations and the ground movements will not be excessive. Common types of retaining walls can be seen at Figure 2.1 below.

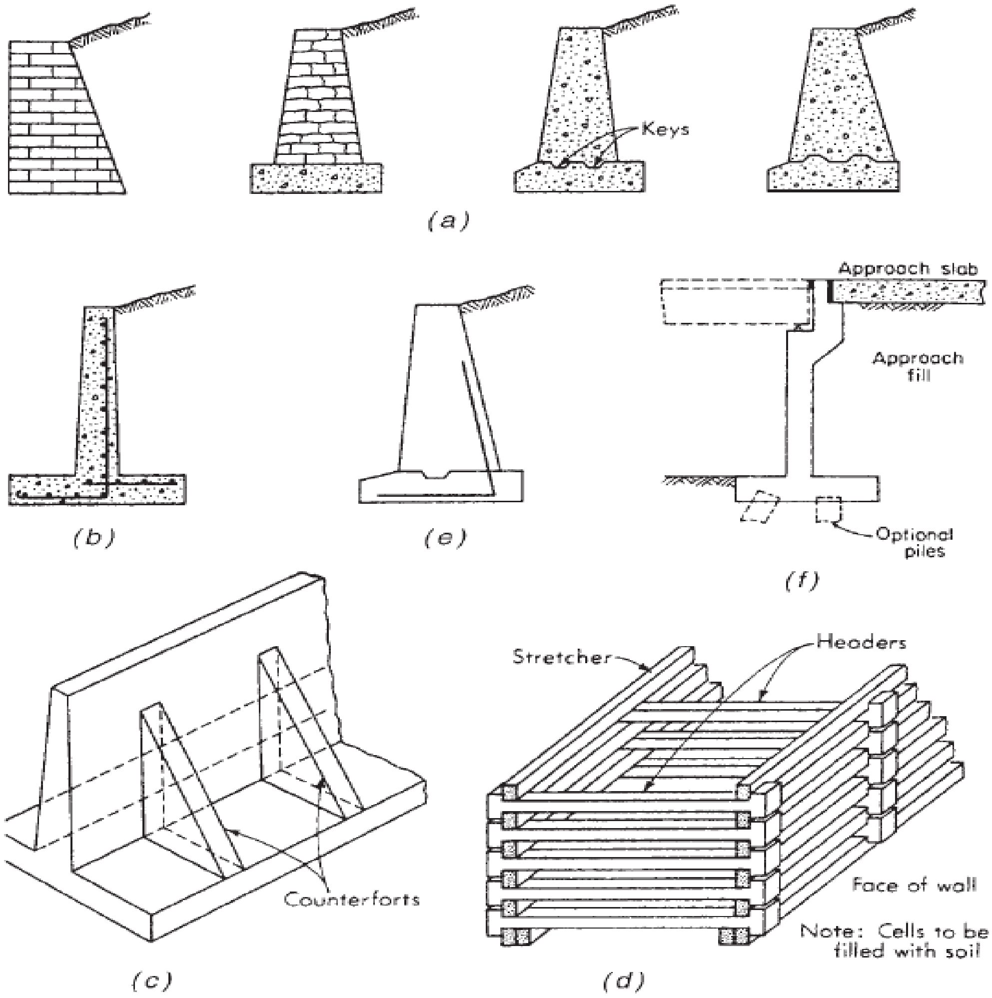

Figure 2.1. Common types of retaining walls. (*a*) Gravity walls of stone, brick, or plain concrete. Weight provides overturning and sliding stability. (*b*) Cantilevered wall. (*c*)Counterfort, or buttressed wall. If backfill covers counterforts, the wall is termed a *counterfort.* (*d*) Crib wall. (*e*) Semigravity wall (often steel reinforcement is used). (*f*) Bridge abutment (Craig, 1987)

Charts below show the flow diagram for Earth Retaining Systems before and during construction. Responsibility flow chart at pre-construction and construction phases for earth retaining systems can be seen in Figure 2.2. and Figure 2.3.

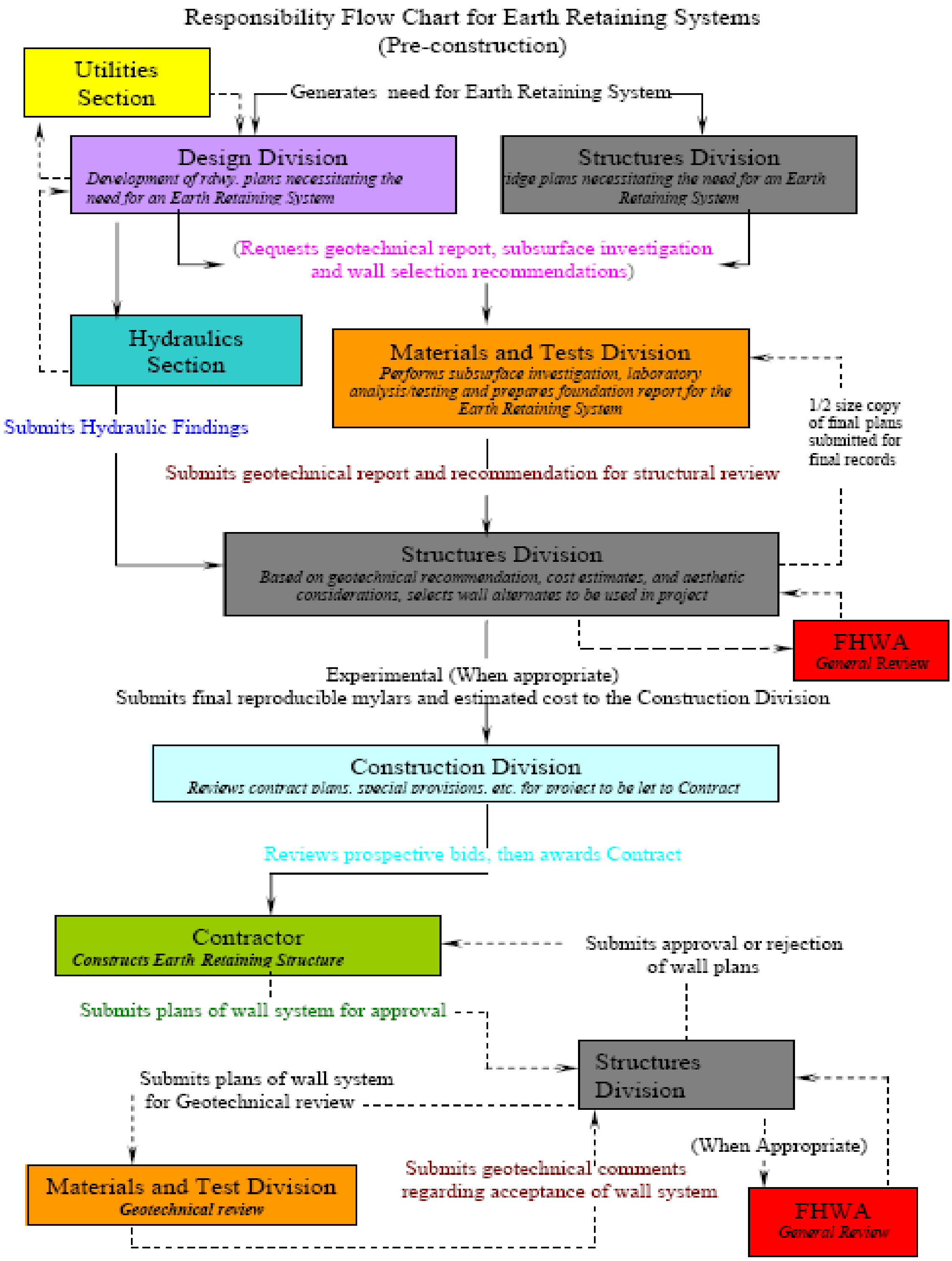

Figure 2.2. Responsibility flow chart (Pre-Construction) (FHWA, 1996)

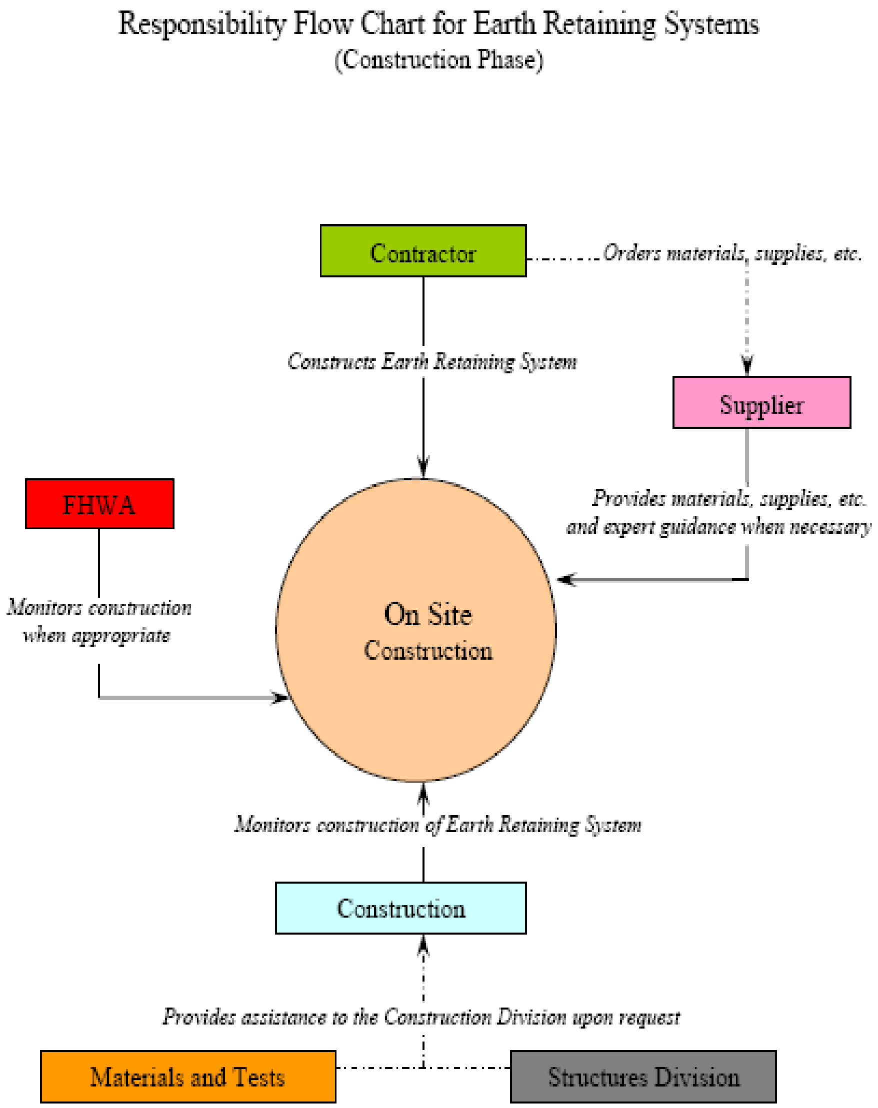

Figure 2.3. Responsibility flow chart (Construction) (FHWA, 1996)

2.2. Selection Criteria of Retaining Structures

In the cases that changing the slope height or inclination do not solve the stability problem, types of retaining structure can be determined according to three main factors listed as follows:

a) Allowable maximum deformation
b) Service life of retaining structure
c) Required space for retaining structure

For preventing adjacent structures and foundations from possible damages, retaining structure must be rigid and keep the deformations at a minimum level. On the other hand service type of retaining structure; whether to be temporary or permanent also plays an important role in system selection (Lambe et al., 1990).

2.3. Geotechnical Investigation

The engineering properties and behavior of backfill, retained soil, and foundation material must be evaluated because these materials are the major sources of both loading and support for any earth retaining system. The evaluations of retained soil and foundation materials are typically made through a geotechnical subsurface investigation and borrow source evaluation and a laboratory or in-situ testing program. The evaluation of backfill material is typically made through a laboratory testing program (Tennessee Department of Transportation, 2006).

2.4. Wall Drainage Systems

Appropriate drainage measures to prevent surface water from infiltrating into the wall backfill should be included in the design of a wall system. During construction, the backfill surface should be graded away from the wall at the end of each day of construction to prevent water from pounding behind the wall and saturating the soil (U.S Department of Transportation, 2006).

The need for drainage in cut wall system applications varies with project requirements. Drainage systems may be omitted in cases where ground-water draw down in the retained soil is prohibited or undesirable. In other cases, drainage is used as a means to control surface-water infiltration and ground-water seepage (U.S. Department of Transportation, 2006). Typical wall drainage systems are shown in Figure 2.4.

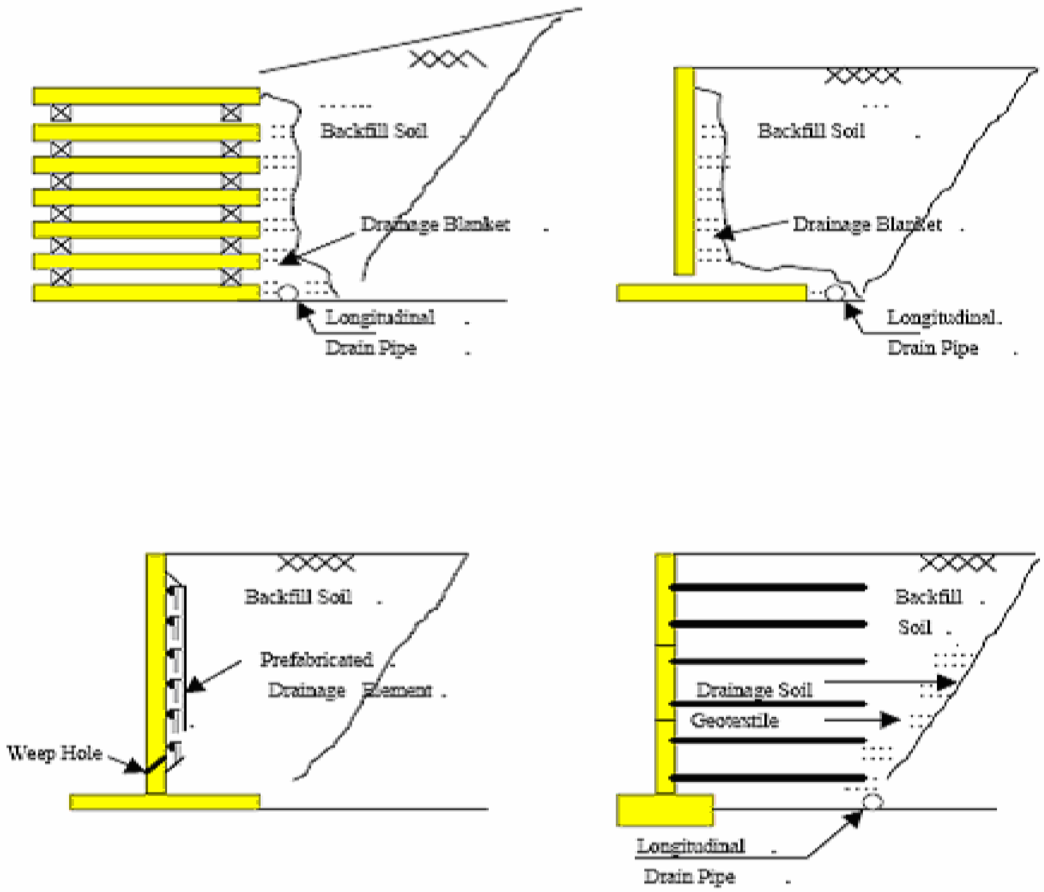

Figure 2.4. Wall drainage systems (FHWA, 1996)

2.5. Construction

During construction there are many factors that affect the loading on a retaining wall. Following are a few one should be aware of:

1) Types of backfill
2) Drainage of backfill material
3) Backfill overloads (heavy equipment)
4) Placement of backfill
5) Type of material beneath footing

Careful planning, study, design, etc. can be rendered useless if the wall is not constructed according to plans & specifications. If existing field conditions do not agree with plans, the engineer and/or geologist should be contacted (Das, 2001).

3. REINFORCED EARTH RETAINING STRUCTURES

3.1. Fundamentals of Earth Reinforcement

Soil is a natural construction material which is durable to certain extent in compression and shear when at a suitable density and water content, but totally incapable of carrying any tension. Similar to the case of Portland cement in reinforced concrete, with the inclusion of tensile reinforcing elements of some sort, a composite structure is produced to combine the best load carrying features of both reinforcement and soil. The resultant composite material is called as *Reinforced Earth* (Lambe and Hansen, 1990).

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An aspect in the success of any reinforced soil is that the two materials should be compatible in terms of surface characteristics, geometry and adherence, so that stresses can be transferred from one to the other. Existing ground, embankments and backfills of retaining walls may be strengthened considerably by the installation of reinforcement elements, in the form of layers or strips or grids, made out of steel, polymers, or plastics (Lambe and Hansen, 1990).

The inclusion of reinforcing elements into the earth, strong in tension, offers an outstanding potential for increasing strength in natural slope stability, for maintaining vertical or steep slopes in embankments and walls, and also for enabling excavation of cuts to withstand without external bracing (Das, 1987).

The field of reinforced soil has been developed significantly during the last two decades, such that a variety of materials and construction techniques have emerged an utilized in very diversified type of foundation engineering projects. Areas of application include retaining walls and systems, bridge abutments, embankments wharf and dikes, dams, army bunkers, stabilization and repair of slopes, improvements in bearing capacity and decreasing settlements of foundations on soft cohesive or loose granular soils, etc. (Das, 1987).

The significant advantage of reinforced soil is their real cost effectiveness over the conventional alternatives, also their ease of construction coupled with a basic simplicity, which provides an attraction to all civil engineers. This is especially true on sites with poor foundation soils that would otherwise require prohibitively expensive site improvement measures (Arı, 1998).

The use of earth reinforcement is so well suited to the needs of high way construction and reconstruction that many of the currently available techniques were specifically developed for highway application. Steep slopes of reinforced soil reduce the required width of new roads and are especially suitable for the widening of existing traffic lanes in constricted rights-of-the way. Reinforced soil is also extremely versatile in application (Das, 1987).

3.2. Earth Reinforcement Concept

The modern concept of earth reinforcement was proposed by Cassagrande who idealized the problem in the form of a weak soil reinforced by high-strength member laid horizontally in Layers (Das, 1987).

The modern methods of soil reinforcement of retaining wall construction were pioneered by the French architect and engineer Henry Vidal in the Early 1960s. His research led to invention and development of reinforced Earth, a system in which steel strip reinforcement is used (Karalı, 2004).

The use of geotextile in retaining walls and slopes after the beneficial effect of reinforcement with geotextile was noticed in highway embankments over weak sub grade. The first geotextile reinforced wall was constructed in France in 1971, and the first structure of this type in United States was constructed in 1974. Since about 1980, the use of geotextile in reinforced soil has increased significantly. On the other hand the use of geogrids for soil reinforcement was developed in 1980 and first application was in 1981 (Karalı, 2004).

Extensive use of geogrid products in the United States started in about 1983 and now they comprise a growing portion of the market. Recently modular block dry cast facing units have gained acceptance due to their lower cost and nationwide availability. These small concrete units are generally applied with grid reinforcement, and the wall system is referred to as modular block wall (MBW). It has been reported that more than 200 such structures have been constructed in the United States, for highway applications till now. The current yearly usage for transportation-related applications is estimated at about 25 projects per year (FHWA, 1996).

3.3. Description of Reinforced Earth Principles

Reinforced Soil Retaining Structures are composite construction materials in which the strength of engineering fill is enhanced by the addition of tensile reinforcement in the form of both steel strips and geosynthetic fabrics or grids. The basic mechanism of Reinforced Soil Structures involves the generation of frictional forces between the soil and the reinforcement. These forces are manifested in the soil in a form of analogous to an increased confining pressure, which enhances the strength of composite (Kesim, 1996).

Also the reinforcement has ability to unify a mass of soil that would otherwise part along the failure surface. The beneficial effects of soil reinforcement derive from:

- The soils increased tensile strength.
- The shear resistance developed from the friction at the soil-reinforcement interfaces (Das, 1994).

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 principle strain direction to composite for soils lack of tensile resistance. The composite material has the following characteristics:

- Stress transfer between the soil and reinforcement takes place continuously along the reinforcement.

- Reinforcements are distributed throughout the soil mass with a degree of regularity and must not be localized.

Stresses are transferred between the soil and reinforcement by friction and/or passive resistance depending on reinforcement geometry (Kesim, 1996).

Friction develops at locations where there is a relative shear displacement and corresponding shear stress between soil and reinforcement surface. Reinforcing elements where friction is important should be aligned with the direction of soil reinforcement relative movement as seen in Figure 3.1. Examples of such reinforcing are steel strips, longitudinal bars in grids, geotextile and some geogrid layers (Kesim, 1996).

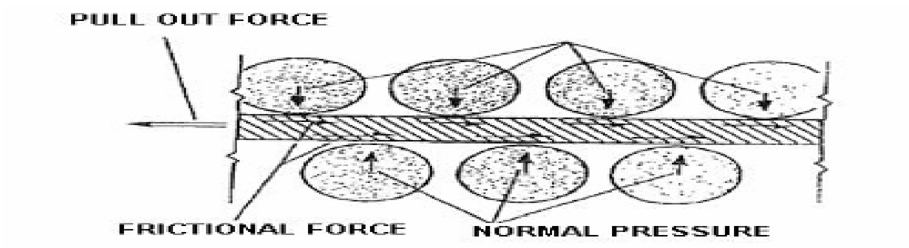

Figure 3.1. Stress transfer by friction (Das, 1987)

Passive resistance occurs through the development of bearing type stresses on "transverse" reinforcement surfaces normal to the direction of soil reinforcement relative moment as seen in Figure 3.2. Passive resistance is generally considered to be the primary intersection for rigid geogrids, bar mat, and wire mesh reinforcements. The transverse ridges on "ribbed" strip reinforcement also provide some passive resistance (Kesim, 1996).

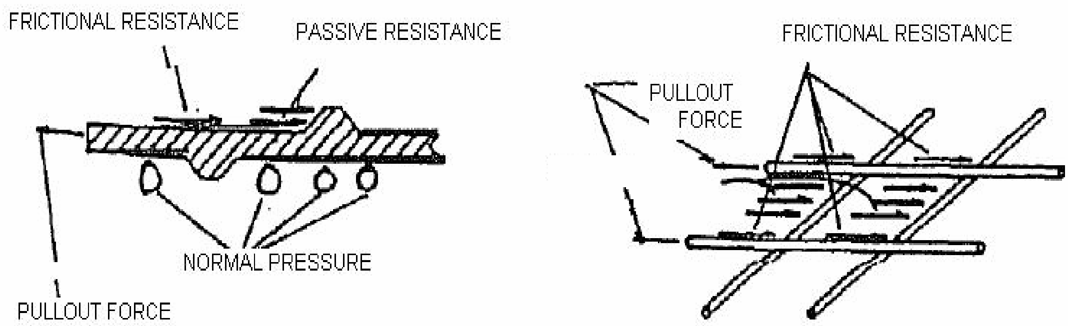

Figure 3.2. Stress transfer by passive resistance (Das, 1987)

The contribution of each transfer mechanism for a particular reinforcement depend on roughness of the surface, normal effective stress, grid opening dimensions, thickness of transverse members, and elongation characteristics of the reinforcement. Equally important for interaction development are soil characteristics, including grain size, grain size distribution, particle shape, density, water content, cohesion and stiffness (FHWA, 1996).

3.4. Components of Reinforced Earth Retaining Wall

The major components comprising a reinforced earth wall are; the backfill soil, reinforcement, facing units and foundation soil (U.S Department of Transportation, 2001). These components are shown in Figure 3.3. below.

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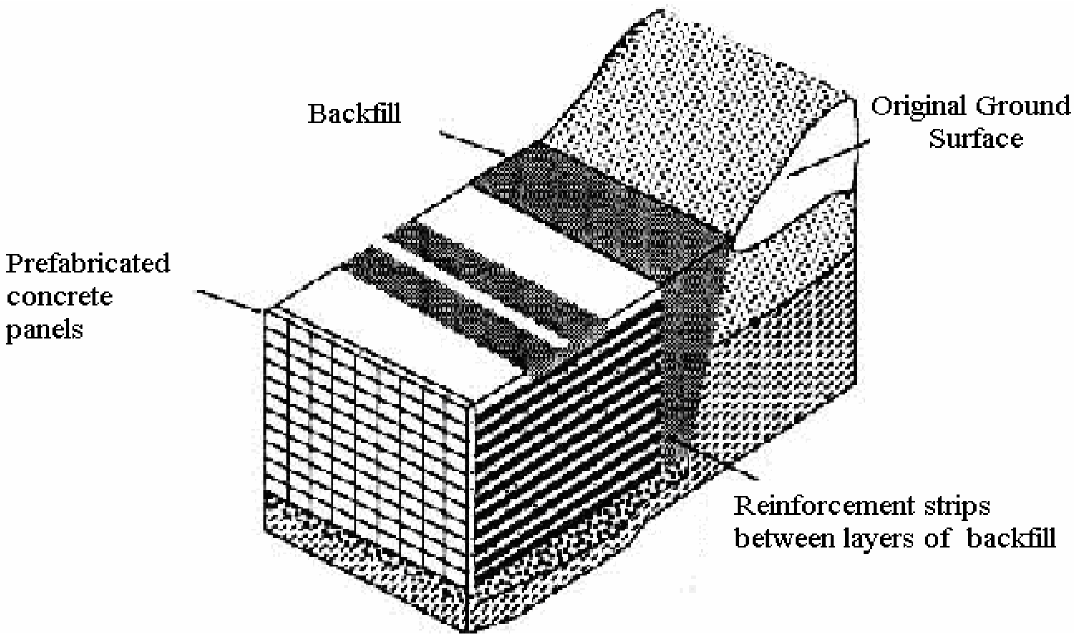


Figure 3.3. Components of Reinforced Earth Retaining Wall (Das, 1987)

3.4.1. Backfill Soil

Reinforced Earth Walls require high quality backfill for durability, good drainage, constructability, and good soil reinforcement interaction, which can be obtained from well-graded, granular materials. Many reinforced soil systems depend on friction between the reinforcing elements and the soil. In such cases, a material with high friction characteristics is specified and required. Some systems rely on passive pressure on reinforcing elements, and, in those cases, the quality of backfill is still critical. These performance requirements generally eliminate soils with high clay contents (U.S Department of Transportation, 2001).

From a reinforcement capacity point of view, lower quality backfills could be used for Reinforced Earth Structures; however, a high quality granular backfill has the advantages of being free draining, providing better durability for metallic reinforcement, and requiring less reinforcement. There are also significant handling, placement and compaction advantages in using granular soils. These include an increased rate of wall erection and improved maintenance of wall alignment tolerances (U.S Department of Transportation, 2001).

The selection criteria of reinforced backfill should consider long-term performance of completed structures, construction phase stability and the degradation environment created for the reinforcements. Much of our knowledge and experience with reinforced earth structures today have proved the efficiency of using cohesionless backfill; but researches are being carried out into the possibility of using cohesive soils as a backfill material (Das, 1987).

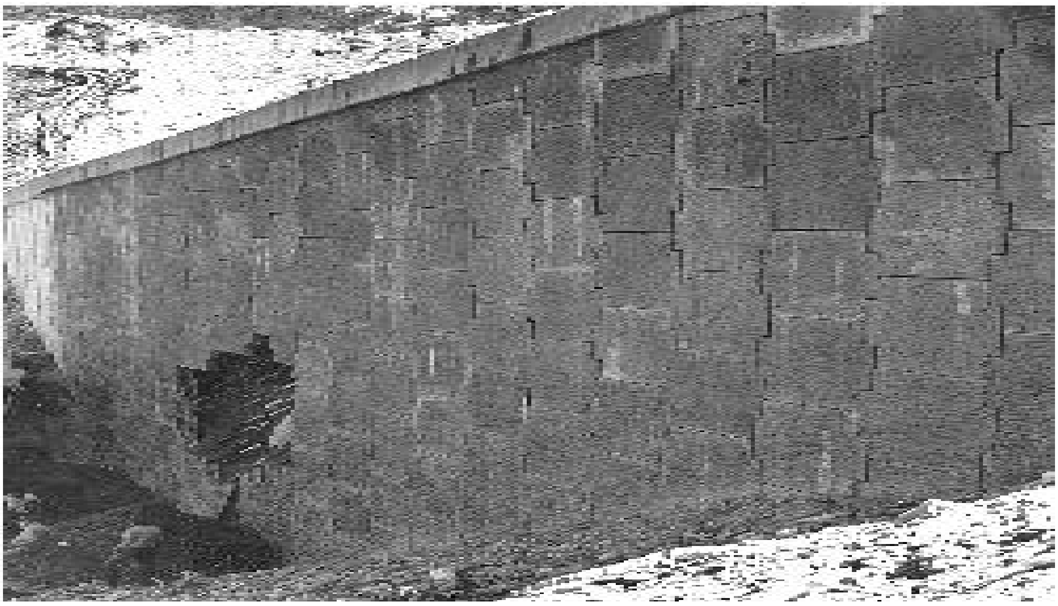


Figure 3.4. Failure of a reinforced earth retaining wall (Koerner, 1998)

The photo shown is Figure 3.4. was taken in the mid 1980's. Shown is the failure of a Reinforced Earth Retaining Wall. This failure was one of the most significant which occurred during construction of the Foothills Parkway in Blount County, Tennessee. Approximately 33 factors were identified by the FHWA as contributing to this failure. The most significant factor was improper placement of backfill. A typical backfill classification chart is shown in Table 3.1. (Das, 1987).

Table 3.1. Backfill classification (Das, 1987)

Wall Backfill Classification	**Common Description**	**UNSC Classification**	**φ range**	**γ range (moist)**	**Comments**
Good	Sand, Gravel, Stone	GW, GP, GM, GC, SW, SP	32-36	100-135pcf	Poor grading lower weight
Moderate	Silty Sands, Clayey Sands	SM, SC	28-32	110-130pcf	Moisture sensitive
Difficult	Silts, Low Plastic Clays	ML, CL, OL	25-30	110-125pcf	Pl<20 LL<40
Bad	High Plastic Silts & Clays, Organics	CH, MH, OH,PT	0-25	50-110 pcf	PI>20 LL>40

3.4.2. Reinforcement

The reinforcement elements can be distinguished into two main types. The first type is metallic reinforcements and typically it is made of mild steel. The steel is usually galvanized or may be epoxy coated. The steel behaves as inextensible reinforcement; deformation of the reinforcement at failure is much less than the deformability of the soil. The geometry of the steel is linear unidirectional. And the other is nonmetallic (geosynthetic) reinforcement; generally polymeric materials consisting of polypropylene, polyethylene, or polyester are used. The deformation of the reinforcement at failure is comparable to or even grater than the deformability of the soil (U.S Department of Transportation, 2001).

3.4.2.1. Metallic Strips. Metallic strips are the types of reinforcement introduced by Henri Vidal (1969) under the trade name Reinforced Earth and have been patented throughout the industrialized countries in the world. The worldwide groups of Reinforced Earth companies are the exclusive licensees for Vidal's technique (Tezcan and Buket, 1999).

Reinforced Earth is a composite material formed from the association of ductile steel elements and granular soil. Reinforced Earth has been used in various types of structures but its main use so far has been for retaining walls and bridge abutments on road schemes.

In the design of Reinforced Earth the cross-sectional area of the reinforcing strip is considered to be the gross area less a sacrificial thickness which is typically 1 or 1.5 mm. Thus, the assumed cross-sectional area of strips is approximately 70 per cent of the area provided. Moreover, the strips are galvanized to a high standard, this protection providing a longer life and ensuring that the corrosion is even (Tezcan and Buket, 1999).

During the design stages of a Reinforced Earth projects, the emphasis is on the arrangement and density of the reinforcing strips. Consideration of the facing panels does not take up a large part of the design time. However, the panels are a very important feature of Reinforced Earth, as they provide a clean physical boundary to the Reinforced Earth mass and are the only visible part of a Reinforced Earth structure (Smith and Unal, 1989). A typical reinforced earth retaining wall construction with metallic strip reinforcement is shown in Figure 3.5.

Figure 3.5. Construction with metallic strips (MSE Handbook, 2006)

3.4.2.2. Reinforcing Planks. Similar to strips except that their form of construction makes them stiff. Planks can be formed from timber, reinforced concrete or prestressed concrete. The dimensions of concrete planks, vary; however, reinforcements with a thickness, t = 100 mm and breadth, b = 200-300 mm have been used. They have to be handled with care as they can be susceptible to cracking (Tezcan and Buket, 1999).

3.4.2.3. Geosynthetic Materials. Geosynthetic products appeared two decades ago as new materials for civil engineering applications. Because of their unique properties as lightweight flexible membranes, the geosynthetics become essential for use in geotechnical applications.

Geosynthetic materials can be divided into two categories; (a) geotextiles and (b) geomembranes. Geotextiles have a fabric structure which is permeable, while geomembranes have a membrane structure designed to allow minimum permeability. Geogrids have a have a grid structure manufactured of synthetic polymers or metallic materials for use in geotechnical engineering. The term geotextiles will include the polymer geogrids and the non-grid (sheet) type geofabrics (Koerner, 1998).

Geotextiles are made of synthetic polymers. The most commonly used polymers in geotextile products are: polyester, polypropylene, polyamide and polyethylene. Geotextiles can be subdivided into two broad classes; (a) the classical geotextiles, and (b) special geotextiles. The classical geotextiles, which are common products of the textile industry, include woven, knitted and nonwoven fabrics. On the other hand, the special geotextiles are products recently developed to be used in combination with, or in place of, classical geotextiles. These products include: nets, grids, mats, webbings and formed plastic sheets (Koerner, 1998).

With this application, the structural stability of the soil is greatly improved by the tensile strength of the geosynthetic material. This concept is similar to that of reinforcing concrete with steel. Since concrete is weak in tension, reinforcing steel is used to strengthen it. Geosynthetic materials function in a similar manner as the reinforcing steel by providing tensile strength that helps to hold the soil in place. Reinforcement provided by geotextiles or geogrids allows embankments and roads to be built over very weak soils and allows for steeper embankments to be built, scheme of an embankment with geosynthetic reinforcement is represented in Figure 3.6. (Koerner, 1998).

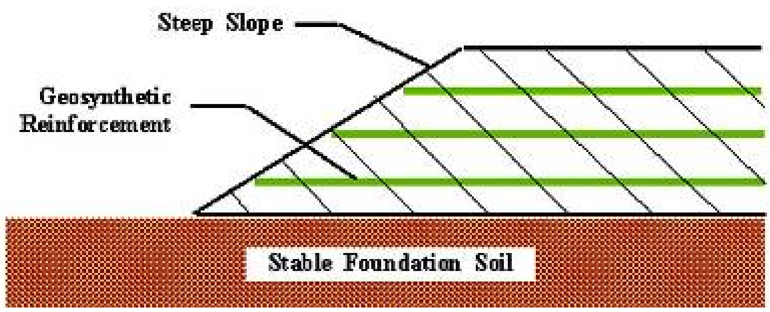


Figure 3.6. Soil reinforcement of an embankment using geosynthetics (Koerner, 1998)

3.4.2.4. Grids and Geogrids. Reinforcing elements formed from transverse and longitudinal members, in which the transverse members run parallel to the face or free edge of the structure and behave as abutments or anchors. The main purpose is to retain the transverse members in position. Since the transverse members act as an abutment or anchor they need to be stiff relative to their length. The longitudinal members may be flexible having a high modulus of elasticity not susceptible to creep (Tezcan and Buket, 1999). A typical metallic grid reinforcement application in a reinforced earth system is seen at Figure 3.7. below (Koerner, 1998).

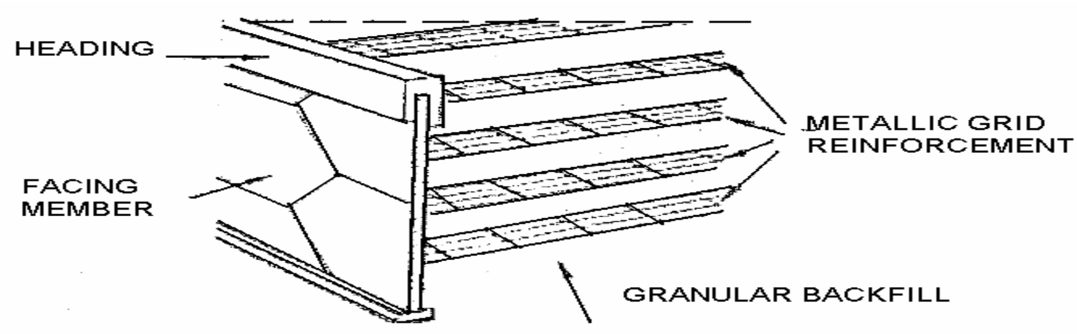


Figure 3.7. Metallic grid reinforcement application in a reinforced earth system (Koerner, 1998)

Grids can be formed from steel in the form of plain or galvanized weldmesh or from expanded metal. Grids formed from polymers are known as geogrids and are normally in the form of an expanded proprietary plastic product. Grids transfer stress to the soil through passive soil resistance on transverse members of the grid and friction between the soil and horizontal surfaces of the grid (FHWA, 1996).

Grid reinforcements made of stable polymer materials may provide good resistance to deterioration in adverse soil and ground water environments. Tensar geogrids are high strength polymer grid reinforcements manufactured from high density polyethylene or polypropylene using a stretching process. Facings can be formed for geogrids by looping reinforcements at the face or by attachment of the reinforcement grids to gabions or concrete panels as shown in Figure 3.8. (Durukan, 1988).

Figure 3.8. Geogrid reinforcement application (Koerner, 1998)

The advantages of using geogrids with strips are as follow (Koerner, 1998);

- Simplicity of facing panels and ease of casting in tails,
- Simplicity of connection of grids using bodkin joint,
- Grids can be quickly rolled out and tensioned, and held in place by fill, compared with difficulty of placing many individual strips,
- Grids are easily cut to length, and require no special treatment at the ends; compare paraweb, which should have the ends sealed to prevent absorption of water which weakens the polyester,
- Grids are supplied in light weight rolls easily carried by one man, roles are clearly labeled. Different sizes and grades look different, so it is no difficult to ensure that the correct type of grid is being used,
- Grids have good ultra-violet resistance, so do not require special storage on site. There is no danger of corrosion due to hard fill scratching protective coatings. The effects of construction on the strength of material are allowed for in the design.

3.4.2.5. Sheet or Sheat Reinforcements. Sheet reinforcement or simply sheats may be formed from metal such as galvanized steel sheet, textile fabrics, or expanded metal not necessarily meeting the criteria for a grid. Continuous sheets of geotextiles laid down alternately with horizontal layers of soil form a composite reinforced soil material with the mechanism of stress transfer between soil and sheet reinforcement being predominantly friction (Smith and Unal, 1989).

3.4.2.6. Rope or Rod Reinforcement. Flexible linear elements having one or more pronounced protrusions or distortions which act as abutments or anchors in the fill or soil. They may be formed from steel, rope, plastic (textile) or combinations of materials such as webbing and tires, steel and tires, steel and tires, or steel and concrete. Reinforcement can be in the form of combinations of materials and material forms such as sheets and strips, grids and strips, or strips and anchors, depending on the requirements (Smith and Unal, 1989).

3.4.2.7. Fiber Reinforcement. A composite construction material with improved mechanical properties can be created by the inclusion of tensile resistant fibers within a soil mass. The engineering use of fiber reinforcements in soil, which is analogous to fiber reinforcement of concrete, is still in the early developmental stages. Materials being investigated for possible use include natural fibers like reeds and other plants, synthetic fibers geotextile threads and metallic fibers like small-diameter metal threads.

Unlike other reinforcements, fibers can potentially provide reinforcement in three directions. The major limitation to use of this method is the difficulties associated with efficiently and economically mixing the fibers uniformly into the backfill. In experimental fiber reinforcement system described by “Leflaive 1982”, continuous geotextile threads are incorporated into a sandy soil mass by a very complex mixing process (Kramer, 2003).

3.4.2.8. Glass Fiber Reinforced Plastic (GRP). Glass fiber reinforced plastic strips have been developed by Pilkington Brothers in 1978. They are formed from a continuous filament of E-glass roving embedded in a thermo-setting polymer. The materials are combined to form an aligned fiber reinforcing strip in the form of a hairpin, the end connection being formed at the loop. The diameter of the anchor hole varies with requirement. GRP does not exhibit plastic deformation (Kramer, 2003).

3.4.2.9. Selection Criteria for Reinforcement. As compared to conventional retaining structures, the geosynthetic reinforced soil systems are more economical and faster to build. In addition, geosynthetics are more flexible which enable them to better withstand differential settlements of the soil mass (Koerner, 1998).

As compared to metallic reinforcement, the geosynthetics are more durable to the chemical attack in the soil, lightweight and easy to handle, and do not need a facing unit during construction. Also, geofabrics have better bonding properties in the soil mass, which allows poor insitu soil be used as the fill material. It is required that the backfill material used in a reinforced earth structure be granular soil with no more than 15per cent finer than 15 micron. This can make a substantial cost difference between metallic reinforced earth and geosynthetic reinforced soil (Koerner, 1998).

As compared with other methods, the construction effort is significantly less in geosynthetic reinforcing systems. This system requires only earthwork and some general labor to place the reinforcement. On the other hand conventional retaining wall systes require concrete work, steel work, forms and other false work in addition to the earth work. Metallic reinforced earth systems require steel work, precast facing work, and some concrete work for the facing foundation in addition to the earthwork (Koerner, 1998).

Disadvantages of geosynthetic reinforcement can be mentioned as follows:

Lack of knowledge and assessment techniques for the long term behavior resulting from creep, chemical and thermal degradation, ultraviolet degradation and the damage caused during construction.

Construction of geosynthetic reinforced slopes in a cut region requires a wider excavation than other retaining structures.

3.4.3. Facing Unit

At a free boundary of Reinforced Earth Retaining Structures, it is necessary to provide some form of barrier so that the soil is contained. This skin can be either flexible or stiff but it must be strong enough to hold back the local soil and to allow attachment of the reinforcements. The types of facing elements used in the different Reinforced Soil Walls systems control their aesthetics because they are the only visible parts of the completed structure. A wide range of finishes and colors can be provided in the facing. In addition, the facing provides protection against backfill sloughing and erosion, and provides in certain cases drainage paths. The type of facing influences the settlement tolerances. Major facing types are explained briefly as follows.

Segmental Precast Concrete Panels are the precast concrete panels, which have a minimum thickness of 140 mm and are of cruciform, square, rectangular, diamond, or hexagonal geometry. Temperature and tensile reinforcement are required but will vary the size of the panel. Vertically adjacent units are usually connected with shear pins. Precast elements can be cast in several shapes and provided with facing textures to match environmental requirements and blend aesthetically into the environment. Retaining structures using precast concrete elements as the facing can have surface finishes similar to any reinforced concrete structure (Helwany, 2001).

Dry Cast Modular Block Wall (MBW) units are relatively small, squat concrete units that have been specially designed and manufactured for retaining wall applications. The mass of these units commonly 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 (Dimension perpendicular to the wall face) of units typically ranges between 200 and 600 mm. 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 the mortar) and in a running bond configuration. Vertically adjacent units may be connected with shear pins, lips, or keys. Recently introduced dry cast segmental block MBW facings raise some concerns as to their durability in aggressive freeze-thaw environments because their water absorption capacity can be significantly higher than that of wet-cast concrete. Historical data provide little insight as their usage history is less than a decade. Further, because the cement is not completely hydrated during the dry cast process, (as is often evidenced by efflorescence von the surface of units), a highly alkaline regime may establish itself at or near the face area, and may become an aggressive aging media for some geosynthetic products potentially used as reinforcements. Freeze-Thaw durability is enhanced for products produced at higher compressive strengths and/or sprayed with a post erection sealant (Helwany, 2001).

Metallic Facings are the original reinforced earth system had facing elements of galvanized steel sheet formed into half cylinders. Although precast concrete panels are now usually used in Reinforced Earth Walls, metallic facings may be appropriate in structures where difficult access or difficult handling requires lighter facing elements (Helwany, 2001).

Geosynthetic Facing, which is various types of geotextile reinforcements, looped around at the facing to form the exposed face of retaining wall. These faces are susceptible to ultraviolet light degradation, vandalism (e.g. target practice) and damage due to fire. Alternately, a geosynthetic grid used for soil reinforcement can be looped around to form the face of the completed retaining structure in a similar manner to welded wire mesh and fabric facing. Vegetation can grow through the grid structure and can provide both ultraviolet light protection for the geogrid and a pleasing appearance (Helwany, 2001).

Gabion Facing (rock-filled wire baskets) can be used as facing with reinforcing elements consisting of welded wire mesh, welded bar mats, geogrids, geotextiles or the double twisted woven mesh placed between or connected to the gabion baskets. Facings using welded wire or gabions have the disadvantages of an uneven surface, exposed backfill material, more tendencies for erosion of the retained soil, possible shorter life from corrosion of the wires, and more susceptibility to vandalism. These disadvantages can, of course, be countered by providing shotcrete or by hanging facing panels on the exposed face and compensating for possible corrosion. The greatest advantages of such facings are; low cost, ease of installation, design flexibility, good drainage (depending on the type of backfill) that provides increased stability, and possible treatment of the face for vegetative and other architectural effects. The facing can easily be adapted and well blended with natural country environment. These facings, as well as geosynthetic wrapped facings, are especially advantageous for construction of temporary or other structures with a short term design life (Helwany, 2001).

Post Construction Facings are for wrapped faced walls, the facing whether geotextile, geogrid, or wire mesh can be attached after construction of the wall by shotcreting, grunting, cast-in-place concrete or attaching prefabricated facing panels made of concrete, wood, or other materials. This approach adds cost but is advantageous where significant settlement is anticipated (Helwany, 2001).

3.4.4. Foundation Soil

The foundation soil of the Reinforced Soil Retaining Wall should satisfy some specifications that are mentioned in the designing manuals. To determine the characteristics of foundation soils, boring may have to be conducted to understand better the geological conditions of the foundation. The depth of investigation should be extended up to a firm soil layer that does not exhibit sign of instability, settlement and liquefaction. For the sites where potential problems have been revealed during pre-investigation, sounding, sampling, and soil testing are required to obtain additional information (Tezcan and Buket, 1999). The determination of engineering properties for foundation soils should be focused on establishment of bearing capacity, settlement potential, and position of ground water levels. Major foundation weakness and compressibility may require the consideration of ground improvement techniques to achieve the adequate bearing capacity, or limiting total or differential settlement (Lambe, 1990).

3.5. Advantages of Reinforced Earth Structures

3.5.1. Technical Advantages

<u>3.5.1.1. Composite Construction Material.</u> The stress transfer between soil and reinforcement creates a composite material with improved structural properties compared to un-reinforced soil. In fact, the use of abundant and relatively inexpensive soil as a

construction material can be greatly extended (Lambe, 1990) as shown by some of the special applications discussed in this chapter.

3.5.1.2. Flexibility. The deformation response characteristics of reinforced soil structures provide technically attractive solutions on sites with poor foundation soils. In comparison with conventional retaining walls, reinforced soil structures are extremely tolerant of large deformations, both laterally and vertically (Lambe, 1990).

The flexibility of reinforced soil structures also allows the use of lower factors of safety for bearing capacity design than conventional more rigid structures. The latter are normally designed to have bearing capacity factors of safety equal to three to restrict settlements to tolerable values, while reinforced soil structures are routinely designed with the bearing capacity factors of safety as low as 1.5 to 2. (Lambe, 1990).

3.5.1.3. Ease of Construction. In general, placement of successive layers of backfill material, reinforcements, and facing elements does not require specialized contractors, skilled labor, or specialized equipment and can be carried out with the same equipment and at a rate comparable to ordinary highway embankment construction (U.S. Department of Transportation, 2001).

Many of the components of the available earth reinforcement systems are prefabricated, thus providing ease of forming and handling and allowing relatively quick construction. Often, only minimal working space is required in front of the earth structure, which is especially advantageous when working along existing highways or restricted areas (Lambe, 1990).

The use of in-situ earth reinforcement to retain excavations offers construction advantages over classical excavation bracing schemes, in that it avoids both obstructions within the excavation, such as cross-lot braces, and the excessive noise associated with driving of sheet piles or soldier piles. Also, although not yet used on large scale, corrosion resistant reinforcement could provide permanent soil restraint, thus reducing the design loads on buried structures (Lambe, 1990).

<u>3.5.1.4. Various Backfill Material.</u> A fairly wide range of backfill materials may be used for reinforced soil structures. Suitable quality backfill material can frequently be found near the construction site and thus need not to be imported. Typically, predominantly granular materials, such as clean sands and gravels, or silty soils have been also used successfully in a few applications. As performance experience is gained, use of these soil types may become more frequent (Kesim, 1996).

Although not yet fully investigated, the admixture stabilizers, such as cement and lime, to improve plastic soils may further enhance the range of materials that can satisfactorily be used for reinforced soil (Han and Gabr, 2001).

As a coherent yet flexible gravity mass reinforced soil structures seem to be particularly well suited for construction in seismically active regions. The structures provide the high degree of structural damping needed to absorb large energy releases associated with earthquakes (Han and Gabr, 2001).

3.5.2. Economic Advantages

Reinforced soil can often provide the most economic retaining wall for embankments constructed under the constraints of limited access or right-of-way. The materials used are often less expensive than those required for a conventional wall; the soil; which comprises by far the largest percentage of volume, is relatively inexpensive, and cost effectiveness may hinge on cost of reinforcements and facing elements. The ease and speed of construction generally associated with the earth reinforcement methods is another source of cost savings relative to conventional walls (Durukan, 1988).

Especially where the construction of rigid retaining walls would require deep foundations (e.g., mountainous areas with unstable slopes and sites underlain by compressible soils), the use of reinforced can result in significant cost savings (Durukan, 1988).

Because of the inherent flexibility, a reinforced soil structure can tolerate large differential settlement and lateral movement. Hence; expensive deep foundations are not required provided overall stability requirements can still be satisfied (Durukan, 1988).

3.5.3. Architectural Advantages

Because the facing elements play only a secondary structural role, a greater flexibility in choice of facing is available to meet aesthetic requirements than is the case of for the normal retaining walls. A wide variety of architectural finishes are available for the facing elements of reinforced soil structures. Available facing arrangements range from concrete panels of various geometric shapes, textures, and colors to provision of vegetation at the exposed face of the soil (Durukan, 1988).

As a summary the following items are listed as the advantages of reinforced earth retaining structures;

- Use simple and rapid construction procedures and do not require large construction equipment.
- Do not require experienced craftsmen with special skills for construction.
- Requires less site preparation than other alternatives.
- Need less space in front of the structure for construction operations.
- Reduce right of way acquisition.
- Do not need rigid, unyielding foundation support because reinforced soil retaining structures are tolerant to deformations.
- Cost effective.
- Are technically feasible to heights in excess of 25 m.

And the following general items may be associated with all reinforced soil structures as disadvantages. :

- Require a relatively large space behind the wall or outward face to obtain enough wall width for internal and external stability.

- Reinforced soil retaining walls require selected granular fill. At sites where there is an absence of granular soil, the cost of importing suitable fill material may render the system uneconomical.
- Suitable design criteria are required to address corrosion of steel reinforcement elements, deterioration of certain types of exposed facing members such as geosynthetics by ultra violet rays, and potential degradation of polymer reinforcement in the ground.

A typical reinforced earth retaining structure application with mono-block facing members at USA is shown in Figure 3.9. below.

Figure 3.9. Reinforced earth retaining structure application at USA (MSE, 2006)

4. AREAS OF APPLICATION

4.1. General Applications

Earth reinforcement is so well suited to the needs of highway construction and reconstruction that many of the currently available techniques were specifically developed for highway applications. Probably two of its most useful applications are for retaining walls and bridge abutments, where they compete favorably, both economically and aesthetically, with reinforced concrete. In some cases, especially when foundation soils are deformable, reinforced soil provides technical advantages over reinforced concrete because of greater flexibility of reinforced soil structures. Any situation requiring an elevation change of more than a meter relatively short distance is potentially suitable for earth reinforcement (Durukan, 1988).

Some applications of reinforced earth retaining structures at USA and Turkey are shown in Figures 4.1. and 4.2.

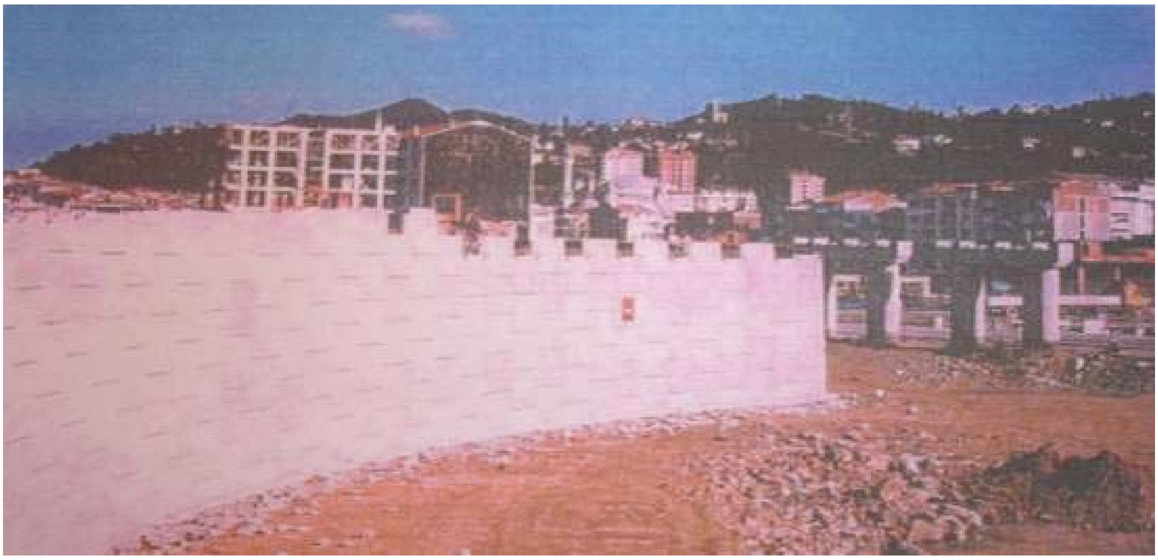


Figure 4.1. Reinforced earth retaining wall construction at Umraniye Istanbul (Demirkan, 2002)

Steep slopes made possible by the use of reinforced soil which also reduce the required width of new rights-of-way. Slope steepening by construction of reinforced soil walls can facilitate the widening of existing traffic lanes in constricted rights-of-way (Tezcan and Buket, 1999).

Reinforcement may be used for construction of new embankments, for the retention of excavations and for the stabilization of slopes. Because of the ease and speed of the construction and since the site preparation requirements are usually minimal, reinforced soil construction normally results in less traffic disruption than conventional construction techniques (Tezcan and Buket, 1999).

Figure 4.2. Another reinforced earth structure application for bridge piers (Tensar, 2006)

Basic concept of earth reinforcement is very simple. Because of the many applications to highway construction, maintenance and improvement, and also to the other earthwork construction, earth reinforcement has been implemented in many forms. Keen competition among the developers and manufacturers of different systems, combined with ongoing study and research by both the developers, manufacturers and independent researchers, has led to rapid technological developments and continued cost reductions (Tezcan and Buket, 1999).

Thus the use of some type of earth reinforcement may provide an effective and economical solution to many transportation corridor situations requiring retaining structures and slope stabilization (Tezcan and Buket, 1999).

Some engineering solutions by reinforced earth method are represented at Figures 4.3., 4.4. and 4.5. consecutively.

Figure 4.3. Solutions by reinforced earth method-retaining walls (Craig, 1987)

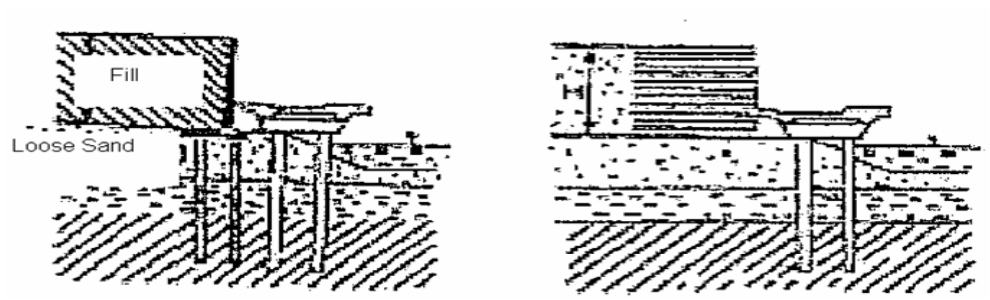


Figure 4.4. Solutions by reinforced earth method -transportation structure (Craig, 1987)

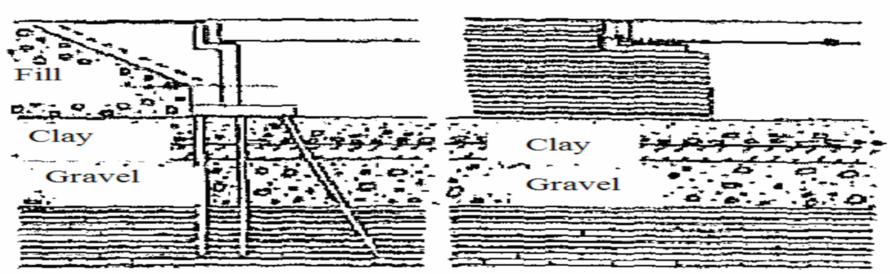


Figure 4.5 Solutions by reinforced earth method -bridge piers (Craig, 1987)

4.2. Special Applications

The most common uses of earth reinforcement techniques have been for retaining walls and bridge abutments. Other applications have included repair of slope failures, foundation rafts, containment dikes, dams, seawalls, bulkheads, and quays. In addition to embankment applications, soil reinforcement has also been used for the improvement of ground in-situ; specific applications include stabilizations of unstable or sliding slopes, retained excavations, and underground chambers. Some special application areas are shown in Figure 4.6.

At the present state of soil reinforcement practice (2007) most applications can be categorized as routine; however, as earth reinforcement methods become more widely used, more specialized uses will undoubtedly be developed. Some special applications of built embankment type earth reinforcement systems are as follows (U.S. Department of Transportation, 2001):

- Marine Structures: Reinforced soil structures have been used for seawalls, dikes, bulkheads, quays and dams.
- Storage Tanks: A sloped wall form of reinforced earth has been developed and used for construction of roof-covered bulk storage facilities.
- Foundation Rafts: A reinforce earth slope was constructed for support of a highway embankment of Pennsylvania Route 202. The slab was required to distribute embankment loads and span over sinkhole cavities at the site. An adapted form of reinforced soil with both longitudinal and transverse strips was used. A similar reinforced soil slab was also constructed in Mercer Country, west Virginia, to spa over sinkhole cavities.
- Containment Dikes: Reinforce soil dikes have been used for secondary containment of hazardous liquids (LNG, crude oil) in tank storage areas. Reinforced soil containment dikes are also built for the Cove Point LNG Terminal in Maryland.

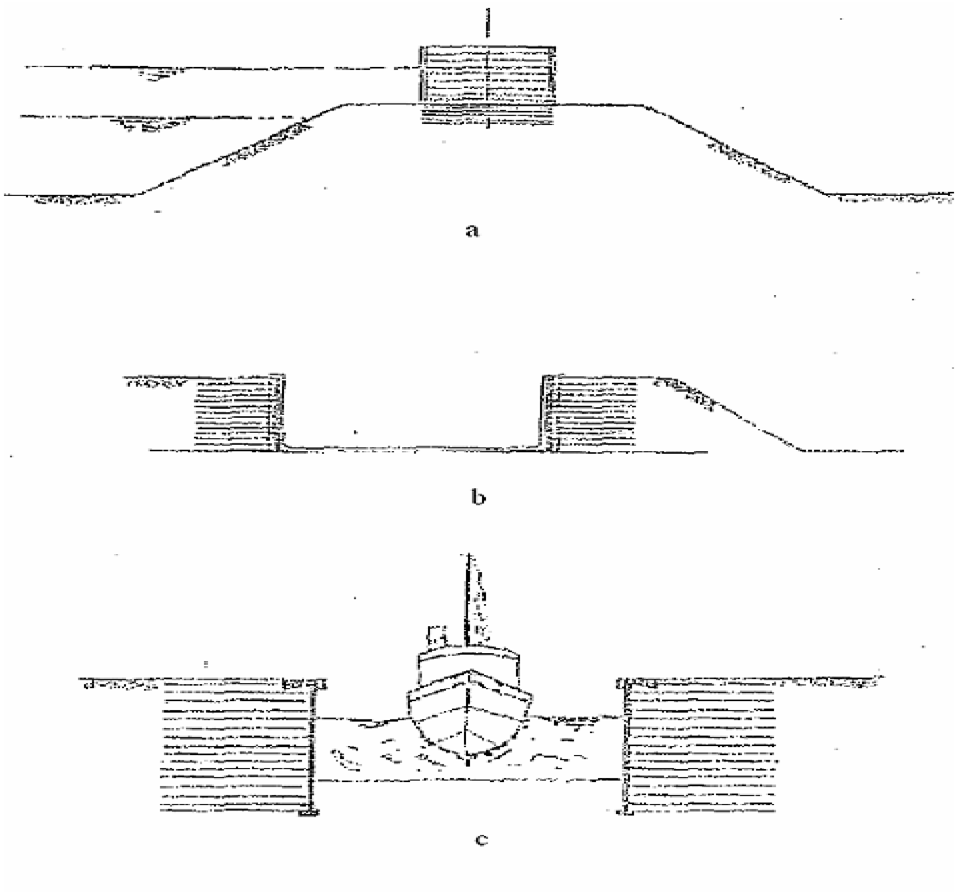


Figure 4.6. Special applications of reinforced earth retaining structures (Craig, 1987)

Earth reinforcement is presently used routinely for construction of retaining walls and abutment structures, for repair of slope failures, for retention of excavations and for stabilization of slopes in-situ (Tezcan and Buket, 1999).

4.3. Available Earth Reinforcing Systems

4.3.1. Websol System

The Websol system of frictional anchored walls comprises thin cladding units in precast concrete, composite plastic frictional anchors connected to the precast units at the face of the structure and passing around anchor bars at the rear, and earth fill. A structure is built up in alternate layers of fill and frictional anchors (East, 2003). Comprising units of Websol system are shown respectively in the Figure 4.7. below.

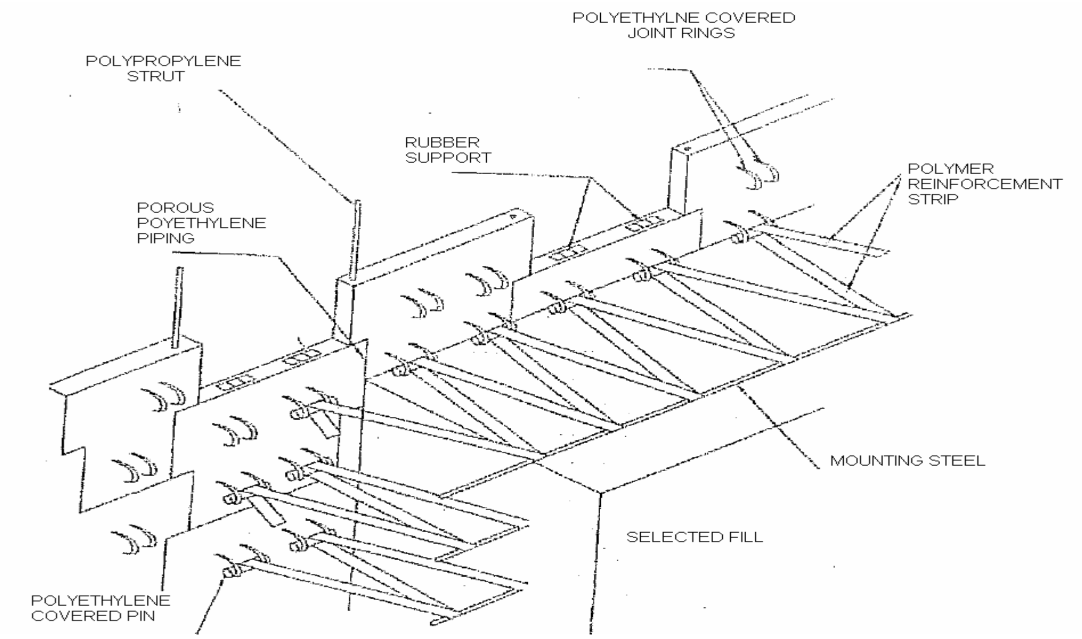


Figure 4.7. Components of websol system (Hansen and Lambe, 1990)

The average rate of construction of websol walls in normal conditions would be approximately 50 square meters per location per day. Thus, completion of walls in the height range five to ten meters will be achieved at the rate of five to ten meters length of wall per day. This will represent an acceleration of construction time by a factor of at least five times when compared with reinforced concrete construction. The rate of output may, where conditions permit, increase to 75-100 square meters per day (East, 2003).

4.3.2. Mechanically Stabilized Earth Walls (MSE)

The earth backfill behind a concrete wall facing can be made to support itself by layering reinforcement within the backfill. The soil itself becomes a self-supporting structure. If metal strips are used as reinforcement, the tensile stress in the soil transfer to the reinforcement through friction. If mesh is used, the transverse wires of the mesh resist the tensile stress in the soil. In either case, the soil itself withstands the compressive and shear stresses. The concrete facing prevents the face of the soil from raveling and provides an attractive facade (Lamberson 1987). Components and construction process of a typical MSE are represented in Figures 4.8. and 4.9. respectively.

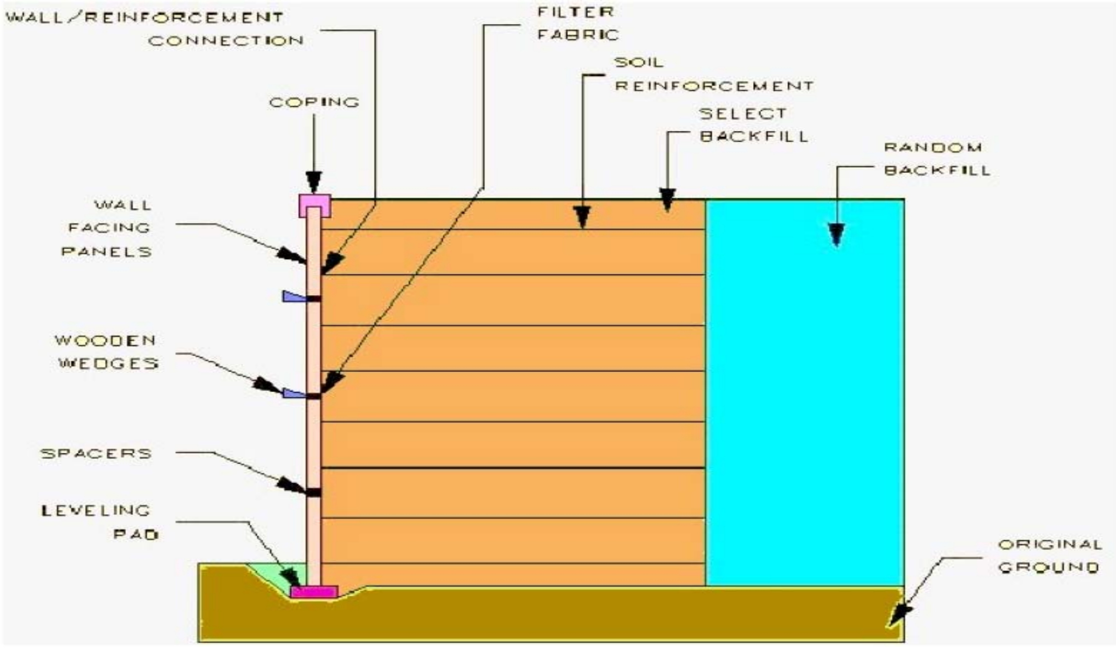


Figure 4.8. Components of MSE wall (Burgess, 1999)

There are many different mechanically stabilized earth walls available in the world. Each system has three main parts: the concrete facing, backfill and reinforcement. Individual systems vary mostly in the type of facing and reinforcement and the way these are connected.

The length of the soil reinforcement varies depending on wall geometry and loading, but generally it must be 70 to 100 percent of the wall height. This usually makes reinforced earth walls too expensive for earth cuts. The excavation behind such a wall would have to be at least as wide as the length of the reinforcement. For walls that support fills, however, reinforced earth wall systems are very cost competitive. According to the Federal Highway Administration they can cost 30 to 50 percent less than conventional cast-in-place walls and provide similar savings in construction time.

Figure 4.9. Installation of a mechanically stabilized earth retaining wall (Tensar., 2006)

4.3.3. VSL Retained Earth

VSL Retained Earth System as described by “Collin” (1986) employs bar-mats similar to that used in Mechanical Stabilized Embankments (MSE) and Georgia Stabilized Embankments (GASE). For VSL Retained Earth the reinforcement mats consist of either W11 or W20 steel bars placed in a rectangular grid with prescribed longitudinal and transversal spacing of 15 cm, and 60 cm, respectively. The width of each mesh may vary, having 4, 5, or 6 longitudinal bars depending on the overall requirements of the site. The facing elements for the three bar-mat systems vary from the hexagonal shape used for the VSL wall to the long narrow beams used in the Mechanical Stabilized Embankment. A VSL wall application at a bridge pier is shown in Figure 4.10.

The internal stability of a VSL Retained Earth Wall deals with the ability of the reinforced soil mass to act as a coherent unit. This is accomplished through a transfer of stress from the soil to the reinforcement. Two criteria must be checked to assure this stability. First, the reinforcement must be strong enough to withstand the stresses induced in it from the soil without rupturing. The reinforcement must also be able to withstand this force without pulling out of the soil (Lamberson, 1987).

Figure 4.10. VSL wall application at USA (Tensar, 2006)

5. METHODS OF CONSTRUCTION

Construction of reinforced soil structures must be of a form determined by the theory and in keeping with the assumed idealization and analysis. Considering the difference between the theoretical form of the structure and an economical prototype, the method of construction throughout the design process has great importance. As the speed of the construction is usually essential to achieve economy, the construction technique should be simple. The aspect of simple construction relevant to reinforced soil structures can be summarized as follows (U.S Department of Transportation, 1997).

- Use materials, which are readily available and easy to use,
- Construct as much as possible with plant at existing ground level to ease access and use natural crust and drainage,
- Design excavations with a level base,
- Use foundations with simple shapes and details,
- Form all surfaces horizontal or vertical,
- If required, anticipate re-use of formwork,
- Design structures to be stable at all stages of construction,
- Fix reinforcement and place soil in one plane at a time,
- Use medium size reinforcement, avoid both small section and heavy large elements, which can be difficult to transport and fix without lifting equipment.

5.1. Basic Construction Methods

Three basic construction techniques, which can accommodate vertical settlements and limited lateral movements within the soil mass are, Concertina Method, Telescope Method, and Sliding Method.

5.1.1. Concertina Method

The construction arrangement of the concertina method was developed by the Vidal (1966). Differential settlements and lateral movements within the soil mass are achieved by the front face of the structure concertinaing in a manner similar to action of a set of bellows.

Some of the largest modern reinforced soil structures have been built using this approach, and it is the form of construction frequently used with geotextiles and related reinforcing materials in both embankments and cuttings. A flexible hoop shaped steel facing unit has been used when the structure is reinforced with strip reinforcements, as have facing units formed aluminum and glass reinforced plastics, although the latter have had limited success due to their lack of robustness (Durukan, 1988).

Geotextiles and related materials often provide their own facing by employing wrap-around techniques. This is particularly useful for structures where distortion of the facing is acceptable. Structures built from fabrics and geogrids or constructed as temporary structures and using the concertina method of construction are particularly prone to distortion of the face. The degree of distortion can not be predicted. An accepted method of overcoming the problem is to cover the resulting structure either with soil or with some form of facing. An alternative is to provide a rolling block against which the compaction plant can act (Durukan, 1988).

5.1.2. Telescope Method

In the telescope method of construction developed by Vidal (1966), the deformations within the soil mass are accommodated by the facing panels closing up and moving forward an amount equivalent to the internal deformations. This is made possible by supporting the facing panels by the reinforcing elements and leaving a discrete horizontal gap between each facing panel, i.e. the facing panels hang from the reinforcing elements. The horizontal gap between each facing panel may be affected by the use of compressible gaskets (Hansen and Lambe, 1990).

Lack of a large enough gap between facing elements can result in crushing and spalling of the units as the fill deforms under the action of gravity and compaction forces. Depending upon the geometry of the structure, quality of the fill material, size of the facing panels and the degree of compaction achieved during construction, the closure between panels will vary. An estimate of the internal movements and distortion of the facing can be made from observations of prototype structures. Horizontal movements of the facing are made up of two components:

- Horizontal movements at the joints
- Tilt of the facing units

Tilt of the facing panels can be significant and may have a marked effect upon the facial appearance of the structure, although other elements of serviceability are unlikely to be effected by tilt of the facing (Helwany and McCallen, 2001).

To accommodate the forward tilt of the facing panels caused mainly by compaction of the fill, it is normal practice to incline the panels backwards between 1 in 20 and 1 in 40 (Helwany and McCallen, 2001).

5.1.3. Sliding Method

In this construction method, differential settlements and compaction within the fill forming a reinforced soil structure can be accommodated by permitting the reinforcing members to slide vertically relative to the facing. Slidable attachments can be provided by the use of grooves, slots, vertical poles, lugs or bolts. Also hybrid structures, using a combination of the telescope and sliding methods of construction have been used successfully with geogrid reinforcement. In this system the reinforcement, which passes through the facing, is attached by means of a slidable connection to king posts, whilst the facing operates in a telescope arrangement (Hansen and Lambe, 1990). The primary advantage is of economy and speed of construction.

In some special conditions there is no need to make allowances for consolidation of the fill, as the degree of movement is limited by the design. This can occur when the foundation beneath the structure is very competent (such as rock) and when the backfill material used is very high quality and well compacted. In these special circumstances the differential movements are very small and the extensibility inherent in polymeric reinforcement (Hansen and Lambe, 1990).

5.2. Damage and Corrosion

Care must be taken that facing elements and reinforcing members are not damaged during construction. Vehicles and tracked plant must not run on top of reinforcement. A depth of fill of 150 mm above the reinforcement is frequently specified before plant can be used (U.S Department of Transportation, 1997).

Reinforcement should be stored in a safe dry environment and non metallic should be stored away from ultra-violet light. With soil nailing and lateral earth support systems, the precautions normally adopted to cater for the corrosion problems of anchor systems relevant (U.S Department of Transportation, 1997).

5.3. Compaction

Compaction of the fill in a reinforced soil structure is desirable as it has a beneficial influence on behavior and increases the efficiency of the structure. Good compaction also reduces internal differential movements, while uniform compaction provides the most stable environmental conditions which are important for durability (U.S Department of Transportation, 1997).

Uniform compaction of fill is achieved by placing the fill in layers varying depth from 100 ~ 300 mm, and compacting the soil using suitable plant moving parallel to the facing or edge of the structure (U.S Department of Transportation, 1997).

6. BEHAVIOR PRINCIPLES OF REINFORCED EARTH RETAINING WALLS

6.1. Working Mechanism of Reinforced Earth System

The main technique of Reinforced Soil system is very well understood and described in detail by Vidal in his publications. A basic type of this mechanism is clarified in Figure 6.1. As shown in Figure 6.1., a single axial load over granular material causes lateral expansions on the compacted materials. Due to this expansion, lateral stresses become more than the half of vertical stresses. On the other hand, if the reinforcing members without any elongation capability are placed inside the soil, these reinforcements prevent lateral stresses with the help of friction between themselves and soil, and the system will act like; a lateral retaining force or load is acting on the member.

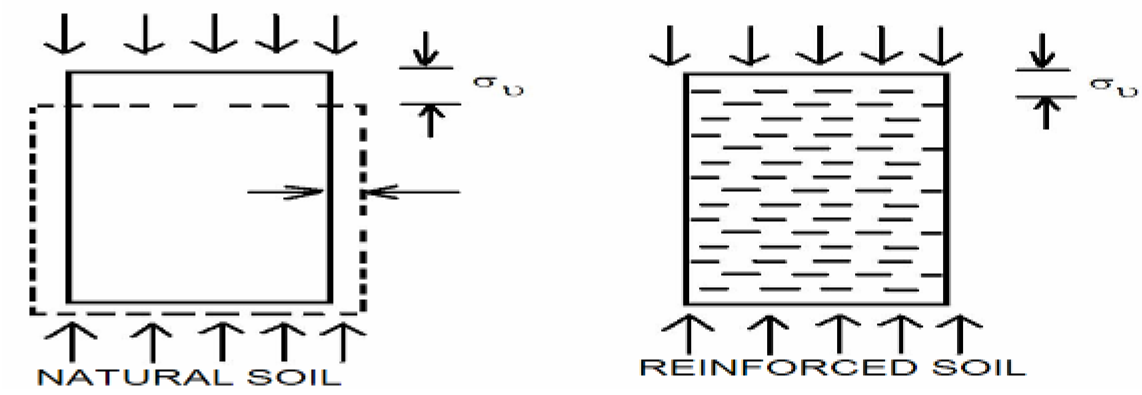


Figure 6.1. Working mechanism of teinforced earth structure (Das, 1987)

This lateral force is equal to passive earth pressure ($K_0 \sigma_V$). Each member of the soil is affected by a lateral pressure equal to K_0 σ_V value; because of this, as the vertical pressures increase the lateral retaining pressures or forces also increase in direct proportion. So for the any value of internal friction angle φ, strain circle is located under the failure envelope at every point. Regarding these facts, any kind of deterioration in balance can occur only regarding to loss of friction between reinforcement and soil or braking off of reinforcements (Smith and Unal, 1989).

There have been some theoretical researches oriented by different researchers for investigating the strength increase in cohesionless soils, and the obtained results are interpreted with 2 theories (Smith and Unal, 1989):

- Pseudo Cohesion Theory
- Equivalent Pressure Theory

6.1.1. Pseudo Cohesion Theory

Pseudo Cohesion concept is defined with respect to one reinforcement strip and two soil particles contacting to this strip, as shown in Figure 6.2. If the angles of contacting forces between reinforcement and particles (T_1, T_2) with the normal planes (α_1, α_2) are smaller than the soil-reinforcement friction angle (arctan f); the reinforcement acts like it is pulling the two particles to each other with a force equal to $dT = T_1 - T_2$. And the particles behave like cohesive particles as long as the friction resistance is not exceeded (Smith and Unal, 1989).

The results of triaxial pressure test on sand particles applied by Sclosser and Long about this subject is shown in Figure 6.3. As it can easily be seen from the Figure 6.3. after a definite value of pressure ($\sigma_3 > 50 - 100$ kN/m2), there observed a constant increment ($\Delta\sigma_1$) in vertical stress at breaking moment.

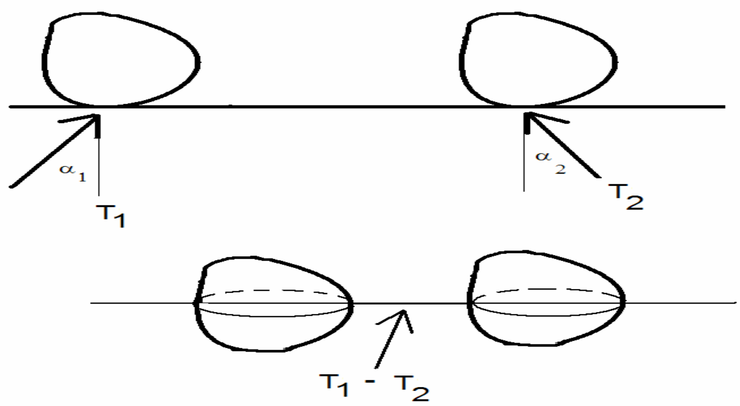


Figure 6.2. Pseudo-Cohesion formation at reinforced soil (Das, 1987)

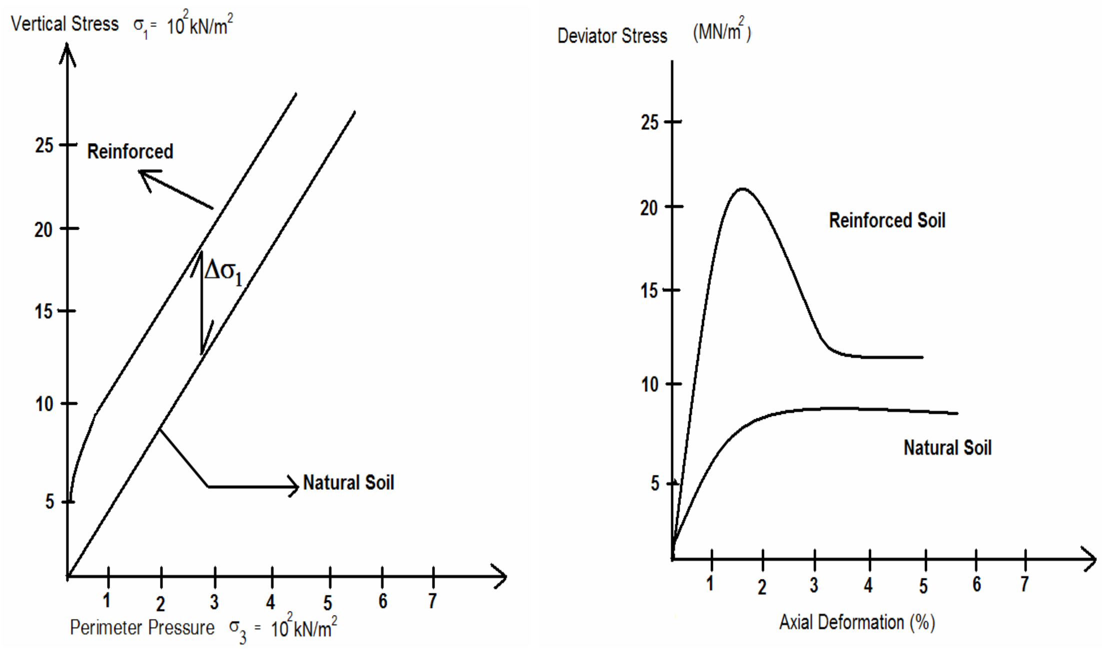


Figure 6.3. Triaxial compression test results for reinforced and un-reinforced sand (Das, 1987)

According to parallelism in the failure envelopes at the breaking point and due to no change in shear strength angles regarding to this parallelism in both reinforced and un-reinforced samples; the observed increment in strength at reinforced sand samples is explained by a visual anisotropic cohesion (C), like in Figure 6.3.

Sclosser ve Long described the failure envelope for reinforced soil as;

$$\sigma'_1 = Kp\sigma'_3 + \Delta\sigma'_1 \tag{6.1}$$

For a C'- φ type soil, the Rankine-Bell equilibrium;

$$\sigma'_1 = Kp\sigma'_3 + 2(Kp)^{1/2} C' \tag{6.2}$$

If we compare these two equations, the pseudo-cohesion term is obtained as;

$$c' = \Delta\sigma'_1 / 2\ (Kp)^{1/2} \tag{6.3}$$

In the equations above, K_p is passive earth pressure coefficient and defined as;

$$K_p = \tan^2(45 + \varphi/2) \tag{6.4}$$

And the same researchers calculated the pseudo-cohesion value for a symmetric axial loaded cylindrical sample with respect to force equilibrium analysis as;

$$C' = T\,(K_p)^{1/2} / 2\Delta h \tag{6.5}$$

Where;

T: Tensile strength of reinforcement.

Δh: Vertical reinforcement spacing

From the above explanation which is also know as LCPC (Laboratorei Central des Ponts et Chausses) cohesion theory, it can be said that the reinforced soil samples will collapse at a higher σ'_{1r} vertical stress value under σ'_3 perimeter pressure than un-reinforced soil samples and pass from ($\sigma_{3,}\ \sigma'_1$) value of Mohr circle.

In this case, the failure envelope is a line which intersects the ζ axis at C' value with an angle of φ, with horizontal plane in ζ- σ coordinate system, as seen in Figure 6.6. As a result of a series of researches which were carried out at New South Wales Technology Institute, existence of pseudo cohesion at cohesionless soils is defended and the theory for reinforcement pullout and reinforcement damaging cases which is also known as NSW cohesion theory was developed (Smith and Unal, 1989).

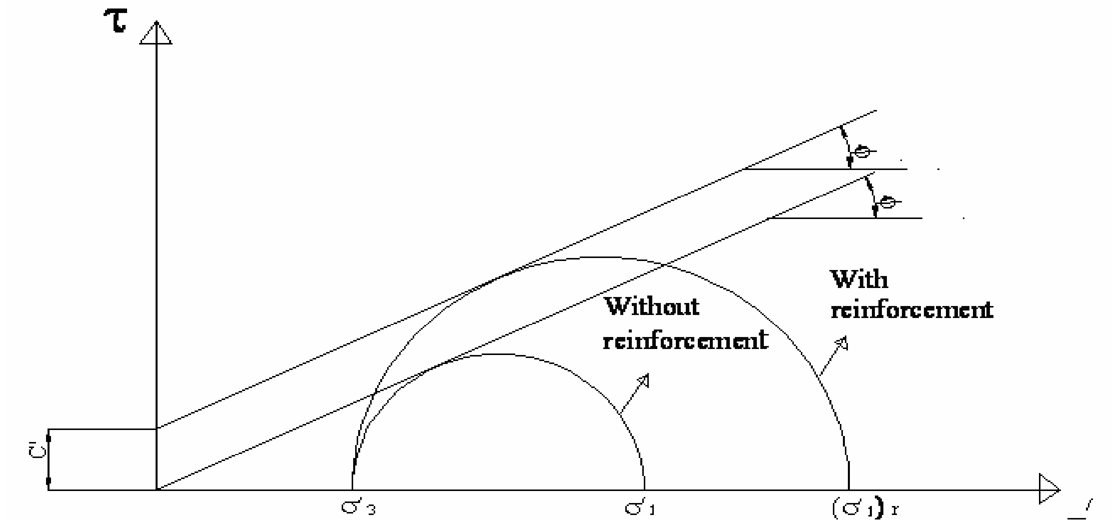


Figure 6.4. Failure envelopes due to LCPC cohesion theory (Smith and Unal, 1989)

According to this theory, in the collapse cases due to reinforcement refraction in reinforced cohesionless soils; the lateral expansion of soil causes a “σ_r” constant pre stress value which is also statically equal to the value of the friction force between soil and reinforcement. The maximum value of this pre stress is defined by the tensile strength of the reinforcement material. The increment of shear strength under a constant pre-stress is obtained by a “C'_r” Pseudo-Cohesion value, as shown in Figure 6.5.

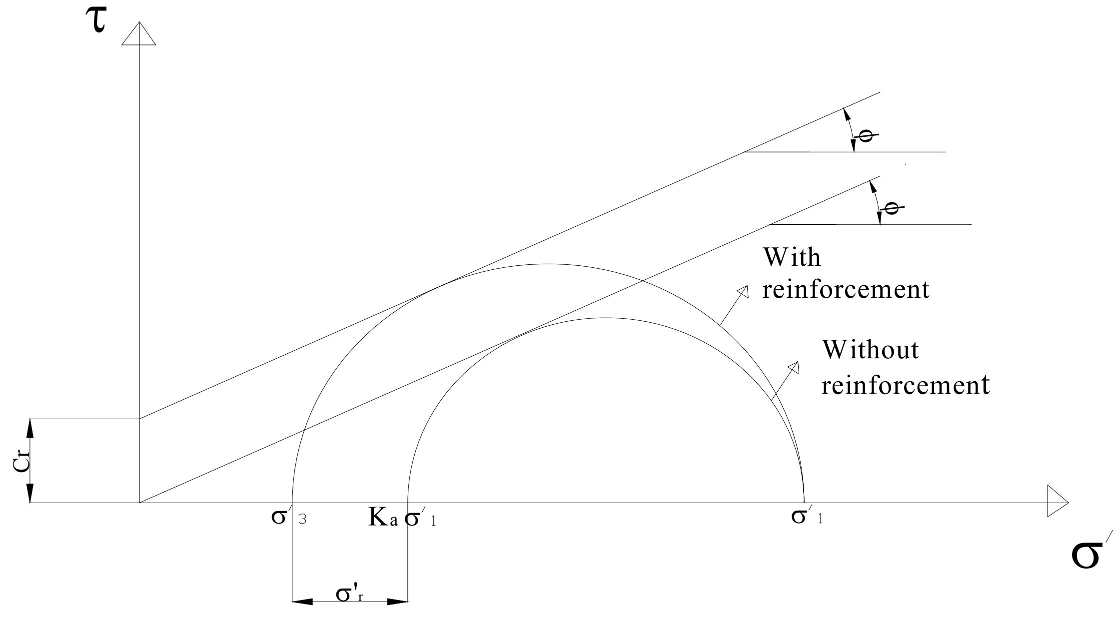


Figure 6.5. Collapse under constant σ_r for reinforced cohesionless soils (Smith and Unal, 1989)

Huastmann defined the Principal Stress Values that will be formed in reinforced soil samples as;

$$\sigma'_3 + \sigma'_r = K_a \sigma'_1 \quad (6.7)$$

$$\sigma'_1 = (\sigma'_3 + \sigma'_r) K_p \quad (6.8)$$

In the Equation (6.7) and Equation (6.8) Pseudo cohesion value is obtained as;

$$C'_r = \sigma'_r (K_p)^{1/2} / 2 \quad (6.9)$$

In considering the strip reinforcing member, constant pre stress value can be calculated as;

$$\sigma'_r = \sigma A/BH \quad (6.10)$$

Where;

K_a, K_p : Active and Passive earth pressure coefficient
A : Strip reinforcement cross section area
σ : Ultimate tension strength of reinforcement
H,B : Length and width of sample

If the calculated σ'_r value in Equation (6.9), is replaced in Equation (6.10), the pseudo cohesion value becomes;

$$C'_r = \sigma A (K_p)^{1/2} / 2BH \quad (6.11)$$

And this is equal to Equation (6.1) which is offered by Sclosser and Long. However, Sclosser and Long claimed that the σ'_1 of the reinforcements has an increasing affect where, Hausmann claimed σ'_3 has a decreasing effect where σ'_1 is constant. There are some incorrect parts of this approach since the soil collapses before rupture of the reinforcements in the cases that lateral pressures are smaller than active pressures (Smith and Unal, 1989).

In the collapse cases due to pull-out of the reinforcement from soil, Hausmann defined a pre stress equation basing on assuming that the friction stresses occurred along reinforcement is proportional to vertical stresses.

$$\sigma'_r = f\,\sigma'_1 \tag{6.12}$$

f: soil-reinforcement friction coefficient

In the cases of collapse cases due to pull out of the reinforcement, reinforcement effect increases the shear strength angle and shown in Figure 6.6. In this case, the reinforced earth shear strength angle (σ'_r) can be calculated as;

$$\text{Sin}\ \varphi'_r = (K_a\text{-}f\text{-}1) / (f\text{-}K_a\text{-}1) \tag{6.13}$$

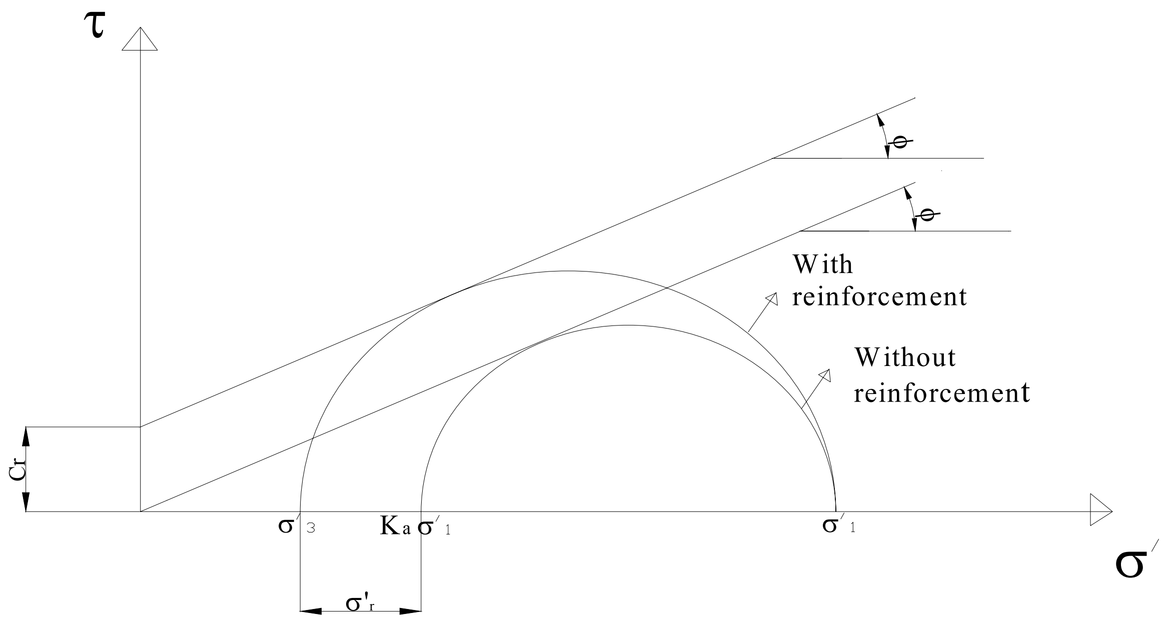


Figure 6.6. Collapse under variable σ_r for reinforced cohesionless soils (Smith and Unal, 1989)

6.1.2. Chapius Equivalent Pressure Equation

In pseudo-cohesion theory, it is assumed that, in reinforced sand samples, laterally applied vertical and horizontal stresses (σ'_1, σ'_3) are distributed uniformly along reinforcement length and principal. However due to friction and shear stresses along reinforcements, σ'_1, σ'_3 are not principal and uniform.

After a series of empirical studies, Chapius stated that the (σ'_{3r}) small prime stress will be bigger than the applied perimeter pressure (σ'_3). This increase in lateral stresses for strip reinforced soil sample can be calculated approximately by the equation;

$$\sigma'_3 = A\ \sigma\ /BH = T/\Delta h \tag{6.14}$$

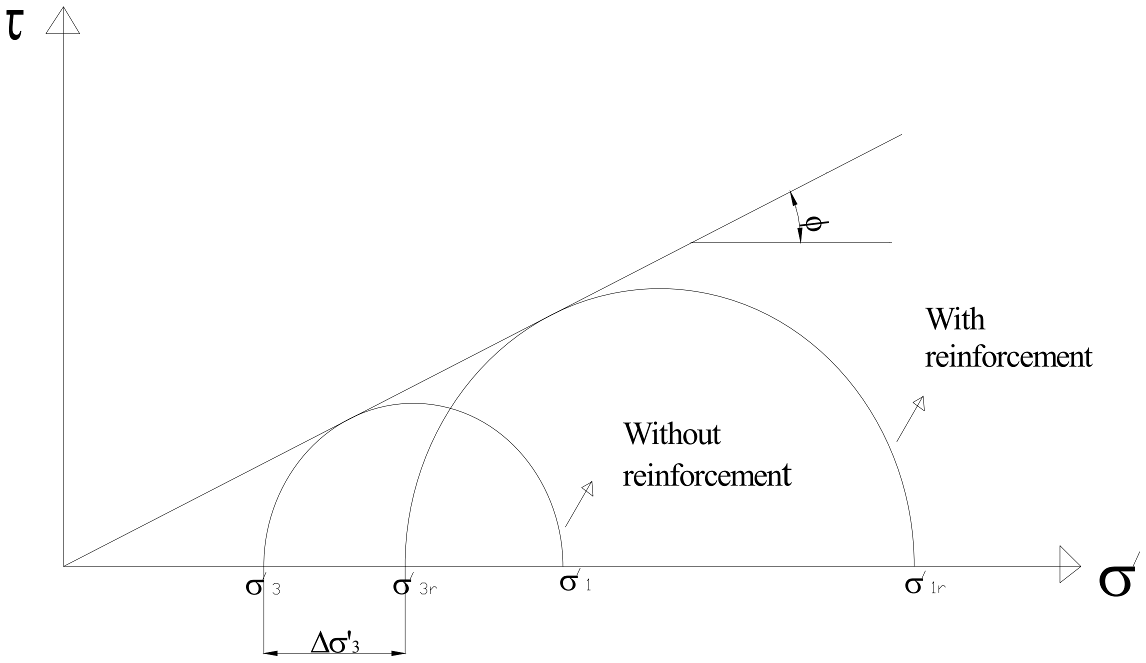


Figure 6.7. Failure envelope due to equivalent pressure theory (Smith and Unal, 1989)

6.1.3. Yang Equivalent Pressure Equation

A more comprehensive and analytic study has been carried out by Yang about this subject. Yang claimed that the vertical effective stress increase observed at failure envelope for reinforced sand samples is caused by equivalent perimeter pressure (σ'_3) and for any applied perimeter pressure (σ'_3), the vertical effective stress at failure moment; equation is advised (Ingoldi,1982; Gray,1986).

$$\sigma'_r = (\sigma'_3 + \Delta\, \sigma'_3)K_p \tag{6.15}$$

As a result, the equivalent pressure can be calculated with Equation (6.16):

$$\Delta\, \sigma_3 = Ka\, \sigma'_{1r} - \sigma'_3 \tag{6.16}$$

6.1.4. Gray and Al-Refeai Equivalent Pressure Equation

After performing triaxial pressure experiments on reinforced sand samples, Gray and Al-Refeai defined the equivalent pressure with Equation (6.17) as;

$$\Delta\sigma'_3 = \sigma'_3\, \Delta\sigma'_1 / \sigma'_1 \tag{6.17}$$

Here;

$\Delta\sigma'_1$: Increase of prime (Major) stress at failure moment ($\sigma'_{1r} - \sigma'_1$)
σ'_1 : The bigger major stress at failure moment for un-reinforced soil
σ'_3 : Applied pressure

And according to our previous knowledge, we can write the equation below for expressing the relation between pseudo-cohesion and equivalent perimeter pressure ;

$$\Delta\sigma'_3 = 2C'/(Kp)^{1/2} \tag{6.18}$$

6.2. Collapse Cases in Reinforced Earth Retaining Walls

Three different collapse cases can be mentioned for reinforced earth retaining walls.

- Exterior stability collapses
- Interior stability collapses
- Facing member collapses

6.2.1. Exterior stability Collapses

This type of collapse is mostly related with dimensions of reinforced earth retaining structure. Like gravity and semi gravity walls, the collapse due to exterior stability can occur in 4 different ways in reinforced earth retaining walls as shown in Figure 6.8. (U.S. Department of Transportation, 2001).

They are;

- Sliding
- Overturning collapse
- Bearing capacity collapse
- Total collapse

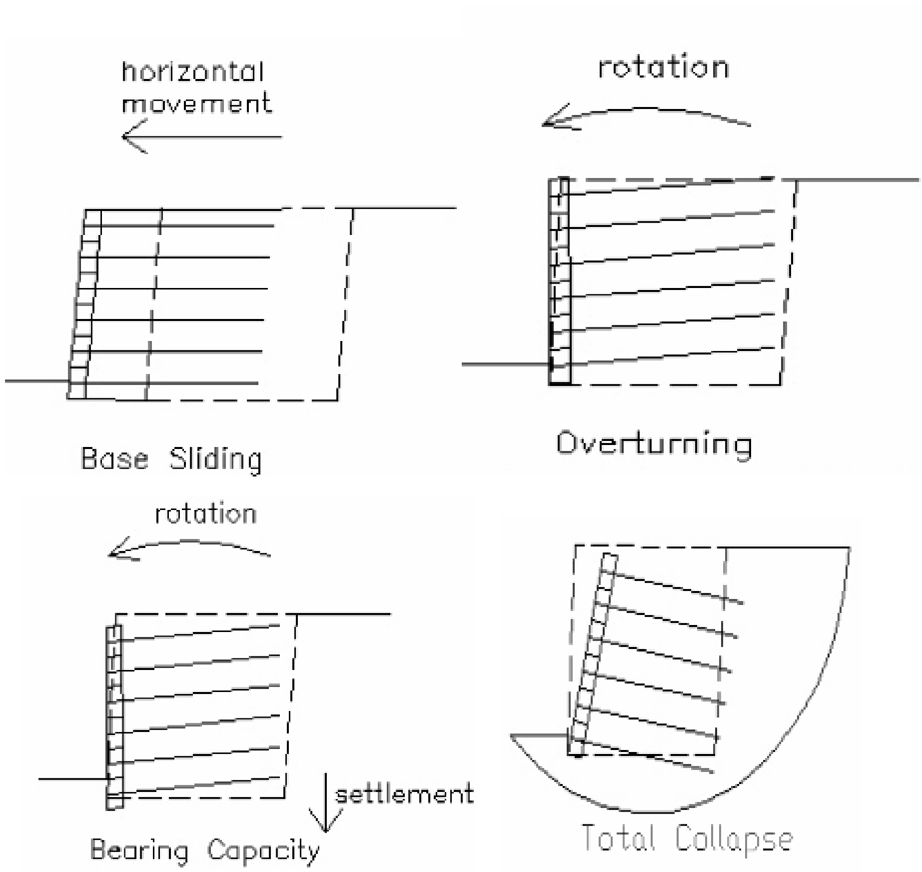


Figure 6.8. Possible exterior stability collapses for reinforced earth structures (Das, 1987)

6.2.2. Interior Stability Collapses

The presence of interior stability at reinforced earth retaining walls can only be possible if the reinforcement can bear the tension, moment and shear forces conducted by soil without rupture, as shown in Figure 6.9-a. Additionally, reinforcements should have joints which will not pull-out from soil, as in figure 6.9-b. On the other hand, in the cases of using geotextile reinforcements, it is possible to have interior sliding at any reinforcement surface if the friction between reinforcement and soil is not sufficient, as in 6.9-c (Burgess, 1999).

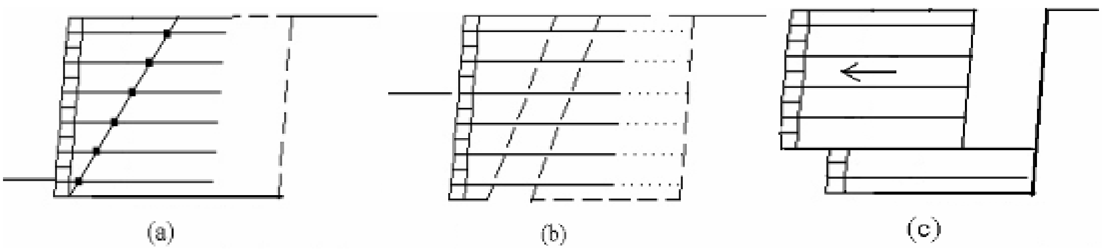


Figure 6.9. Possible internal stability collapses for reinforced earth structures (Das, 1987)

6.2.3. Facing Member Collapses

It is possible to observe facing member collapses if the facing members are not designed properly or the joints between reinforcement and facing member is not adequate Most of these collapses are investigated under the heading of local stability collapses. Some of the local stability analyses are shown in Figure 6.10. below (Burgess, 1999).

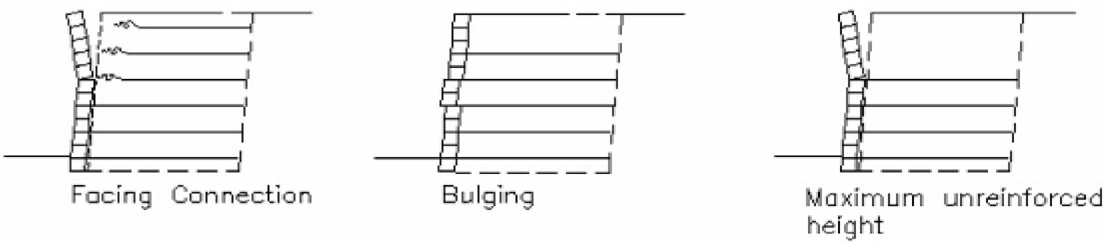


Figure 6.10. Possible facing member collapses for reinforced earth structures (Das, 1987).

7. DESIGN AND ANALYSIS METHODS

7.1. Design Features of the Reinforced Earth System

Reinforced Earth Retaining Wall System has been made up of three components mainly. These are;

- Fill Material,
- Reinforcement,
- Facing Member.

For a perfect engineering design, these three components mentioned above should be combined and a serial numbers of stability and safety researches should be realized. This research can be divided into 2 different categories as below:

- Exterior Stability Analysis
- Interior Stability Analysis

In the real design, it has been assumed that the structure behaves as a unique mass. Collapse in exterior stability should not been observed by occurrence of one or some of the mentioned mechanisms .Sliding of the reinforced earth mass on the surface or at any level, overturning of the structure, collapse due to bearing capacity or loosing the strength of foundation surface due to different deformation values or shrinking of the foundation soil, (Tezcan and Buket, 1999).

7.2. Analysis Methods of Reinforced Earth System

For the last ten years, different methods and researches have been performed and it has been reached a general opinion that, a complete design approach includes below mentioned analyses:

- Working Stress Analysis
- Deformation Evaluation
- Limit Equilibrium Analysis

7.2.1. Working Stress Analysis

A working stress analysis includes the following items;

- Selection of the reinforcement placement and soil properties and stress control in the stable soil mass as in harmony with the results.
- Local stability evaluation and estimation of the progressive collapse in each reinforcement level.

7.2.2. Deformation Evaluation

A deformation response analysis should take into account the expected performance of the lateral and vertical movement of the structure. In addition to this, it should be able to be inspected the effect and change of the reinforcement over the structure performance. Lateral deformation analysis is the most difficult and not certain performance analysis. It has been realized approximately in a lot of circumstance and it has been assumed that common safety coefficients for interior and exterior stability hold deformations in tolerable amount. Lateral deformation analysis with traditional settlement calculations has been obtained with different settlement calculations as direct to wall surface and from the wall surface to the end of reinforced soil mass diagonally. The results can affect the selection of surface element and facing member joint (U.S. Department of Transportation,2001).

7.2.3. Limit Equilibrium Analysis

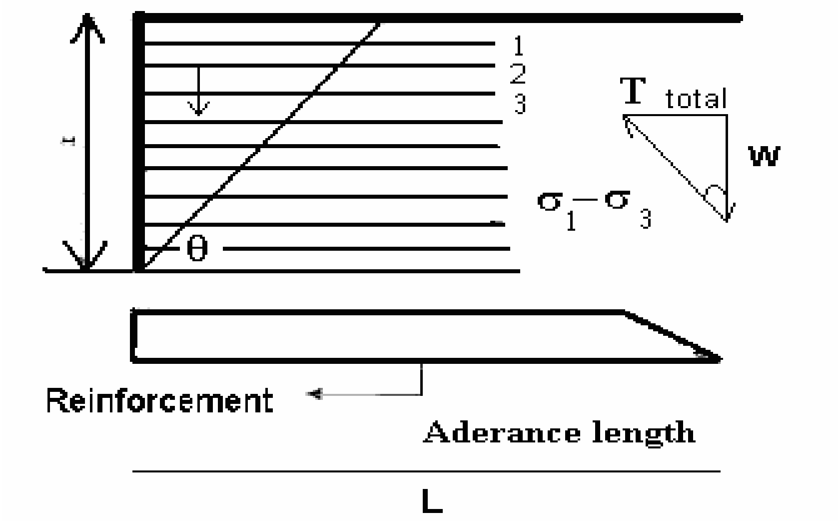
Lots of approaches have been developed on analysis and design of reinforced earth retaining walls. All the methods that can be accepted as usual design methods are based on the limit equilibrium analysis that has been applied on reinforced earth structures under collapse circumstance. In this analysis, it has been searched of potential collapse planes inside the soil mass. Almost all types of sliding surfaces have been able to be analyzed (dagger, circular, logarithmic spiral shapes) (Bedal, 1997).

In this method, it has been taking into account that the situation is generally as rupture and pull-out of the reinforced earth structures in its interior stabilization. Other forces that have been taking into account are; vertical soil force, horizontal soil force, strength in reinforcement and resistance of the reinforcement pull-out beside the potential collapse surface. There are lots of studies published after Vidal's reinforced earth theory about limit equilibrium analysis. The oldest two analysis methods have been advised by Schlosser and Vidal (1969). One of these methods are based on Coulomb's theory and the others are based on Rankine's theory (Bedal, 1997).

In the first method, there has been Ø angle behind the pull-out dagger surface as seen in Figure 7.1. Total tension strength in the reinforcement by the dagger equilibrium has been defined in terms of Ø angle as follows:

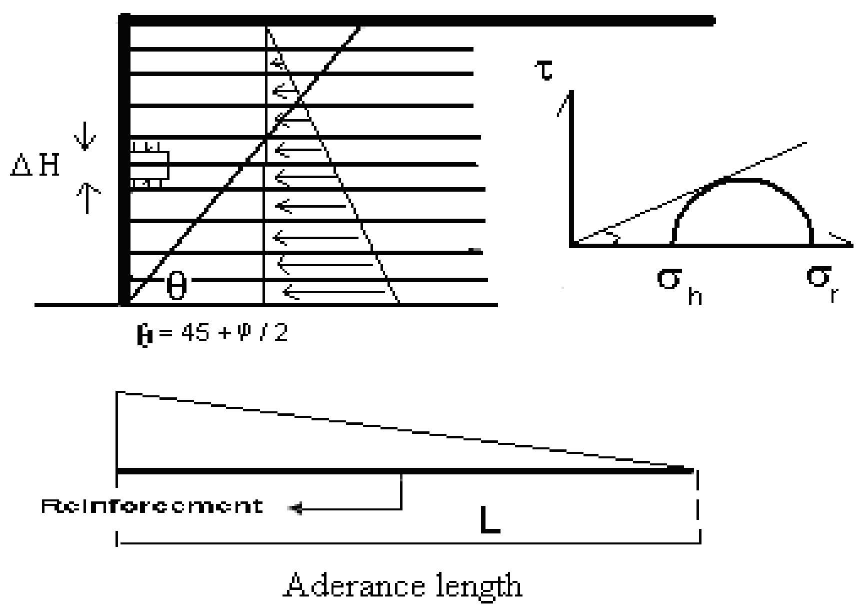
$$T = 0.5\, \gamma\, H^2 \cot\theta \tan(\theta - \varphi) \tag{7.1}$$

Equation 7.1. shows that maximum T force occurs at $\theta = 45 + \varphi/2$ angle.



Distribution of tension stress along the reinforcement

a) Method as to Coulomb Analysis (Das, 1987)



Distribution of tension stress along the reinforcement

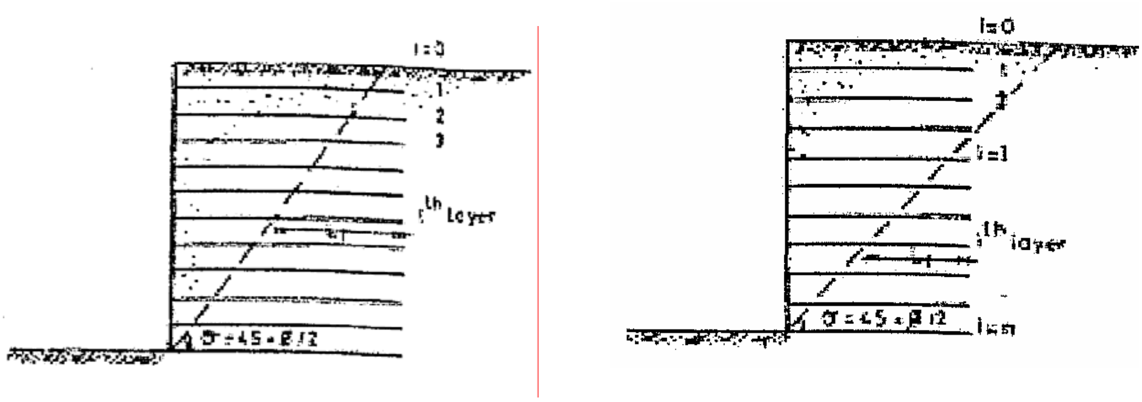
b) Method as to Rankine Analysis (Das, 1987)

Figure 7.1. Limit equilibrium design methods on reinforced earth structures

Stress distribution on reinforcement is assumed to be constant except the zone called as; adherence length at the reinforcement end, as seen in Figure 7.2. (Kramer, 2003).

In the second method, a representative critical soil member that has been affected by horizontal and vertical fundamental stresses had been taken into account as seen in Figure 7.2. Tension stress at any reinforcement at any depth is assumed to be equal to horizontal pressure that has been applied to the wall area in the same depth (Kramer, 2003).

Tension strength in reinforcement is calculated by using one of these methods and checks are being done according to two collapse conditions. Maximum tension in reinforcement should be smaller than the allowable tension stress and pull-out resistance in effective length should be adequate as seen in Figure 7.2.



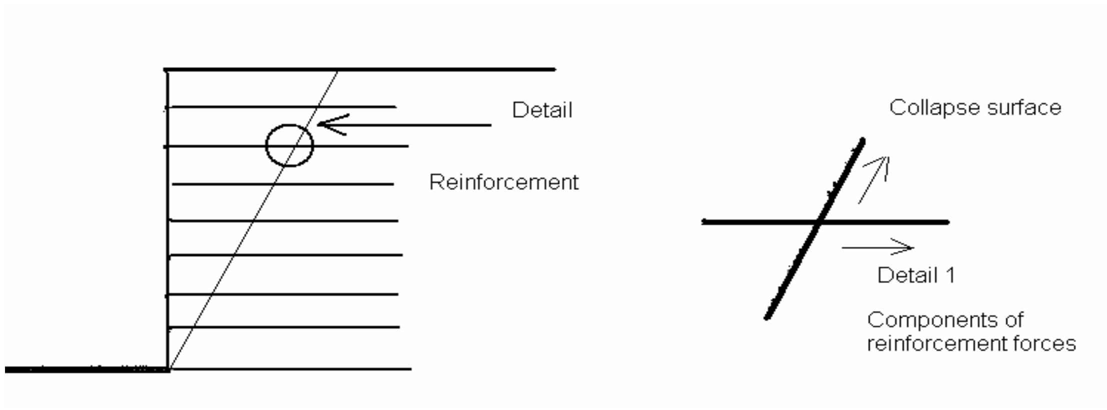
a) Collapse plane intersects all reinforcements b)Collapse plane doesn't intersects all reinforcements

Figure 7.2. Effective reinforcement lengths (Kramer, 2003)

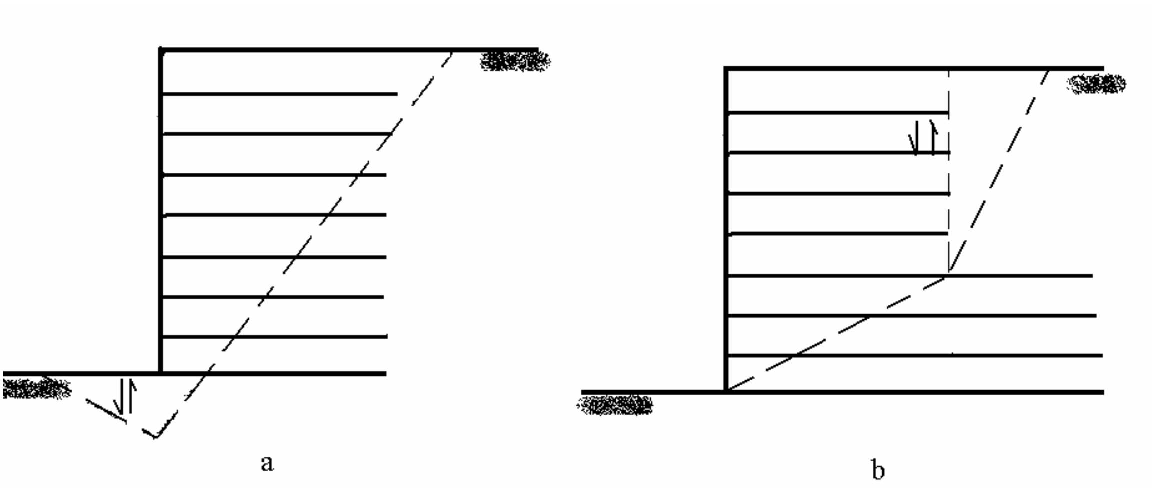
Lee (1973) had observed that Coulomb analysis reached better collapse prediction on reinforcement earth models that he used both Coulomb and Rankine analysis. Rankine analysis is more conservative and shows collapse values where Coulomb analysis is stable. Juran and Schlosser (1978) stated that they obtained exaggerated estimations on reinforcement stresses by using Rankine analysis.

Two types of limit analyses have been developed. First one is based on laboratory methods and collapse mechanism observed on structures exactly. Local equilibrium at any level where reinforcement exists and restricted by collapse surface should be taken into account. It is assumed that all sliding resistance occurs along the collapse plane. Limit analysis of local equilibrium on active zone shows the maximum tension stress on each reinforcement. This approach has been developed by Juran (1977) and arranged by Schlosser and Segrestin (1979). Many summaries of Limit Equilibrium Analysis have been given by Fahim (1983), Loke (1989) and Ho (1993).

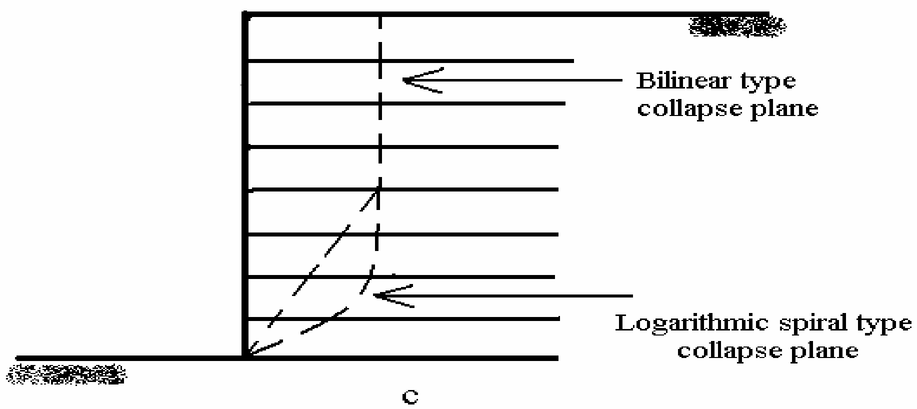
Second type of limit analysis is similar to classical slope stability. Reachable pull-out, sliding and tension resistances are needed to be taken into account in case of exceeding the potential sliding surface. In this method, some hypotheses are proposed according to collapse mechanism and collapse surface type (plate, dagger, circular, non circular, logarithmic or spiral) (Kramer, 2003).



a. Plane type collapse surface



b. Two pieces dagger type collapse surface



c. Logarithmic Spiral Type Collapse Surface

Figure 7.3. Collapse surfaces used in limit equilibrium analysis (Kramer, 2003)

Main differences among various limit equilibrium analyses are location, shape of the collapse plane and magnitude of the reinforcement strength in the analysis. In addition to that, there are different identifications about shear stress on the soil, tension and pull-out resistance of the reinforcement (Kramer, 2003).

As observed in experimental studies, in contrary with common expectations, maximum tension stress in reinforcement is not on the surface as in lots of observations, but it occurs beneath the surface and effective reinforcement length depends on potential collapse surface. The most important restrictive characteristic of limit equilibrium design method is the necessity of maintaining deformation frequency between reinforcement and the soil. Additionally, in these methods it can not to be taken into account of ground swelling and the extension on the structural stability of the reinforcement.

7.3. Exterior Stability Analysis in Reinforced Earth Structures

In reinforced earth retaining walls, there are four possible exterior stability mechanisms as in classical gravity and semi gravity retaining walls (sliding, overturning, bearing capacity and total collapse). Safety factors used in exterior stability are smaller than the RC and gravity retaining walls because of the flexibility of reinforced earth structures and sufficient site performance. In addition to that, flexibility of reinforced earth structures prevents the probability of overturning of structure due to collapse at a considerable amount. Exterior stability calculation steps that should be followed in reinforced earth retaining walls are shown in Table 7.1.

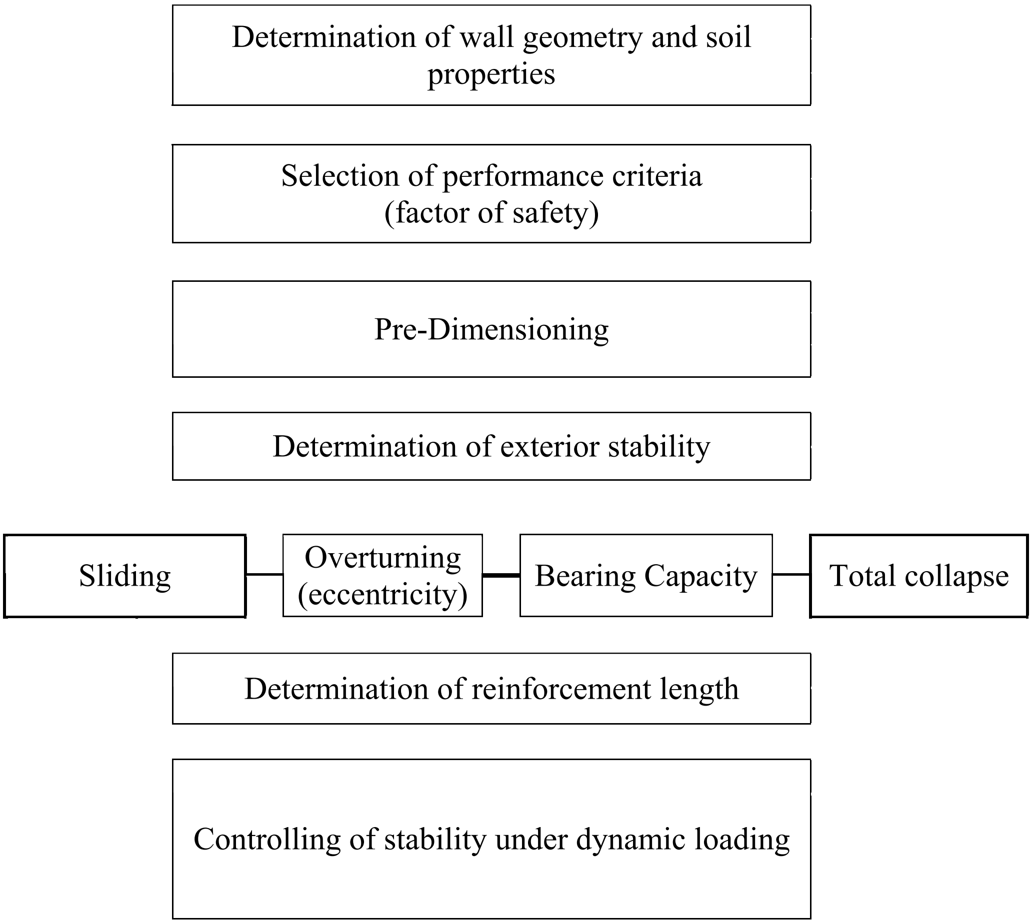


Table 7.1 Exterior stability calculation steps in reinforced earth structures

7.3.1. Determination of Wall Geometry and Soil Properties

Items mentioned below should be defined and clarified by the designer:

- Height of the wall and its slope
- Soil surcharge load, dead loads and live loads
- Dynamics loads
- Engineering properties of foundation Soil (γ,c,φ)
- Determination of engineering properties of selected fill material (γ,c,φ)
- Determination of engineering properties of Backfill Material (γ,c,φ)
- Determination of groundwater condition

7.3.2. Selection of Performance Criteria

Performance criteria that should be defined in exterior stability are as follows:

- Numbers of exterior stability safety factor (sliding, bearing capacity, over turning)
- Number of general safety factor (Total collapse)
- Maximum differential settlement
- Maximum lateral shifting
- Number of safety factor in case of dynamic loading
- Lifetime of the design

7.3.2.1. Factor of Safety. According to Specification of American Highways AASHTO (published in 1996), minimum factor of safety values that should be used in design of reinforced earth structures on exterior stability are shown in Table 7.2. below.

Table 7.2 Exterior Stability F.O.S. values (FHWA., 1996)

Feature of Exterior Stability	**Factor of Safety**
Sliding	> 1.5
Base eccentricity	> L/6 (soil), L/4 (rock)
Bearing capacity	>2.5
Total collapse	> 1.3
Stability for dynamic loading condition	> 75% of static safety factor

7.3.2.2. Horizontal and Vertical Settlement Occurred in Structure. There is no method that has been approved about estimation of horizontal settlement of reinforced earth structures while most of it happens during the construction of the structure (Kramer, 2003).

Horizontal settlement of reinforced earth surface during construction can be estimated roughly as to the ratio of reinforcement length –wall height coefficient (L/H) and reinforcement stretchiness (FHWA, RD 89-043). As shown in Figure 7.4., as the 0.5 theoretical limits of L/H increases to 0.7, settlement ratio is going to decrease 50 per cent. Additionally, in Figure 7.4. it can be seen that structural horizontal settlement assumed in reinforced earth structure constructed with polymer (stretched reinforcement) is three times higher than the reinforced earth structure constructed with metal striped reinforcements (not stretched reinforcement).

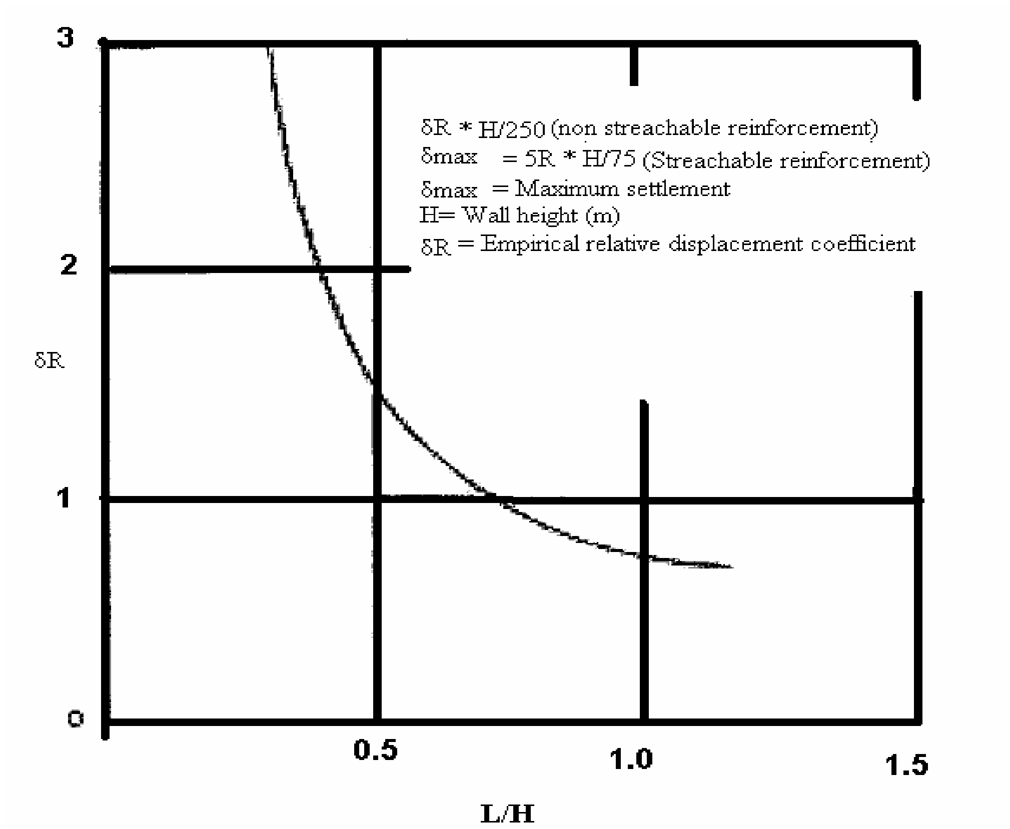


Figure 7.4. Empirical curve that shows possible horizontal settlements during construction of reinforced earth structures (FHWA, RD 89-043)

<u>7.3.2.3.Allowable Settlement Values in Precast Surface Panels.</u> Reinforced earth structures have features of allowable deformation values along the wall and in the direction of front perpendicular side. In spite of this, bad foundation conditions sometimes obstruct application of this feature. Usage of sufficient joint and/or slip joints in places of large

differential settlements occur (bigger than 1/100) will prevent cracking of panels. This situation may effect the selection of surface element and its design (U.S Department of Transportation, 2001).

Square panels which have the same surface area with long rectangular panels can better tolerate differential settlements. The minimum joint widths and allowable differential settlements to be used in panels of surface area with less than 4,5m^2 are shown in Table 7.3.

Table 7.3. Relationship between connection point and differential limit settlement which has been used in reinforced earth structures precast panels (FHWA RD 89-043).

Joint Width	**Limit Differential Settlement**
20 mm	1/100
13 mm	1/200
6 mm	1/300

Limit of differential settlement is in a range of 1/500 in reinforced earth structures where high panels are used entirely. Although the limit settlement in reinforced earth structures where welded wire mesh has been used is 1/50, this limit is 1/200 in structures where mono-block surface members are used (U.S Department of Transportation, 2001).

7.3.2.4. Design Life of Reinforced Earth Structures. Service lifetime of reinforced earth retaining structures should be determined as taking into consideration of all other long-term harmful influences of environmental factors that effects leaking, metallic corruption and establishing of reinforced earth structure. Permanent retaining should be designed up to 75 years of lifetime for many applications. For temporary retaining structures generally, service lifetime is 36 months or less than that. Design of structures with higher safety levels and/or with long-term economical lifetime (more than 100 years) takes places where; much damage happens in case of demolitions of structures, like bridge piers, housings and critical office buildings (U.S Department of Transportation, 2001).

7.3.3. Preliminary Design of Reinforced Earth Retaining Walls

Dimensioning of the structure should be started with determining of the thickness of the fill layer (toe fill) since the reinforced earth construction advances from downward to upward and this case will be active from start to finish of the construction. Minimum fill depth should be clarified in case of absence of rock at foundation level in order to obtain necessary bearing capacity and to take into account factors like freezing, shrinking, swelling clays and earthquake. Minimum fill depth is required to be 0,5 m. for a soil of possible shrinking, swelling and freezing potential and in case of taking into account the seismic activity and general stability. Minimum width is 1,2m. Minimum fill heights advised by American Highways for special conditions are shown below in Table 7.4. (U.S Department of Transportation, 2001).

Table 7.4. Minimum toe fill heights (U.S Department of Transportation, 2001)

Slope of Wall Front Face	**Minimum Toe Fill Heights**
Steep Walls	**H/20**
Steep Bridge Piers	**H/10**
1/3 slopes	**H/10**
1/2 slopes	**H/7**
2/3 slopes	**H/5**

Toe fill creates a passive resistance against lateral effect of the wall on toe area but this resistance is neglected due to possible excavation in front of the wall at a later time, since it is assumed that fill will not stay in its place all the time.

Initial value for reinforcement length is advised to be taken as L=0,7H or bigger than 2,5 m. Required reinforcement length is increased in case of observing surcharge and sloped fill. Minimum reinforcement length can be used as 2 m. in spite of AASHTO advises this value as 2,4 m. for walls with heights less than 3 m. In all cases it should be taken into account that length of reinforcement should provide the total collapse stability.

7.3.4. Preliminary Design Criteria as to French Ministry of Transportation

Various geometric data that French Ministry of Transportation uses in criteria can be seen below on Figure 7.5. Here H1 represents the total surface length of the reinforced earth structure, H represents mechanical wall height that will be used in calculations, D represents fill thickness and L represents the length of reinforcement (French Ministry of Transportation, 1980).

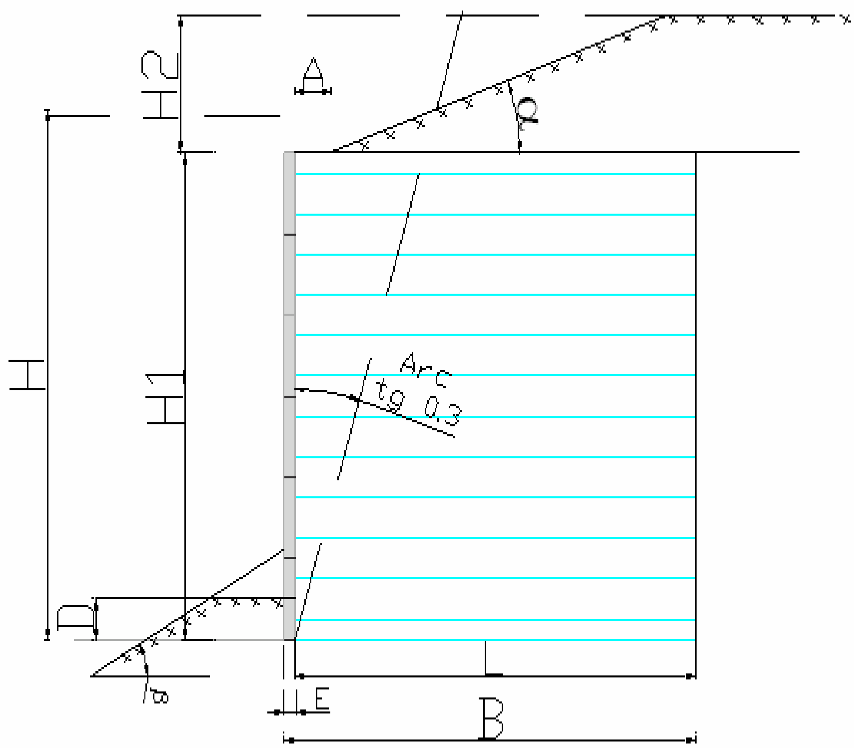


Figure 7.5. Geometric Definitions

French Ministry of Transportation has instructed to use the “D” fill in reinforced earth structures for the reasons mentioned below:

- To prevent possible local settlements near to surface
- To prevent punching of foundation soil
- To prevent piping

Fill thickness (D) depends on freezing depth and piping risks where water concentration exists. But in all cases minimum fill thickness (D_{min}) should not be less than 0, 40 m.

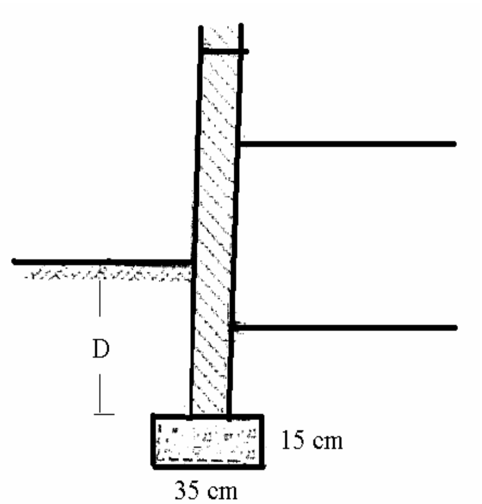
In order to prevent stress increase under surface members and not to exceed the bearing capacity on the foundation soil, minimum fill thickness (D_{min}) is shown as a function of reference stress and wall slope (β) in Table 7.5. (French Ministry of Transportation, 1980).

Table 7.5. Minimum fill thickness values due to reference tension and wall slopes (French Ministry of Transportation, 1980)

Wall slope	**D_{min}/q_{ref}**
Steep walls (β=0°)	1.5*10-3
β=18° (cotg β =3/1)	3*10-3
β=27° (cotg β =2/l)	4.5*10-3
β=34° (cotg β =3/2)	5.5*10-3

French Ministry of Transportation advises that minimum fill thickness as mechanical height (H), should be used as the same values as in American Highways shown in Table 7.3. (French Ministry of Transportation, 1980) in addition to Table 7.4.

Panels that are used at surface, settle on the under buried concrete piers with thickness of 0, 15 m and width of 0, 35 m (Figure 7.8). Additionally, the fill in front of the wall should be extended for 1 m laterally (Figure 7.9).



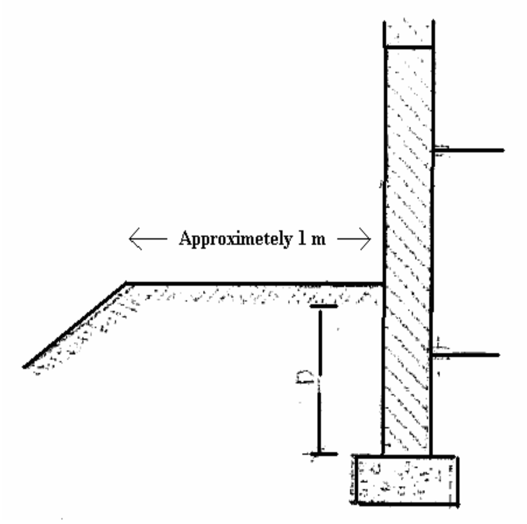


Figure 7.6 .Concrete under surface panel Figure 7.7. Minimum width of toe fill

(French Ministry of Transportation, 1980)

Reinforced earth structures like retaining wall, bridge side pier and dam body, it is demanded that the reinforcement length is to have a value equal or bigger than 0,7H. Taking into account of general stability, soil reinforcement adherence and standardization of reinforcement length, it is defined that the length of reinforcement “L” should not be smaller than 0, 7 H value.

For reinforced earth structures especially for bridge piers, two conditions mentioned below should be achieved.

$L \geq 7m$

$L \geq (0,6H + 2)$ m ($H \leq 20$)

In some cases, due to site conditions, length of reinforcement can be less than the minimum value. For such cases, it should be noted that length of reinforcement is progressive.

7.3.5. Calculation of Lateral Earth Pressure Coefficients

In case of stability analysis on vertical faced walls, it should be accepted that reinforced earth structure behaves as a rigid mass and lateral pressures that affect the system is considered to have influence on a vertical plate in back of the wall.

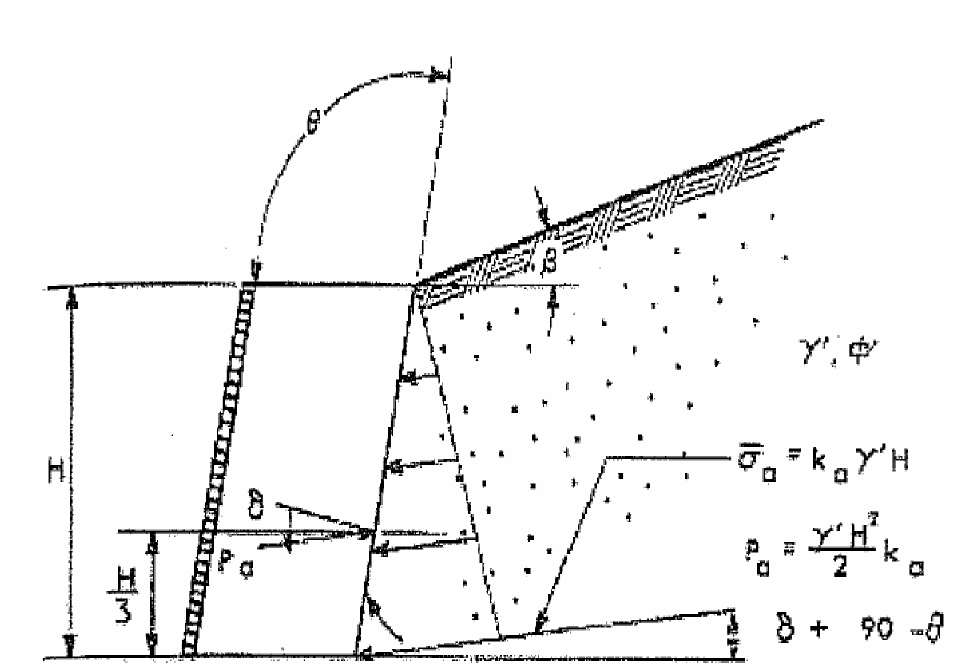


Figure 7.8. Basic concepts in lateral pressure coefficient calculations (U.S Department of Transportation, 2001).

* All angles are positive

γ'	=Effective unit weight
φ	=Effective interior friction angle
K_a	= Active lateral pressure coefficient
δ	= Wall friction angle
P_a	= Resultant lateral force
σ_a	= Lateral pressure
H	= Wall height
B	= Surcharge slope
θ	=Horizontal angle of wall surface

Walls with an angle of 8° or less to horizontal surface (0 < 98°) should be called as steep walls and active lateral pressure coefficients in these walls should be calculated as to back slope angle condition like equation 7.2. and 7.3.

In case of the back slope is horizontal; (θ<98° and β=0, Figure 7.11.)

$$K_a = \tan^2(45 - \varphi / 2) \tag{7.2}$$

In case of back slope is not horizontal then; (θ<98° and β ≠ 0, Figure 7.12)

$$K_a = \cos\beta \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \right] \tag{7.3}$$

In case of a wall with horizontal surface angle of 8° degrees or more (θ ≥ 98°, β = 0 or β ≠ 0) active lateral pressure coefficient can be calculated by Coulomb theory (Equation 7.4., Figure 7.10.).

$$K_a = \frac{\sin^2(\theta + \phi)}{\sin^2\theta \sin(\theta - \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right]^2} \tag{7.4}$$

Backside slope with traffic surcharge case (β=0)

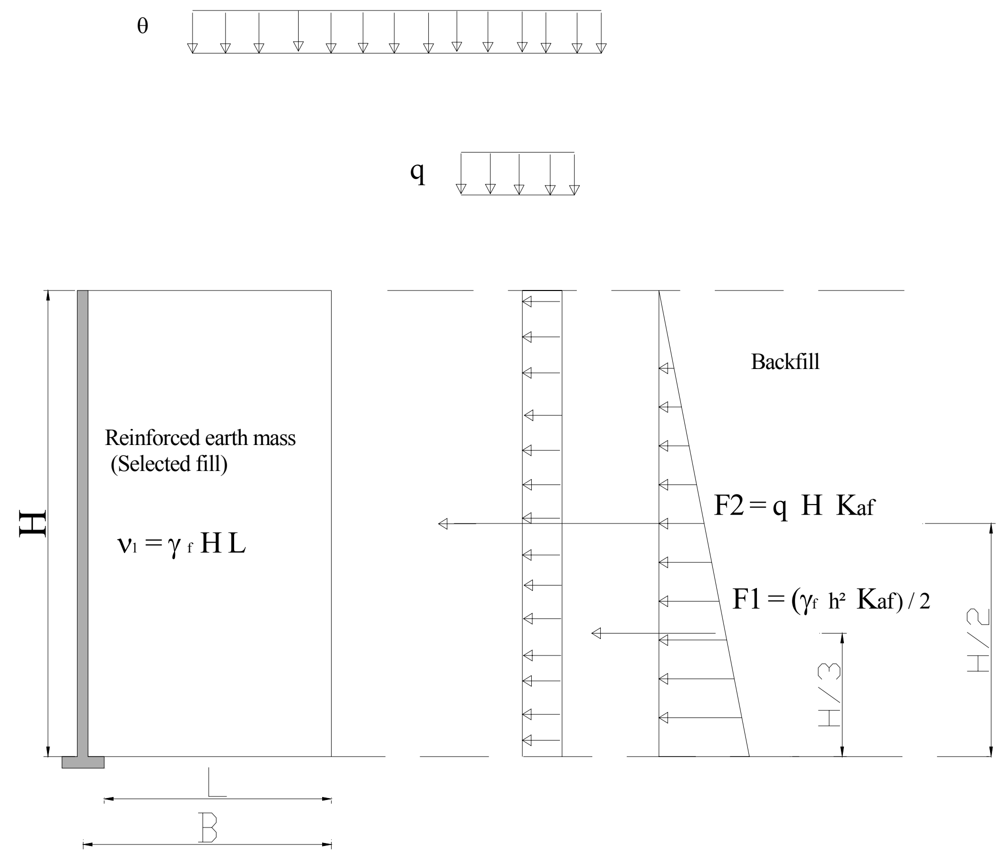


Figure 7.9. Exterior stability in case of the back slope is horizontal (β=0) and existance of surcharge (U.S Department of Transportation, 2001)

q	= Traffic surcharge load in eccentricity and pull-out investigations
θ	= Traffic surcharge load in bearing capacity and total collapse investigations
e	= Eccentricity
V	=Weight of the reinforced earth mass
L	=Width of the reinforced earth
B	= Width of reinforced mass including thickness of surface element
F_1	= Lateral earth force
F_2	= Lateral earth force due to surcharge
K_{af}	=Lateral earth pressure coefficient
γ_f	= Unit weight of fill material
φ_f	= Interior friction angle of fill material

Inclined back slope condition;

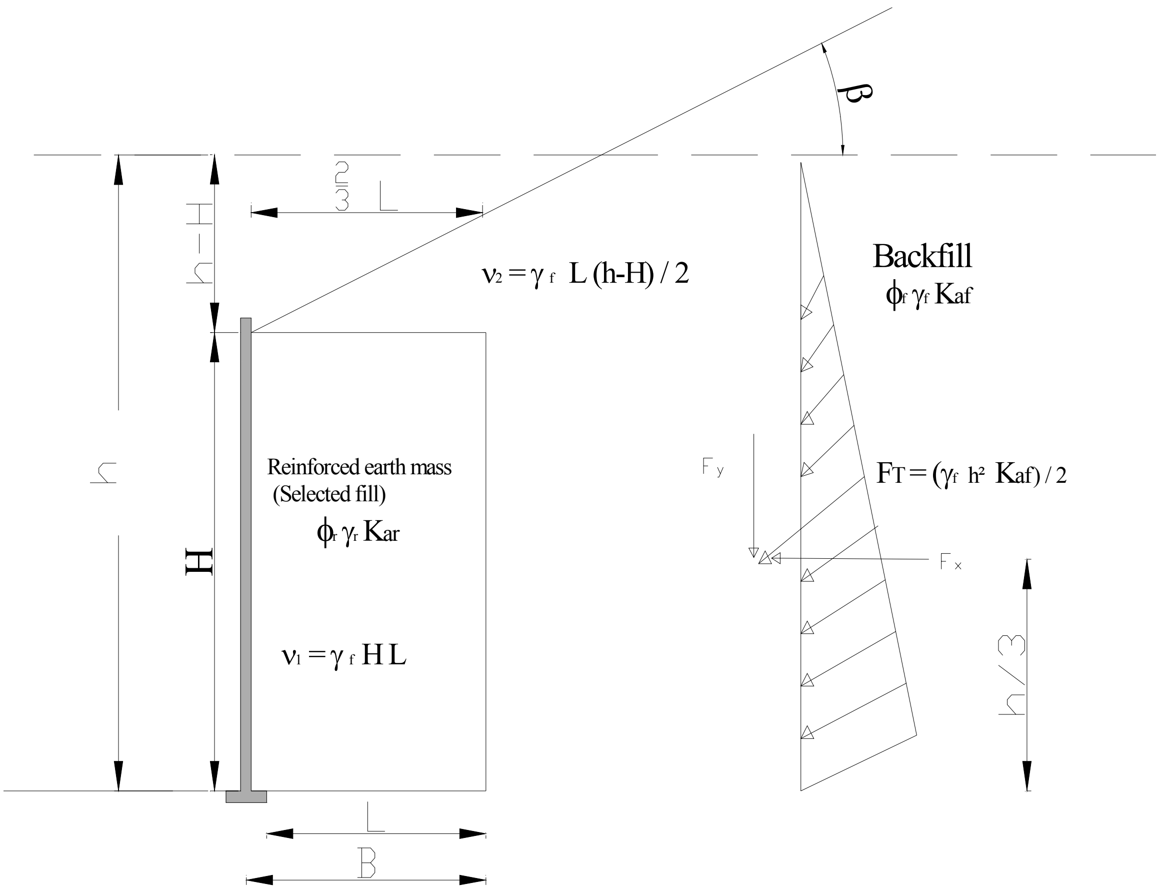


Figure 7.10. Exterior stability in case of backside slope is inclined (U.S Department of Transportation, 2001)

V_1 = Weight of reinforced earth mass

F_T = Resultant lateral ground force

F_v = Vertical components of lateral ground force

F_H = Horizontal component of lateral ground force

Dimensions of facing member and weight can be added into calculations of sliding and over turning in case of using thick surface elements like concrete blocks. In this case width of reinforced earth structure can be taken as B instead of L (U.S Department of Transportation, 2001).

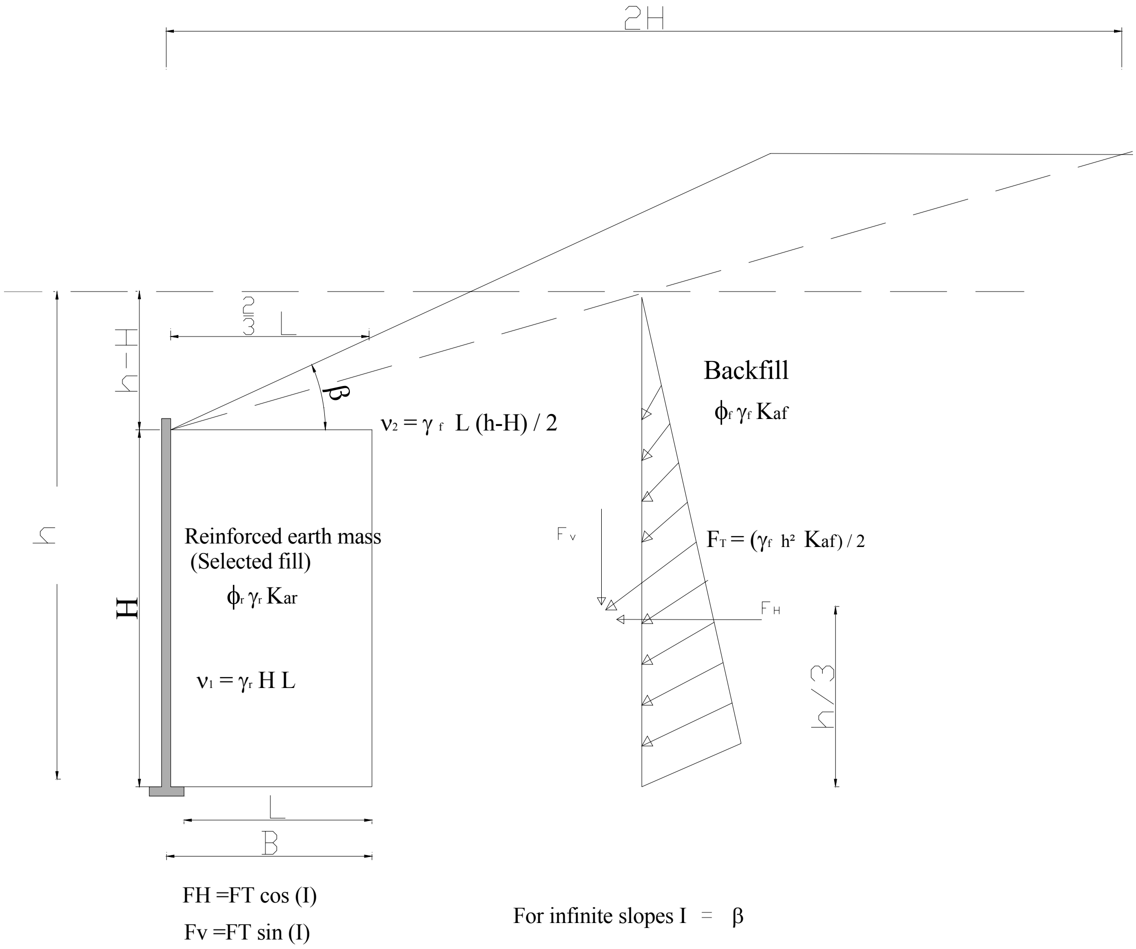


Figure 7.11. Exterior stability in case of backside slope is declined (U.S Department of Transportation, 2001)

7.3.6. Base Pressure Calculation

All definitions about the calculations for base pressure under the wall are shown in Figure 7.12. In these calculations, weights of surface elements have been neglected generally. Calculation steps of vertical surface pressure as follows;

1. F_T: Resultant lateral ground force is calculated by Equation (7.5)

$$F_T = \frac{1}{2} K_{af}(\varphi, \beta)\gamma_f \, h^2 \tag{7.5}$$

2. Moment of resultant vertical forces ($R = V_1 + V_2 + F_T \sin\beta$) as to central point (C) of the system should be equal to the moment of forces that affect the system as to point C. Eccentricity is calculated from this equation.

This approach has been advised by Meyerhof and is assumed to be a uniform pressure on the floor under eccentric loading (L -2e) (Figure 7.14.).

Eccentricity as to Meyerhof approach;

$$e = \frac{F_T(\cos\beta)\,h/3 - F_T(\sin\beta)L/2 - V_2(L/6)}{V_1 + V_2 + F_t \sin\beta} \tag{7.6}$$

3. Eccentricity is accepted to be less than L/4 for rocky foundation soils and less than L/6 for non rocky foundation soils. If eccentricity is bigger than the limit values, then longer reinforcement lengths should be chosen.

4. Equivalent vertical base pressure is calculated by Equation (7.7).

$$\sigma_v = \frac{V_1 + V_2 + F_t \sin\beta_T}{L - 2e} \tag{7.7}$$

5. Effects of surcharge and other exterior loads are added to σ_v.

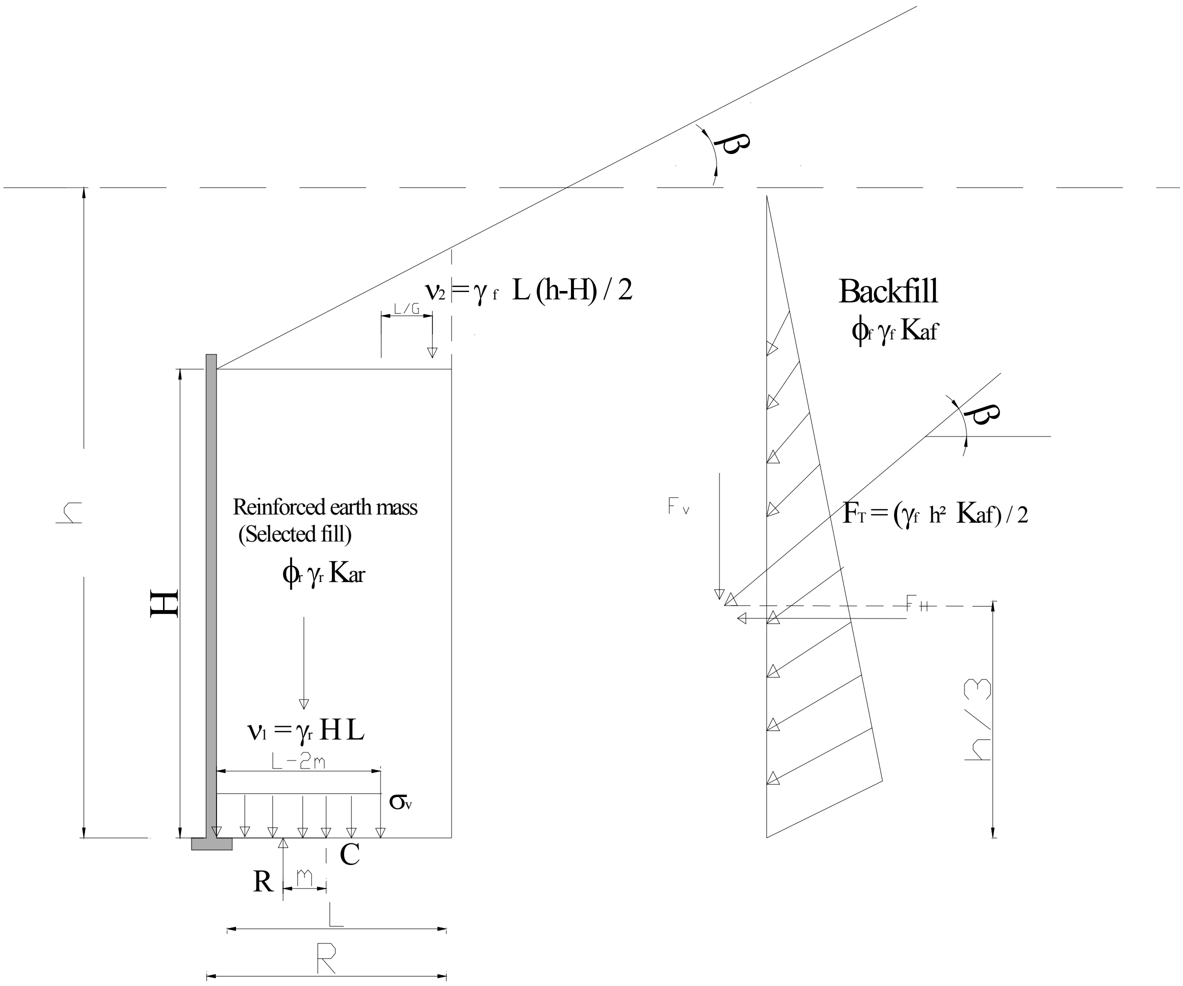


Figure 7.12. Calculation of base pressure under the wall
(U.S Department of Transportation, 2001)

7.3.7. Sliding Check

Wall dimensions that are defined in preliminary design should be checked at base against sliding. In order to maintain the sliding stability, the condition mentioned in Equation (7.8) should be achieved.

$$FS_{sliding} = \frac{\sum horizontal\ resisting\ forces}{\sum horizontal\ driving\ forces} = \frac{\sum P_R}{\sum Pd} \geq 1.5 \tag{7.8}$$

Here;

$FS_{sliding}$ = Safety factor against sliding
ΣP_d = Sum of Driving forces
ΣP_R = Sum of Resisting forces

Passive resistance of the toe fill is neglected as taking into consideration that the fill has been talked of will be removed up till the end of structure's lifetime for some reason. Resistance of facing system against sliding is also neglected.

If extra surcharge loads exist on the site, then these loads should be added to the existing dead and dynamic surcharge loads (Figure 7.10 - 7.14)

In case of a declined backfill presence, sliding investigation steps should be as follows.

1. Earth force is calculated;

$$F_T = \frac{1}{2} K_{af(\varphi,\beta)} \gamma_f h^2 \qquad h = H + L \tan\beta \qquad (7.10)$$

2. Driving forces are calculated;

$$Pd = F_H = F_T \cos\beta \qquad (7.11)$$

3. To determine the most critical condition on the base, interior friction angle (φ) is selected from the smallest alternatives mentioned below:

   - Interior friction angle of foundation soil (φ_f)
   - Interior friction angle of selected fill material (φ_r)
   - Friction angle between soil and reinforcement (ρ)

4. Sum of Resisting forces t are calculated

$$P_R = (V_1 + V_2 + F_T \sin\beta)\,\mu \tag{7.12}$$

Here;

μ = min [tan φ_f, tan φ_r or (for continuous reinforcement) tanρ]

Vertical exterior loads increase resistance of reinforced earth mass against base sliding. For this reason, if these loads are permanent then they should be taken into consideration in calculations (like dynamic traffic loads).

5. It should be checked that condition of equation of 7.8 is satisfied.

6. If this condition is not satisfied then length of reinforcement (L) is increased and the calculations are repeated.

7.3.8. Bearing Capacity Check

Preventing the bearing capacity collapse occurrence becomes possible in the cases that the base stress (σ_v) calculated as to Meyerhof distribution, does not exceed the allowable bearing capacity. Allowable bearing capacity is obtained by dividing the ultimate bearing capacity to factor of safety against bearing capacity.

$$\sigma_v \leq q_{all} = \frac{q_{ultimate}}{FS_{bearing}} \tag{7.13}$$

Factor of safety values of less than 2 for bearing capacity can be used only with the presence of geotechnical analysis that acceptable settlement calculations are achieved (U.S Department of Transportation, 2001).

In case of declined backfill (surcharge) presence, steps of calculations of bearing capacity are as follows (Figure 7.11. and Figure 7.14.);

1. Eccentricity is calculated by Equation (7.6). It is checked that eccentricity is less than L/6 for non rocky foundation soil and L/4 for rocky foundation soil.
2. Meyerhof base pressure σ_v is calculated by Equation (7.7)
3. Ultimate bearing capacity is calculated by using classical soil mechanics methods ($q_{ultimate}$). Ultimate bearing capacity where there is no ground water impact is calculated by Equation (7.14).

$$q_{ult} = c_f \, N_{cf} + 0.5(L)\gamma_f \, N_{tf} \qquad (7.14)$$

Here;

c_f = Foundation soil cohesion

γ_f = Foundation soil unit weight

N_c = Dimensionless Bearing capacity coefficients

Dimensionless bearing capacity coefficients can be obtained from American Highways Specification ASSHTO 1996.

7.3.9. Overturning check

Stability against over turning of reinforced earth structures is realized by checking the moment equilibrium or base eccentricity.

In Section 7.2.5., by eccentricity control with Equation (7.6), check of over turning of the structure is achieved. In this mentioned check, calculated eccentricity is controlled for L/6 for non rocky foundation soil and L/4 for rocky foundation soil.

If the eccentricity exceeds the limit values, then the longer length of reinforcement should be selected (Figure 7.9.).

Check of over turning stability in reinforced earth structure is a check of moment equilibrium (Figure7.13.). In this check it is desired that the ratio of total moment as to toe point A of resisting forces that maintain stability (ΣM_R) to the moment of driving forces (ΣM_O) is higher than the over turning safety value.

$$FS_{overturning} = \frac{\sum M_R}{\sum M_O} = \frac{Sum\ of\ the\ moments\ that\ resist\ to\ overturning}{Sum\ of\ the\ moments\ that\ lead\ to\ overturning} \geq 2 \qquad (7.15)$$

If a declined backfill slope exists (surcharge), ΣM_R is calculated by Equation (7.17).

$$\sum M_R = (F_T \sin\beta)\, L + V_1\,(L/2) + V_2\,(2L/3) \qquad (7.16)$$

If a declined backfill slope exists (surcharge), ΣM_O is calculated by equation 7.18

$$\sum M_0 = (F_T \cos\beta) \times (h/3) \qquad (7.17)$$

7.3.10. Total Collapse Check

Total collapse can occur by existence of a possible sliding surface that passes through either inside the reinforced earth mass or outside. The first stage is an exterior stability case and on design of this type, classical slope stability principals are in use. The second is an interior stability case and special design methods have been created in order to use in design of reinforced earth structures (Tezcan and Buket, 1999).

Most of the general slope stability techniques, based on conventional limit equilibrium method (Section 7.2.3.), can be used in design of reinforced earth structures. One of these is modified Bishop Method.

In the control of exterior stability analysis of reinforced earth mass, analysis should be performed according to conventional soil mechanics methods assuming that structure behaves as a weight, a mass body.

For activation of interior stresses of reinforced earth mass, it should be inclined over sliding reinforcements or inclined to definite distance to the free end of the reinforcements. For instance, while first circle in Figure 7.13. includes interior stability evaluation as well as exterior stability evaluation, circles 2 and 3 will activate the tension stresses in some reinforcements.

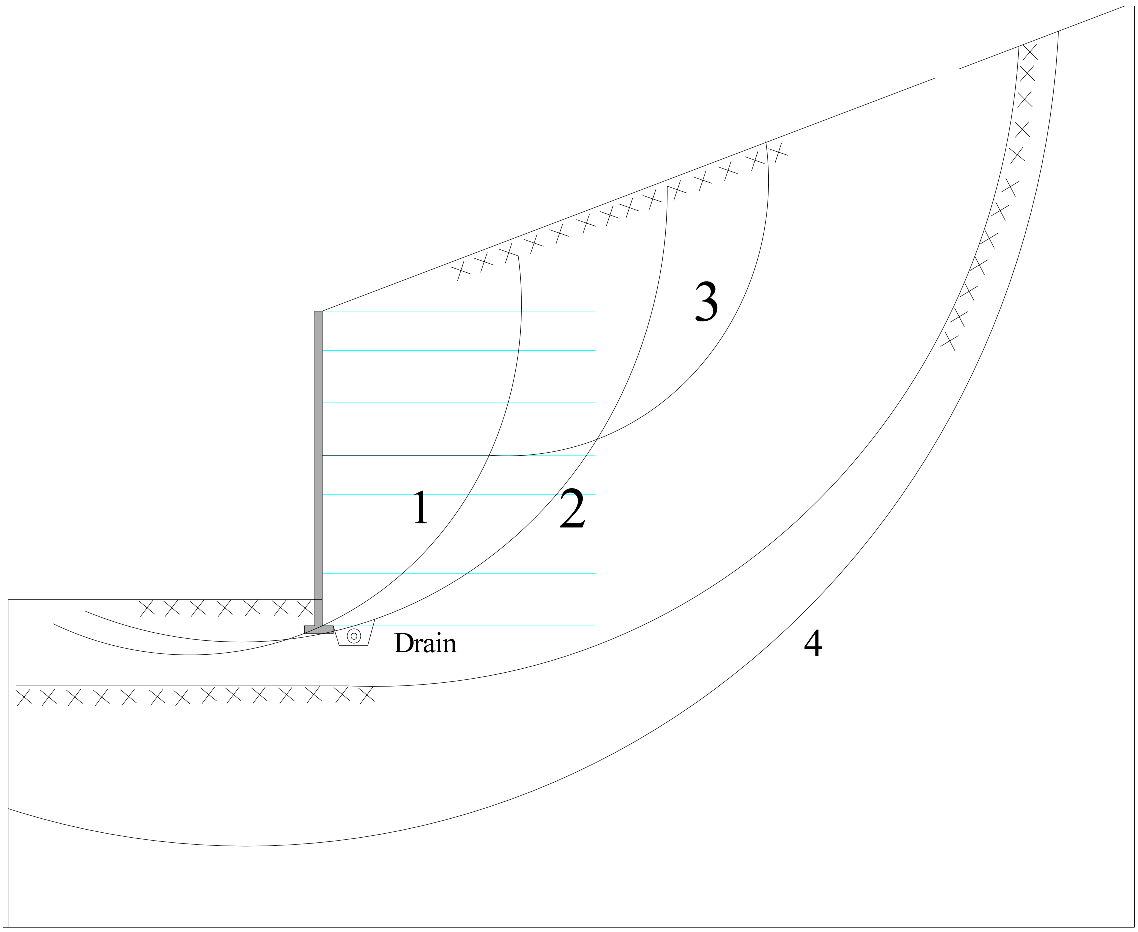


Figure 7.13. Possible sliding (failure) circles that should be taken into account during calculations of total collapse analysis (Tezcan and Buket, 1999)

There occurs a design method where interior and exterior stabilities are investigated at the same time and a sliding surface that passes through the reinforced earth mass (Figure 7.14.). This method is based on classical Bishop Method and especially used for reinforced earth structures on steep slopes, reinforced earth structures in dams and in complicated geometrical shaped reinforced earth structures (Tezcan and Buket, 1999).

Principles of this method are shown in Figure 7.14. Center of circular sliding surface base is divided by slices which crosses the reinforcement and have a width of “b”. Collapse can happen by pull-out or rupture of the reinforcement. Consequently, for stability; maximum tension stress which occurs in the reinforcement should not be equal to sliding resistance and limit tension stress.

For a selected possible sliding circle, factor of safety against collapse (FS_{total}) can be calculated as follows.

$$F_S = A / B$$

$$A = \sum_{i}^{n} \left[\left(b_i c' + W_i \tan \phi'_w - b_i u_i + W_i \tan \phi'_w \right) \times \frac{F \sec \alpha}{F + \tan \phi'_w \tan \alpha} + T_i \cos \alpha_i \right]$$

$$B = \sum_{i}^{n} W_i \sin \alpha_i$$

Here;

b_i = Width of the soil block at the level i.

c' = Cohesion,

W_i = Weight of soil block at the level i,

ϕ_w = Interior effective friction angle,

υ_I = Cavity water pressure at the level i.

α_i = Angle of tangent of sliding circle at the level i,

T_i = Minimum tension resistance-(pull-out resistance) of the reinforcement at level i.

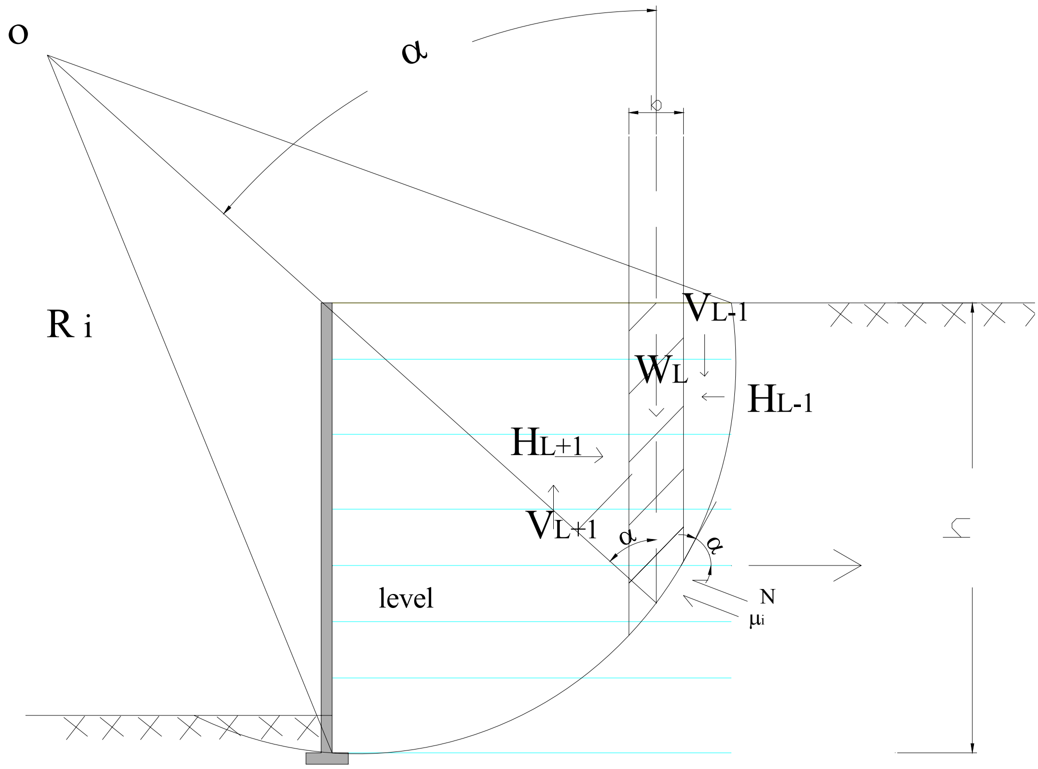


Figure 7.14. Sliding circle method (Tezcan and Buket, 1999)

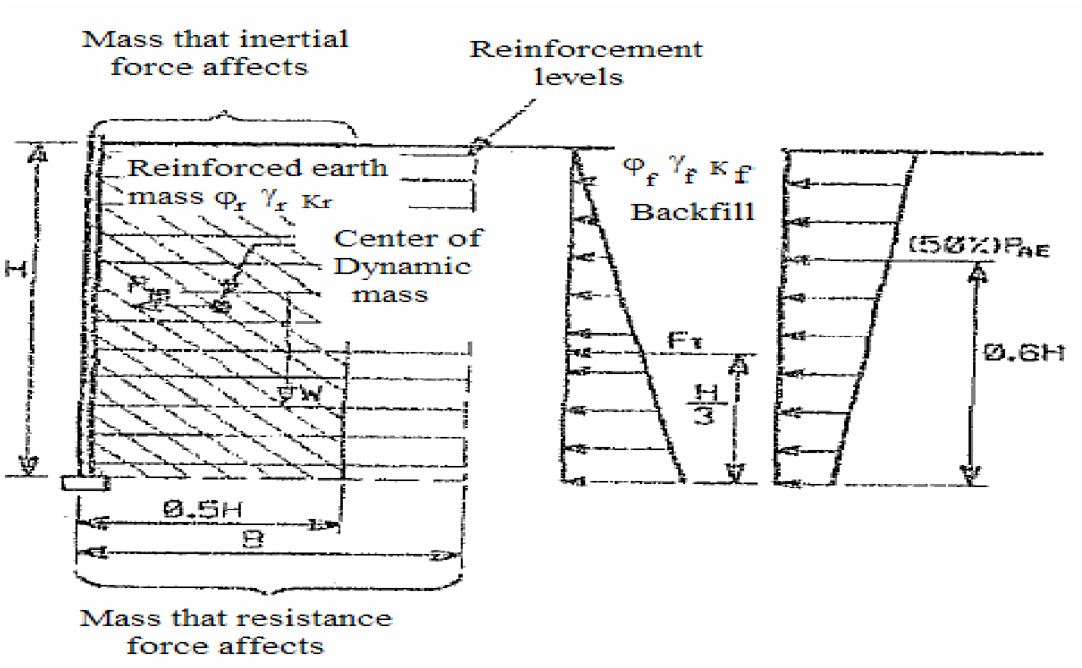
All possible sliding surfaces should be checked in total collapse check. That would be more practical to use computer programs for this type of analysis. Safety factor against total collapse in reinforced earth structures (FS_{total}) is the same as traditional retaining structures. It is required to have 1.5 for general loading condition, and for loadings including earthquake combination it can be reduced to 1.25 (Tezcan and Buket, 1999).

7.3.11. Exterior Stability Checks for Dynamic Loading Case

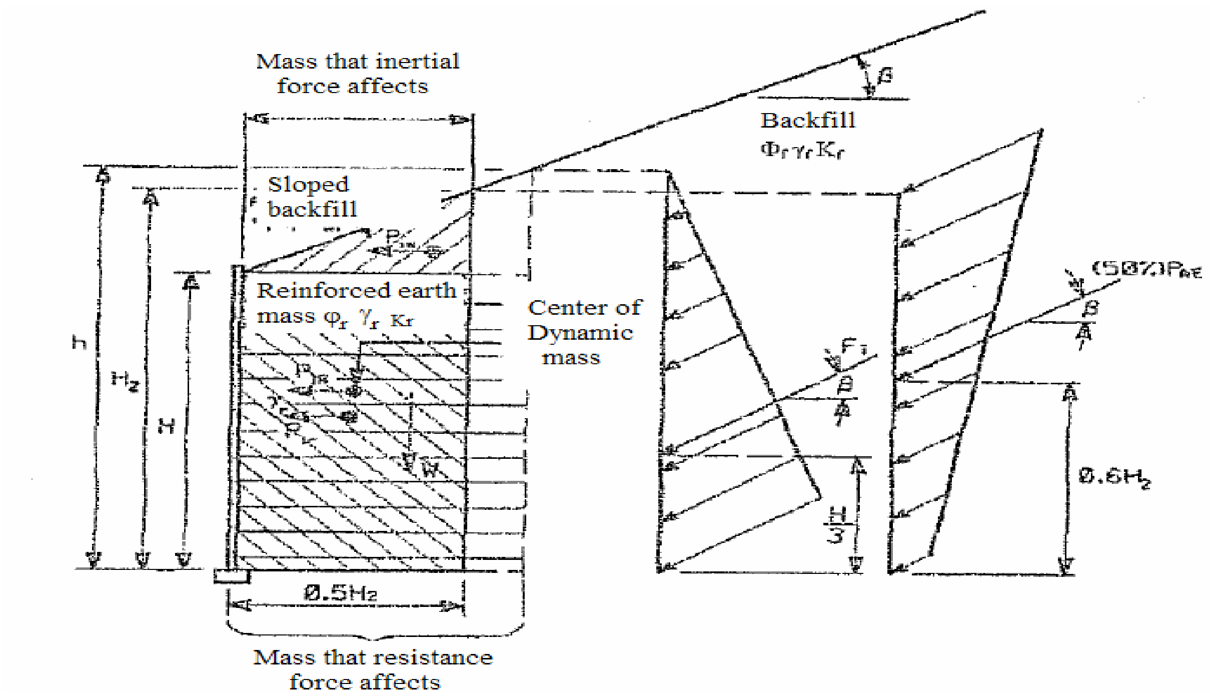
In spite of using lots of different materials for reinforced earth structures, it is made of a reinforced earth area placed at the back of reinforced earth walls that holds the ground. In case of an earthquake, reinforced wall has been influenced by dynamic ground effect at the back of reinforced earthed area and static forces in reinforced earth in addition to inertia forces. The wall should be designed so that it is avoided from exterior insensitivity (failure of reinforced area to sliding and rotation type). Exterior stability analysis has been mentioned for reinforced earth structures with different approaches below. Fundamentals of these 3 methods are based on Mononobe-Okabe method (Kramer, 2003).

In addition to present static loads on reinforced earth structures during earthquake, dynamic lateral affect (P_{AE}) is occurred at back of the fill zone. In addition, reinforced earth is subjected to lateral inertial force "$P_{IR} = M\ A_m$". M is weight of active piece that assumed to have width of 0.5H; and A_m is the lateral maximum acceleration on system. (U.S Department of Transportation, 2001).

P_{AE} force is calculated by pseudo-static Mononobe- Okabe analysis (Figure 7.15.) and this force should be added to other static loads that affect the wall (surcharge, weight, static stress), and then exterior stability of the structure is evaluated. Allowable minimum factor of safety value should be 75 per cent of static factor of safety value.



a) Horizontal Backfill condition



b) Inclined backfill condition

Figure 7.15. Exterior stability analysis under dynamic loading on reinforced earth structures (U.S Department of Transportation, 2001)

As to American Highways Specification, steps mentioned below should be followed in determining exterior stability under dynamic loading.

- Peak lateral ground acceleration is selected based on design seismic load
- Ground acceleration coefficient can be determined from AASTHO Section 1A.
- Maximum acceleration that will occur on the wall should be calculated (A_m)

$$A_m = (1.45\text{-}A)\ A \tag{7.22}$$

Here;

A = Maximum ground acceleration coefficient, AASHTO Section 1 A,

A_m =Maximum wall acceleration coefficient at the middle of the wall mass (peak acceleration)

- Lateral inertia force P_{IR} and seismic dynamic effect P_{AE} are calculated

$$P_{IR} = 0.5\, A_m\, \gamma_r\, H^2 \tag{7.23}$$

$$P_{AE} = 0.375\, A_m\, \gamma_f\, H^2 \tag{7.24}$$

- 50 per cent of P_{AE} force, whole amount of P_{IR} inertial force and other static forces are subjected to the structure. The reason for using reduced P_{AE} is that the possibility of affecting of maximum values of P_{AE} and P_{IR} at the same time is very low.

- If there exists a sloped backfill, then inertia and dynamic lateral force are defined as a function of "H_2" height (Figure 7.15.)

$$H_2 = H + \frac{\tan\beta \times 0.5\, H}{(1 - 0.5\tan\beta)} \tag{7.25}$$

In case of sloped backfill, P_{AE} force is determined by Mononobe-Okabe analysis where the lateral acceleration k_h equals to A_m and vertical acceleration k_v equals to zero. In calculation of P_{AE} force, H_2 height will be used. In case of sloped backfill, P_{AE} and P_{IR} equations will be as follows (U.S Department of Transportation, 1998).

$$P_{IR} = P_{ir} + P_{is} \tag{7.26}$$

$$P_{ir} = 0.5 A_m \gamma_f (H_2) H \tag{7.27}$$

$$P_{is} = 0.125 A_m \gamma_f (H_2)^2 \tan\beta \tag{7.28}$$

$$P_{AE} = 0.5 \gamma_f (H_2)^2 \Delta K_{AE} \tag{7.29}$$

Here, P_{ir} occurs due to acceleration of reinforced earth mass with inertial force width of 0,5H_2 and P_{is} inertial force, occurs due to acceleration sloped backfill (surcharge) on reinforced earth mass. P_{IR} force affects to center of the force components P_{is} and P_{ir} (Figure 7.15.). General explanation of total seismic ground pressure coefficient K_{AE} due to Mononobe-Okobe is defined as in the equation below.

$$K_{AE} = \frac{\cos^2(\phi - \xi - 90 + \theta)}{\cos\xi \cos^2(90-\theta)\cos(I+90-\theta+\xi)\left[1+\sqrt{\dfrac{\sin(\phi+I)\sin(\phi-\xi-I}{\cos(I+90-\theta+\xi)\cos(I-90+\theta)}}\right]^2} \tag{7.30}$$

I = Slope of backfill =β (Figure 7.8. and figure 7.12.)

ς = arctan (K_h/(l-K_v))

φ = Ground interior friction angle

θ = Angle of wall surface with horizon (Figure 7.8.)

In order to complete the design, sliding stability, eccentricity and bearing capacity stability controls mentioned in previous sections should be achieved. All dynamic factor of safety values calculated in these checks should be confirmed as that they are equal to or more than the 75 per cent of static factor of safety values and the eccentricity is within L/3.

As mentioned in 1998 AASHTO Highways Bridge Specification, Mononobe-Okobe base pressure at the back of the wall, permits permanent seismic base pressure reducing due to lateral movement of the wall to the outside. Mentioned reduced seismic base pressure is calculated with K_h coefficient occurred by allowable reduced lateral acceleration. Reduced K_h coefficient is calculated by 'Newmark Sliding Block Analysis'. Reduced K_h coefficient can be used for any weighted or semi-weighted retaining wall if conditions mentioned below are confirmed (U.S Department of Transportation, 2001).

- Wall system or any earth structures supported by wall, should be able to permit horizontal settlement that occurred by sliding of the mentioned structure.
- Sliding ability of the wall should be limited by earth friction at base and minimum passive earth effect.
- If wall functions are supported against sliding as in bridge side piers, then top of the wall should not be limited.

As explained in AASHTO, in case of horizontal settlement is allowable for values bigger than 250 A mm; K_h in Mononobe-Okabe Analysis shall be able to decreased to 0.5A in weight, side weight and bridge pier structures .If there is no information about historical data of Kavazanjian ground movement, ground speed is assumed to be 30A and a definition and simplified Newmark Analysis has been created for K_h. In mentioned definition, maximum lateral acceleration, Am at the center of reinforced earth mass has been used and Kh is calculated by the formula shown below;

$$K_h = 1.66\, A_m \left(\frac{A_m}{d} \right)^{0.25} \tag{7.31}$$

Here, d is horizontal settlement of the wall in mm. This equation is not used in case the horizontal settlement is less than 25 mm (1 inch) and bigger than 200 mm (8 inch). It is advised to use this reduced acceleration equation only for reinforced earth structures that behave as a rigid block. Interior reaction of reinforced earth structures to horizontal settlement is more complicated and in dynamic loading it is not clear that how much decrease will be generated in acceleration due to allowable horizontal settlement.

Generally, reduced ground pressures which are equal to 50-100 mm (2-4 inch) settlements have been used at wall designs in applications at active seismic zones. But, allowable settlement is dependent on the nature of the wall, supporting items and those in front of the wall. Equation (7.31) is defined as K_h, $A_{m.}$.

Peak ground acceleration "A" of this simplified approach; is advised not to be used in walls higher than 15m and structures with complicated geometrical body (U.S Department of Transportation, 2001).

7.3.12. Exterior Stability Analysis Methods as to Turkish Disaster Regulations

Since it is accepted that wall is a weight that behaves as a rigid body in exterior stability analysis of reinforced earth structures, it is required to confirm the items defined in Turkish Disaster Regulations (1997).

Total active pressure P_{at} and total passive pressure P_{pt} that occurred in retaining structures during earthquakes have been calculated by Mononobe-Okabe and this calculation format has been used in Disaster Regulations in countries like Japan, India and Russia (Ozden et al., 1995).

In the definition of dynamic and active earth pressure forces, static and passive ground pressure coefficients and total active and passive earth pressure coefficients are mentioned. For this reason, definition of these terms should be clarified first (Ozden et al., 1995).

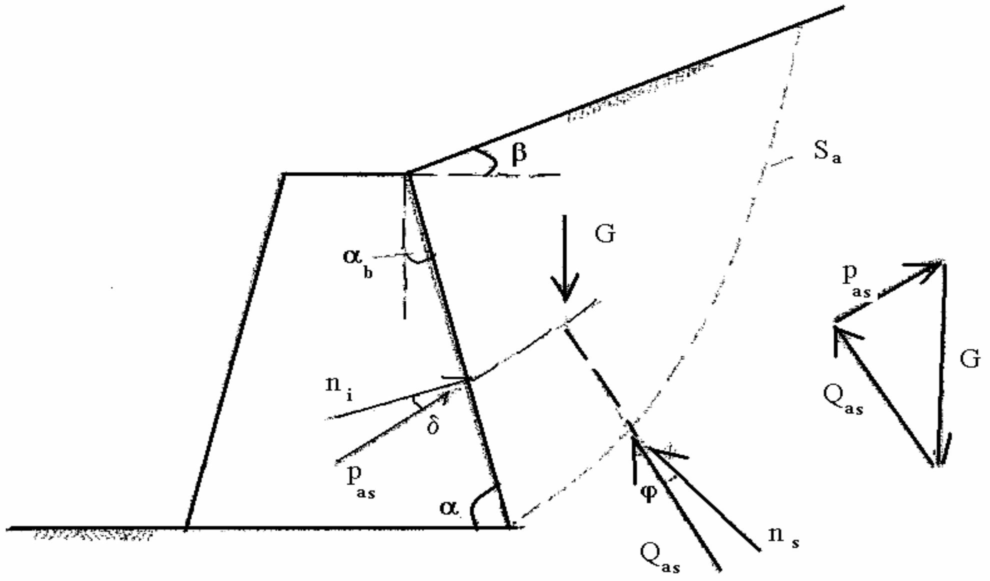


Figure 7.16. Earth daggers occurred in case of static active position (Das, 1987)

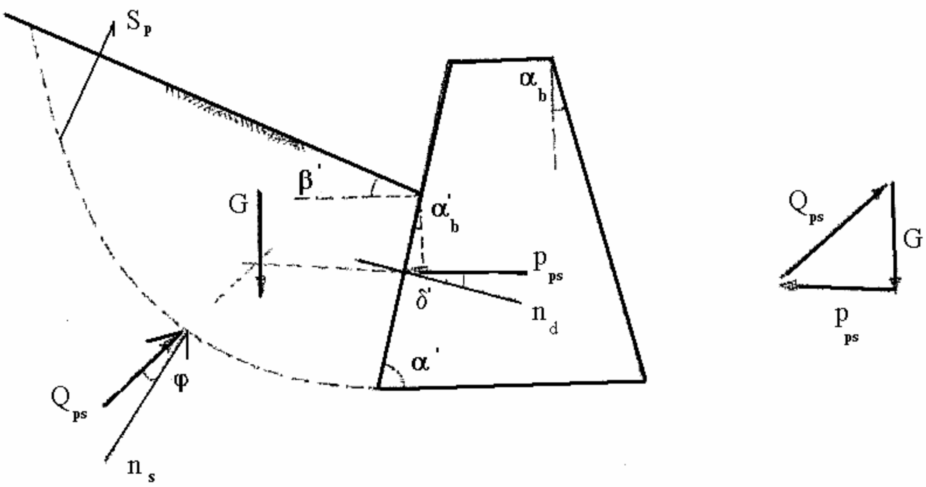


Figure 7.17. Earth Dagger in case of static passive position (Das, 1987).

Here:

β', β = Slope of surface soil

α, α' = Angle with horizon of the back side of the wall

$\alpha_h, \alpha_{h'}$ = Angle of the backside of the retaining structure with vertical

n_i = Normal of backside of the retaining structure

G = Weight of the dagger

S_a = Normal on sliding surface

Q_{as} = Resultant of sliding stresses

n_s = S_a's normal on Q's application point

δ, δ' = Friction angle of backside of retaining structure

φ = Interior friction angle of foundation soil

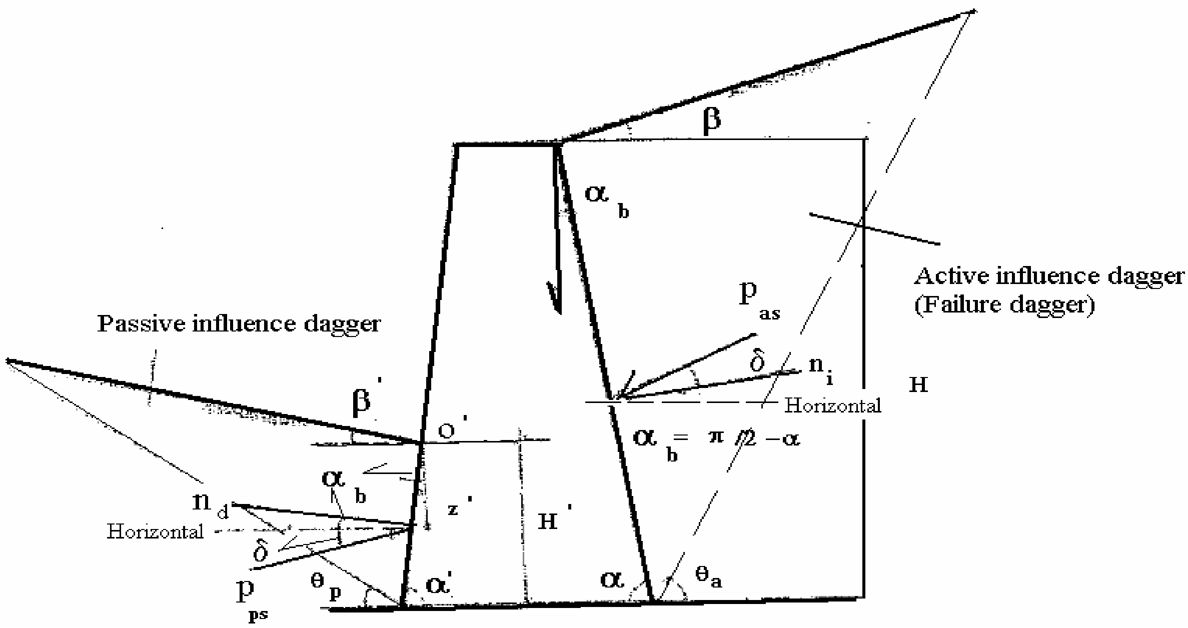


Figure 7.18. Lateral stresses occurred in gravity retaining wall on static active and passive position (Das, 1987).

Here;

p_{as} = Static active earth influence

P_{ap} = Static passive earth influence

γ = Unit weight of the soil or fill

H, H' = Distance of upper soil surfaces from bottom level of member of retaining member and contact points

K_{as} and K_{ps} coefficients can be calculated by Coulomb method mentioned in Equations (7.32) and (7.33).

$$Kas = \frac{\cos^2(\phi - \alpha_b)}{\cos^2\alpha_b \cos(\delta + \alpha_b)\left[1 + \sqrt{A\ /B}\ \right]^2} \tag{7.32}$$

$$Kps = \frac{\cos^2(\phi + \alpha_b)}{\cos^2\alpha'_b \cos(\delta - \alpha'_b)\left[1 - \sqrt{A'\ /B'}\ \right]^2} \tag{7.33}$$

Here;

$$A \qquad = \sin(\varphi + \delta) \times \sin(\varphi - \beta) \tag{7.34}$$

$$B \qquad = \cos(\alpha_b - \beta) \times \cos(\delta + \alpha_b) \tag{7.35}$$

$$A' \qquad = \sin(\varphi + \delta') \times \sin(\varphi - \beta') \tag{7.36}$$

$$B' \qquad = \cos(\delta' - \alpha_b') \times \cos(\beta' - \alpha_b') \tag{7.37}$$

After obtaining the earth coefficients for static, active and passive cases then it is possible to keep on with calculation of these coefficients for dynamic case. In this calculations type, only active dynamic "P_{ad}" and passive "P_{ap}" pressures are taken into account. Accordingly P_{ad} and P_{ap} pressures in Figure 7.19., 7.20., 7.2. rotation is defined as follows:

$$P_{ad} = \pm\ \gamma z\, E\, a \tag{7.38}$$

$$P_{ad} = \pm\ \gamma z'\, Ep \tag{7.39}$$

Here;

P_{ad} =Dynamic active earth influence

P_{ap} =Dynamic passive earth influence

γ = Unit weight of reinforced earth fill

E_a = 3*(1-z/H)*($K_{at} - K_{as}$) (7.40)

E_p = 3*(1-z/H)*($K_{pt} - K_{ps}$) (7.41)

K_{as} = Static active earth pressure coefficient

K_{ps} = Static passive earth pressure coefficient

K_{at}=Total active earth pressure coefficient

K_{pt}=Total passive g earth pressure coefficient

K_{at} is calculated as follows;

$$K_{at} = \frac{(1 \pm C_v)\cos^2(\phi - \lambda - \alpha_b)}{\cos\lambda \, \cos^2\alpha_b \cos(\delta + \alpha_b + \lambda)\left[1 + \sqrt{A_d / B_d}\right]^2} \qquad (7.42)$$

Here;

C_h = Horizontal equivalent earthquake coefficient

C_v = Vertical equivalent earthquake coefficient

$$\arctan\left[\frac{C_h}{1 \pm C_v}\right] = \lambda \quad \text{(dry soils)} \qquad (7.43)$$

$$\arctan\left[\frac{C_h}{1 \pm C_v}\right] = \lambda \quad \text{(below GWT)} \qquad (7.44)$$

γ_s = Unit weight of saturated soil (7.45)

γ_b = Unit weight of soil under water (7.46)

$$A_d = \sin(\varphi + \delta) \times \sin(\varphi - \lambda - \beta) \tag{7.47}$$

$$B_d = \cos(\alpha_b - \beta) \times \cos(\delta + \alpha_b + \lambda) \tag{7.48}$$

K_{ap} total passive pressure coefficient is calculated by the equation seen below.

$$K_{pt} = \frac{(1 \pm C_v)\cos^2(\phi - \lambda + \alpha_b')}{\cos\lambda\ \cos^2\alpha_b' \cos(\delta' - \alpha_b' + \lambda)\left[1 - \sqrt{A_d'/B_d'}\right]^2} \tag{7.49}$$

Here;

$$A_d' = \sin(\varphi + \delta') \times \sin(\varphi - \lambda + \beta') \tag{7.50}$$

$$B_d' = \cos(\beta' - \alpha_b) \times \cos(\delta - \alpha_b + \lambda) \tag{7.51}$$

In case that the soil is saturated, δ/2 instead of δ and δ'/2 instead of δ' should be taken in equations mentioned before. Signs of coefficients K_{at}, K_{pt} and C_v should be selected in accordance with λ so that minimum negative results are obtained.

P_{ad} and P_{pd} dynamic pressures should be added to the calculations to affect most negatively as taking into account that earthquake is a reversely effect.

Horizontal equivalent earthquake coefficient C_h;

In earth retaining walls that work vertically as free console; formula below mentioned is used;

$$C_h = 0.2\ (I + 1)\ A_o \tag{7.52}$$

Here;

I = Structure importance coefficient (It is generally 1 but it is used up to 1.2 and depends on structure type)

A_o = Maximum ground acceleration (Table 7.5.)

Table 7.5 Maximum ground acceleration coefficients

Earthquake Zone	**Maximum ground acceleration coefficient(Ao)**
]	0.4g
2	0.3g
3	0.2g
4	0.1g

g : gravity (9.81 m/s2)

Building slabs in horizontal directions or earth retaining structures and elements that were supported by anchorages are calculated by the formula seen below;

$$C_h = 0.3(I + 1)\ A_0 \qquad (7.53)$$

Vertical equivalent earthquake coefficient C_v has been defined in equations mentioned before, but this coefficient should be taken as zero in structure slabs in horizontal direction and in supported basement floors (Specification for Structures to be Built in Disaster Areas, 1998).

$$C_v = 2C_h/3 \qquad (7.54)$$

According to this explanation, dynamic active P_{ad} and dynamic passive P_{pd} pressures or as seen in Equations (7.38) and (7.39), distributed along back side of the wall as H height and along the front face of the retaining wall as H' height as a form of second degree parabola. This distribution is shown in Figure 7.19.

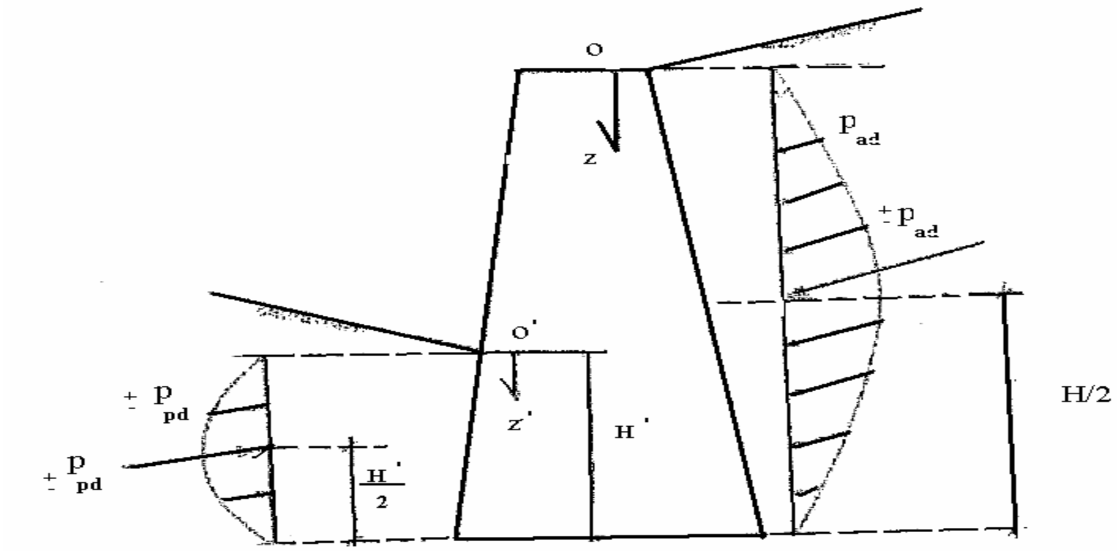


Figure 7.19. Distribution of active and passive ground influences (Das, 1987)

Resultants of P_{ad} and P_{pd} pressures, in other words; integration of dynamic active influence P_{ad} and dynamic passive influence P_{pd} are calculated by using Equation (7.55) and Equation (7.56) as shown below (Ozden et al., 1995);

$$P_{ad} = \int_0^H P_{ad} dz \times 1 = \frac{1}{2} \times \gamma\, H^2\, (K_{at} - K_{as}) \tag{7.55}$$

$$P_{pd} = \int_0^{H'} P_{pd} dz' \times 1 = \frac{1}{2} \times \gamma\, H'^2\, (K_{pt} - K_{ps}) \tag{7.56}$$

In case of layered soils, Equation (7.55) and (7.56) can be used. In this case, by using K_{ad} or K_{pd} coefficients for each layers, depths z or z' will be considered always downwards from free surface. Components of additional dynamic active and passive pressure values for each layer can be found by integrating Equation (7.55) and (7.56) along the related layers (Specification for Structures to be Built in Disaster Areas, 1998).

In addition to static earth pressures, calculated safety coefficient against sliding by taking into account of dynamic earth pressures mentioned in Equations (7.38), (7.39), (7.62) and (7.63), should be at least 1.1 and safety coefficient against over turning should be at least 1.3 (Specification for Structures to be Built in Disaster Areas, 1998).

<u>7.3.12.1. Exterior stability Analysis Method for Dynamic Loading Case by Steven L. Kramer.</u> Although alternative seismic exterior stability analysis method that submitted by Steven L. Kramer in Geotechnical Earthquake Engineering Earthquake Book (1996) principally has similarities with American Highways Specification criteria's, it can differ in some formulations.

To evaluate the exterior stability, a reinforced earth wall is taken into consideration in many ways like a gravity wall. As shown in Figure 7.20., it is assumed that force applied to the reinforced zone is made up of only self weight "W" and static ground influence P_A. Earthquake load is represented as dynamic ground influence P_{AE} and inertial force P_{IR} pseudo-statically. Exterior stability of any wall design can be evaluated by the procedure below (Kramer, 2003).

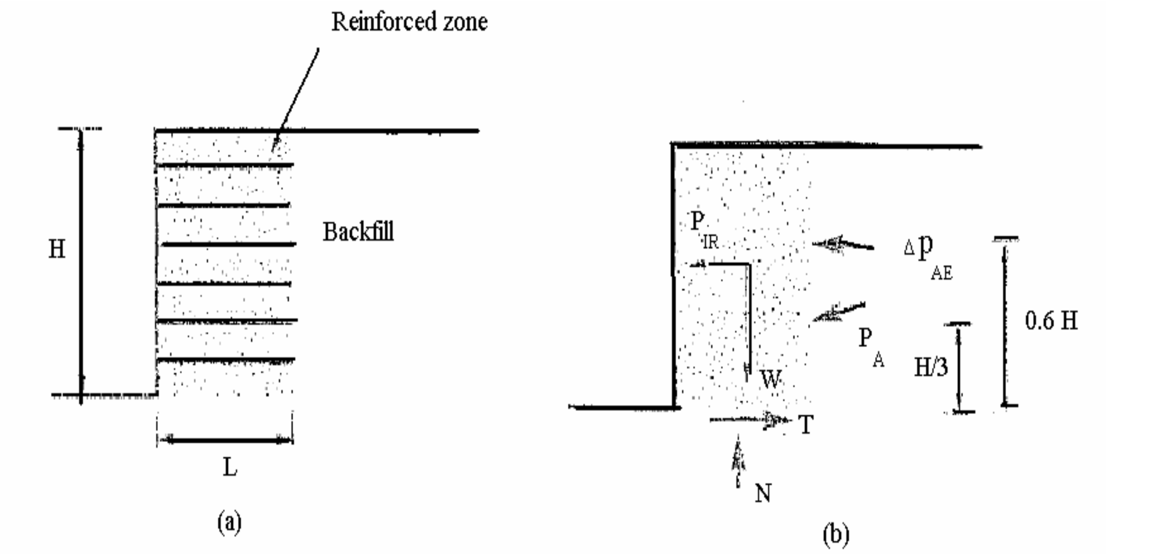


Figure 7.20. (a) Geometry and rotation for reinforced earth wall - (b) Static and pseudo-static forces applied to reinforced zone (Kramer, 2003)

- Peak ground acceleration (a_{max}) is calculated
- Peak acceleration in center of gravity of reinforced zone is calculated
- Dynamic ground influence is calculated

Here;

$\gamma^{(b)}$ = Unit weight of backfill

- Inertial force affecting on reinforced zone is calculated by Equation 5.59

$$P_{IR} = \frac{a_c \gamma^{(r)} H L}{9} \tag{7.58}$$

$\gamma^{(r)}$ = Unit weight of the soil in reinforced zone (selected fill)

- Exterior stability should be checked against sliding and rotation for static forces that affect reinforced earth, by adding P_{av} and 50 per cent of P_{IR}. (The reason for using of reduced value of P_{IR} is low probability of realization of maximum values of ΔP_{AE} and P_{IR} at the same time). During the design for dynamic loading, safety coefficients against sliding and over turning are assumed to be 75 per cent or higher than the safety coefficients accepted for minimum static loading (Kramer, 2003).

7.3.12.2. Active and Passive Ground Influences Due to Uniform Distributed External Loading During Earthquake. In addition to static earth pressure, change of uniform distributed exterior loading along soil height as active and passive earth pressure in case of an earthquake is defined in equation 7.60 and 7.61 (Figure 7.20. and 7.24.).

$$q_{ad}(z) = 2 q K_{ad} (1 - z / H) \cos \alpha_b / \cos(\alpha_b - \beta) \tag{7.60}$$
$$q_{pd}(z) = 2 q' K_{pd} (1 - z'/H') \cos \alpha'_b / \cos(\alpha'_b - \beta') \tag{7.61}$$

Here;

q_{ad} (z) = Change of active stress that occurred behind of the wall due to q uniform distributed load during earthquake with respect to "z" depth (Figure 7 .24.).

q_{pd} (z) = Change of passive stress that occurred in front of the wall due to q' uniform distributed load during earthquake with respect to "z" depth (Figure 7 .24.).

q = Uniform distributed load behind the wall
q' = Uniform distributed load in front of the wall

In case of uniform soil properties, integration of Equations (7.60) and (7.61) along the height of soil in addition to static pressure, components of active (positive) and passive (negative) earth pressures Q_{ad} and Q_{pd} and Z_{cd} that shows depth of this component forces from ground surface is defined in Equation 7.63. Also same distribution is shown in Figure 7.24. (Specification for Structures to be Built in Disaster Areas, 1998).

$$\begin{aligned} Q_{ad} &= q\, K_{ad}\; H\, B_a \\ Q_{pd} &= q'\, K_{pd}\; H'\, B_p \end{aligned} \tag{7.63}$$

Here;

$$\begin{aligned} B_a &= \cos \alpha_b \,/\, \cos(\alpha_b - \beta) \\ B_p &= \cos \alpha'_b \,/\, \cos(\alpha'_b - \beta') \end{aligned} \tag{7.64}$$

As seen in Figure 7.21. Z_{cd} is H/3, in active case and H'/3 in passive case

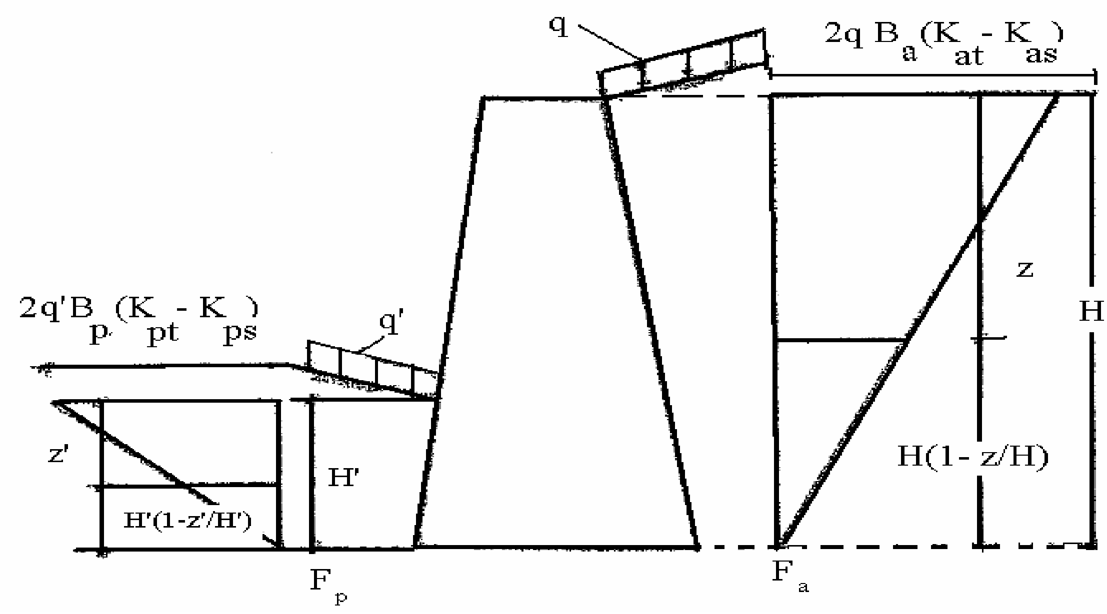


Figure 7.21. Active and passive influences at the backside and in front of the wall in case of earthquake (Specification for Structures to be Built in Disaster Areas, 1998)

7.4. Interior Stability Analysis of Reinforced Earth Structures

Interior stability collapse is observed in two types in reinforced earth structures:

- Tension stress in reinforced structures (and shear stress in rigid reinforcement) gets so big that, the reinforcement can lengthen or rupture. So, this causes the structure to collapse or move extremely. This type of collapse cases are called as collapses of stretching and rupture.
- Tension stress occurred in reinforcement can be bigger than the pull-out resistance that prevents it to pull-out from the mass. This situation causes increase in shear stress and a large movement in the structure which will probably ends up with collapse. These types of collapses are called as pull-out collapses (U.S Department of Transportation, 1998).

Interior stability design steps are explained below.

- Selection of reinforcement type (stretchable or not stretchable),

- Determination of the position of critical collapse surface
- Determination of reinforcement intervals as harmonious with facing member
- Calculation of maximum tension stress at all reinforcement levels for static and dynamic conditions
- Calculation of maximum tension stress at facing connection points
- Rupture resistance at all reinforcement levels

Steps in calculation of interior stability are shown schematically in Figure 7.22. (U.S Department of Transportation, 2001).

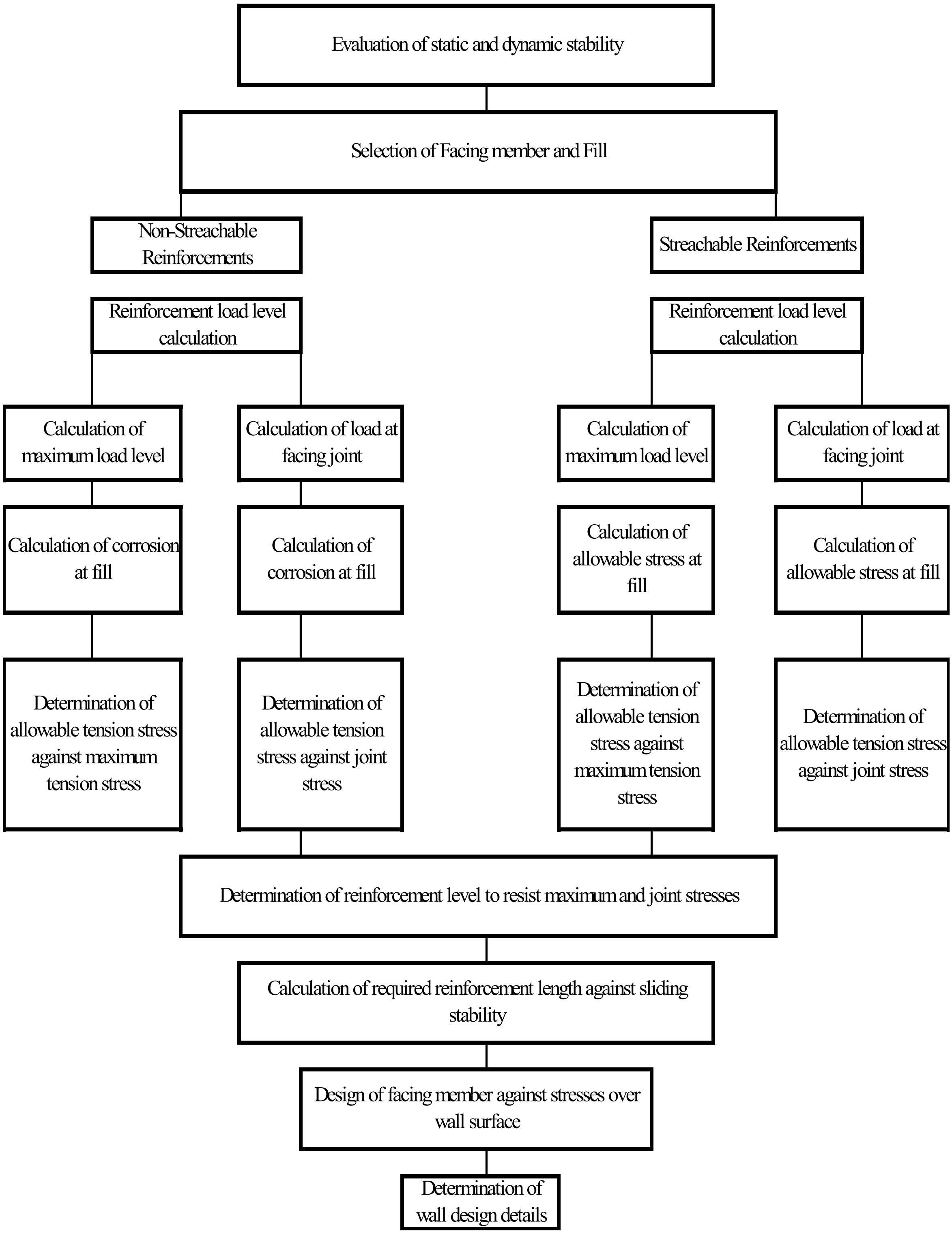


Figure 7.22. Interior stability calculation steps in reinforced earth structures (U.S Department of Transportation, 2001)

7.4.1. Factor of Safety Values for Interior Stability

As to American Highways Specification it is required safety coefficient against pull-out to be taken as 1.5 and as to rupture is 3.3. Also, safety coefficient of pull-out under dynamic loading is to be taken al least 1.1 (U.S Department of Transportation, 2001).

In dynamic cases, it should be checked that allowable tension resistance should be equal to 75 per cent of total tension stress. It should be also checked in pull-out resistance for each reinforcement layer that, safety coefficient that prevents pull-out collapse of critical collapse surface should be at least 7 per cent of minimum static factor of safety value under total loading (Kramer, 2003).

As to TS 7994 where bearing capacities of structures are checked, safety factor against pull-out is to be taken as 2 (TS 7994, 1995).

7.4.2. Calculation of Maximum Tension Stress at Reinforcement Levels

Latest studies conducted on reinforced structures has revealed that maximum tension stress occurred in reinforcement is related with density, elasticity module and stretchiness of reinforcement. Depending on these studies, relationship between types of reinforcement and shroud load (height of wall) has been developed as shown in Figure 7.23. K/K_a ratio in non stretchable reinforcements decrease till the wall height reaches to 6 m. but; this becomes constant after the wall height of 6 m. American Highways Specification AASHTO advises to get K_r coefficient in lateral pressure calculation that will be obtained from Figure 7.23. (U.S Department of Transportation, 2001).

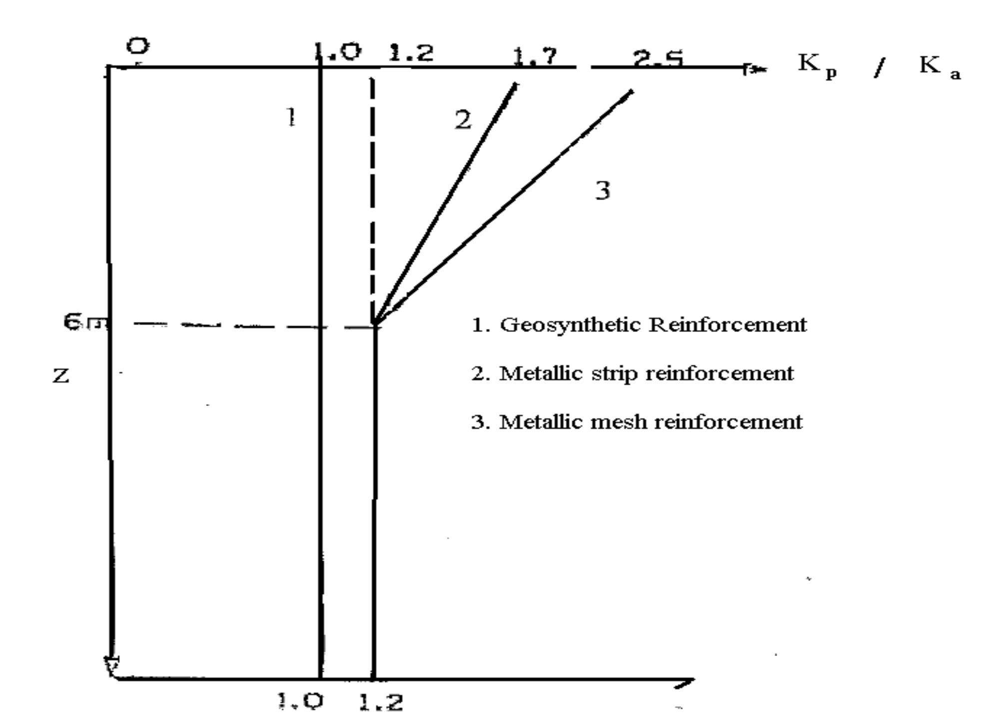


Figure 7.23. Lateral pressure coefficient ratios due to wall height (U.S Department of Transportation, 2001)

Lateral pressure coefficient that is required to be used in calculations (K_r) is defined as active pressure coefficient (K_a) as shown in Figure 7.24. Active lateral pressure coefficient can be found by Coulomb approach. In case zero wall friction and zero backfill angle (β), Ka value can be found by Equation (7.65)

$$K_a = \tan^2(45 - \frac{\phi'}{2}) \tag{7.65}$$

In cases that the angle of wall surface with horizontal is equal or higher than (θ) 8 degrees, Ka can be obtained with the equation below;

$$K_a = \frac{\sin^2(\theta + \phi')}{\sin^3\theta\left[1 + \frac{\sin\phi'}{\sin\theta}\right]^2} \tag{7.66}$$

The maximum load that will occur on system should be calculated by following the steps mentioned below.

$$\sigma_h = K_r \, \sigma_v \, + \Delta\sigma_h$$
$$\sigma v = \gamma_r \, Z + \sigma_2 + q + \Delta\sigma_v \qquad (7.67)$$

Here;

γ_r = unit weight of fill material

K_r = Obtained value from Figure 7.23. (In accordance with function $K_{(z)}$)

$K_r\ \sigma_y$ = Horizontal stress due to systems own weight

$\Delta\sigma_h$ = Possible additional horizontal surcharge load

$\gamma_r Z$ = Vertical stress due to soils own weight at depth Z

q = Possible uniform distributed load over the wall

σ_2 =Vertical stress due to possible fill over the wall

$\Delta\sigma_v$ = Possible additional vertical surcharge load

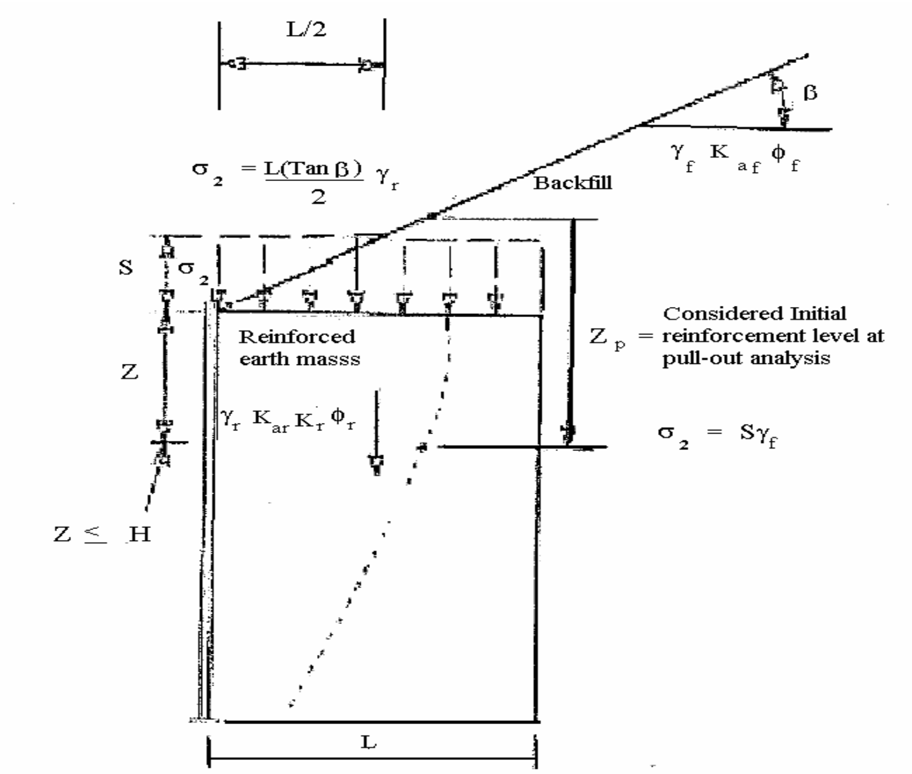


Figure 7.24. Calculation of lateral stress in case of sloped backfill

- In all reinforcement levels, maximum tension stress *(T_{max})* in lateral intervals S_v and reinforcement unit lateral intervals should be counted.

$$T_{\max} = \sigma_h S_v \tag{6.70}$$

During calculation of T_{max} stress in different reinforcement types like; geogrid, metallic strip and wire mesh, a lining ration named (R_c) is defined and this ratio is calculated by the equation seen below. This mentioned ratio equals to one "1"for all shroud reinforcements like geotextiles (U.S Department of Transportation, 2001).

$$R_c = b / S_h$$
$$T_{\max} = \frac{\sigma_h S_v}{R_c} \tag{6.71}$$

Here;

S_h	= Lateral interval between reinforcements
v	= Vertical intervals between reinforcements
R_c	= Lining ratio
B	= Gross weight of reinforced member

7.4.3. Rupture Check at Interior Stability

For rupture stability check , it is required that allowable tension bearing (T_{all}) must be higher than the maximum tension bearing mentioned in Section 7.4.2 (T_{max}) and at Equation (7.73). Safety factor against rupture in the mentioned equation as American Highways Specification is 1.3, as to TS 7994 this number is 2. T_{all} is calculated differently in strip reinforcements and in Geosynthetic reinforcements (U.S Department of Transportation, 2001)..

$$T_{all} = \frac{T_u}{GS_{rupture}} \geq T_{\max} \tag{7.73}$$

7.4.3.1. Allowable Tension Resistance Calculation of Metallic Strip Reinforcement. Allowable tension bearing capacity is calculated by Equation (7.74) (Kumbasar and Kip, 1999).

$$T_{al} = \frac{T}{GS_{rupture}} \qquad (7.74)$$
$$T = \sigma_e \times b \times d$$

Here;

T = Maximum reinforcement tension stress,

σ_e = Steel reinforcement tension resistance

b = Width of reinforcement

d = Reinforcement thickness as taking into consideration of corrosion

7.4.3.2.Allowable Tension Resistance Calculations of Geosynthetic Reinforcements. Allowable tension resistance can be determined by partial safety factor approach (Bonaparte and Berg, 19S7). Reduced coefficients (RF) are used in calculation of T_{all} in order to control a possible shrinking deformation in polymer and to spread geosynthetic material and to take into account of biological and chemical circumstances. Different reducing coefficients are advised to be used due to connection points. Long term tension resistance and allowable tension resistances of geosynthetics are calculated by the formula seen below (U.S Department of Transportation, 2001).

$$T_{all} = \frac{T_u}{GS_{rupture}} \qquad (7.75)$$

$$T_u = \frac{T_{ultimate}}{RF} \qquad (7.76)$$

$$RF = RF_{sh} + RF_{ly} + RF_{cv} \qquad (7.77)$$

Where;

T_{all} = Allowable tension resistance

T_u = Long term tension stress of geosynthetic reinforcement

$T_{ultimate}$ = Ultimate tension resistance of geosynthetic reinforcement

RF = Total reducing factor

RF_{sh} = Shrinking reducing factor

RF_{ly} = Laying reducing factor

RF_{cv} = Reducing factor against environmental effects (biological and chemical).

Shrinking reducing factor:

Typical values of this reducing factor as to polymer types which is advised due to possible shrinking features of geosynthetics is shown in Table 7.6. Real shrinking behavior of geotextile materials is determined by some tests as to ASTMD 5262.

Table 7.6. Shrinking Reducing Coefficients as to polymer types (U.S Department of Transportation, 1998)

Type of Polymer	Shrinking Reducing Factor
Polyester	2.5-2.0
Polypropylene	5-4 -1
Polyethylene	5-2.5

Laying Reducing Factor;

Laying reducing factor (RF_{ly}) is used because of damages that might occur during laying of geosynthetic reinforcements. It changes between 1.05 and 3. Determination of real laying factors of geosynthetic materials is found by some serial tests. Unit weight of 270g/m^2 geosynthetics are advised to be used in order to make the damage minimum during laying of geosynthetic (U.S Department of Transportation, 1998).

Reducing Factor Against Environmental Effects

Durability reducing factor depends on tension cracks, chemicals, micro-organisms and sensitivity against biological effects. Due to existence of various types, qualities and geometries of polymers, each type of geosynthetics has a different type of resistance against to environmental effects. This reducing factor changes between 1.1 and 2.

American Highways Specification says that this factor can be taken as 7 in case of obtaining the conditions mentioned below;

- Granular material should be used in fill (sand, gravel)
- PH value of backfill should be between 4.5 and 9
- Site temperature should be less than 30 degree
- Aggregate size in fill should be maximum 20mm
- Maximum wall height is 10 m.
- Used geosynthetics should have properties mentioned in AASHTO M 288 standards (U.S Department of Transportation, 1998).

7.4.4. Pull-Out Check

Below mentioned pull-out criteria should be obtained in order to prevent reinforcement to skim off from the fill (U.S Department of Transportation, 2001).

$$T_{\max} \leq \frac{1}{FS_{pull-out}} CF^{*} \gamma Z_p L_e C R_c \alpha \tag{7.78}$$

$FS_{pull-out}$ = Safety factor against pull-out (1.5),

T_{max} = Maximum tension stress occurred in reinforcements

C =Number of surfaces (for strips, geosynthetic and plates is 2)

α = Measurement correction factor (Table 7.8)

F’ =Friction coefficient (Table 7.8)

R_c = Number of Liner

γZ_p = Shroud pressure

L_e = Effected reinforcement length

Table 7.7. Measurement correction factors and friction factors as to American Specification (U.S Department of Transportation, 2001)

Type of reinforcement	**Assumed F***	**Assumed α**
Geogrid	0.8 tanφ	0.8
Geotextile	0.67 tanφ	0.6

Length of reinforcement in resistance zone (Le) can be defined by the explanation mentioned below. As to American Specification, mentioned length is asked to be bigger than 1m (U.S Department of Transportation, 2001).

$$L_e \geq \frac{1.5\,T_{\max}}{CF^{*}\gamma\, Z_p R_c \alpha} \geq 1m \tag{7.80}$$

In case of satisfying the conditions said in Equation 7.80 for each reinforcement level, horizontal intervals can be decreased, reinforcement with higher tension stress can be used or length of reinforcement can be increased adequately in order to decrease T_{max} (U.S Department of Transportation, 2001).

Total length of reinforcement L is calculated as to Equation (7.81)

$$L = L_a + L_e \tag{7.81}$$

Here;

L = Total reinforcement length,

L_a = Length of reinforcement left on the active zone (Figure 7.27.)

L_e = Effected reinforcement length

Below said definitions can be made for L_a as to Figure 7.27.

For reinforced earth retaining structures where there are vertical wall surfaces, horizontal wall backfill and stretchable reinforcement; formula below is used.

$$L_a = (H\text{-}Z) \tan (45\text{-}\varphi'/2) \tag{7.82}$$

For section between bottom of reinforced earth walls where non stretchable bars are used and H/2 level, below seen formula is used;

$$L_a = 0.6(H\text{-}Z) \tag{7.83}$$

For the zone above H/2 level that non stretchable reinforcement is used, below seen formula is used.

$$L_a = 0.3H \tag{7.84}$$

Longest reinforcements are advised to be used in each reinforcement levels during construction but in case of obtaining interior stability, it is possible to change reinforcement length gradely.

7.4.5. Interior Stability Analysis Methods under Dynamic Loading

In addition to present static loads affecting the system, inertial forces P_I that affects the system laterally is occurred in reinforced earth retaining structures during dynamic loading. (Figure 7.30.) Mentioned loading causes dynamic increases in maximum tension stress in reinforcements. It is assumed that location and slope of maximum tension forces line does not change during dynamic loading. An interior stability calculation step under dynamic loading is summarized below. Geometries and terms mentioned in these steps are shown in Figure 7.25. (U.S Department of Transportation, 2001).

Maximum acceleration occurred on the wall and inertial force P_I for unit width is calculated.

$$P_I = A_m W_A \tag{7.85}$$

$$A_m = (1.45\text{-}A)A \tag{7.86}$$

Here;

P_I = Inertial force,

A = Maximum acceleration coefficient

A_m = Maximum acceleration on the wall

W_A = Weight of active zone in reinforced earth structure

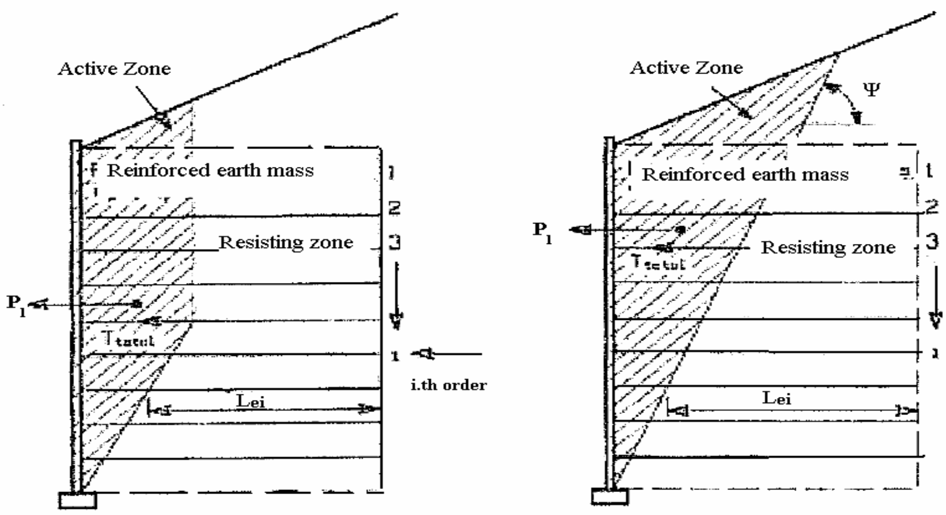


Figure 7.25. Inextensible & Extensible reinforcement conditions

Where,

L_{ei} = Active reinforcement length at order i

T_{max} = Maximum loading that affected to each reinforcement by static forces

T_{md} = Loads that applied on each reinforcement due to dynamic loads

- Maximum horizontal static force that affects reinforcements is calculated (T_{max}). Before this calculation, horizontal stress (σ_h) should be calculated.

$$\sigma_h = K\,\varsigma_v + \Delta\sigma_h = K\,\gamma\,Z + \Delta\sigma_h\,K + \Delta\sigma_h \tag{7.87}$$

$$T_{\max} = S_v \sigma_h \tag{7.88}$$

- T_{md} dynamic increment forces are calculated;

$$T_{md} = P\frac{L_{ei}}{\sum_{i=1}^{n}(L_{ei})} \tag{7.89}$$

- Maximum tension force is calculated as to formula shown below;

$$T_{total} = T_{\max} + T_{md} \tag{7.90}$$

Rupture and pull-out safeties of reinforcements should be checked again as to the limitation of "safety factor for dynamic loading should be minimum 75 per cent of allowable safety factor (U.S Department of Transportation, 2001).

In order to obtain rupture safety of steel reinforcements, the condition below seen is to be proven.

$$T_{al} \geq \frac{T_{total}\,(0.75)}{R_c} \tag{7.91}$$

For rupture safety in geotextile reinforcements, below mentioned controls are needed to be checked for static and dynamical controls.

For static case;

$$T_{\max} \leq \frac{S_{rs}\ R_c}{(0.75)\ RF \times FS_{rupture}} \tag{7.92}$$

For dynamic case;

$$T_{md} \leq \frac{S_{rt}\ R_c}{(0.75)\ FS_{rupture} \times RF_{ly} \times RF_{cr}} \tag{7.93}$$

Ultimate resistance of Geosynthetic;

$$T_{final} = S_{rs} + S_{rt} \tag{7.95}$$

For rupture safety of all reinforcements at dynamic case mentioned below condition should be satisfied (U.S Department of Transportation, 2001).

$$T_{total} \leq \frac{P_r\ R_c}{0.75 \times FR_s} = \frac{C(0.80F^*)}{0.75 \times 1.5} \times \gamma\ Z' \times L_e \times R_c \times \alpha \tag{7.96}$$

7.4.6. Simplified Interior Stability Analysis Method for Geosynthetic Reinforced Earth Retaining Walls

Method submitted for geosynthetic reinforcements by Robert M. Koerner in his book, named 'Design with Geosynthetic' is much quick and gives easier solution possibility compared to method advised in American Highways Specification (Afatoglu, 2004).

According to method advised by Koerner, design steps for geosynthetic reinforced retaining walls are defined below;

1. As to Rankine's formula after calculating Ka pressure coefficient , horizontal and vertical stresses formed by live and earth loads should be calculated with respect to depth "z ", as seen in the Figure 7.26.

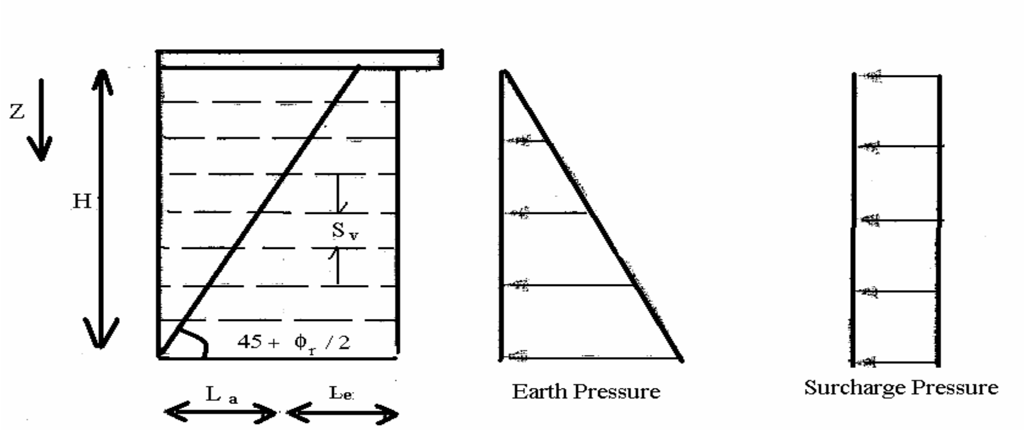


Figure 7.26. Interior stability investigations in Koerner Method (Das, 1987)

2. Allowable tension stress of geosynthetic (T_{all}) is determined by obtaining ultimate tension resistance of geosynthetic ($T_{ultimate}$) and reducing factors.

Robert M. M. Koerner in his book named Design with Geosynthetics' for geosynthetic reinforcement walls advises laying reducing factor RF_{ly} to be 1.1 to 2, shrinking reducing factor RF_{sh} to be 2 to 4, chemical effect reducing factor to be RF_{chem} 1 to 1.5, biological reducing factor RF by 1 to 1.3 (Afatoglu, 2004).

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

$$\sigma_h = K_a \times \sigma_v \tag{7.98}$$

$$T_{al} = \left[\frac{1}{RF_{ly} + RF_{sh} + RF_{chem} + RF_{sh}} \right]$$

3. Equation 7.100 is applied for different depths. First of all, it should be started from the depth of z =H and later on by reducing this definite amounts, required horizontal reinforcement intervals should be determined. $FS_{rupture.}$ is advised to be taken as 1.3 (Afatoglu, 2004).

$$S_v = \frac{T_{all}}{\sigma_h \, (FS_{rupture})} \tag{7.100}$$

4. Effective reinforcement length (Le) and active reinforcement length (La) is determined. Le is advised to be more than 3 m.

$$L_e = \frac{S_v \, \sigma_{nh} \, (FS_s)}{2(C + \gamma \, z \tan \delta)} \tag{7.101}$$

$$L_a = (H - z) \tan (45 - \frac{\phi}{2}) \tag{7.102}$$

Here, tanδ is said as “friction coefficient between geosynthetic and the fill”.

5. L_o bending length should be calculated. The minimum value of L_o is 1 m. If the calculated value is less than 1m., and then it is assumed to be 1 m.

$$L_o = \frac{S_v \, \sigma_{ult} \, (FS_s)}{4(C + \gamma \, z \tan \delta)} \tag{7.103}$$

8. DESIGN OF A MODEL REINFORCED EARTH RETAINING WALL

8.1. Introduction

12 meter height reinforced earth retaining wall is going to be constructed on a second degree earthquake zone. Reinforced earth retaining wall is being constructed at toe zone of an unstable natural slope. Aim of the construction is to increase the stability of the present slope and to gain the area that is needed for the highway that passes through the toe zone.

In this section, assuming that the mentioned structure is going to be constructed in geometry shown in Figure 8.1.; design has been carried out under static and dynamic loadings as to specifications of U.S Department of Transportation.

Formulas and conditions mentioned in analysis which has been given in Section 7. were repeated where they were used; and referred to section number that presented earlier. Final situation of the site after proposed construction is shown in Figure 8.2.

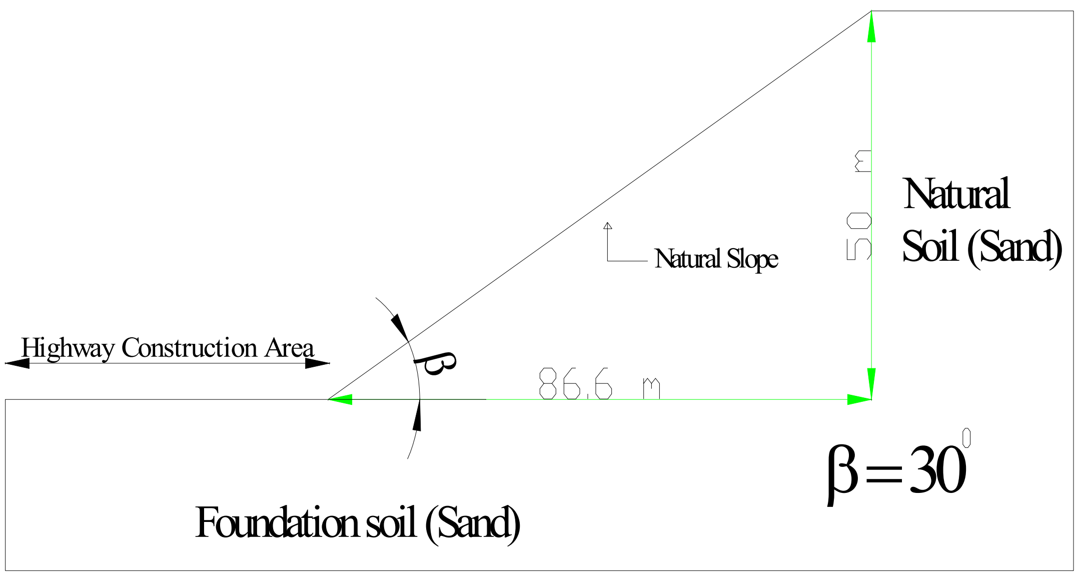


Figure 8.1. Initial geometry of the natural slope

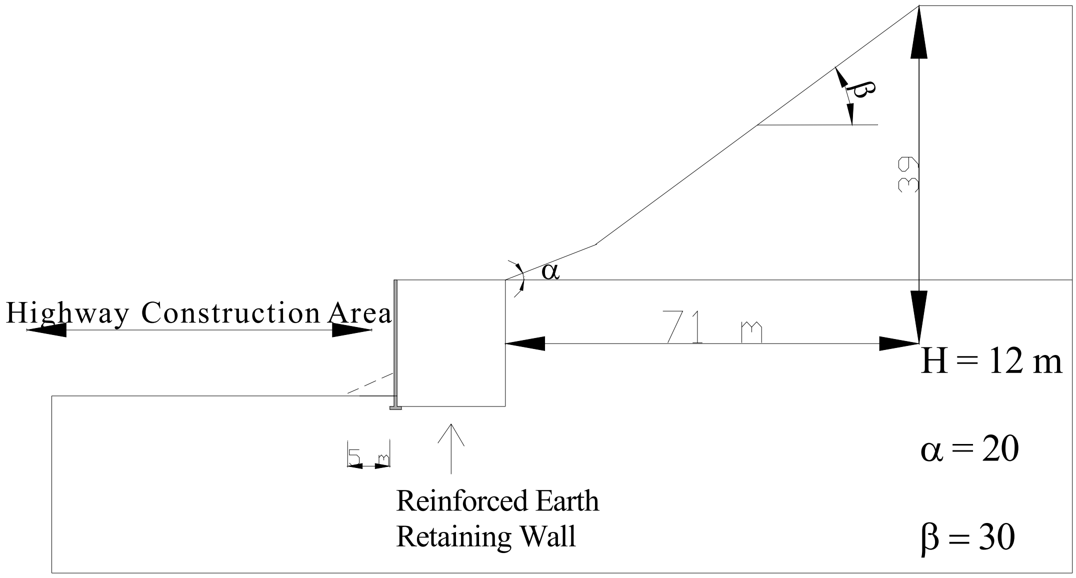


Figure 8.2. Proposed geometry for construction

8.2. Properties of Natural Soil and Selected Fill Material

Engineering parameters of Natural soil and selected fill material are represented in Table 8.1.

Table 8.1. Properties of natural soil and fill material

	Property	Symbol (Turkish literature)	Symbol (USA literature)	Value	Unit
Natural Soil	Soil interior friction angle	φ	φ_f	28	^{0}
	Cohesion	c	c_f	7	kN/m^2
	Modulus of elasticity	E	E	60000	kN/m^2
	Saturated unit weight	γ_d	γ_f	20	kN/m^3
	Natural dry unit weight	γ_n	γ_f	19,50	kN/m^3
	Horizontal permeability	k_x	k_x	1	m/s
	Vertical permeability	k_y	k_y	1	m/s
	Possion ratio	ν	ν_f	0.4	
Fill Material	Soil interior friction angle	φ	φ_r	40	^{0}
	Cohesion	c	c_r	0	kN/m^2
	Modulus of elasticity	E	E	100000	kN/m^2
	Saturated unit weight	γ_d	γ_f	20,50	kN/m^3
	Natural dry unit weight	γ_n	γ_f	20	kN/m^3
	Horizontal permeability	k_x	k_x	1	m/s
	Vertical permeability	k_y	k_y	1	m/s
	Possion ratio	ν	ν_r	0.4	

8.3. Properties of Reinforcement

In reinforced earth construction, materials produced by “Tensar Earth Technologies” company with codes ‘Structural Geogrid UX 1600 MSE’ and ‘Structural Geogrid UX 1700 MSE’ have been used.

8.4. Properties of Facing Unit

On the facing of reinforced earth structures, mono-blocks of 40 cm thick reinforced concrete structural members that their heights can be arranged according to reinforcement levels have been used.

8.5. Total Collapse Analysis for Natural Slope

Reason for constructing reinforced earth retaining wall on toe area of natural slope is to increase the safety against total collapse of the slope. For this reason, first of all it should be checked whether the natural slope has the required collapse safety against total failure.

In the current system (natural slope), it is very difficult to define where critical sliding circle passes that causes total collapse. So, it will be more suitable to perform total collapse analysis by computer programs for better results.

In section 10 of this study, total collapse analysis of the natural slope will be performed with Plaxis software under the title of “Model-1”. Also at the same section, total collapse analysis will be repeated with slice method as to the critical sliding circle obtained by Plaxis analysis.

Total collapse analysis results achieved by Swedish Slices Method in Equation (8.1) for a potential sliding circle passes through toe zone of the reinforced earth structure are shown in Figure 8.3. and in Table 8.2.

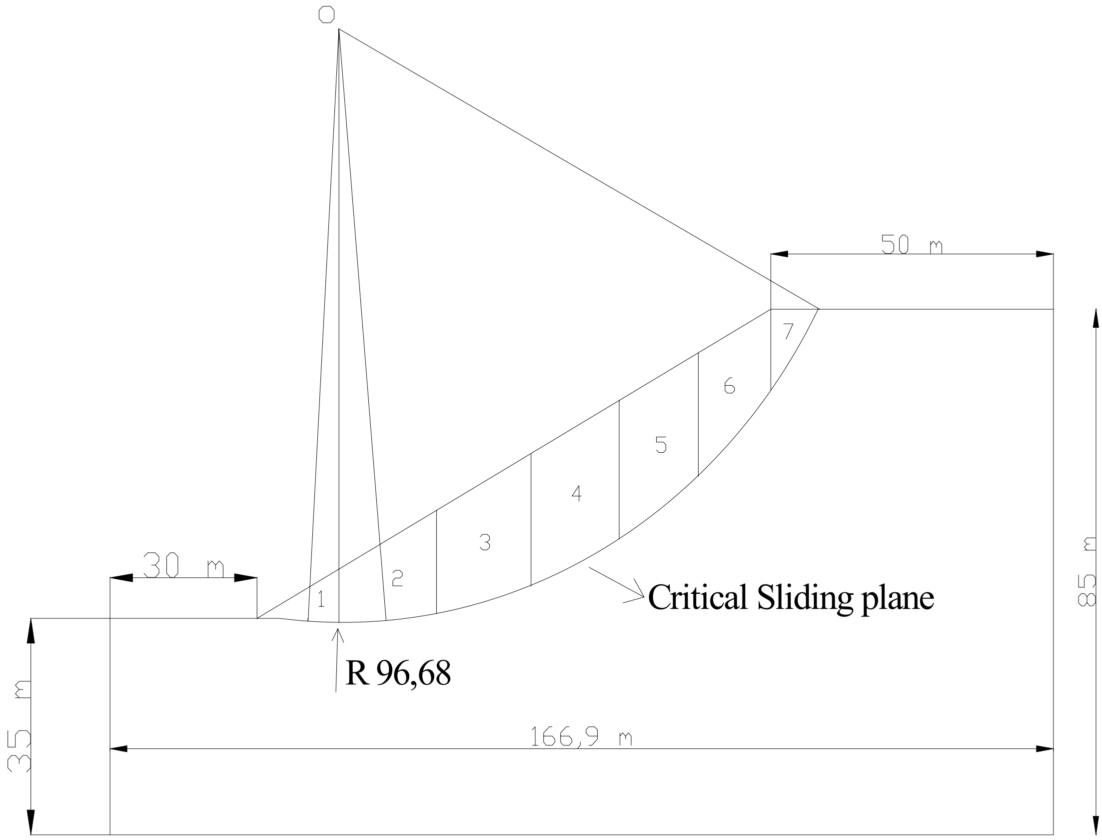


Figure 8.3. Calculation slices and assumed sliding circle

$$FS_{total} = \frac{\sum W(\cos a) \times \tan \phi_f + c \times l}{\sum W(\sin a)} \tag{8.1}$$

Here;

W = Weight of sliding circle (kN)

α = Angle of slice midpoint with the vertical line passes through the center of failure circle.

φ_f = Interior friction angle of natural soil (35^0)

c = Cohesion

I = Arc length of sliding circle (m)

FS_{total} = Factor of safety for total collapse

R = Radius of Sliding circle

Calculations for each slice are summarized in Table 8.2.

Table 8.2. Calculation table for total collapse of natural slope

Slice No	Slice Area(m2)	a(°)	W(kN)	Wsina (kN)	Wcosa (kN)
1	52	-8	1008	-140	998
2	179	4	3481	242	3472
3	272	16	5305	1462	5099
4	328	25	6367	2691	5770
5	337	36	6591	3874	5332
6	279	49	5441	4106	3569
7	78	56	1521	1260	851
Total				13495	25091

$$FS_{total} = \frac{\sum W(\cos a) \times \tan \phi_f + c \times l}{\sum W(\sin a)} = \frac{25091 \times \tan 35 + 0}{13495} = 1!$$

FS_{total} = 1! – Failure condition

Decision: Safety factor of total collapse analysis for selected sliding circle has been found at collapse limit. So there is a risk of total collapse for selected sliding circle.

Total collapse stability, does not satisfies the required safety for selected sliding circle. Between the sections 8.6 - 8.12, by constructing a reinforced earth structure; it has been worked to increase the slope stability. Also at section ten of this study, total collapse analysis is also performed by Plaxis finite element software.

8.6. Preliminary Design

8.6.1. Determination of Initial Length of Reinforcement

For determining initial reinforcement length, according to American Highways Specification as said in Section 7.3.3.1., below seen equation should be satisfied.

$$L = 0.7H \geq 2.5 \text{ m.}$$

$$H=12 \text{ m} \Rightarrow L=8.4 \text{ m.}$$

Decision: It is decided to start to calculations with L = 9 m.

8.6.2. Determination of Height of Toe Fill

As to definitions and conditions mentioned in Section 7.3.3.1., height of toe fill "D" for steep reinforced earth walls, should be bigger than H/20 and 0.5m.

$$D=H/20 \geq 0.5$$

$$H=12 \text{ m} \Rightarrow D=0.6 \text{ m.}$$

Decision: It is decided to take D=l m.

8.6.3. Determination of Width of Toe Fills

As to definitions and conditions given in Section 7.3.3., toe fill width 'B' should be bigger than 1.2 m. for reinforced earth walls.

Decision: In order create a working space inside the wall it is asked to B=5

8.7. Exterior Stability Calculations of Model Reinforced Earth Retaining Wall

8.7.1. Calculation of Lateral Pressure Coefficient for Selected Fill Material

Lateral active pressure coefficient that used in reinforcement for selected fill material is calculated as below mentioned formulas in Section 7.3.4. and in Figure 7.11.

Here;

θ = Angle of wall surface with horizon = *90° < 98°* (Steep wall)

δ = Wall friction angle= 0°

β = Surcharge slope = 0°

φ' = Effective interior friction angle = *40°*

For steep walls;

Equation (7.2) => $K_a = \tan^2 (45 - \phi/2)$ and $K_{ar} = K_a$ => $K_{ar} = 0.36$

8.7.2. Calculation of Lateral Pressure Coefficient for Natural Soil

Lateral active earth pressure coefficient that used at back of the slope and foundation is calculated with respect to tables and definitions mentioned in Section 7.3.4 and in Figure 7.11.

θ = Angle of wall surface with horizon = *90° < 98°* (Steep wall)

δ = Wall friction angle= 0 degree

β = Surcharge slope = 20 degrees

φ' = Effective interior friction angle = *28°*

For steep walls;

Equation (7.3) => $$K_a = \cos\beta \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \right] = K_a = K_{af} = 0.321$$

8.7.3. Calculation of Static External Loads

Calculation of external static loads affecting on system due to its weight in accordance with definitions at section 7.3.4. and Table 8.4. are as follows;

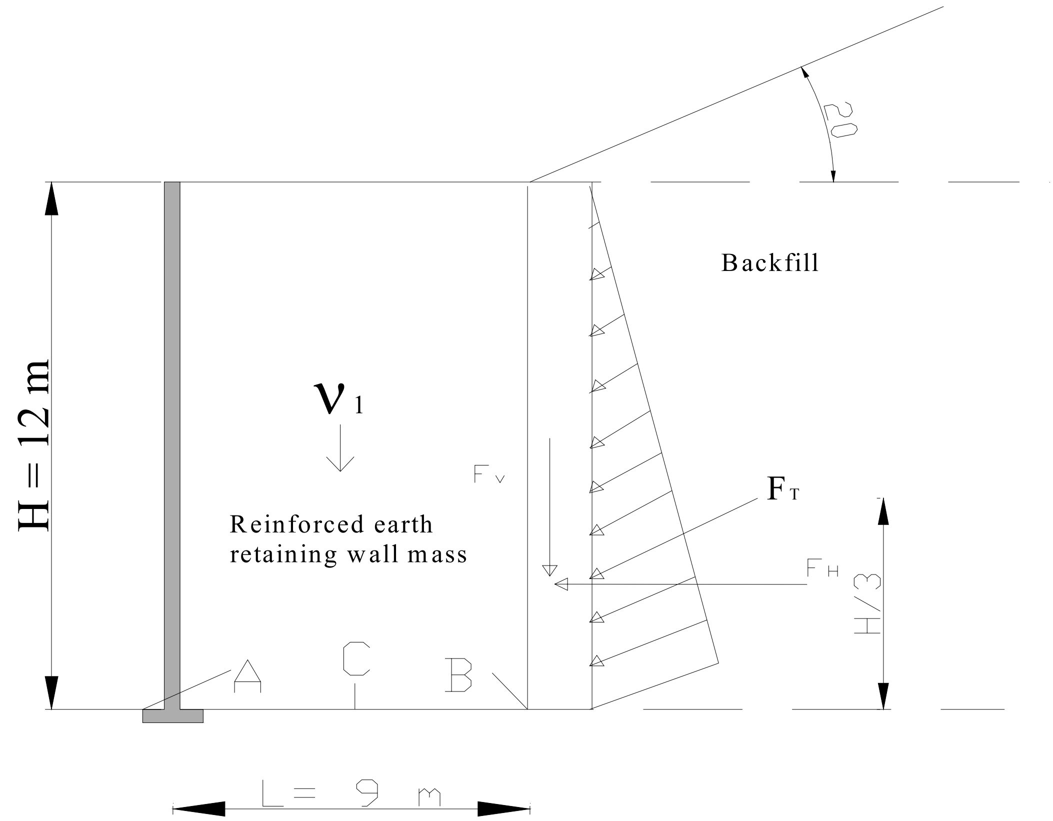


Figure 8.4. External loads that affect system

Reinforced earth retaining wall mass (V_1):

Figure 7.12. => $V_1 = 2160$ kN/m

Resultant force (F_T) :

Equation (7.5) => $F_T = 451$ KN/m

Resultant vertical force (F_v):

Figure 7.13. => $F_v = 154$ KN/m

Resultant horizontal force (F_H):

Figure 7.14. => $F_H = 424$ KN/m

8.7.4. Calculation of Eccentricity and Base Pressure under the Wall

As it is mentioned in section 7.3.5., according to Meyerhof bearing capacity approach, moment of resultant force (R) as to the point "C" should be equal to the moment of exterior forces on system as to the same point.

Equation (7.6)=> $e = \dfrac{F_T(\cos\beta)\,h/3 - F_T(\sin\beta)L/2}{V_1 + F_T \sin\beta} \Rightarrow e = 0.43\,m$

Meyerhof base pressure is calculated with Equation (7.7) as follows;

Equation (7.7) => $\sigma_v = \dfrac{V_1 + F_T \sin\beta}{L - 2e} = 284\,kN/m^2$

8.7.5. Bearing capacity check

Bearing capacity check in accordance with definitions at section 7.3.7. and Equation (7.13) is mentioned below. According to FHWA regulations, it is required for obtained bearing capacity value to be more than 2.5.

Equation (7.13) => $\sigma_v \le q_{all} = \dfrac{q_{ult}}{FS_{b.cap}}$

Equation (7.14) => $q_{ultimate} = c_f\, N_c + 0.5(L)\gamma_f\, N_{\gamma f}$

For $\Phi = 28^0$ => $N\gamma = 48.03$ => $q_{ultimate} = 4214kN/m^2$ and $q_{all} = 1685\ kN/m^2$

Check: $\sigma_v = 284\ kN/m^2 < q_{all} = 1685\ kN/m^2$

Decision: Dimensions of Reinforced earth wall are sufficient in the meaning of bearing capacity safety.

<u>8.7.5.1. Eccentricity Check.</u> Eccentricity control, in accordance with definitions at section 7.3.5. and Equation (7.6) is mentioned below. According to FHWA regulations, it is required for obtained eccentricity to be less than L/6 for non-rocky soils.

Equation (7.6) => e = 0.43 < L/6 = 1.5m

Decision: Dimensions of Reinforced earth wall are sufficient in terms of eccentricity safety.

<u>8.7.5.2. Sliding Check.</u> Wall dimensions that have been found in preliminary design should be checked at base which is most critical area against sliding. In order to maintain the sliding stability, below mentioned conditions in Equation (7.8) must be satisfied.

Equation (7.8) $\Rightarrow FS_{sliding} = \dfrac{\sum horizontal\ resisting\ forces}{\sum horizontal\ sliding\ forces} = \dfrac{\sum P_R}{\sum Pd} > 1.5$

Equation (7.11) $\Rightarrow \sum P_R = F_H = F_T \cos\beta = 424\, kN$

Equation (7.12) $\Rightarrow \sum Pd = (V_1 + F_T \sin\beta)\,\mu = 1144\, kN$

$\mu = \min[\tan\phi_f, \tan\phi_r] = \tan 28 = 0.53$

Equation (7.8) $\Rightarrow FS_{sliding} = 2.7 > 1.5$

Decision: Dimensions of reinforced earth structure is sufficient as to safety of static sliding.

8.7.6. Overturning Check

Checking of reinforced earth structures against over turning is a check of moment equilibrium as seen in Figure 7.14. In this check it is asked to maintain that the ratio of total resisting moments (ΣM_R) to the driving moments (*ΣM_0*) as to toe I bigger than $FS_{overturning}$ by Equation (7.16).

$$\text{Equation (7.16)} \Rightarrow FS_{overturning} = \frac{Sum\ of\ the\ moments\ that\ resist\ to\ overturning}{Sum\ of\ the\ moments\ that\ lead\ to\ overturning} \geq 2$$

$$\text{Equation (7.17)} \Rightarrow \sum M_R = (F_T \sin\beta)L + V_1(L/2) = 11106kNm$$

$$\text{Equation (7.18)} \Rightarrow \sum M_0 = (F_T \cos\beta) \times (h/3) = 1696\,kNm$$

$$FS_{overturning} = 6.55 > 2$$

Decision: Dimensions of reinforced earth structures are sufficient as to static over turning.

8.8. Exterior Stability Analysis under Dynamic Loading

As to calculations and checks achieved in section 8.7., it is confirmed that H=12m and L = 9m wall dimensions are sufficient for safety of static conditions.

In this section, sufficiency of these dimensions from dynamic loading point of view has been evaluated. During this evaluation, it has been referred to the equations and definitions in Section 7.3.10.1. After seismic calculations, total collapse, eccentricity and sliding checks are repeated as to American Highways Specification. Safety factor required in these checks should be 75 per cent of the one in static case. In calculations; it was assumed that the structure is constructed in second degree earthquake zone.

8.8.1. Calculation of Peak Acceleration

Calculation of peak acceleration that affects the system is done with Equation (7.22).

$$\text{Equation (7.22)} \Rightarrow A_m = (1.45\text{-}A)\ A$$

As it is clear in Equation (7.22), A_m depends on maximum base acceleration coefficient ‘A’. According to Table 7.6., maximum base acceleration coefficient A (A_o) should be 0.3g since the structure lays in second degree earthquake zone.

Table 7.6. $\Rightarrow$ Second degree earthquake zone $\Rightarrow$ A=0.3g

Equation (7.22) $\Rightarrow$ $A_m = 0.345$

8.8.2. Calculation of Dynamic Forces

During an earthquake, in addition to present static forces, there occurs a dynamic impact (P_{ae}) at the back of the fill of reinforced earth structure. In addition to that, reinforced earth mass is affected by inertial force $P_{IR} = M\ A_m$. Here, M is the weight of the active mass in the reinforced earth that is assumed to have a base width of 0.5H, and A_m is the maximum acceleration at reinforced earth as seen from Figure 8.5.

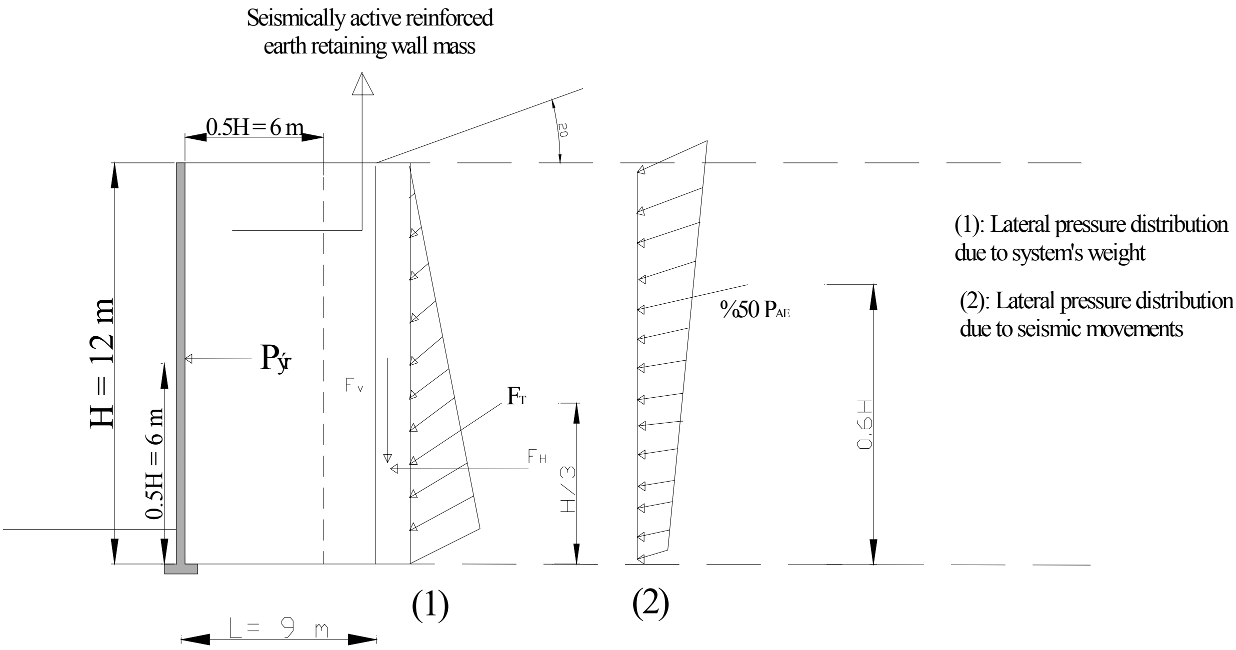


Figure 8.5. Calculation of exterior loads affecting the system due to seismic movement

$$\text{Equation (7.23)} \Rightarrow \quad P_{IR} = 0.5\, A_m\, \gamma_r\, H^2 = 496\, kN/m$$

$$\text{Equation (7.23)} \Rightarrow \quad P_{AE} = 0.5\, \gamma_f\, (H_2)^2\, K_{AE} \ \text{(sloped backfill)}$$

Here: K_{AE} is total seismic pressure coefficient and calculated with Equation 7.30

Equation (7.30) $\Rightarrow K_{AE} = 0.64$

$\Rightarrow P_{AE} = 898$ kN/m

8.8.3. Sliding Check under Dynamic Loading

Adding the effects of dynamic loads in Section 8.6.2. to the static loads calculated in static sliding check, the sliding check under dynamic loading is done as shown below.

For dynamic case;

$$\sum P_R = F_H + P_{IR} + 0.5 P_{AE} \times \cos 20 = 1341\, kN$$

$$\sum Pd = (V_1 + F_T \sin\beta + 0.5 P_{AE} \times \sin 20)\,\mu = 1431\, kN$$

$$\sum FS_{sliding} = \frac{\sum horizontal\ resisting\ forces}{\sum horizontal\ sliding\ forces} = \frac{\sum P_R}{\sum Pd} \geq (1.5 \times 0.75) = 1.125$$

$FS_{sliding} = 1.07 < 1.125$ (75 per cent of $FS_{sliding}$ at static case)

Decision: System does not comply with required safety values under dynamic loading against sliding. For this reason, its resistance against sliding must be increased. Length of reinforcement of 9 m. should be increased and check should be repeated.

8.8.4. Eccentricity Check

Eccentricity formula by adding the dynamic loads that affects to Equation (7.6) has been found as shown below. Eccentricity calculated by Equation (8.2) is asked not to be greater than L/3 (3 m) in non rocky soils.

Equation (8.2) $\Rightarrow$

$$e = \frac{F_T(\cos\beta)H/3 + 0.5P_{AE}(\cos\beta)0.6H + P_{IR}\,0.5H - F_T(\sin\beta)L/2 - 0.5P_{AE}(\sin\beta)L/2}{V_1 + F_T\sin\beta + 0.5P_{AE}(\sin\beta)}$$

e = (6325/2467) = 2.65< 3 m

Decision: Dimensions of reinforced earth walls are adequate for safety under dynamic loading as to eccentricity.

8.8.5. Bearing Capacity Check

Bearing capacity check is achieved by Equation (8.2) seen below under dynamic loading.

Equation (8.2) $\Rightarrow$

$$\sigma_v = \frac{V_1 + F_t\sin\beta + 0.5P_{AE}(\sin\beta)}{L-2e} \Rightarrow \sigma_v = 666\,kN/m^2$$

Allowable base pressure stress (q_{al}) was calculated before as 1685 kN/m^2

$$\sigma_v = 666 kN/m^2 < 1685 kN/m^2$$

Decision: Dimensions of reinforced earth walls are adequate as to bearing capacity safety under dynamic loading.

8.8.6. Evaluation of Stability Results under Static and Dynamic Loading.

Checks of the wall H=12m and L=9m for static and dynamic loading are preformed in Section 8.5. and 8.6. and summarized in Table 8.3. As can be seen in the table, wall dimensions are not enough to satisfy the required safety against sliding in dynamic loading case. So, for this reason, there should be some precautions against wall sliding risk.

Table 8.3. Results of exterior stability analyses under static and dynamic loading for the wall of H=12 m. and L=9 m.

Controls	Value	Acceptance condition	Result
Eccentricity control	e = 0.43m	<L/6=1.5 m.	Dimensions are adequate
Bearing capacity check	σ_v = 284 kN/m^2	σ_v<q_{all}=1685kN/m^2	Dimensions are adequate
Sliding check	$FS_{sliding}$ = 2.7	>1.5	Dimensions are adequate
Over turning check	$FS_{overturning}$ = 6..55	>2	Dimensions are adequate
Seismic sliding	$FS_{sliding}$ = 1.07	<1.125	**Dimensions should be increased**
Seismic Bearing Capacity	σ_v=666 kN/m^2	σ_v<q_{all}=1685kN/m^2	Dimensions are adequate
Seismic eccentricity control	e=2.65m	<L/3=3 m.	Dimensions are adequate

Decision: For this design, solution has been found by increasing the reinforcement length. Instead of 9 m., 11 m. reinforcement length should be used and the sliding check should be repeated. This increase in reinforcement length will cause an increase in weight of reinforced earth mass and consecutively in resistance against sliding.

8.9. Exterior Stability Calculations for Model Wall of L=11 m

For the wall of H= 12m and L= 11m, sliding check under dynamic loading should be repeated where 9 m length of reinforcement does not satisfy safety conditions.

8.9.1. Sliding Check under Dynamic Loading (L=11m)

For the dynamic loading case where L = 11 m

$$\sum P_R = F_H + P_{IR} + 0.5P_{AE} \times \cos 20 = 1701\,kN$$

$$\sum Pd = (V_1 + F_T \sin\beta + 0.5P_{AE} \times \sin 20)\,\mu = 1341\,kN$$

$$\sum FS_{sliding} = \frac{\sum horizontal\ resisting\ forces}{\sum horizontal\ sliding\ forces} = \frac{\sum P_R}{\sum Pd} \geq (1.5 \times 0.75) = 1.125$$

$FS_{sliding}$ = $1.16 > 1.125$ (The 75 per cent of FOS against sliding for static case)

Decision: Safety against sliding at dynamic loading is satisfied by increasing the length of reinforcement by 2 meters and having a total reinforcement length of 11m.

8.9.2. Evaluation of Results

All of the analysis for the wall L = 19m are repeated in accordance with the definitions at Section 8.5. and 8.6. for the wall of L = 11m. Obtained results are shown in Table 8.4.

Table 8.4. Results of exterior stability for static and dynamic loading for the wall of H=12m and L=11 m

Controls	Value	Acceptance condition	Result
Eccentricity control	e=0.30m	<L/6=1.83 m.	Dimensions are adequate
Bearing capacity check	σ_v =268 kN/m^2	σ_v<q_{all}=1685kN/m^2	Dimensions are adequate
Sliding check	$FS_{sliding}$ =3.82	>1.5	Dimensions are adequate
Over turning check	$FS_{overturning}$= 8.66	>2	Dimensions are adequate
Seismic sliding	$FS_{sliding}$=1.16	<1.125 (1.5*0.75)	Dimensions are adequate
Seismic Bearing Capacity	σ_v = 392kN/m^2	σ_v < q_{all} = 1685 kN/m^2	Dimensions are adequate
Seismic eccentricity control	e=2.36m	<L/3=3.66 m.	Dimensions are adequate

Decision: Safety against all types of failures at dynamic and static loading cases is satisfied by increasing the length of reinforcement by 2 meters and having a total reinforcement length of 11m. Decided dimensions of H = 12m and L = 11m should be investigated by total collapse analysis which will be performed at the end of the interior stability analysis.

8.10. Static Interior Stability Calculations

8.10.1. Introduction to Interior Stability Calculations

Reinforced earth wall geometry has been decided to be as seen in Figure 8.6. after exterior stability calculations with respect to static and dynamic loading. At interior stability calculations, by determining lateral reinforcement intervals, reinforcement length and adequacy of its strength will be checked.

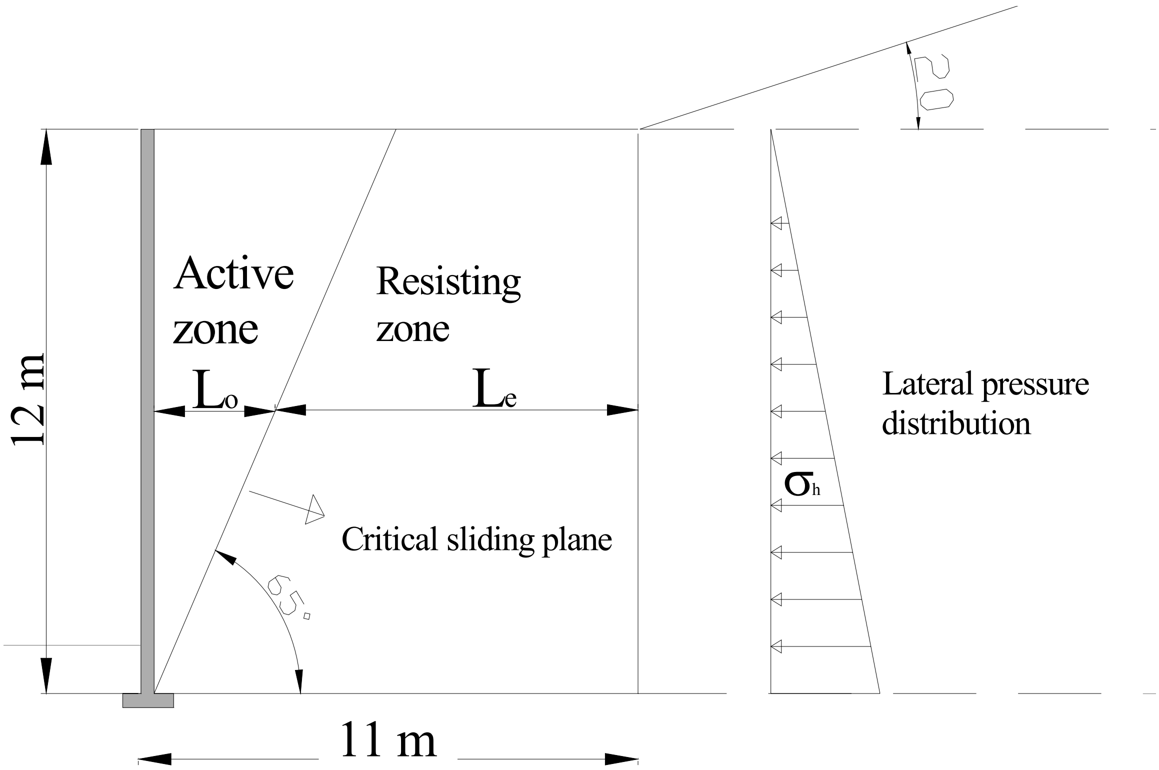


Figure 8.6. Dimensions of reinforced earth structures for interior stability analysis

Details of rupture and pull-out checks under static and dynamic loading have been explained in Section 7.4. During analysis, referred equation and section is mentioned.

In all interior stability calculations, American Highways Specification has been taken into account.

In construction of reinforced earth structure, materials with code 'Structural Geogrid UX1600 MSE' and 'Structural Geogrid UX1700 MSE’ were used. Properties of reinforcement are shown in Table 8.5.

Table 8.5. Geogrid properties used in reinforced earth structures

Property	UX1600 MSE	UX 1700 MSE	Unit
Elasticity modulus	1750	2350	kN/m^2
Allowable long term tension resistance inside sand (T_u)	59.9	75.1	kN/m^2
Allowable tension bearing for long term uniformly graded sand.(T_u)	58.2	73	kN/m^2

In specification of U.S Department of Transportation, safety factor against pull-out is required to be at least 1.5 and safety factor against rupture is to be 1.3. In addition to that, in dynamic case, allowable tension stress of the reinforcement should be equal to 75 per cent of the total tension stress of the reinforcement.

8.10.2. Calculation of Horizontal Pressure Coefficient

As to American Highways Specification, K_r in Figure 7.27. should be used in calculation of lateral pressure coefficient.

$$\text{Figure 7.27} \Rightarrow K_a / K_r = 1 \Rightarrow K_a = K_r$$

$$\text{Equation (7.66)} \Rightarrow K_a = K_r = \tan^2(45 - \phi'/2) = 0.36$$

8.10.3. Calculation of Horizontal and Vertical Stresses

Horizontal and vertical stress values are calculated as a function of depth ‘z’ in Equation (7.68) and (7.69).

$$\text{Equation (7.69)} \Rightarrow \sigma_v = \gamma_r Z = 20 \times z = 20z$$

$$\text{Equation (7.68)} \Rightarrow \sigma_h = K_r \sigma_v = 20 \times 0.217 \times z = 4.34\,z$$

8.10.4. Calculation of Allowable Tension Resistance

As explained in Section 7.4.4.2., allowable reinforcement tension resistance is calculated by Equation 7.76-7.78. Since long term tension stress in sand (T_u) is known for geogrid material (UX 1600 MSE) that used in Table 8.5., there is no need to use reduction factors for geogrid.

Equation (7.66) $\Rightarrow$ $$T_{\max} = \frac{T_u}{FS_{rupture}} = (59.6 / 1.3) = 46.07 kN / m$$

8.10.5. Determination of Vertical Intervals of Reinforcement and Static Rupture Check

Here first of all, it is asked that the allowable reinforcement tension resistance (T_{all}) should be greater than maximum allowable tension pressure resistance (T_{max}) (Equation 7.73). Safety factor against rupture in this equation is 1.3 as to American Highways Specification. By this control, safe vertical spacing between reinforcements is determined.

Equation (7.73) $\Rightarrow$ $$T_{all} = \frac{T_u}{FS_{rupture}} > T_{\max}$$

Firstly T_{max} is calculated by Equation (7.70) – (7.72)

Equation 7.71 $\Rightarrow$ R_c = b/S_h (It is advised to get 0.8 as value of R for geogrid reinforcements)

$$R_c = 0.8.$$

Equation (7.72) $\Rightarrow$ $$T_{\max} = \frac{\sigma_h S_v}{R_c} \Rightarrow 46.07 = \frac{4.34z \times S_v}{0.8} \Rightarrow S_v = \frac{36.86}{4.34z}$$

Maximum depth for S_v= 1 meter

$$1 = \frac{36.86}{4.34z} \Rightarrow z = 8.46\,m$$

Maximum depth for S_v= 0.5 meter

$$0.5 = \frac{36.86}{4.34z} \Rightarrow z = 16.98\,m$$

Decision: It is decided that first 13 orders of geogrid are placed by 0.5m intervals and followed by placement of 5 geogrids by 1m intervals and totally 18 orders of reinforcements as seen in Figure 8.7.

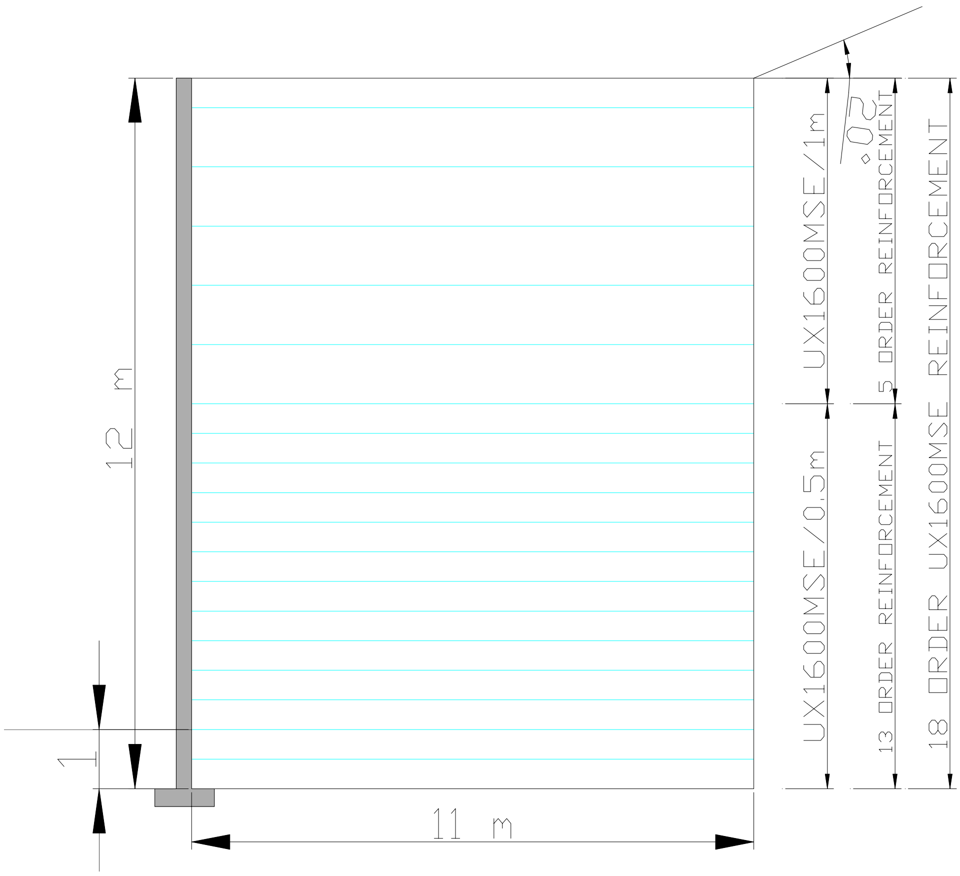


Figure 8.7. Geogrid reinforcement placements on reinforced earth structures

8.10.6. Control of Effective Reinforcement Length and Static Pull-Out

Details of effective reinforcement length and static pull-out controls are explained in Section 7.4.5. As said in that section, in order reinforcement not to pull out from the fill, required reinforcement length for satisfying the condition said in Equation (7.79) should be used.

$$\text{Equation (7.79)} \Rightarrow T_{\max} \leq \frac{1}{FS_{pull-out}} F^{*} \gamma\, Z_{p}\, L_{e} C R_{c} \alpha$$

Here;

$FS_{pull\text{-}out}$	= Safety value against pull-out =1.5,
T_{max}	= Maximum tension stress in reinforcement = 4.34z,
C	= Number of surfaces = 2,
F	= Friction coefficient = 0.67 (Table 7.8),
α	= Factor of measuring correction = 0.8 (Table 7.8),
R_c	= Number of lining = 0.8,
γZ_p	= Lining pressure = 20z (It includes distributed static loads but excludes dynamic loads like traffic loads), Figure 7.30
L_e	= Effective reinforcement length

When the above mentioned values are placed into Equation (7.79)

$$L_e = \frac{6.51 S_v}{17.15} = 0.38 S_v$$

Moreover, the reinforcement length inside the active region (L_o) is calculated by Equation (7.82).

$$\text{Equation (7.82)} \Rightarrow L_a = (H\text{-}Z) \tan(45 - \varphi'/2) = (12\text{-}z)*0.47$$

Above mentioned L_a and L_e equations are calculated for each reinforcement level at Table 8.6.

Table 8.6. Control of effective reinforcement length and static pull-out check

Reinforcement Level	Depth (m)	Spacing (S_v) m	L_e required	L_e min (m)	L_a (m)	L_e existing (m)	L (m) Decision
16	0.5	0.75	0.28	1	5.41	5.6	11
15	3.5	1	0.38	1	4.00	7.01	11
14	4.5	0.88	0.33	1	3.53	7.48	11
13	5.5	0.75	0.28	1	3.06	7.95	11
12	6	0.75	0.28	1	2.82	8.18	11
11	6.5	0.75	0.28	1	2.59	8.42	11
10	7	0.75	0.28	1	2.35	8.65	11
9	7.5	0.75	0.28	1	2.12	8.89	11
8	8	0.75	0.28	1	1.88	9.12	11
7	8.5	0.75	0.28	1	1.65	9.36	11
6	9	0.75	0.28	1	1.41	9.59	11
5	9.5	0.63	0.24	1	1.18	9.83	11
4	10	0.5	0.19	1	0.94	10.06	11
3	10.5	0.5	0.19	1	0.71	10.3	11
2	11	0.5	0.19	1	0.47	10.53	11
1	11.5	0.75	0.28	1	0.24	10.77	11

Decision: As it is apparently seen from table, investigated reinforcement length L= 11 is able to satisfy the L_e effective reinforcement length for pull-out safety at each reinforcement level. As a result, determined dimension and reinforcement distribution for interior stability is statically safe.

8.10.7. Interior Stability Check under Dynamic Loading

In case of dynamic loading, in addition to present static loads, P_I inertial force that affects the system horizontally occurs in reinforced earth retaining structure as can be seen in Figure 7.31. and Figure 8.8. Mentioned loads cause dynamic increases in maximum tension stress at reinforcement. It is assumed that the slope and location of maximum tension force line are not changed during dynamic loading. Stability checks under dynamic loading are explained in Section 7.4.6.

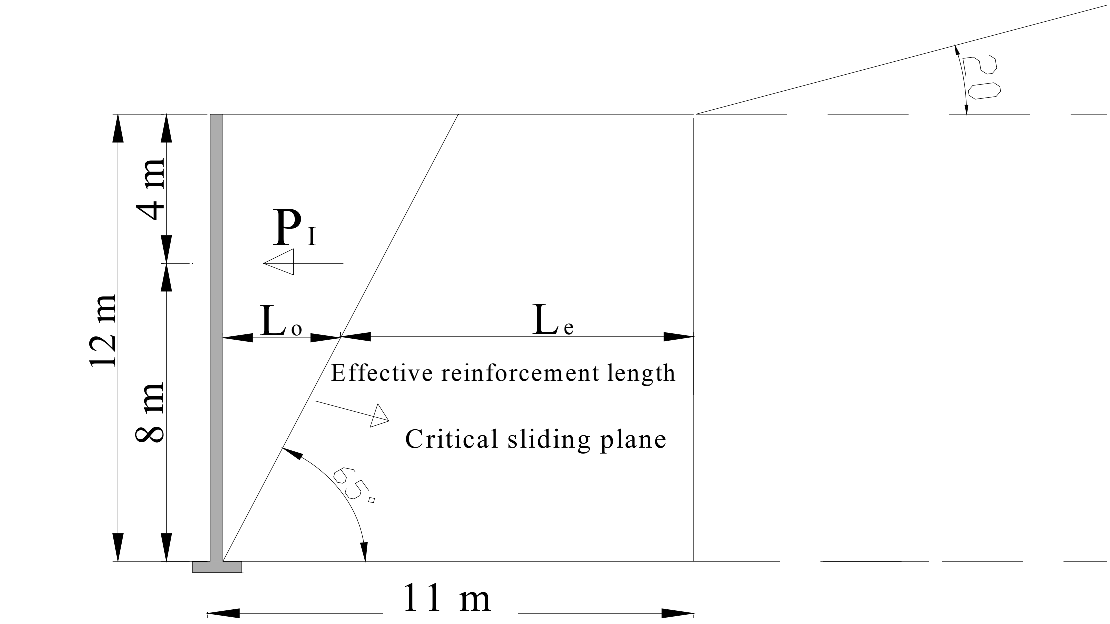


Figure 8.8. Influence point of PI inertial force

8.10.7.1.Calculation of P_I Inertial Force.

$$\text{Equation (7.85)} \Rightarrow P_I = A_m W_A$$

Here;

W_A is the weight of active zone, and A_m is the maximum acceleration on the wall.

$A_m = 0.345$ and $W_A = 750$ kN/m $\Rightarrow$ $P_I = 258.75$ kN/m

8.10.7.2.Calculation of Total Stress at Reinforcement Levels. Total stress occurred in each reinforcement level T_{total} is calculated as adding T_{max} by using Equation (7.88) that comes from static effects to T_{md} by using Equation (7.89) that comes from dynamic effect.

Equation (7.88) $\Rightarrow$ $T_{\max} = S_v \; \sigma_h$ = 5.425z*S_v

Equation (7.89) $\Rightarrow$ $$T_{md} = P \frac{L_{ei}}{\sum_{i=1}^{n} (L_{ei})}$$

Equation (7.90) $\Rightarrow$ $T_{total} = T_{\max} + T_{md}$

Where;

P_1 = Inertial force due to weight of active region

L_{ei} = active reinforcement length in order i

T_{max} = Affection load on reinforcement in unit width due to dynamic forces

T_{max}, T_{md} and T_{total} values for each reinforcement levels are represented in Table 8.7.

Table 8.7. T_{max} , T_{md} and T_{total} values for each reinforcement levels

Reinforcement Level	Depth (m)	Spacing (S_v) m	L_e existing	T_{max} (kN/m)	T_{md} (kN/m)	T_{total} (kN/m)
16	0.5	0.75	5.02	2.03	8.68	10.71
15	1.5	1	5.54	8.14	9.58	17.72
14	4.5	0.88	7.1	21.36	12.28	33.64
13	5.5	0.75	7.62	22.38	13.18	35.55
12	6	0.75	7.88	24.41	13.63	38.04
11	6.5	0.75	8.14	26.45	14.08	40.52
10	7	0.75	8.4	28.48	14.52	43.01
9	7.5	0.75	8.66	30.52	14.97	45.49
8	8	0.75	8.92	32.55	15.42	47.97
7	8.5	0.75	9.18	34.58	15.87	50.46
6	9	0.75	9.44	36.62	16.32	52.94
5	9.5	0.63	9.7	32.21	16.77	48.98
4	10	0.5	9.96	27.13	17.22	44.35
3	10.5	0.5	10.22	28.48	17.67	46.15
2	11	0.5	10.48	29.84	18.12	47.96
1	11.5	0.5	10.74	31.19	18.57	49.76

8.10.7.3. Rupture Check under Dynamic Loading. To obtain the safety condition under dynamic loading, condition mentioned below should be satisfied.

$$T_{all} = \frac{T_u}{FS_{br} \times 0.75} > T_{\max}$$

From Table 8.6., T_{max} is obtained as T_{max} = 52.94 kN/m

$$T_{all} = \frac{59.9}{1.3 \times 0.75} = 61.44 kN / m > 52.94 kN / m$$

Decision: Vertical reinforcement distance is sufficient to satisfy rupture safety under dynamic loading.

8.10.7.4. Pull-Out Check under Dynamic Loading. To satisfy the safety condition under dynamic loading, condition mentioned in Equation (7.95) should be satisfied. Resisting forces against pull-out are placed at the right hand side of the equation and total lateral forces leading to pull-out are placed at the left hand side of the equation.

$$\text{Equation (7.95)} \Rightarrow T_{total} \leq \frac{P_r\, R_c}{0.75\times FS_{pull-outg}} = \frac{C(0.80F^*)}{0.75\times 1.5}\times \gamma\, Z_p \times L_e \times R_c \times \alpha$$

$FS_{pull\text{-}out}$ = Safety factor against pull-out = 1.5

T_{max} = Maximum tension stress in reinforcement = 4.34z

C = Number of surfaces = 2

F = Friction coefficient = 0.67 (Table 7.8)

α = Factor of measuring correction = 0.8 (Table 7.8)

R_c = Number of covering = 0.8

γZ_p = Lining pressure = 20z (It includes distributed static loads but excludes dynamic loads like traffic loads)

Total and resisting forces are calculated for each reinforcement level which can be seen in Table 8.8.

Table 8.8 Dynamic pull-out check

Reinforcement Level	Depth (m)	Spacing (S_v) m	L_e existing	T_{max} (kN/m)	T_{md} (kN/m)	T_{total} (kN/m)
16	0.5	0.75	5.02	2.03	8.68	10.71
15	1.5	1	5.54	8.14	9.58	17.72
14	4.5	0.88	7.1	21.36	12.28	33.64
13	5.5	0.75	7.62	22.38	13.18	35.55
12	6	0.75	7.88	24.41	13.63	38.04
11	6.5	0.75	8.14	26.45	14.08	40.52
10	7	0.75	8.4	28.48	14.52	43.01
9	7.5	0.75	8.66	30.52	14.97	45.49
8	8	0.75	8.92	32.55	15.42	47.97
7	8.5	0.75	9.18	34.58	15.87	50.46
6	9	0.75	9.44	36.62	16.32	52.94
5	9.5	0.63	9.7	32.21	16.77	48.98
4	10	0.5	9.96	27.13	17.22	44.35
3	10.5	0.5	10.22	28.48	17.67	46.15
2	11	0.5	10.48	29.84	18.12	47.96
1	11.5	0.5	10.74	31.19	18.57	49.76

Decision: As indicated in Table 8.8., safety occurred for pull-out check even for the reinforcement at top level where it is hard to obtain safety against pull-out. Effective reinforcement length is sufficient for pull-out safety under dynamic loading.

8.11. Total Collapse Analysis for Model Reinforced Earth Retaining Wall

After completing interior and exterior stability analysis for reinforced earth retaining walls then, total collapse analysis where construction is evaluated generally should be carried out.

In the present system, it is very difficult to determine where critical sliding circle passes. For this reason, analyses performed by computer programs will give more realistic results.

Total collapse analysis of present system has been investigated by Plaxis software in Section 10. Also at the same section, total collapse analysis has been repeated by slice method for critical sliding circle found by Plaxis analysis.

In Figure 8.9. and in Table 8.9., total collapse analysis calculation steps have been represented for sliding circle passes through the toe of reinforced earth structure by Swedish Slice Method.

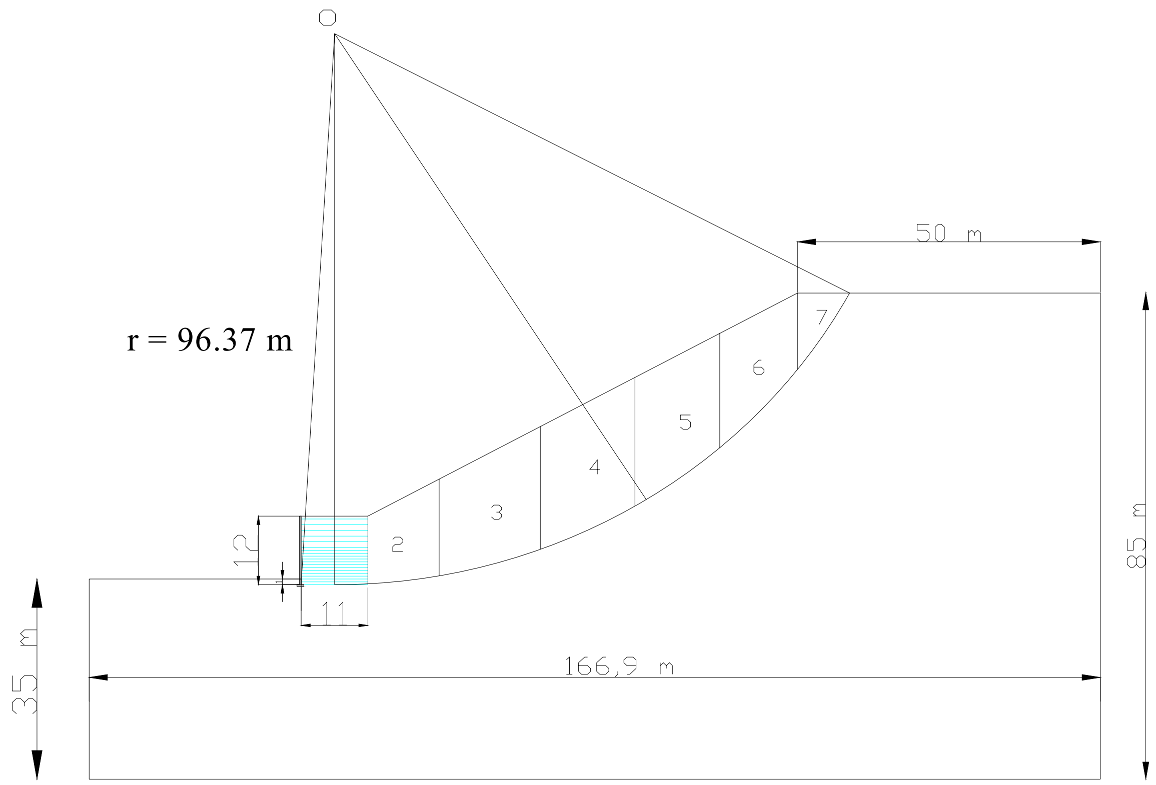


Figure 8.9. Total collapse analyses after construction (Swedish Slice Method)

As seen in Figure 8.9., slope has been divided into seven calculation slice. Failure circle intersects only the bottom reinforcement. According to Swedish Slice Method, safety factor against total collapse is calculated by Equation (8.3) as seen below;

Equation 8.3 $\Rightarrow FS_{total} = \dfrac{\sum W(\cos a) \times \tan \phi_f + c \times l + \sum l_d \times T_{all}}{\sum W(\sin a)}$

Here,

W = Weight of failure circle (kN)

a = Angle of slice center with vertical line passes through the center of failure circle "O"

φ_f = Interior friction coefficient of soil (28^0)

I_d = Vertical distance of the point that is crossed by failure circle to the point "O"

T_{all} = Allowable tension stress of reinforcement (46.07kN/m)

FS_{total} = Factor of safety against total collapse (1.5)

r = Radius of failure circle (96.37m)

Calculations for each slice are summarized at Table 8.9.

Table 8.9. Calculation table for slice method

Slice No	Slice Area m^2	$a(^0)$	W (kN)	Wsina (kN)	Wcosa (kN)
1	89	-3	1736	-90	1734
2	194	4	3783	264	3774
3	272	16	5305	1462	5099
4	327	25	6376	2691	5770
5	338	36	6591	3874	5332
6	279	49	5441	4106	3569
7	78	56	1521	1260	851
Total				13567	26129

$$FS_{total} = \frac{\sum W(\cos a) \times \tan \phi_f + \sum l_d \times T_{al}}{\sum W(\sin a)} = \frac{26129 \times \tan 28 + 46.07 \times 96.37}{13567} = 1.351$$

$$FS_{total} = 1.351$$

It is calculated that the safety factor obtained from total collapse analysis for selected sliding circle is 1.35 and this is a very satisfactory value since our slope was at collapse case at its initial condition; but total collapse analysis achieved by hand should be repeated for different potential sliding circles or can be checked by different computer programs. Total collapse analysis of present system for different conditions will be studied by Plaxis software in section ten under the title of “Model-2”. Also at the same section, total collapse investigation will be repeated with slice method for critical sliding circles that found by Plaxis analysis.

8.12. Completed Model Reinforced Earth Retaining Wall

Final state of reinforced earth retaining wall designed according to specification of U.S Department of Transportation, is shown in Figure 8.10.

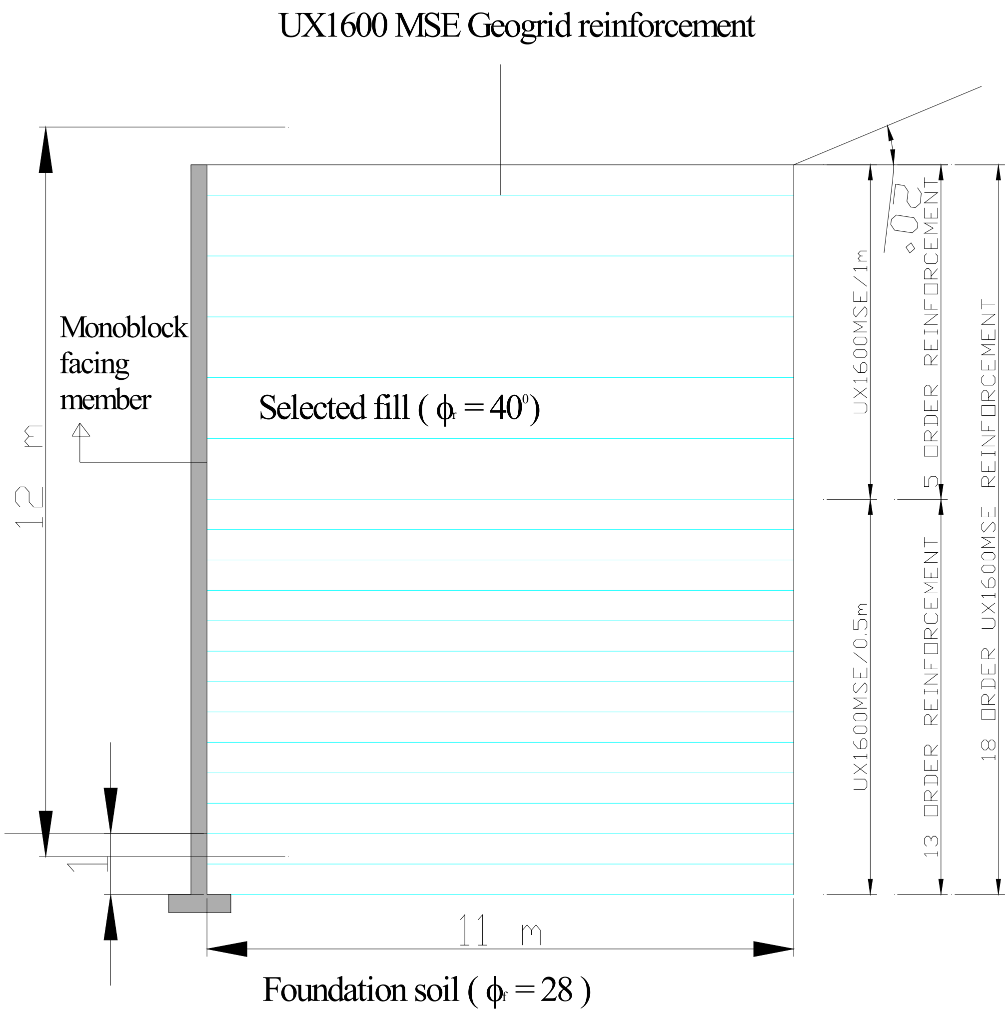


Figure 8.10. Final geometry of proposed reinforced earth retaining wall

9. FINITE ELEMENT METHOD AND PLAXIS SOFTWARE

9.1. Finite Element Method

The FEM is a computer-aided mathematical technique for obtaining approximate numerical solutions to the abstract equations of calculus that predicts the response of physical systems subjected to external influences.

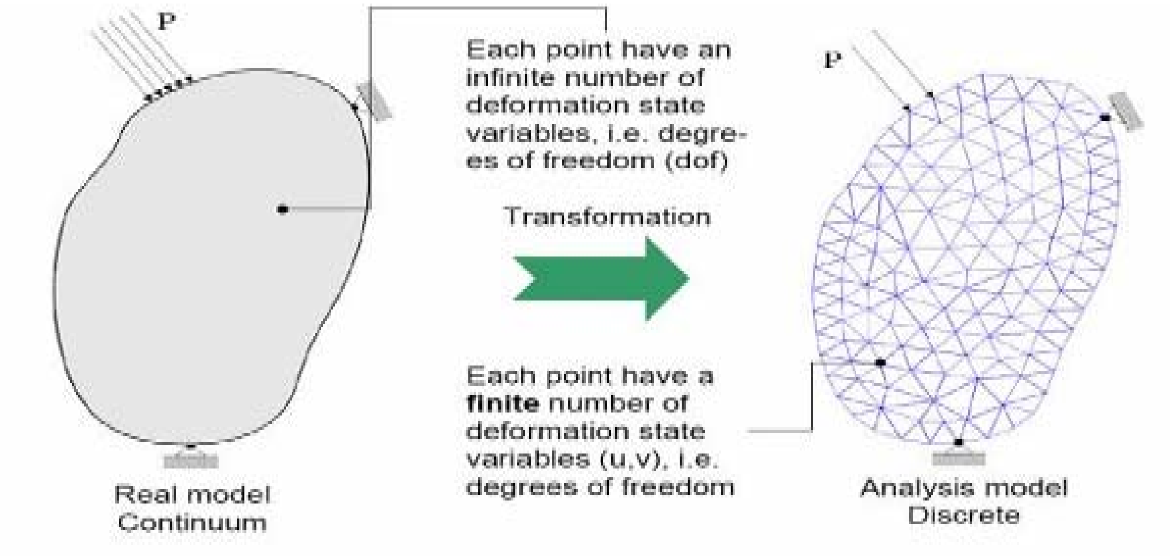


Figure 9.1. Transformation

- Divide a continuum with infinitely degrees of freedom in to finite elements with a given number of degrees of freedom.
- An element is geometrical defined by a number of nodes in which the elements are connected. The directions a node can move in is termed degrees of freedom (dof).

Following conditions must always be satisfied:

- Equilibrium conditions

$$\sum F_x = \mathrm{m}a_x \ \sum F_y = ma_y \sum F_z = ma_z$$

$$\sum M_x = I_x a_x - (I_y - I_z)\omega_y \omega_z$$

$$\sum M_y = I_y a_y - (I_z - I_x)\omega_z \omega_x$$

$$\sum M_z = I_z a_z - (I_x - I_y)\omega_x \omega_y$$

- Compatibility conditions

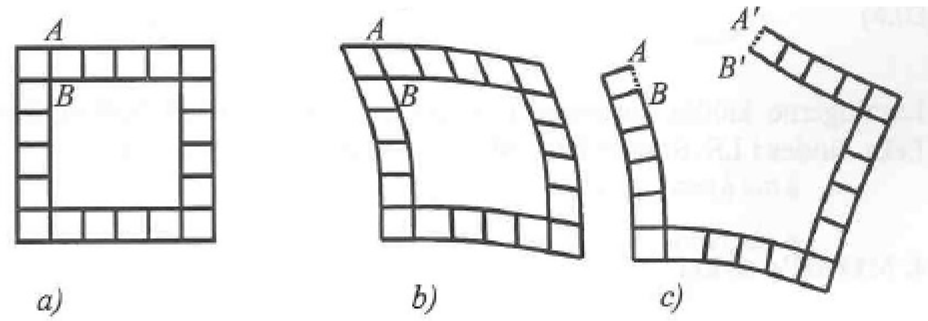


Figure 9.2. Compatibility conditions

- Constitutive conditions

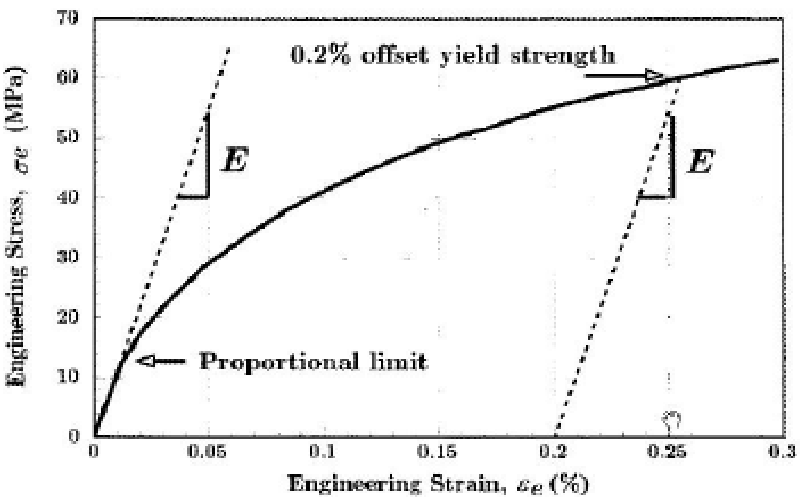


Figure 9.3. Constitutive conditions

- Boundary conditions

General advantages of the Finite Element Method can be listed as follows;

- Irregular Boundaries
- General Loads
- Different Materials
- Boundary Conditions
- Variable Element Size
- Easy Modification
- Dynamics
- Nonlinear Problems (Geometric and/or Material)

And disadvantages can be listed as follows;

- An approximate solution
- An element dependent solution
- Shape quality of elements affect the solution, e.g. poorly shaped elements (irregular shapes) reduce accuracy of the FE solution
- Element density affect the solution, i.e. the element size should be adjusted to capture gradients. Example: Plate with a circular hole.
- Errors in input data

9.1.1. Fundamental Analysis Steps in Finite Element Method

1. Member selection
2. Selection of approaching method
3. Determination of material behavior
4. Obtaining member equations
5. Obtaining system equation by combining member equations
6. Obtaining primary unknowns by solving the system equations
7. Obtaining secondary unknowns by primary unknowns
8. Evaluation of results

9.2. The Plaxis Software

The Plaxis software grew out of research conducted at Delft University of Technology in late 1970's on the use of finite element methods for geotechnical design. The initial brief was to develop an easy-to-use finite code for analysis of river embankments on the soft soils of lowlands of Holland.

Plaxis is a finite element package specifically intended for analysis of deformation and stability in geotechnical engineering projects. Geotechnical applications require advanced constitutive models for the simulation of the nonlinear and time dependent behavior of soils. The modeling of the soil itself is an important issue; many geotechnical engineering projects involve the modeling of the structures and the soil. Plaxis is equipped with special features to deal with the numerous aspects of complex geotechnical structures (Brinkgreve and Vermeer, 1998)

A geometry model in Plaxis is a representation of a real problem and consists of points; lines and clusters. Lines are used to define the physical boundaries of the geometry, the model boundaries and discontinuities in the geometry. Clusters are areas that are fully enclosed by lines. Plaxis automatically recognizes clusters based on the input geometry lines. Within the cluster, the soil properties are homogenous. Actions related to clusters apply to all elements in the cluster. A two dimensional plain strain model is used for the structures with a (more or less) cross-sections and corresponding stress-state and loading scheme over a certain length perpendicular to the cross section.

The creation of geometry model, a finite element mesh can automatically be generated, based on the composition of clusters and lines in the geometry model. The finite element mesh has distinguished as elements, nodes, stress points. During the automatic generation of mesh, clusters are divided into triangular elements in Plaxis. The default is 6 node-elements and also 15 node-elements is available for more accurate calculations of stress and failure loads. The 6 node triangular element provides a second order interpolation for displacements. The element stiffness matrix is evaluated by numerical integration using a total three stress points. A 6 node element is defined by 6 nodes shown in Figure 9.4. during a finite element calculation;

displacements are calculated at the nodes. In contrast to displacements, stresses are calculated at the stress points.

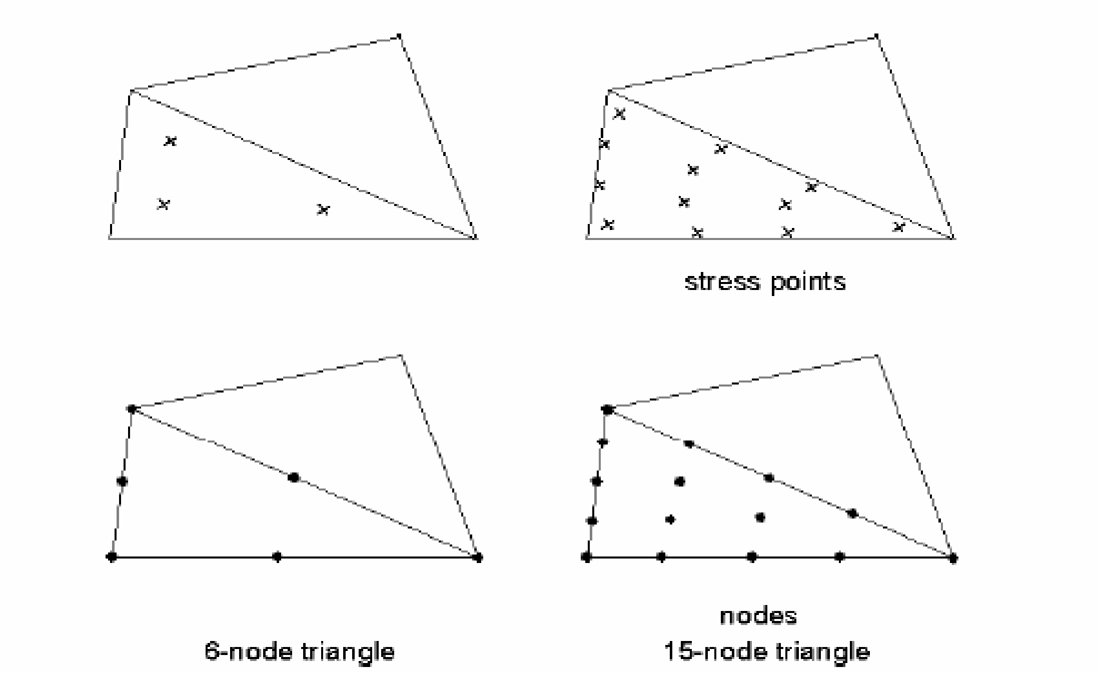


Figure 9.4. Position of nodes and stress points on the element

9.2.1. The Plaxis Software Modules

There are 4 modules in Plaxis software to define and analyze the model. An "Input" module for entering the graphical and numerical data of the problem, a "Calculation" module where analyses are defined and performed, an "Output" module where analyze results are represented graphically and a "Curve" module which allows us to generate curves based on the obtained results.

9.2.2. Geotextile Material Definition in Plaxis Software

In Plaxis program, geotextiles are modeled as materials that are only able to get tension force. The only needed material property for geotextile definition is elastic axial rigidity (EA). Possible deformations and axial forces observed in geotextile can be determined after analyses.

9.2.3. Factor of Safety Calculation (Phi-c Reduction)

In Plaxis software, there exists a "Phi-c reduction" option in order to calculate factor of safety against sliding. In this analyze, soil parameters "tan Φ" and "c" are reduced gradely in order to determine the collapse moment. With this option, it is also possible to determine the safety factor against collapse (sliding-total collapse) for any stage of construction phase.

ΣM_{sf} total multiplier is used in order to obtain soil bearing parameter in any phase of construction.

$$\sum M_{sf} = \frac{\tan \phi_i}{\tan \phi_r} = \frac{c_i}{c_r} \tag{9.1}$$

In the above equation "i" represents the given values at material property definitions and "r" represents the reduced values used in the analyses. $\Sigma M_{sf} = 1$ is used at initial stages of the analyses to use reduced material properties and then by reducing the tan Φ and c parameters gradely, it is satisfied that the structure reaches collapse state. The safety factor at this collapse state is equal to the value of ΣM_{sf} at collapse moment. In the analyses performed at the 10th section of this study, "Phi-c Reduction" method is used to determine the safety against total collapse.

10. ANALYSIS OF MODEL REINFORCED EARTH RETAINING WALL BY PLAXIS SOFTWARE

10.1. Analysis Performed by Software Plaxis

In this section, analysis done by Plaxis Version 8 for natural slope and model reinforced earth walls can be found as follows;

1. Model 1: Collapse analysis of natural slope
2. Model 2: Design and analysis of model reinforced earth retaining wall (Reinforcement: Tensar UX1600 MSE geogrid)
3. Model 3: Design and analysis of model reinforced earth retaining wall (Reinforcement: Tensar UX1700 MSE geogrid)
4. Model 4: Design and analysis of model reinforced earth retaining wall with reinforced slope (Reinforcement: Tensar UX1600 MSE geogrid)

Model 1: Stability of natural slope with 30 degrees angle as shown in Figure 8.1 whose instability was proved by slice method is checked by Plaxis software.

Model 2: In this model; in order to increase the stability of unstable slope shown in Model 1 and create a safe working place for a highway construction at toe zone of natural slope, a model reinforced earth retaining wall of 12m height and 11m width with UX1600 MSE reinforcement has been constructed at toe zone of natural slope (Figure 10.1). Analyses of proposed model reinforced earth retaining wall have been preformed by Plaxis software. Considering that the natural slope lays in a second degree earthquake zone, A_0 has been selected as 0.3g.

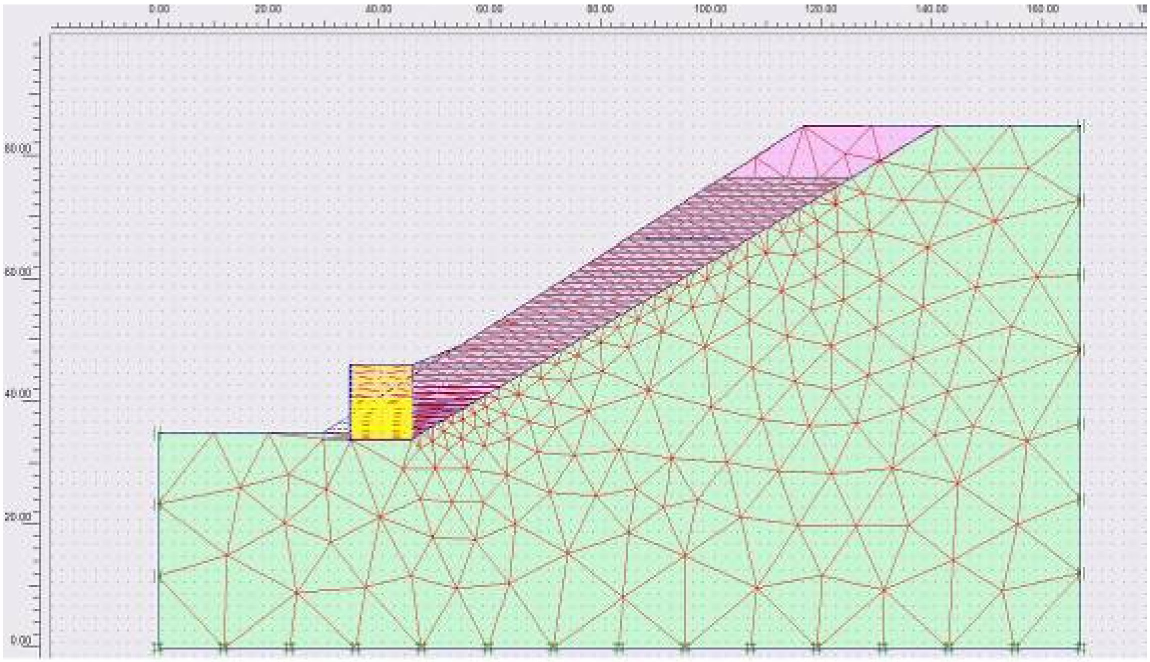


Figure 10.1. Working space and boundary conditions for reinforced earth retaining wall

Model 3: In this model, analysis of reinforced earth retaining wall of which's analysis has been realized in Model 2 is repeated by using of 'Tensar UX1700 MSE' geogrid" reinforcement in order to investigate the influence of reinforcement stiffness to the system.

Model 4: In last part of this section, the slope at backside of the wall in Model 2 was reinforced with Tensar UX1600 MSE geogrid and Plaxis analysis has been performed. Aim of this analysis is to search influence of reinforced slope to the system.

10.2. Engineering and Material Properties of Structural Members and Soil at Plaxis Analyses

Material, soil and structure properties used in 4 different analyses at Plaxis software are represented in Table 10.1.

Table 10.1. Material, soil and structural member properties

		Property	Plaxis Symbol	Unit	Value			
					Model 1	Model 2	Model 3	Model 4
Natural Soil		Soil interior friction angle	φ	^{0}	28	28	28	28
		Cohesion	c_{ref}	kN/m^2	7	7	7	7
		Modulus of elasticity	E_{ref}	kN/m^2	60000	60000	60000	60000
		Saturated unit weight	γ_{sat}	kN/m^3	20	20	20	20
		Dry unit weight	γ_{dry}	kN/m^3	19.5	19.5	19.5	19.5
		Horizontal permeability	k_x	m/s	1	1	1	1
		Vertical permeability	k_y	m/s	1	1	1	1
		Dilatency angle	Ψ	^{0}	5	5	5	5
		Poisson ratio	ν		0.4	0.4	0.4	0.4
Fill material	Selected Fill	Soil interior friction angle	φ	^{0}	40	40	40	40
		Cohesion	c_{ref}	kN/m^2	2	2	2	2
		Modulus of elasticity	E_{ref}	kN/m^2	100000	100000	100000	100000
		Saturated unit weight	γ_{wet}	kN/m^3	20.5	20.5	20.5	20.5
		Dry unit weight	γ_{dry}	kN/m^3	20	20	20	20
		Horizontal permeability	k_x	m/s	1	1	1	1
		Vertical permeability	k_y	m/s	1	1	1	1
		Dilatency angle	Ψ	^{0}	10	10	10	10
		Poisson ratio	ν		0.4	0.4	0.4	0.4
Reinforcement	UX1600	Stretching rigidity	EA	kN/m^2	1800			
	UX1700	Stretching rigidity	EA	kN/m^2	2350			
Facing member	Monoblock Precast	Thickness	d	m	0.4	0.4	0.4	0.4
		Stretching rigidity	EA	kN/m^2	$8x10^6$	$8x10^6$	$8x10^6$	$8x10^6$
		Bending rigidity	EI	kNm^2/m	$1x10^5$	$1x10^5$	$1x10^5$	$1x10^5$
		Weight	W	kN/m/m	1.6	1.6	1.6	1.6
		Poisson ratio	ν		0.2	0.2	0.2	0.2

10.3. Acceptances and Assumptions Applied in Plaxis Analyses

- Linear deformation model 'Plane Strain' has been used.
- Triangular finite elements with 15 node points have been used.
- Worked with m, kN, day in SI unit system.
- 'Mohr-Coulomb' failure criteria was selected for analyses.
- Material behaviors have been taken into account as drainage type.
- For 'G' sliding modulus, value that Plaxis software gave; has been used.
- In this study, taking into account that dimensions of finite elements affect the result directly and due to this, very fine mesh system has been selected for better results.
- Intermediate surface member ‘Interface’ has not been used in any stage of the analysis.
- For soil diletancy angle (Ψ), Φ-30 angle that has been advised by Plaxis method has been used.
- Although fill material is sand, instead of c = 0 value, in order to prevent possible ignorable faults during analyses, c = 2 kN/m^2 has been used.
- Reinforced earth retaining structure starts 5m inside from the toe of slope as seen in Figure 10.1.
- Since the analyses achieved contain declined surfaces, gravity loading process has been applied for initial condition instead of K_0 procedure.
- The value taken into account for factor of safety analysis is ΣM_{sf} value and the ΣM_{sf} - total displacement graphics obtained from selected points on slope.

10.4. Control Criteria for Evaluation of the Results

While considering Plaxis analyses, the safety values at Table 10.2. should be taken into account.

Table 10.2. Control criteria for plaxis analyses

Controls	**Allowable value**	**Unit**
Wall maximum Horizontal displacement	120	mm
Wall maximum Settlement	75	mm
Wall maximum differential Settlement	32	mm
Wall maximum angular Settlement	L/60 = 183	mm
Bearing capacity check	1685	kN/m^2

10.5. Plaxis Analyses

10.5.1. Model 1 – Analysis Definitions

In this analysis, stability investigation for the model natural slope shown in Figure 8.1. is performed. The purpose of analysis is to determine the stability state of the natural slope at initial stage. Slope and foundation soil are formed by the same type of the material which's properties are represented at Table 10.1.

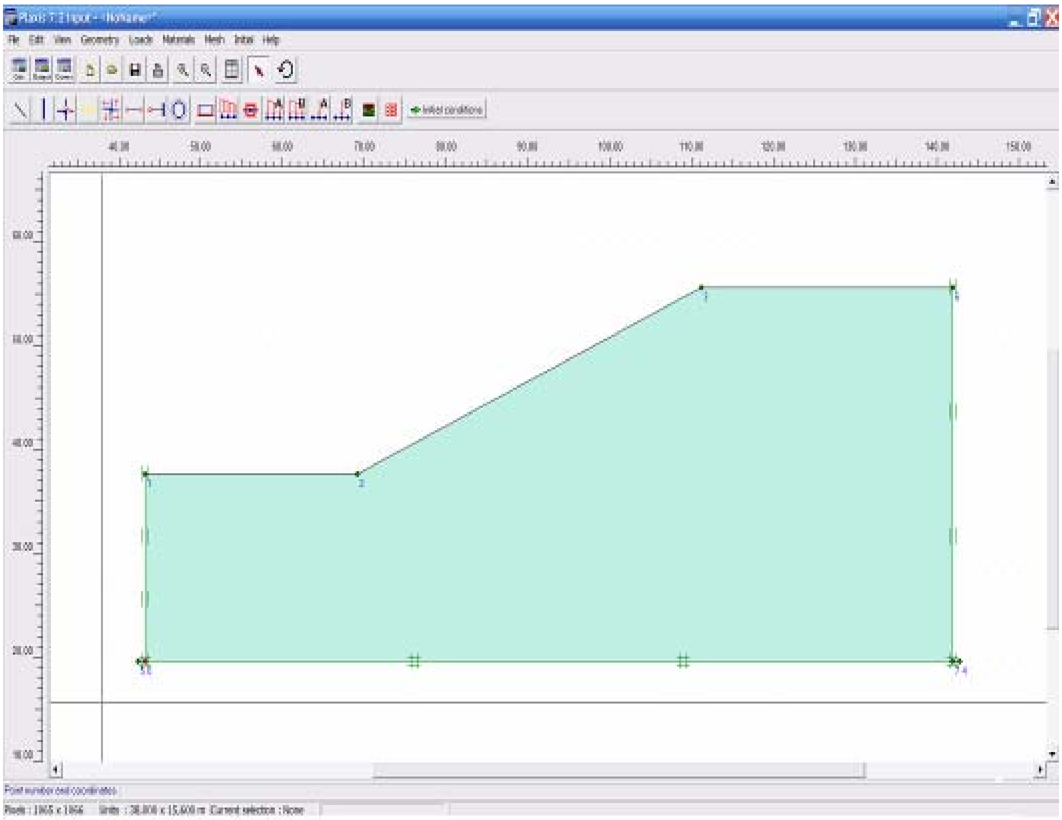


Figure 10.2. Plaxis working area boundaries for Model 1

Angle of natural slope is 30 degrees and sufficient working space has been left at the bottom and upper points where the slope ends. Also it has been accepted to be adequate to take 35 m for thickness of foundation soil.

In this analysis, as considering dimensions of finite elements affects the result directly, it has been worked with very fine element network (mesh) in order to obtain more sensitive results.

For stability analysis of slope shown in Figure 10.2. with Plaxis, steps mentioned below were followed.

Design stages;

- All steps in design have been realized in input mode. First of all, boundaries of working area were determined from the window of general adjustments (bottom boundary; 0, top boundary; 85m, left boundary; 0, right boundary; 185 m.

- Geometry shown in Figure 10.2 is entered in input phase.
- Boundary conditions are determined by 'Standard Fixities' command (Figure 10.2).
- By 'Material Sets' command, material named 'Natural Soil' is generated with the properties given in Table 10.1 and this material is assigned to slope and foundation soils.
- By 'Generate Mesh' command, finite element network is created and it is properly refined (Refine Mesh command)
- In 'Initial Conditions' command, Ko procedure is ignored and skipped to the calculation phase.

Analysis steps;

- Analysis steps are performed in 'Calculation' module. This problem is made up of 2 stages. For the first step gravity loading method is selected for initial conditions. To define the initial conditions; from 'Loading Input' menu, $\sum M_{weight} = 1$ should be selected while 'Total Multipliers' command is active. By this application we let the Plaxis to consider the soils own weight in analysis. As a feature of weight loading, 'Ignore undrained behavior' command is to be selected at this stage.

- At the second step of the analysis, collapse check of the slope is done by the method 'Phi-c reduction'. For this analysis from the mode 'Calculation type', 'Load Adv. Number of steps' should be selected. Also, due to the features of weight loading applied at previous step, "Reset displacements to zero" command should be selected.

- After completing the second step of the analysis, points that are asked to be drawn should be defined earlier. In this analysis, points given in Figure 10.3. have been determined for safety coefficients and settlement graphics.

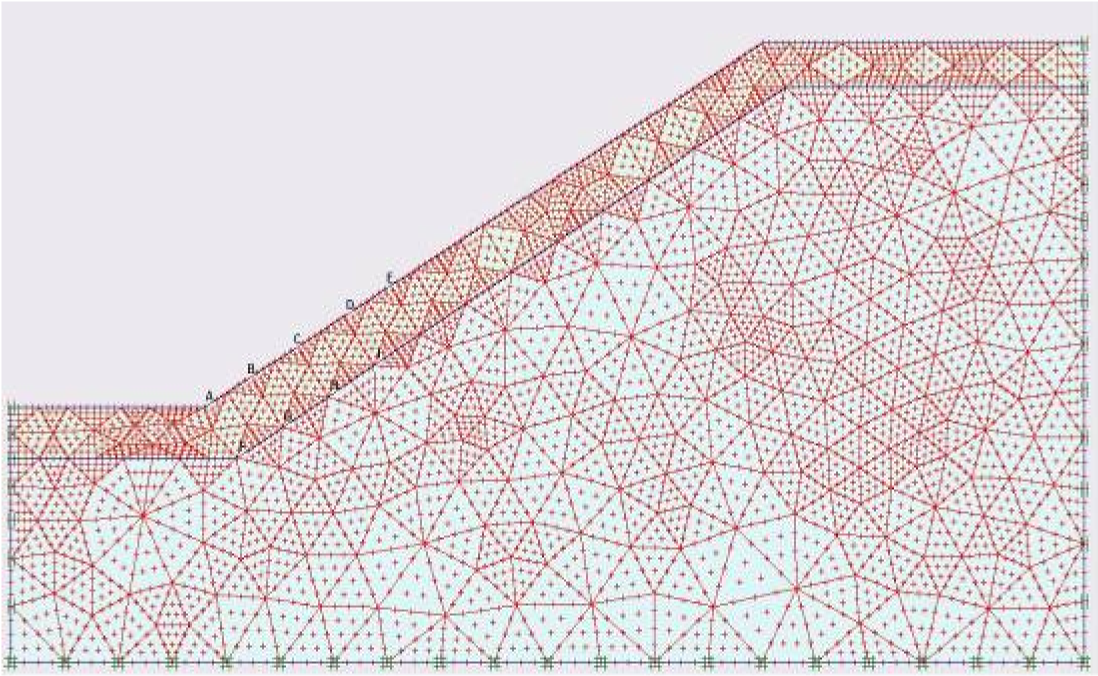


Figure 10.3. Connectivities

<u>10.5.1.1. Model 1 – Results of Analysis for Natural Slope.</u> Obtained results from analysis of Model 1 are represented at Table 10.3.

Table 10.3. Analysis results of Model 1 by Plaxis

Analysis result	Value	Unit
Maximum effective stress	1590	kN/m^2
Maximum total stress	1590	kN/m^2
Factor of safety(ΣM_{sf})	**1.053**	

Graph of ΣM_{sf} – Total displacement at A, B, C, D points that their locations are determined at Figure 10.3. and can be seen at Figure 10.4. At mentioned figure, curves are getting horizontal at 1.05 M_{sf} value which means the safety factor for the slope is 1.05.

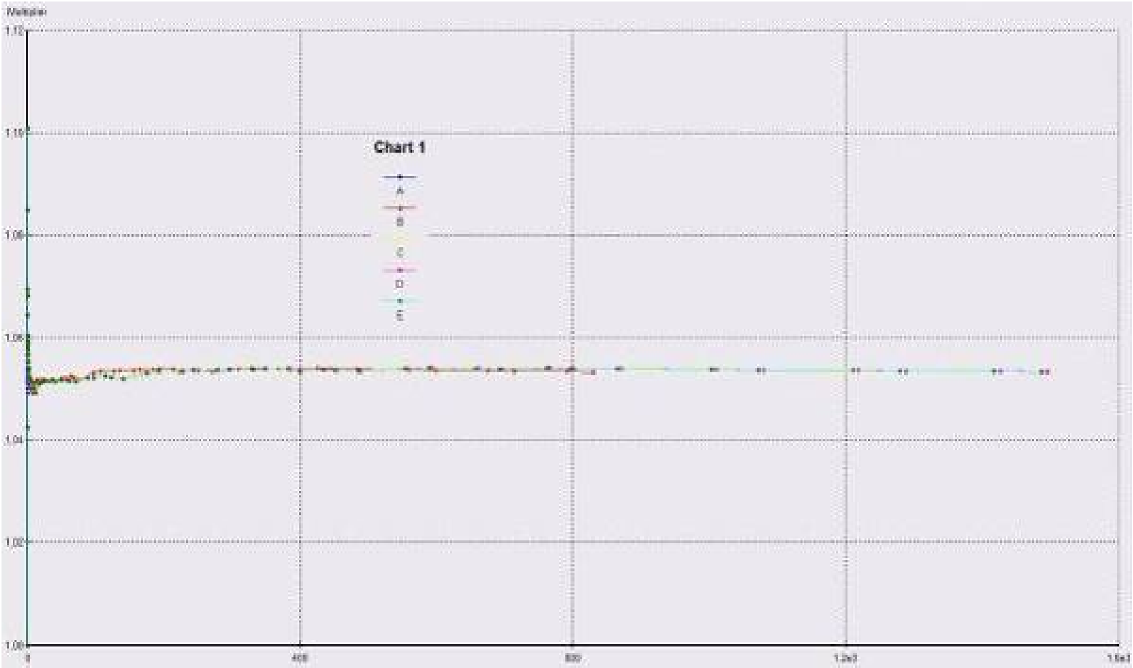


Figure 10.4. M_{sf} –Displacement curve for Model 1

“Total Incremental Displacement” graph where sliding circles of natural slope can be seen; and Total stresses occurred at natural slope are represented at figures below. By Plaxis software, it is possible for sliding circles to be displayed by shadings or arrows.

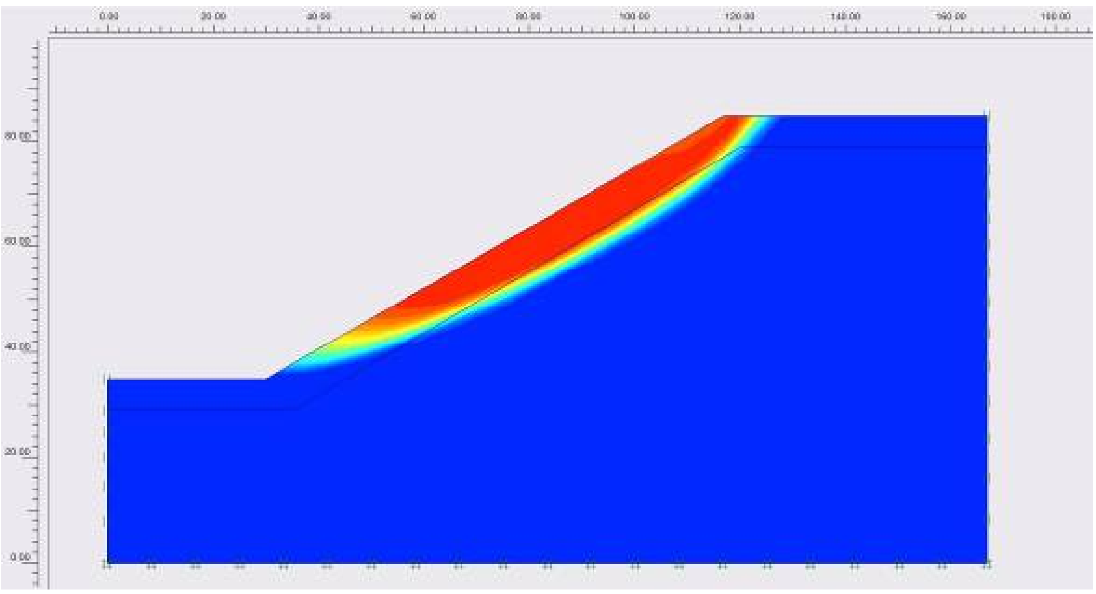


Figure 10.5-(a). Total incremental displacements by shadings for Model 1

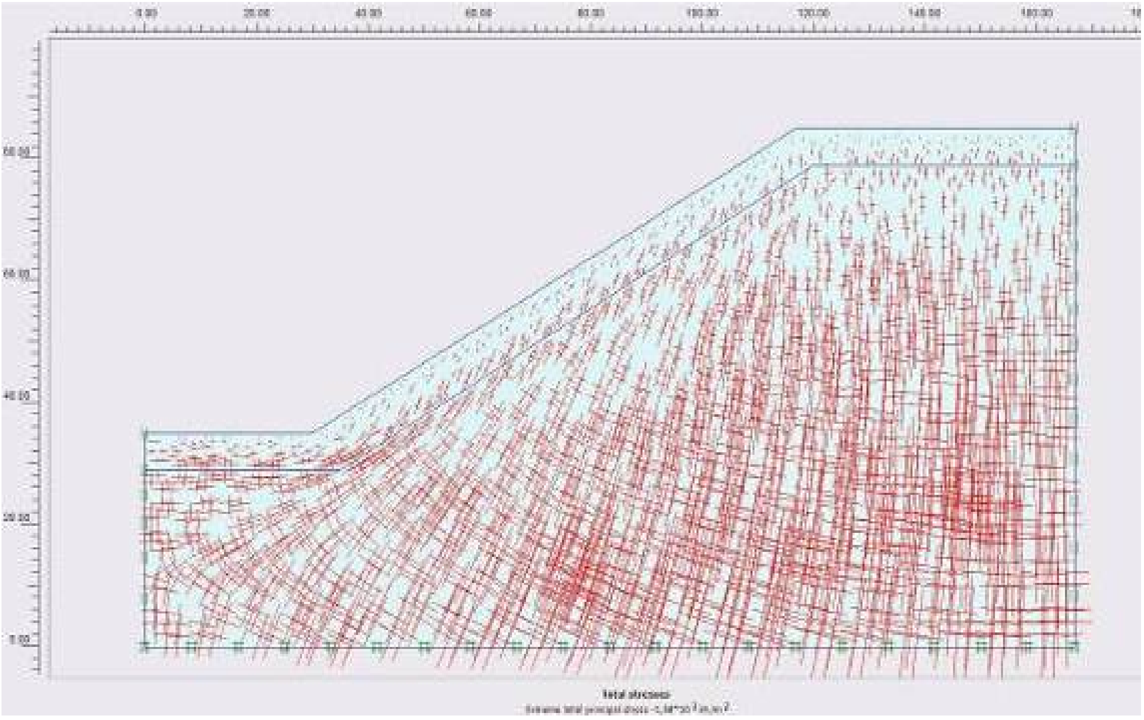


Figure 10.5-(b). Total stresses for Model 1

10.5.2. Model 2 – Analysis Definitions

In this analysis, stability investigations of reinforced earth retaining wall given in Figure 10.6. have been achieved. Aim of this analysis is to make design and analysis of a model reinforced earth retaining structure by Plaxis software, and to determine the change in stability of natural slope after construction.

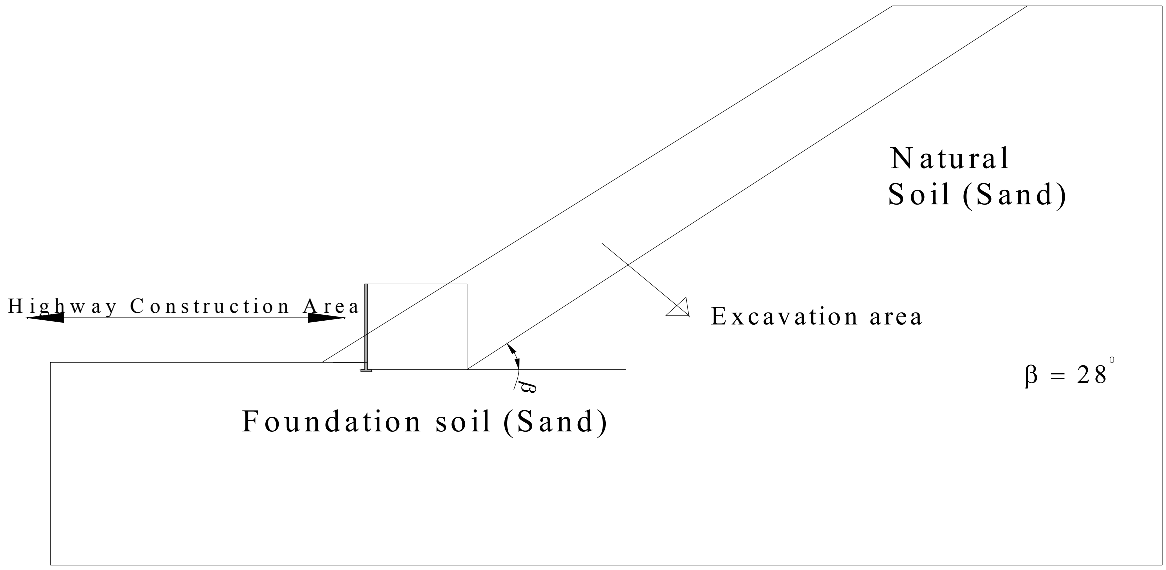


Figure 10.6. Position of model reinforced earth structure analyzed in Model 2

- Properties of materials used in reinforced earth structure are given on Table 10.1.
- Just like in the analysis done by hand, reinforce earth retaining wall starts 5m inside from toe area of slope.
- There is a toe fill at 1 m. depth in front of reinforced earth retaining structure.
- For a safer construction, back of reinforced earth structure has been excavated with 28^0 degree angle as seen in Figure 10.6.
- All geometrical properties of reinforced earth structure (H=12m and L=11m) have been shown in Figure 10.7.

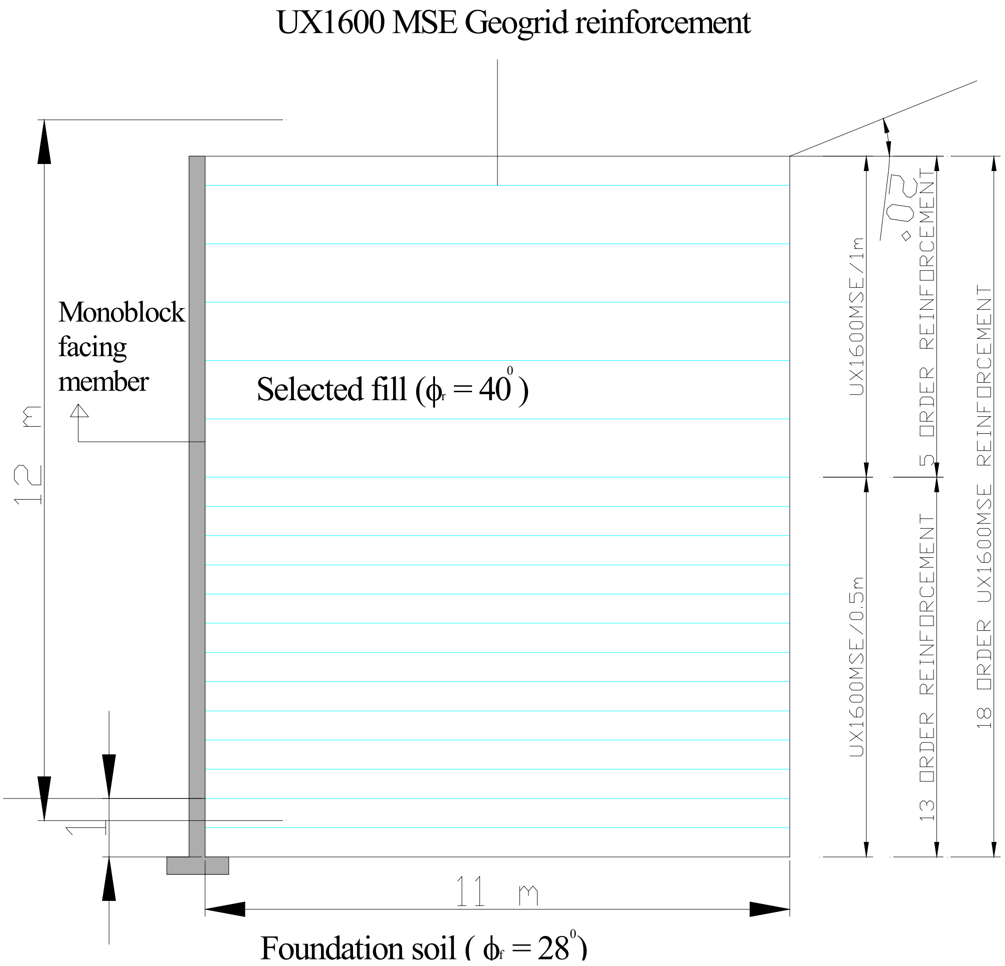


Figure 10.7 Properties of reinforced earth retaining system in Model 2 analysis.

Design stages;

- Establishing working page
- Generating geometry before and after the construction
- Assigning of geometries as structural members (facing members and geotextiles)
- Determining the boundary conditions
- Generating soil and structural members
- Generating finite element network (mesh)
- Determination of initial geometry conditions

Analysis stages;

- 1st Phase; Sum of analysis is made up of 34 phases in Model 2. In first step, weight loading is applied like in Model 1.
- 2nd phase; Definition of first excavation step (in order to allow settlement limitation, excavation was completed in 8 stages instead of 1 and after each excavation stage, displacements are reset to zero).
- From 3rd phase to 9th phase; definitions of excavation stages
- 10th phase; completion of 1st stage of reinforced earth retaining wall. (In this phase, facing member of first stage, selected fill, geotextile and backfill are activated)
- 11th phase; completion of 2nd stage of reinforce earth retaining wall. (similar applications as to 1st stage are realized for 2nd stage)
- 12th phase; completion of front fill
- 13th phase; completion of 3srd stage of reinforced earth retaining wall.
- From 14th phase to 29th phase; completion of construction stages of reinforced earth retaining wall (with this phase, all of the stages of reinforced earth retaining wall are generated).
- 30th phase; completion of 1st stage of upper level wall backfill. (the part of the backfill which remains above the wall level is refilled in 3 stages)
- From 31st phase to 33rd phase; completion of constructions stages of upper level wall backfill (with this phase, construction of reinforced earth retaining wall is completed).
- 34th phase; Slope stability analysis (Phi-c reduction) definition.
- 35th phase; Earthquake analysis according to conditions of second degree earthquake zone
- 36th phase; Slope stability analysis due to seismic condition definition.

Construction Stages of Model Wall can be seen in Figure 10.8. below,

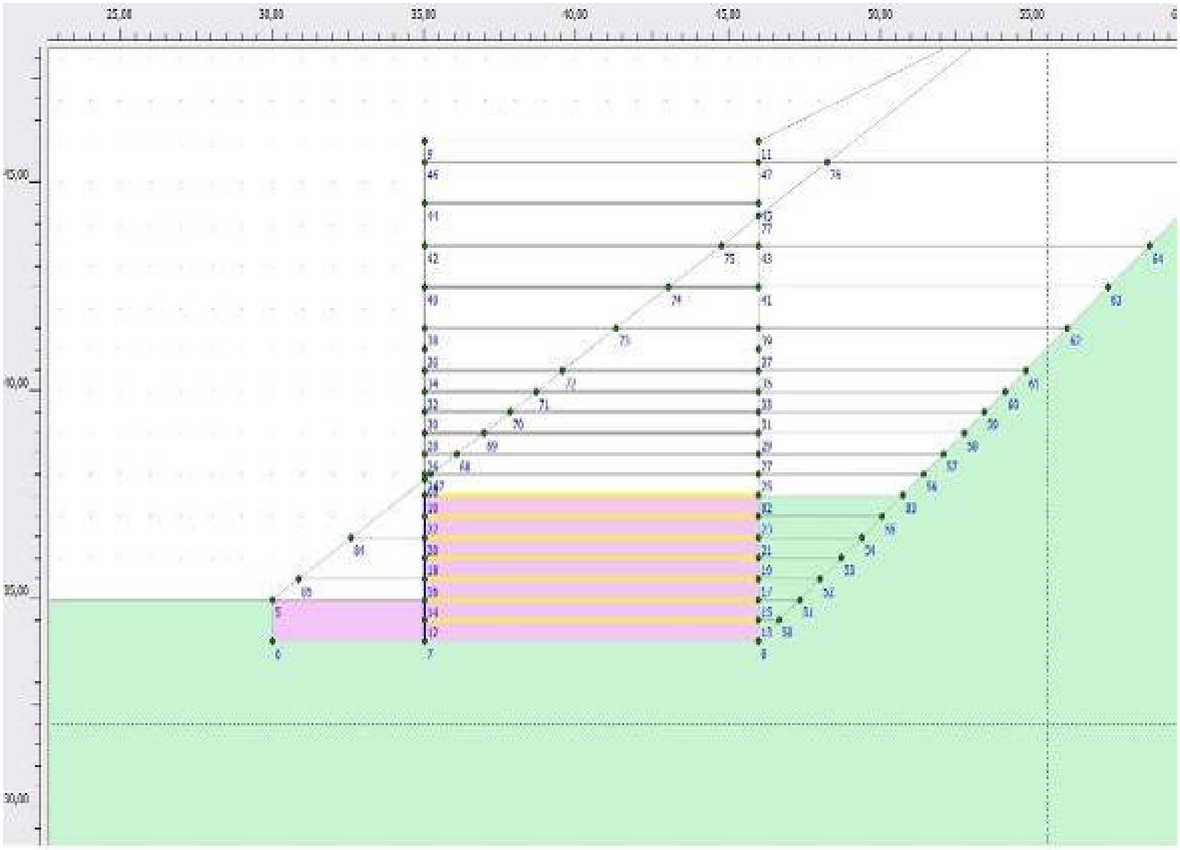


Figure 10.8. Construction stages

After the above mentioned phase, before starting the analysis, the points in the Figure 10.9. are selected in order to draw graphics. Here; point A is upper left corner of the wall, B is the lower left corner, C is the lower right corner and D, E, F, G, H and I are the points on the slope for stability calculations.

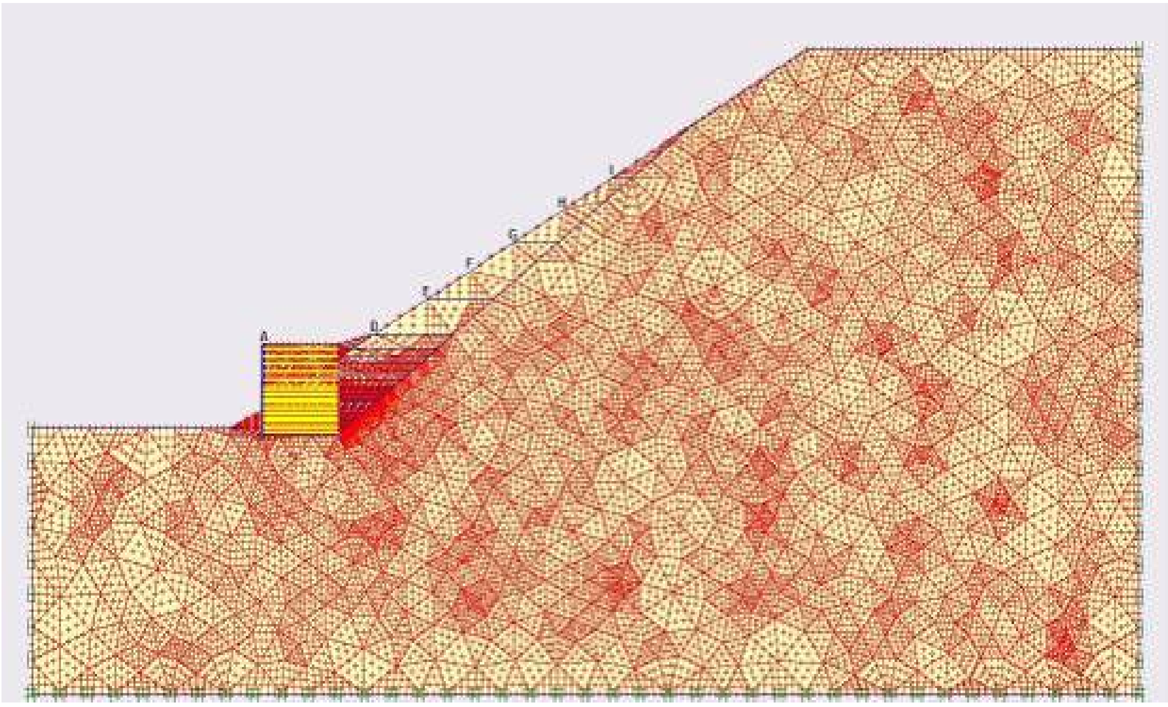


Figure 10.9. Selected points for curve generation

10.5.2.1.Analysis Results by Plaxis Software. Obtained displacement, stress and safety factor values after analysis of Model 2 are represented in Table 10.4.

Table 10.4. Analysis results

Analysis result	**Value**	**Unit**
Maximum total displacement (Slope at the back of wall)	84	mm
Maximum horizontal displacement (Slope at the back of wall)	58	mm
Maximum vertical displacement (Slope at the back of wall)	60	mm
Maximum effective stress	1590	kN/m^2
Maximum total stress	1590	kN/m^2
Factor of safety(ΣMsf)	1,33	
Factor of safety (ΣMsf)-Dynamic case	1,04	

ΣM_{sf} - total displacement graphics of points A, B, C and D are shown at Figure 10.10. At the mentioned figure, curves are getting horizontal at 1.33 M_{sf} value which means the safety factor for the slope is 1.33.

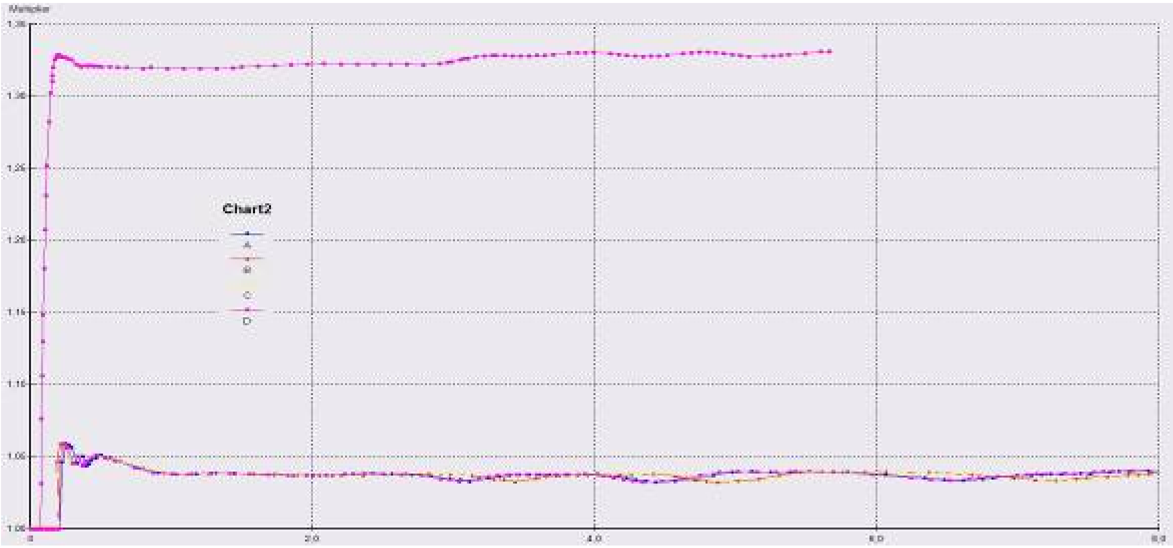


Figure 10.10. M_{sf}-Displacement chart for Model 2

"Total Increments" graph where critical sliding circles can be seen is represented at Figure 10.11.

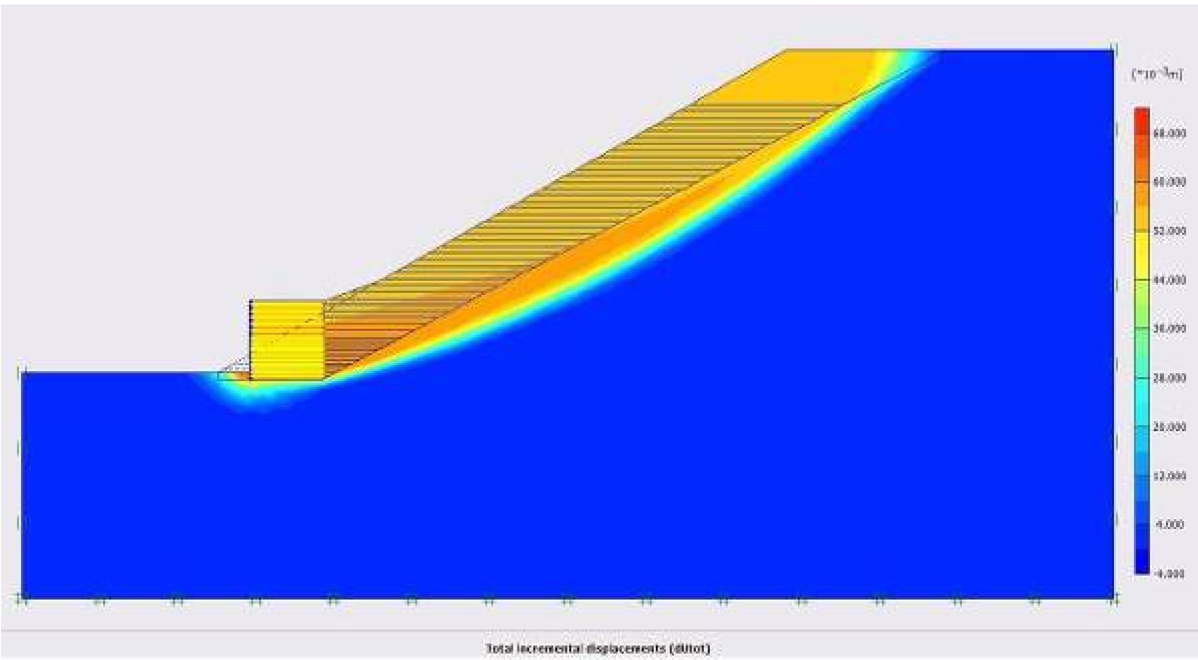


Figure 10.11. Total Increments

Results and controls for reinforced earth retaining wall can be mentioned in the following text. Control criteria for Plaxis analyses applied on reinforced earth retaining walls were given in Table 10.2. At Table 10.5., comparison of the mentioned criteria and obtained results after analysis are represented.

Table 10.5. Comparison of results

	Controls	**Calculation method**	**Obtained value**	**Allowable value**	**Unit**	**Evaluation**
Displacement control	Wall maximum horizontal displacement	Maximum horizontal displacement of point A	29	120	mm	In acceptable limits
Settlement Controls	Wall maximum settlement	Maximum of the settlements occurred at points B and C	58	75	mm	In acceptable limits
	Wall maximum differential settlement	Difference of the settlement values at point B and C	12	32	mm	In acceptable limits
	Wall maximum angular settlement	The ratio of the difference between maximum vertical settlement values of points B and C to the wall width (L)	$1*10^{-3}$	L/60 = 183	mm	In acceptable limits
Bearing capacity Control	Bearing capacity check	Maximum stress at wall base	425	1685	kN/m^2	system is safe as to required bearing capacity conditions
Rupture control	Allowable tension resistance of reinforcement	Maximum of tensions stress occurred at reinforcements	10	46	kN/m	There is not any risk of rupture for reinforcements

Considered base cross-section and settlement values of some points over this section at settlement control of Model 2 are represented below.

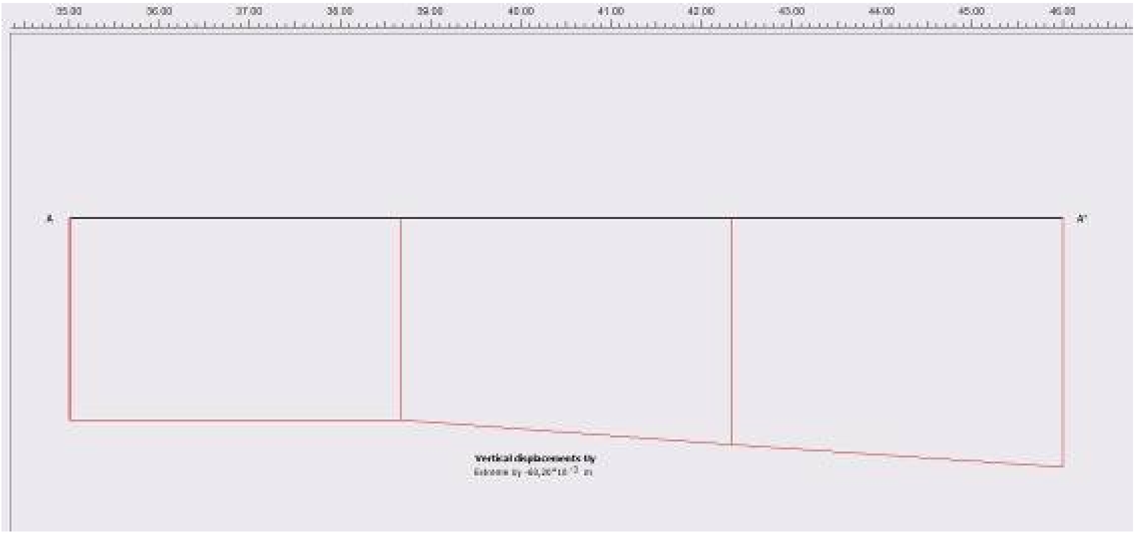


Figure 10.12 Vertical displacement distribution at section A-A’ between points B-C

Table 10.6. Settlement values between points B and C

X [m]	Y [m]	U_y [m]
35,001	33,997	-0,049
35,006	33,997	-0,049
35,006	33,997	-0,049
38,671	33,997	-0,049
42,335	33,997	-0,055
45,999	33,997	-0,060

In Model 2 analysis, bottom section area that is to be taken into account in control of bottom collapse is shown in Figure 10.13.

Figure 10.13. Maximum effective stress

Table 10.7. Stress values between points B and C

X [m]	Y [m]	σ'_N [kN/m ^{2}]
35,001	33,997	-388,394
35,006	33,997	-388,133
35,006	33,997	-303,966
38,671	33,997	-270,776
42,335	33,997	-271,858
45,999	33,997	-291,617

10.5.3. Model 3 – Analysis Definitions

The only difference of this analysis from Model 2 is that ‘Tensar UX1700 MSE' material is used in reinforced earth structure instead of 'Tensar UX1600 MSE in Model 2. All other features of this analysis are same as Model 2. Aim of this analysis is to investigate influence of reinforcement stiffness to the system’s stability. In order to do that UX1700 MSE geogrid with EA values of 2350 kN/m has been used instead of UX1600 with 1800 kN/m EA value.

<u>10.5.3.1.Results of Analysis with Plaxis Software.</u> Obtained displacement, stress and safety factor values from the analysis of Model 3 are represented at Table 10.8.

Table 10.8. Analysis results

Analysis results	**Value**	**Unit**
Maximum total displacement	84	mm
Maximum horizontal displacement	58	mm
Maximum vertical displacement	60	mm
Maximum effective stress	1590	kN/m^2
Maximum total stress	1590	kN/m^2
Factor of safety (ΣM_{sf})	1,35	
Factor of safety (ΣMsf)-Dynamic case	1.07	

ΣM_{sf} - total displacement graphics of points A, B, C, D and E are shown at Figure 10.14. At mentioned figure, curves are getting horizontal at 1.35 M_{sf} value which means the safety factor for the slope is 1.35

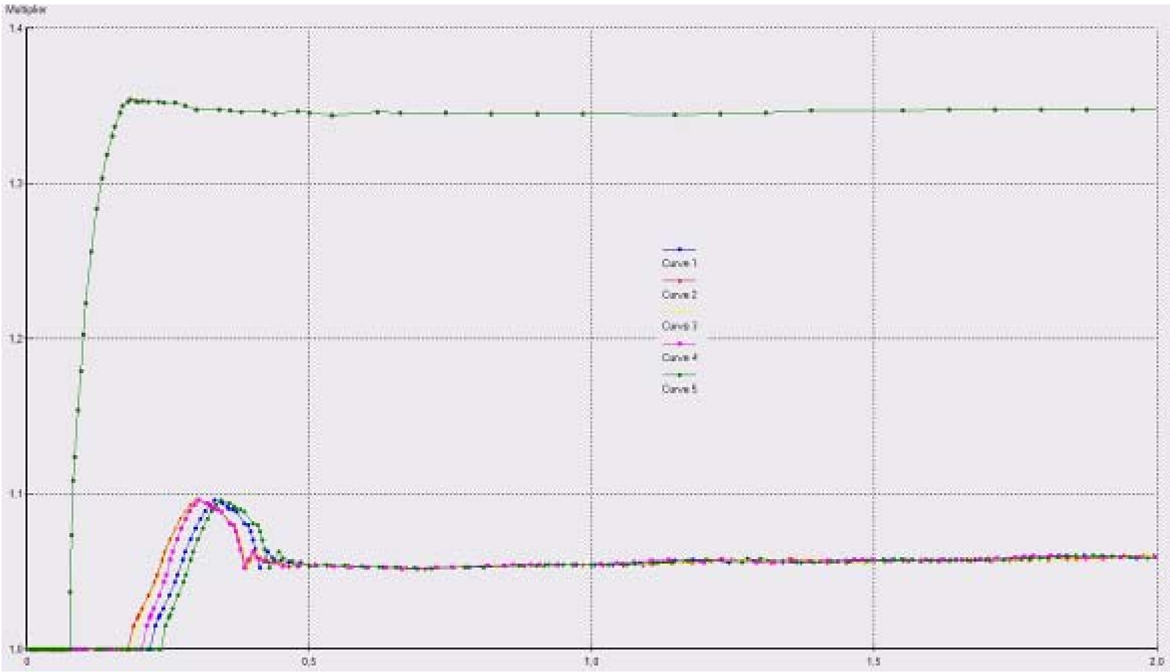


Figure 10.14. M_{sf}- Displacement Curve for Model 3

"Total Increments" graph where critical sliding circles can be seen is represented at Figure 10.15.

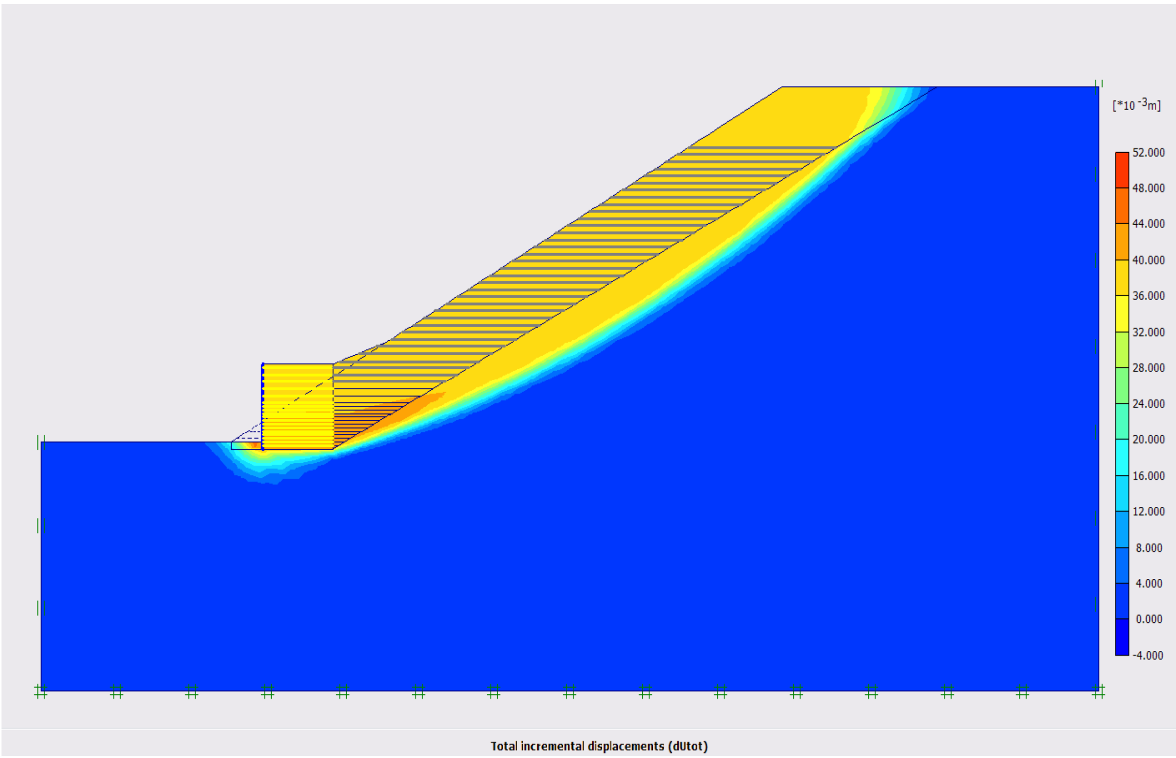


Figure 10.15. Total increments

Results and Controls of Reinforced Earth Retaining Wall:

Control criteria for Plaxis analyses applied on reinforced earth retaining walls were given in Table 10.2. At Table 10.9., comparison of the mentioned criteria and obtained results after analyze are represented.

Table 10.9. Comparison of results

	Controls	**Calculation method**	**Obtained value**	**Allowable value**	**Unit**	**Evaluation**
Displacement control	wall maximum horizontal displacement	Maximum horizontal displacement of point A	21	120	mm	In acceptable limits
Settlement Controls	wall maximum settlement	Maximum of the settlements occurred at points B and C	60	75	mm	In acceptable limits
	wall maximum differential settlement	Difference of the settlement values at point B and C	11	32	mm	In acceptable limits
	wall maximum angular settlement	The ratio of the difference between maximum vertical settlement values of points B and C to the wall width (L)	$1*10^{-3}$	L/60 = 183	mm	In acceptable limits
Bearing capacity Control	Bearing capacity check	Maximum stress at wall base	309	1685	kN/m^2	system is safe as to required bearing capacity conditions
Rupture control	allowable tension resistance of reinforcement	maximum of tensions stress occurred at reinforcements	8	46	kN/m	There is not any risk of rupture for reinforcements

Considered base cross section and settlement values of some points over this section at settlement control of Model 2 are represented below. Figure 10.16.

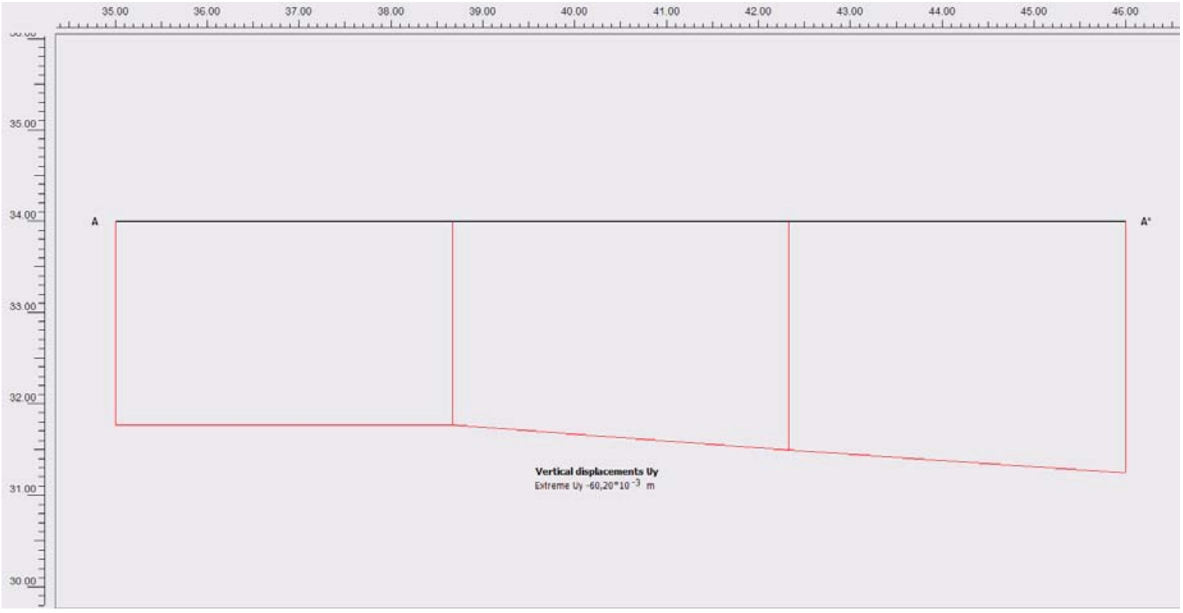


Figure 10.16 Vertical displacement distribution at section A-A' between points B-C

Table 10.10. Settlement values between points B and C for Model 3

X [m]	Y [m]	U_y [m]
35,003	33,998	-0,049
38,669	33,998	-0,049
42,334	33,998	-0,055
45,999	33,998	-0,060
45,999	33,998	-0,060
46,002	33,998	-0,060

Base section that is taken into account in control of bottom collapse analysis in Model 3 is shown in Figure 10.17. below.

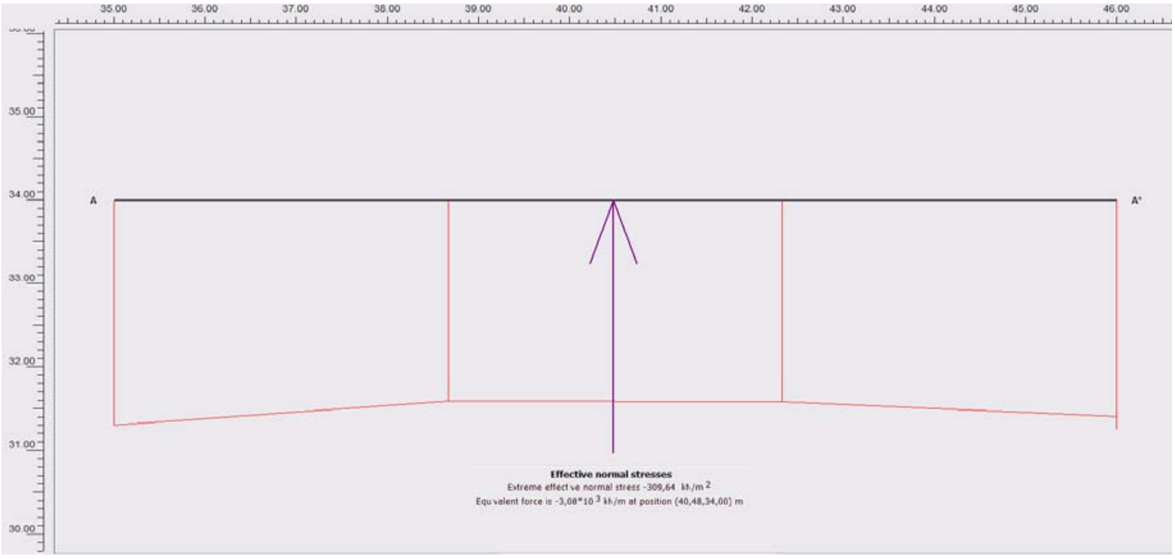


Figure 10.17 Effective stress distribution at section A-A' between points B-C

Table 10.11. Stress values between points B and C for Model 3

X [m]	Y [m]	σ'_N [kN/m²]
35,003	33,998	-303,843
38,669	33,998	-270,576
42,334	33,998	-271,717
45,999	33,998	-291,694
45,999	33,998	-309,640
46,002	33,998	-309,604

10.5.4. Model 4 – Analysis Definitions

In Model 4, in addition to the Model 2, a reinforced slope has been added to the back of the wall as seen in Figure 10.18. Aim of this analysis is to determine the effect of reinforced slope to the stability of system by using Tensar UX1600 MSE geogrid reinforcement.

In this analysis, properties of wall geometry, reinforcement and fill material are the same as in Model 2, but 'Tensar UX1600 MSE' reinforcement is placed inside the slope at the back of the wall which was generated during wall construction, by 1m intervals of 14 orders as can be seen in Figure 10.18. Also at the back of the wall and at the reinforced fill, natural soil material had been used. Plaxis calculations and graphics for this analysis are shown in Figures 10.18 and 10.19.

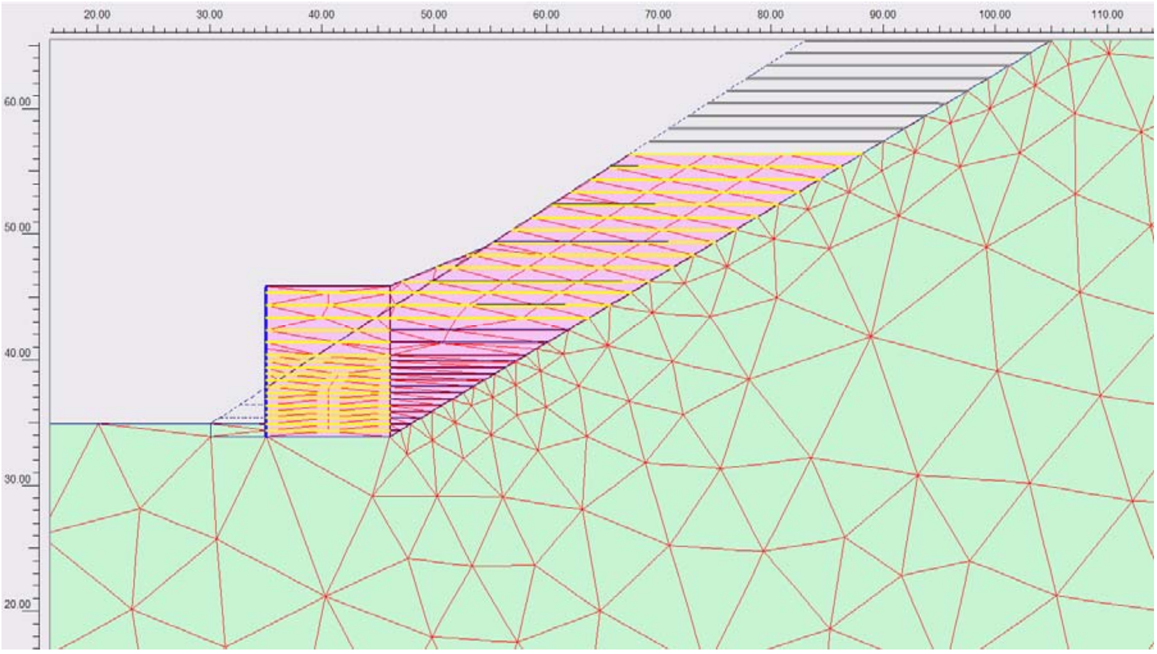


Figure 10.18 Boundary conditions and structure geometry for Model 4

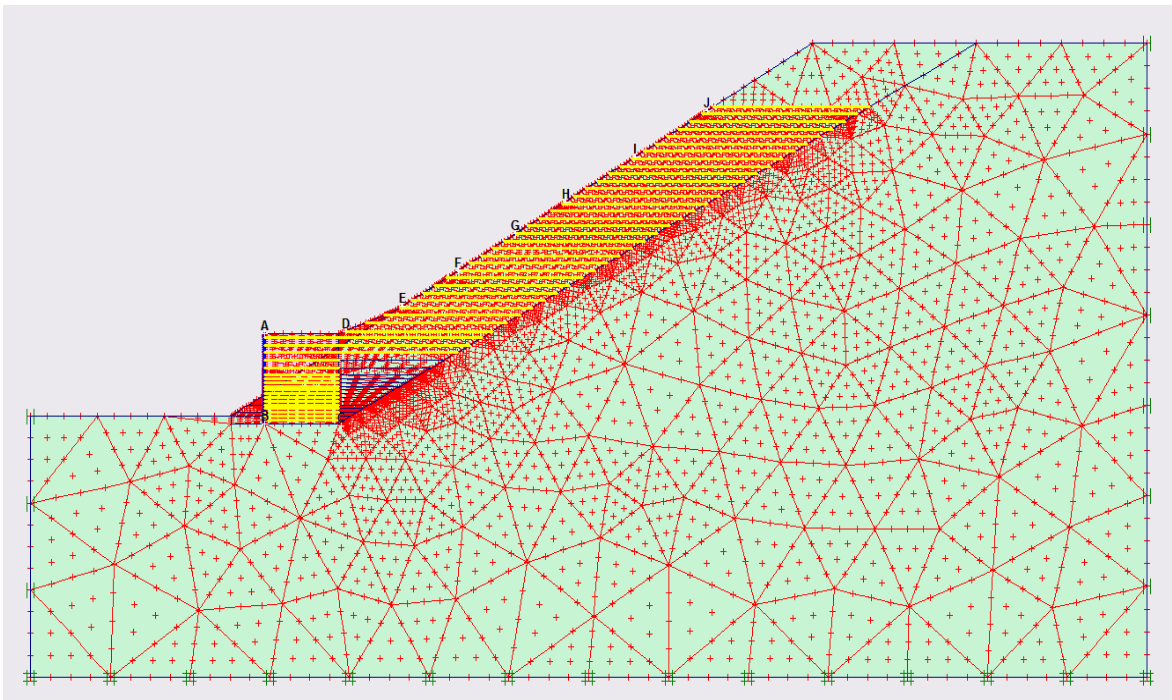


Figure 10.19. Connectivities for Model 4

<u>10.5.4.1.Results of the Analyses with Plaxis Software.</u> Obtained displacement, stress and safety factor values from the analyze of Model 4 are represented at Table 10.12.

Table 10.12. Analysis results

Analysis result	**Value**	**Unit**
Maximum total displacement (Slope at the back of wall)	84	mm
Maximum horizontal displacement (Slope at the back of wall)	58	mm
Maximum vertical displacement (Slope at the back of wall)	60	mm
Maximum effective stress	1590	kN/m^2
Maximum total stress	1590	kN/m^2
Factor of safety(ΣM_{sf})	1,4	
Factor of safety (ΣM_{sf})-Dynamic case	1.1	

ΣM_{sf} - total displacement graphics of points A, B, C, D, E, F, G, H, I and J are shown at Figure 10.20. At mentioned figure, curves are getting horizontal at 1.4 M_{sf} value which means the safety factor for the slope is 1.4.

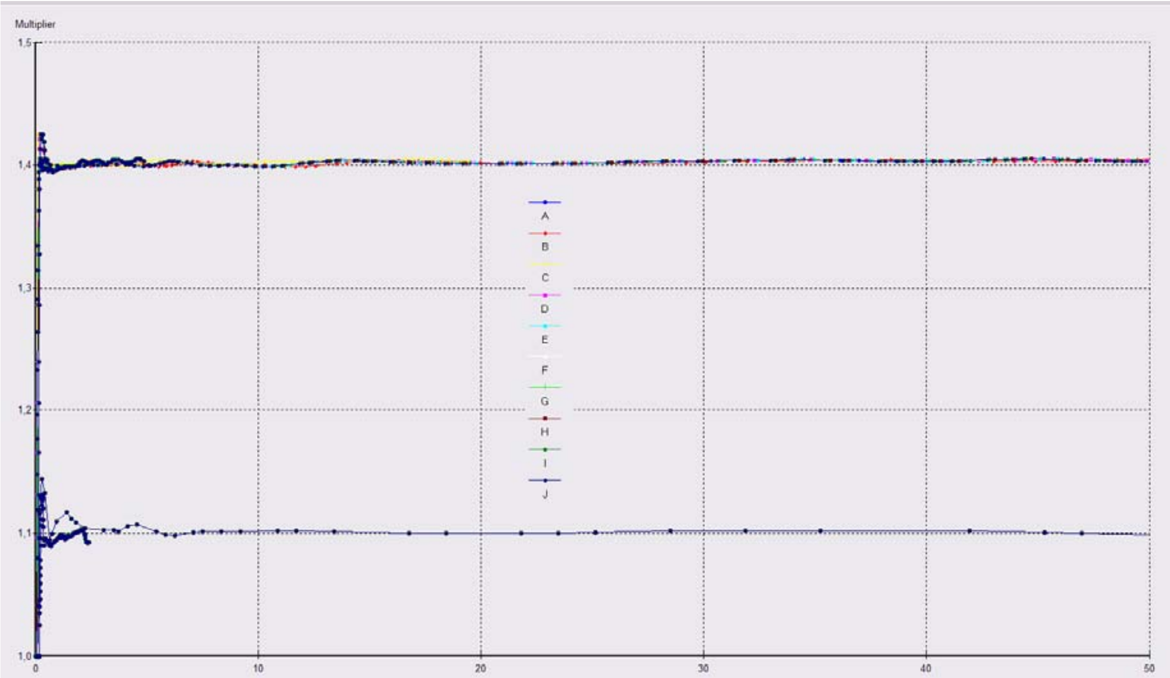


Figure 10.20. ΣM_{sf} - total displacement graphics for selected points

“Total Incremental Displacements” graph where sliding circles can be seen is represented at Figure 10.21.

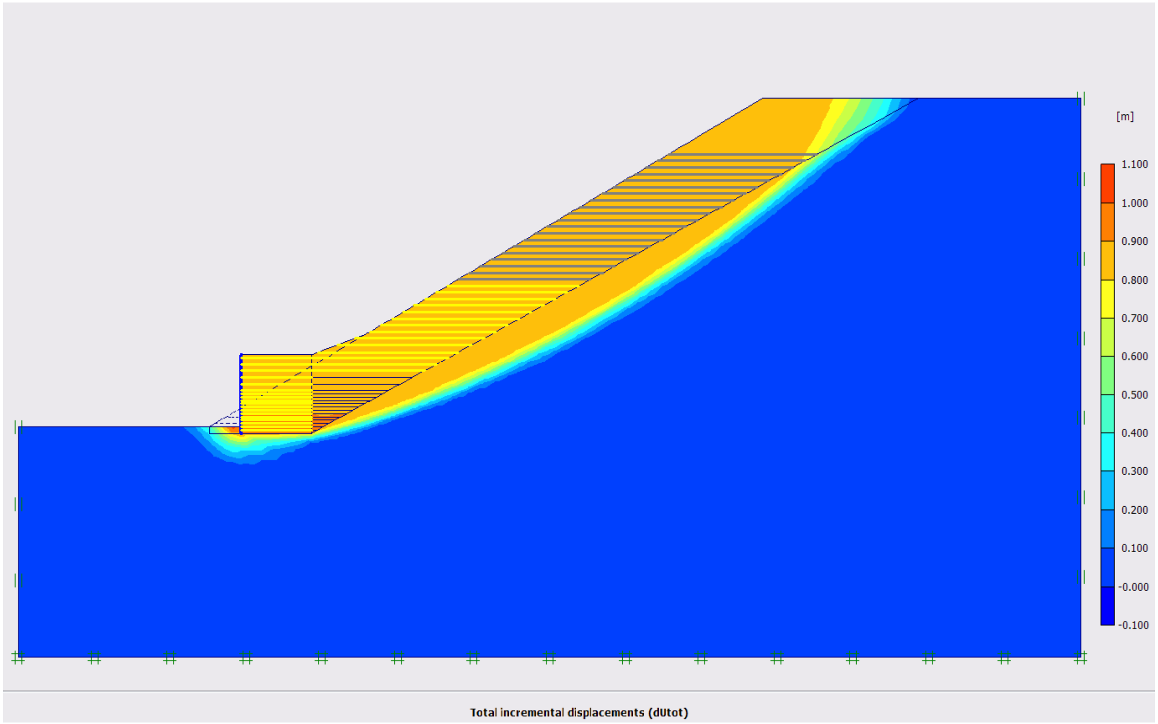


Figure 10.21. Total incremental displacements

Control criteria for Plaxis analyses applied on reinforced earth retaining walls were given in Table 10.2. and 10.7. At Table 10.9., comparison of the mentioned criteria and obtained results after analyses are represented.

Table 10.13. Comparison of results

	Controls	Calculation method	Obtained value	Allowable value	Unit	Evaluation
Displacement control	wall maximum horizontal displacement	Maximum horizontal displacement of point A	17	120	mm	In acceptable limits
Settlement Controls	wall maximum settlement	Maximum of the settlements occurred at points B and C	61	75	mm	In acceptable limits
	wall maximum differential settlement	Difference of the settlement values at point B and C	12	32	mm	In acceptable limits
	wall maximum angular settlement	The ratio of the difference between maximum vertical settlement values of points B and C to the wall width (L)	$1*10^{-3}$	L/60 = 183	mm	In acceptable limits
Bearing capacity Control	Bearing capacity check	Maximum stress at wall base	380	1685	kN/m^2	system is safe as to required bearing capacity conditions
Rupture control	allowable tension resistance of reinforcement	maximum of tensions stress occurred at reinforcements	10	46	kN/m	There is not any risk of rupture for reinforcements

Bottom section that is taken into account in settlement control and settlement values at some points on this section for analysis of Model 4 are shown in Figure 10.22. below.

Figure 10.22. Vertical displacement distribution at section A-A' between points B-C

Table 10.14. Settlement values between points B and C for Model 4

X [m]	Y [m]	U_y [m]
35,001	33,997	-0,048
35,006	33,997	-0,048
35,006	33,997	-0,048
38,670	33,997	-0,049
42,335	33,997	-0,055
45,999	33,997	-0,060
45,999	33,997	-0,060
46,003	33,997	-0,061

Bottom section that is taken into account in control of bottom collapse analysis in Model 4 is shown in Figure 10.23. below.

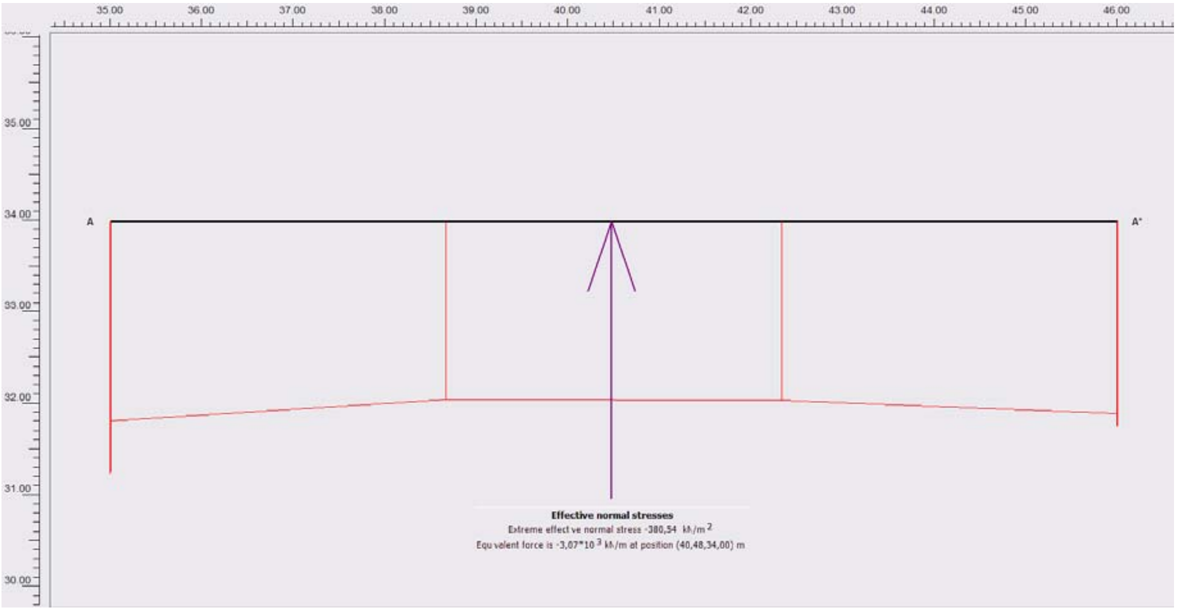


Figure 10.23. Effective stress distribution at section A-A’ between points B-C

Table 10.15. Stress values between points B and C for Model 4

X [m]	Y [m]	σ'_N [kN/m ^{2}]
35,001	33,997	-380,539
35,006	33,997	-380,325
35,006	33,997	-301,994
38,670	33,997	-269,808
42,335	33,997	-271,382
45,999	33,997	-291,178
45,999	33,997	-309,557
46,003	33,997	-309,497

10.6. Total Collapse Analysis for Natural Slope and Model Reinforced Earth Retaining Wall

10.6.1. Swedish Slice Method for Natural Slope

In Section 8.5 total collapse stability of a model slope in Figure 8.3. has been calculated for any possible sliding circle that passes through the toe zone. But as explained before, it is very difficult to determine the most critical sliding circle by hand calculations and it is strongly encouraged to use computer softwares in order to determine the location of sliding circle more accurately.

Plaxis finite element software described in Section 9 is one of these programs. Total collapse stability of mentioned slope has been investigated by Plaxis software under the title of Model 1. At the end of this analysis, location of critical sliding circle of natural slope is determined as shown in Figure 10.24. and calculation stages of Swedish Slice Method are represented at Table 10.16. Since the location of failure circle is determined with Plaxis software, the total collapse analysis of the natural slope by Swedish Slice method can be preformed more accurately.

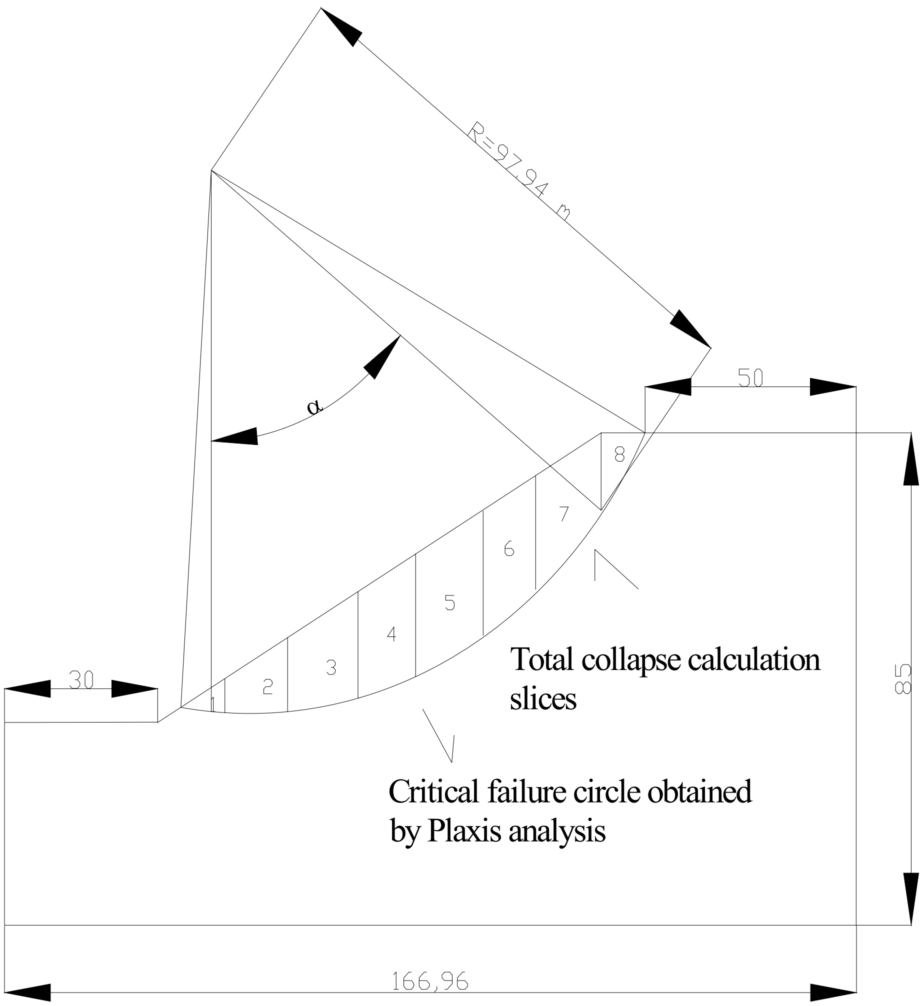


Figure 10.24. Swedish Slice method with failure circle obtained by plaxis

Table 10.16. Calculation stages in Swedish Slice method

Slice No	Slice Area m^2	$a(^0)$	W (kN)	Wsina (kN)	Wcosa (kN)
1	6	-3	120	-6	119
2	54	3	1080	56	1078
3	164	10	3280	570	3230
4	234	20	4680	1600	4379
5	259	31	5180	2667	4440
6	234	44	4680	3251	3366
7	96	50	1960	1470	1234
8	22	60	440	381	220
Total				9989	18084

$$FS_{total} = \frac{\sum W(\cos a) \times \tan \phi_f \ + c \times l}{\sum W(\sin a)}$$

$$FS_{total} = 1.1$$

Factor of safety against total collapse had been calculated as 1.053 for the same sliding circle by Plaxis Software.

10.6.2. Swedish Slice Method for Reinforced Earth Retaining Wall

Stability against total collapse of model reinforced earth retaining wall that was studied on Section 8.9 was calculated as to any possible sliding circle that passes through toe zone. In this section, the same check is repeated for the real critical circle that has been obtained by Plaxis analysis of Model 2 and final calculation table is represented at Table 10.15.

Repeat of total collapse analysis of reinforced earth retaining wall as to critical sliding circle determined by Plaxis Software is shown in Figure 10.25 and Table 10.17.

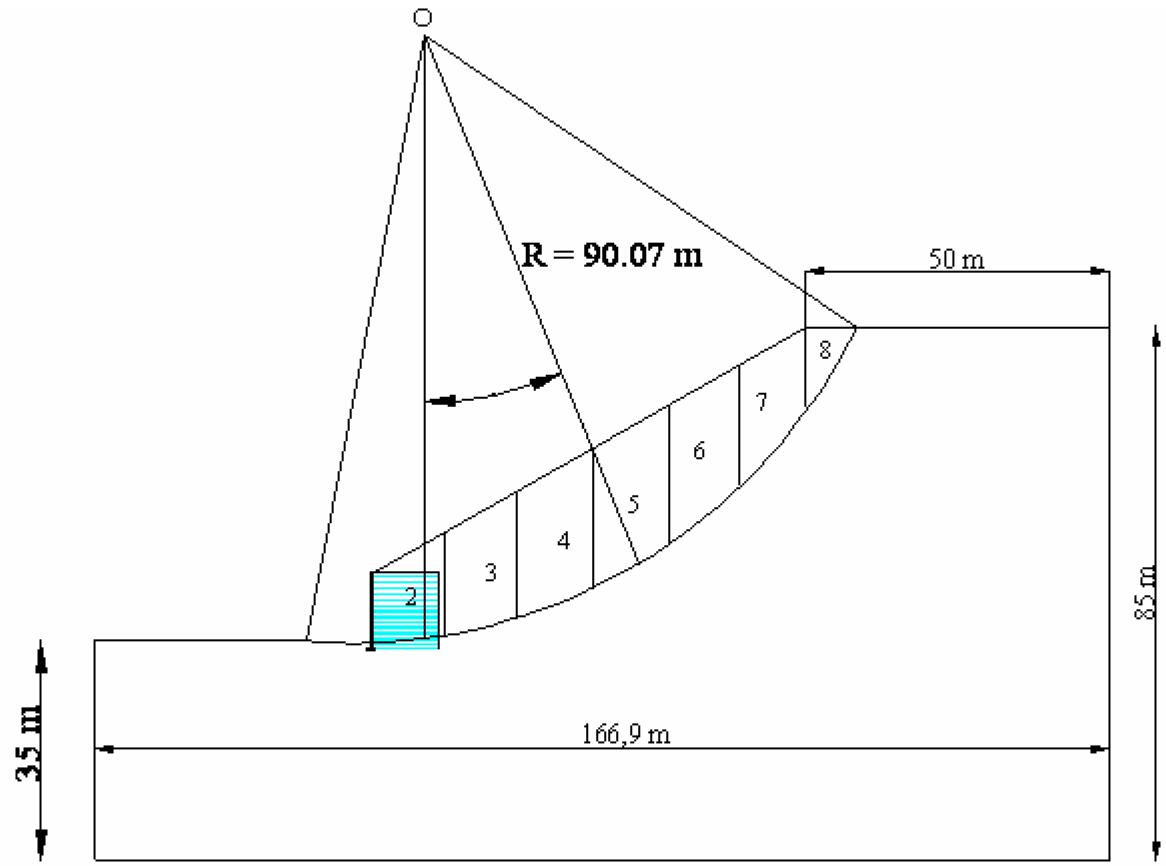


Figure 10.25. Final position of reinforced earth retaining wall as to slice method

Table 10.17. Total collapse calculation stages

Slice No	Slice Area m2	a(^{0})	(kN)	Wsina (kN)	Wcosa (kN)
1	4	-8	80	-11	79
2	98	-3	1960	-102	1957
3	190	5	3800	331	3785
4	268	14	5360	1296	5200
5	317	25	6340	2679	5745
6	314	36	6280	3691	5080
7	209	48	4180	3106	2796
8	53	64	1060	952	464
Total				134638	25106

$$FS_{total} = \frac{\sum W(\cos a) \times \tan\phi_f + \sum l_d \times T_{al}}{\sum W(\sin a)} = \frac{25106 \times \tan 28 + 46.07 \times 90.07}{13638} = 1.28$$

$$FStotal = 1.28$$

Factor of safety against total collapse had been calculated as 1.33 for the same sliding circle by Plaxis Software previously.

10.7. Comparison of Results

After determining wall dimensions by considering preliminary design criteria of related specifications and regulations, exterior and interior stability analysis were performed in order to investigate stability condition of proposed model wall.

Obtained results of analysis under static and dynamic loading cases are represented in Table 10.18. below. All of the allowable values for stability controls are obtained form regulations of U.S Department of Transportation in 2001.

Table 10.18. Results of analysis under static and dynamic loading cases

		Analysis results of model Reinforced Earth Retaining wall			
For H=12m and L=9m interior and exterior stability checks	**Static and seismic exterior stability checks**	**Inspections**	**Value**	**Allowable Value**	**Result**
		Eccentricity check	e=0.43	<L/6=1.5 m	Wall dimensions are sufficient
		Bearing Capacity Check	$\sigma_v = 284$ kN/m^2	$\sigma_v < q_{all} = 1685$kN/m"	Wall dimensions are sufficient
		Sliding Check	$FS_{sliding} = 2.7$	>1.5	Wall dimensions are sufficient
		Over turning check	$FS_{overturning} = 6.55$	>2	Wall dimensions are sufficient
		Seismic sliding	$FS_{sliding} = 1,07$	<1.125 (1.5*0.75)	**Wall dimensions should be increased**
		Seismic bearing capacity	$\sigma_v = 666$ kN/m^2	$\sigma_v < q_{all} = 1685$ kN/m^2	Wall dimensions are sufficient
		Seismic eccentricity check	e = 2.65m	<L/3=3 m	Wall dimensions are sufficient
For H=12m and L=11m interior and exterior stability checks	**Static and seismic exterior stability checks**	**Eccentricity check**	e=0.30m	<L/6=1.83 m	Wall dimensions are sufficient
		Bearing Capacity Check	$\sigma_v = 268$ kN/m^2	$\sigma_v < q_{all} = 1685$ kN/m^2	Wall dimensions are sufficient
		Sliding Check	$FS_{sliding} = 2.9$	>1.5	Wall dimensions are sufficient
		Over turning check	$FS_{overturning} = 8..66$	>2	Wall dimensions are sufficient
		Seismic sliding	$FS_{sliding} = 1.16$	<1.125 (1.5*0.75)	Wall dimensions are sufficient
		Seismic bearing capacity	$\sigma_v = 3,92$ kN/m^2	$\sigma_v < q_{all} = 1685$ kN/m^2	Wall dimensions are sufficient
		Seismic eccentricity check	e=2.36m	<L/3 = 3.66 m	Wall dimensions are sufficient
	Static and seismic interior stability checks	**Static rupture check**	$FS_{sliding} = 1.9$	>1.3	Length of reinforcement is sufficient
		Static pull-out check	$L_e = 5.60$m	>1 m	Length of reinforcement is sufficient
		Seismic breaking check	$FS_{rupture} = 1.5$	>0.975	Length of reinforcement is sufficient
		Seismic pull-out check	$L_e = 5.02$m	>1 m	Length of reinforcement is sufficient

First exterior stability checks for mentioned structure of 12m height have been achieved by taking L=0.7H with wall width of 9 m. as a preliminary design principle. In all exterior collapse controls, sufficient factor of safety values were obtained. In checks for H=12 and L= 9 m cases under dynamic loading, safety was not found sufficient only for sliding checks where most critical situation occurred ($FS_{sliding}$=1.07 < 1.125). For this reason, width of wall has been increased to improve safety of system against sliding (L=11m). So, weighted structure can resist sliding better.

In second part of this study, analysis of the model reinforced earth retaining wall which's dimensions were determined by hand calculations and the natural slope at its initial condition, has been performed by Plaxis Software (Model-2) in order to observe the influence of mode wall to the systems stability. Moreover, the reinforcement type has been changed (Model-3) and backslope of the model wall has been reinforced (Model-4) in order to observe the influence of these variable parameters to the systems stability. Results of the analyses are shown in Table 10.15 below and again, all of the allowable values for stability controls are obtained form regulations of U.S Department of Transportation, 2001.

Table 10.19. Results of static analysis performed by Plaxis software

Definitions for checks			Values: Model 1	Values: Model 2	Values: Model 3	Values: Model 4	Values: Allowable Value (FHWA,1996)	Values: Unit
General checks	Maximum horizontal displacement			84	84	84	120	mm
General checks	Maximum vertical settlement			60	60	60	75	mm
General checks	Maximum effective stress		1590	1590	1590	1590		kN/m^2
	Slope stability check		1,05	1,33	1,35	1,40		
Checks for reinforced earth retaining wall	Displacement check	maximum horizontal displacement of wall		29	21	17	120	mm
Checks for reinforced earth retaining wall	Settlement checks	maximum settlement of wall		60	60	60	75	mm
Checks for reinforced earth retaining wall	Settlement checks	maximum differential settlement of wall		11	11	12	32	mm
Checks for reinforced earth retaining wall	Settlement checks	maximum angular settlement of wall		$1*10^{-3}$	$1*10^{-3}$	$1*10^{-3}$	L/60 = 183	mm
Checks for reinforced earth retaining wall	Bearing capacity checks	Check for bearing capacity		388	385	380	1685	kN/m^2
Checks for reinforced earth retaining wall	Checks for rupture	Rupture check for reinforcement		10	8	10	46	kN/m

11. CONCLUSIONS

In evaluation of engineering structures, problem of slope stability and required retaining structures for them have been one of the major factors that engineers have to deal with.

In which conditions, a slope that is stable in its current state can be formed, in which conditions retaining structures are needed, and which retaining structure type should be selected. All these questions affect all progress of the project, so appropriate decisions must be made.

Flexible retaining walls are preferred comparing to rigid and semi-rigid types because of insufficiency of bearing capacity of foundation soil for traditional walls, work difficulty near watery zones and expensive rescue methods for other types, and easy construction, necessity of forming temporary wall and making economy of them by re-using.

Included in flexible retaining structure types, reinforced earth retaining walls have been used frequently in transportation structures for the last 20 years. Aim of using steel, polymer strip or geogrids is to supply tension resistance that does not exist in the soil. One of the major advantages of this type of retaining structures are; to be large mass and ability of being raised on slope or in front of the slope before excavation is held if necessary. Economical applications can be achieved on loose soils since there is not any matter of bearing capacity problem in those walls due to their significant widths. Reinforced earth retaining structures are generally held successfully and also they can be applied progressively and confidently as displacements commonly do not exceed allowable limits when exposed by extreme earthquake accelerations.

This study was performed in order to maintain the principles of analysis and design of reinforced earth retaining walls from the view of static and dynamic loading.

After stability controls of 12m height model reinforced earth retaining wall, it was seen that L=0.7H principle at preliminary design is sufficient for static design but for dynamic loading, length of reinforcement should be brought to at least 0.9H levels.

Also in studied problem, it has been observed that 20 per cent increase in reinforcement length causes a 10 per cent increase in sliding stability.

In second part of this study, analysis of the model reinforced earth retaining wall which's dimensions were determined by hand calculations and the natural slope at its initial condition, has been performed by Plaxis Finite Element Software.

First analysis achieved by Plaxis (Model l) had been performed in order to determine the stability condition of natural slope against total collapse and this factor of safety value was found as 1.053 which means, the natural slope is in failure condition.

In the second analysis achieved by Plaxis software (Model 2), analysis has been performed with respect to the geometry and reinforcement features that were determined previously in order to increase stability of slope.

At the end of the analysis, it has been observed that factor of safety against total collapse increased significantly from 1.053 to 1.33. Additionally, checks done at the end of analysis for bearing capacity, reinforcement rupture risk and settlement have been found as over-safe as seen in Table 10.2.

In the third analysis performed by Plaxis software (Model 3), influence of reinforcement stiffness to the system's stability was investigated. For this reason, analysis performed in Model 2 was repeated by using "Tensar UX1700 MSE Geogrid" reinforcement with stretching rigidity (EA) of 2350 kN/m instead of "Tensar UX1600 MSE Geogrid" reinforcement with stretching rigidity (EA) of 1800 kN/m.

At the end of Model 3 analysis, it was found that increase in EA causes a small decrease in tension stress on reinforcement. No considerable change was observed in settlement and stability values.

Last analysis performed by Plaxis (Model 4) aims to increase the factor of safety value against total collapse that was calculated in Model 3. As to this aim, a reinforcing application has been added at excavated back slope area by installing 'Tensar UX1600 MSE' reinforcements with 14 orders and 1m intervals. At the end of analysis, factor of safety against total collapse was increased from 1.35 to 1.40.

Main reason of the influence of reinforced slope to system is; leaving most of the reinforcements at outside of sliding circle. So it proves that, while searching solution for slope stability, it is absolutely needed to determine the sliding type (deep or shallow) and to designate the place that sliding circle passes correctly. Since the factor of safety value has been significantly increased from 1.0 (Failure condition) to 1.4 levels, then it can be said that the slope has been taken into safe position against collapse. It is also observed that the reinforced slope also causes a 50 per cent decrease at horizontal displacements of retaining wall as seen in Table 10.2.

At the last section of this study, total collapse analyses performed in Plaxis and Swedish Slice Method for reinforced earth retaining walls were compared. At the beginning of the study, total collapse analysis had been achieved by slice method for a possible sliding circle that passes through the toe zone of natural slope and system with reinforced earth retaining wall. Sliding circles counted in these analyses were not known as critical circles. They were just thought to be critical since they pass through the toe. For this reason, total collapse safety values were not certain. But at the last part of the study, by calculations done with Plaxis software, location of critical sliding circles for both natural slope and slope after construction of reinforced earth structure were determined. As a result, critical sliding circles found for both two cases are repeated correctly in analyses done by Swedish slice method. Factor of safety value for natural slope was found as 1.053 by Plaxis and for the same sliding circle, it was found as 1.0 by Swedish Slice Method. Similarly, for reinforced earth retaining wall model studied, factor of safety value against total collapse was found as 1.33 by Plaxis and 1.35 by Swedish Slice Method.

Very close results achieved in two analyses shows that the analyses performed are consistent.

REFERENCES

Afataoğlu, H. A., 2004, *Yumuşak Kil Zemine Oturan Kum Dolgunun Taşıma Gücü Güvenliğinin Geotekstiller ve Taş Kolonlarla Arttırılması*, M.S Thesis, İTÜ

Specification for Structures to be Built in Disaster Areas, 1998. İnşaat Mühendisleri Odası İzmir Şubesi Yayın No: 25, İzmir.

American Association of State Highway and Transportation Officials (AASHTO), 1967, *Manual of Geotechnical Investigations*, National Press Building, Washington.

American Association of State Highway and Transportation Officials (AASHTO),1996, *Standard Specifications for Highway Bridges*, 16th Edition, Washington.

Arı, S., 1998. Toprakarme Sistemi ve Türkiyede ki Uygulamaları, *İnşaat Mühendisleri Odası teknik Dergi*, İstanbul.

Bedal, A. M., 1997. *Finitie Element Analysis of Geosynthetic Reinforced Soil Retaining Wall Subjected To Seismic Loading*, Ph.D. Dissertation, University of Mississippi.

Bowles, E. J., 1997. *Foundation Analysis and Design*, McGraw-Hill Companies, Inc., Singapore.

Burgess, C. G. P., 1999. *Two Full Scale Model Geosynthetic Reinforced Segmental Retaining Wall*, Royal Military College of Canada Kingston, Ontario.

Coduto, D. P., 19994. *Foundation Design Principles and Practices*, Prentice Hall, California.

Craig, R. F., 1987, *Soil Mechanics*, ELBS, Berkshire

Das, B. M., 2001, *Principles of Geotechnical Engineering*, Thomson. California

Das, B. M., 1987, *Advanced Soil Mechanics*, McGraw-Hill Companies, Inc, Singapore

Demirkan, M. M., 1999, *Seismic Response of Reinforced Soil Retaining Structures*, M.S Thesis, Boğaziçi University.

Durukan, Z. S., 1988, *Reinforced Soil Retaining Walls*, M.S Thesis, İTÜ.

East İnşaat ve Dış Tic. A.Ş., 2003, *Yol ve Otoyol İnşaatlarında Polimer Şerit Donatılı Toprak Duvar Uygulamaları için Genel Tasarım Yöntemleri*, Ankara.

French Ministry of Transportation, 1980, *Reinforced Earth Structures Recommendation and Rules of The Art.*

GeoSyntec Consultants, 1997, *Earth Retaining Systems,* Geotechnical Engineering Circular No: 2, Federal Highway Administration, FHWA-SA-96-038, Washington.

Güner, E., 1994, Geoteknik Mühendisliğinde Geosentetikler, *Aylık Teknoloji Dergisi*, İstanbul.

Han, J., Gabr, M. A., 2001, "Numerical Analysis of Geosytnthetic Reinforced and Pile Supported Earth Platforms Over Soft Soil", *Journal of Geotechnical and Geoenvironmental Engineering.*

Helwany, M. B., D, McCallen., 2001, "Seismic Analysis of Segmental Retaining and Effects of Facing Details", *Journal of Geotechnical and Geoenvironmental Engineering.*

Karalı, Y., 2004, *Donatılı Zemin İstinat Duvarlarının Deprem Davranışlarının Çeşitli Deprem Yönetmeliklerine göre İncelenmesi*, M.S Thesis, İTÜ.

Kesim, R. S., 1996. *Donatılı Zemin Yapılarının Sistem Davranış Özellikleri*, M.S Thesis, İTÜ.

Koerner, M. R., 1998, *Design with Geosynthetics*, Prentice Hall.

Kramer, L. S., Çev. Katabalı Kamil, 2003, *Geoteknik Deprem Mühendisliği*, Fersa Matbaacılık, Ankara.

Kumbasar, V., Kip, F., 1999, *Zemin Mekaniği Problemleri*, Çağlayan Basımevi, İstanbul.

Lambe, P. C., Hansen, L. A., 1990, *Design and Performance of Earth Retaining Structures*, American Society of Civil Engineers Geotechnical Publication No: 25, Washington

Ling, L., Victor, N. K., Huabei, L., 2004, Analyzing Dynamic Behavior of Geosynthetic-Reinforced Soil Retaining Walls, *Journal of Geotechnical and Geoenvironmental Engineering.*

Mifari Construction Products Company Publication, 2003, *Guidelines for the Design Specification of Geosytnthetic Reinforced Soil Reatining Structures.*

Ozden, K., A, Trupia., İ, Eren. ve T, Öztürk., 1995, Betonarme İstinat Duvarları ve Perdeleri, *İTÜ Yaınları*, İstanbul.

Smith, R. J. H., İ, Ünal., 1989, “Donatılı Toprak Projelendirme ve Malzemelere İlişkin Notlar”, *İnşaat Mühendisleri Odası 10. Teknik Kongre Bildirileri Kitabı* Cilt 2, Ankara.

Sowers, G. B., and Sowers, G. F. (1970), *Introductory Soil Mechanics and Foundations,* Macmillan, New York.

Tennessee Department of Transportation, 2006, *Earth Retaining Structures Manual,* Tennesse.

Tensar benchmark website, http://www.Tensar.com, access date: 14.02.2007

Tezcan, S. S., Buket, Z. S., 1999, *Design of Reinforced Soil Retaining Wall Including Seismic Performance Principles*, Türk Deprem Vakfi, İstanbul.

Tezcan, S. S., Buket, Z. S., 1999, *Donatılı Zemin İstinat Duvarları için Sayısal Örnekler*, Türk Deprem Vakfi, İstanbul.

TS 7994, 1990, *Zemin Dayanma Yapıları; Sınıflandırma, Özellikleri ve Projelendirme Esasları*, Ankara.

U.S. Department of Transportation, 1997, *Geotechnical Earthquake Engineering for Highways* Volume 1 Design principles, Geotechnical Engineering Circular No: 3, FHWA, Washington.

U.S. Department of Transportation, 1998, *Geosynthetic Design and Construction Guidelines Participant Notebook*, Publication No: FHWA HI-95-038, National Highway Institute Course No: 13213, FHWA, Washington.

U.S. Department of Transportation, 2001, *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines,* Publication No: FHWA-NHI-00-043, FHWA Washington.