

BEHAVIOR OF REINFORCED EARTH STRUCTURES UNDER EARTHQUAKES

by

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ABSTRACT

BEHAVIOR OF REINFORCED EARTH STRUCTURES UNDER EARTHQUAKES

This study is focused on presenting a literature review and evaluation of types and general concepts of all earth reinforcement structures, especially reinforced earth retaining walls with metallic strip reinforcements, their applications, design methods, components, behavior under seismic loads. Case studies including behavior of structures during previous earthquakes and an example of applied project are investigated. In addition introducing numerical analysis to Reinforced Earth walls and estimating the displacements formed in the structures.

Design calculations were carried out for a 6.73m high Reinforced Earth wall, under seismic loading with maximum horizontal acceleration ($a_0/g = 0.4$). By using VALDEZ software which uses limit state approach; solutions including strip length, reinforcement density were determined. Internal and external stability analyses were evaluated. These length and densities were used for 1m width of wall in PLAXIS software. Horizontal and vertical displacements, axial forces on the reinforcements were calculated. The deformations and forces were in the applicable limits.

Reinforced Earth structures have proven to be safe and flexible in the presence of seismic events throughout the world. Current design codes apply a very conservative approach, especially in the determination of external stability. A plastic deformation approach be taken instead in the design of Reinforced Earth structures; whereby consideration for seismic design will be based on wall height, ground acceleration, and allowable deformation. The wall height will determine how much reliance needs to be paid to seismic design, with lower height walls being less restrictive than moderate to tall walls.

ÖZET

DONATILI ZEMİN DUVAR YAPILARININ DEPREMDE DAVRANIŞI

Bu çalışma, donatılı zemin duvar yapılarının her tipine ve genel özelliklerine ilişkin literatürde yer alan bilgilerin sunulması, özellikle metal şeritlerle donatılandırılmış istinat duvarlarının uygulamalarına, dizayn metodlarına, bileşenlerine ve deprem yükleri altındaki davranışlarına odaklanmıştır. Söz konusu yapıların daha önceki depremlerde nasıl davrandığını kapsayan vaka analizleri ve depremselliğin ciddi rol oynadığı bir bölgede projesi yapılmış bir duvar örnek olarak incelenmiştir. Ayrıca, donatılı zemin (toprakarme) duvarların numerik analizlerinin nasıl yapıldığından bahsedilmiş ve bu tür yapılarda meydana gelen deplasmanlar değerlendirilmiştir.

6.73m yüksekliğe sahip donatılı zemin duvarı için, maksimum yatay yer ivmesi 0.4 ($a_0/g = 0.4$) olacak şekildeki deprem yükü altında dizayn hesapları yapılmıştır. Hesapları limit durum yaklaşımına göre yapan VALDEZ programı kullanılarak, şerit uzunluğu ve donatı yoğunluğunu içeren sonuçlara ulaşılmıştır. İç ve dış stabilite analizleri yapılmıştır. Elde edilen şerit uzunluğu ve yoğunluğuna ait değerler, PLAXIS programında 1m genişlik için yapılan hesaplarda kullanılmıştır. Yatay ve dikey deplasmanlar ile şeritlerde oluşan aksel kuvvetler hesaplanmıştır. Söz konusu deformasyonların ve kuvvetlerin kabul edilebilir sınırlar içinde olduğu görülmüştür.

Donatılı zemin duvar yapıları, dünya çapında meydana gelen depremlere bakıldığında güvenli ve esnek yapılar olduklarını kanıtlamışlardır. Özellikle dış stabilite analizleri açısından, günümüzde kullanılan dizayn kodlarının çok güvenli tarafta kaldığını gösterir bir yaklaşıma sahip olduğu görülmektedir. Donatılı zemin duvarların deprem yükleri altındaki dizaynı için, duvar yüksekliği, yer ivmesi ve kabul edilebilir deformasyonlar baz alınarak; plastik deformasyon yaklaşımı ile hesaplar yapılabilir. Duvar yüksekliği, sismik dizayn yapılırken ne derece bir hassasiyete gereksinim duyulacağı konusunda belirleyici olacaktır, alçak duvarlar yüksek duvarlara kıyasla daha az kısıtlayıcı olacaktır.

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LIST OF SYMBOLS / ABBREVIATIONS

a	Bedrock acceleration in an earthquake
a_m	Maximum horizontal Acceleration
A	Area, cross sectional area
A_b	Base area of a pile
A_c	Cross sectional area of reinforcements minus estimated corrosion losses
b	Width of a reinforcing element
B	Width of the unit reinforcing element at modal tests
B'	Effective width of the reinforcement at modal tests
b_f	The width of a footing
c	Cohesion in terms of total stress
c'	Effective cohesion, apparent anisotropic cohesion
c_f	Cohesion of foundation soil
c_u	Undrained shear strength
c_v	Coefficient of consolidation
c_r	Added cohesion to the system
C	Perimeter of reinforcing strip or bar
d	Depth, diameter, subsoil deflection
D	Wall embedment, effective width of applied stress with depth
D_a	Average diameter of a pile
E	Young's modulus
e	Eccentricity
e_s	Reinforcement efficiency
F	Friction factor, tensile force developed by the reinforcements
F*	Pullout resistance factor
FS	Factor of safety
g	Acceleration due to gravity
h_s	Height of surcharge
H	Wall height, height of the unit reinforcing element
H'	Wall height modified to include uniform or sloping surcharge
K	Stress ratio

K_a	Active earth pressure of the retained fill
K_{ab}	Lateral earth pressure coefficient based on Coulomb theory and peak angle of internal friction
K_s	Horizontal restitution coefficient
K_0	Coefficient of earth pressure for at rest condition
l	Length of footing
L	Length of reinforcement
L_a	Length of reinforcement in the active zone
L_e	Embedded length of reinforcement to resist pullout
L_t	Length of reinforcement required for internal stability
L'	Effective length of the reinforcement
M	Moment, mass, earthquake magnitude
M_d	Driving moment
M_{TR}	Resisting moment due to vertical component of thrust
M_{WR}	Resisting moment due to weight of mass above base
n	Number of reinforcement layers
N_c	Bearing capacity factor
N_γ	Bearing capacity factor
P_a	Resultant of active earth pressure
P_{AE}	Dynamic horizontal thrust
P_b	Resultant of active earth pressure due to the retained backfill
P_{base}	Tip resistance capacity of a pile
P_h	Concentrated horizontal surcharge load
P_I	Horizontal inertial force
P_q	Resultant of active earth pressure due to the uniform surcharge
P_r	Available pullout resistance
$P_{skinfriction}$	Frictional capacity of pile shaft surface
P_u	Ultimate capacity of pile
P_v	Concentrated vertical surcharge load
Q	Surcharge load
Q_a	Allowable bearing capacity
Q_{ult}	Ultimate bearing capacity
R	Resultant force, occurrence risk of an earthquake

R_c	Reinforcement coverage ratio
R_v	Resisting force
S_h	Horizontal spacing of reinforcement strips
S_R	Stiffness factor of reinforcement
S_v	Vertical spacing between reinforcements
T	Tension of the reinforcement, applied pullout force, sum of the tensile forced from each reinforcement cut by the failure plane
T_a	Allowable tension per unit width of reinforcement
T_m, T_{max}	Maximum tensile force in the reinforcement per unit length along the wall
T_1	First component of maximum tensile force
T_2	Second component of maximum tensile force due to inertia
T_0	Tensile force at the connection of reinforcement to facing
V	Sum of vertical forces on reinforced fill
W_{opt}	Optimum water content
W	Vertical force due to the weight of the fill
W'	Weight of surcharge
z, z'	Depth below a reference level
Z	Depth to reinforcement layer
Z_{ave}	Distance from ground surface to midpoint of bar in the resisting zone
α	Maximum ground acceleration coefficient, scaling factor, angle of inclined failure plane with the horizontal, reduction factor for adhesion of the column surface
α_m	Maximum wall acceleration coefficient
β	Slope of soil surface
γ	Unit weight
γ_b	Unit weight of backfill
γ_r	Unit weight of reinforced zone
Δ	Change in some parameter or quantity
ΔB	Horizontal spacing of reinforcements at model tests, effective width of the reinforcement
ΔH	Vertical spacing of reinforcements at model tests

ΔL	Effective length of the reinforcement
ϕ	Angle of internal friction
ϕ_b	Angle of internal friction for drained condition of retained backfill
ϕ_f	Angle of internal friction of foundation soil
ϕ_r	Angle of internal friction of reinforced backfill
ϕ'	Effective angle of internal friction
ϕ_u	Angle of internal friction for undrained condition
ϕ_r	Induced friction angle
ε	Strain
λ	Inclination of earth pressure resultant relative to the horizontal when retained soil is also horizontal
λ_b	Inclination of earth pressure resultant relative to the horizontal when retained soil is at slope β
μ	Friction coefficient along the sliding plane
θ	Face batter of reinforced wall section
σ_h	Horizontal stress
σ_1'	Vertical principle stress in the soil
σ_3'	Horizontal principle stress in the soil
σ_r'	Induced prestress caused by the reinforcement
$\sigma_{r,max}'$	Maximum value of induced prestress caused by the reinforcement
σ_v	Vertical overburden stress
τ	Magnitude of shear stress acting between the soil and the reinforcement
ξ	Geometric scaling factor, reduction factor for end bearing column
δ	Displacement, geometrical coefficient, angle of bond stress, soil-column friction angle
δ_R	Relative displacement
MSEW	Mechanically Stabilized Earth Wall

1.INTRODUCTION

1.1. Concept of Earth Reinforcement

Mechanically Stabilized Earth Wall (MSEW) is a generic term that includes reinforced soil which is a term used when multiple layers of inclusions act as reinforcement in soils placed as fill. It is a gravity type retaining structure (Figure 1.1). Reinforced Earth is a trademark for a specific reinforced soil system. It is a construction material consisting of a frictional backfill material and linear reinforcing strips, generally placed horizontally.

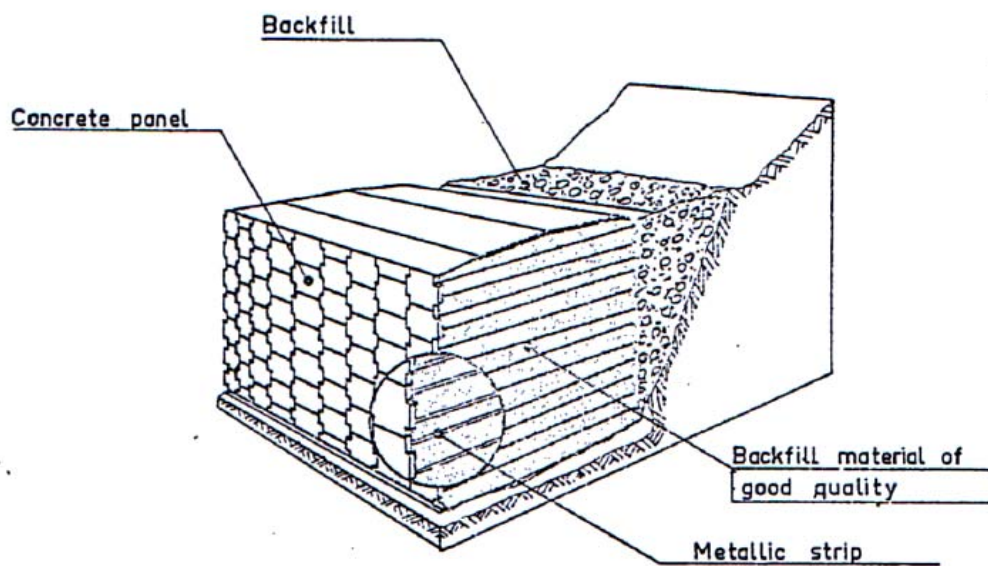


Figure 1.1 Cut-away view of a Mechanically Stabilized Earth Wall

Reinforced retaining walls are used where a slope is uneconomical or not technically feasible. Retaining walls with reinforced backfills are very cost-effective, especially for higher walls; also, they are more flexible and thus more suitable for poor foundation conditions. The concept was developed in France by Henri Vidal in the mid-1960s. His system, called Reinforced Earth. Steel strips or ties are used to reduce the earth pressure against the wall face (Figure 1.2). The design and construction of Vidal-type reinforced

earth walls are now well established, and many thousands have been successfully built throughout the world in the last 35 years.

A reinforced retaining wall consists of horizontal layers of reinforcing elements embedded in the backfill, and a wall facing. Reinforcing elements are typically metallic (strips, wire meshes, or rebar grids) or polymeric (strips, sheets, or grids). The primary function of the wall facing is to prevent loss of backfill from between the reinforcing elements. The facing is typically made of precast concrete architectural elements connected to the reinforcing elements. For MSE walls reinforced with sheets, grids, and wire mesh, the reinforcement can also be used as the wall facing by extending it beyond the face and wrapping the extra length back into the wall.

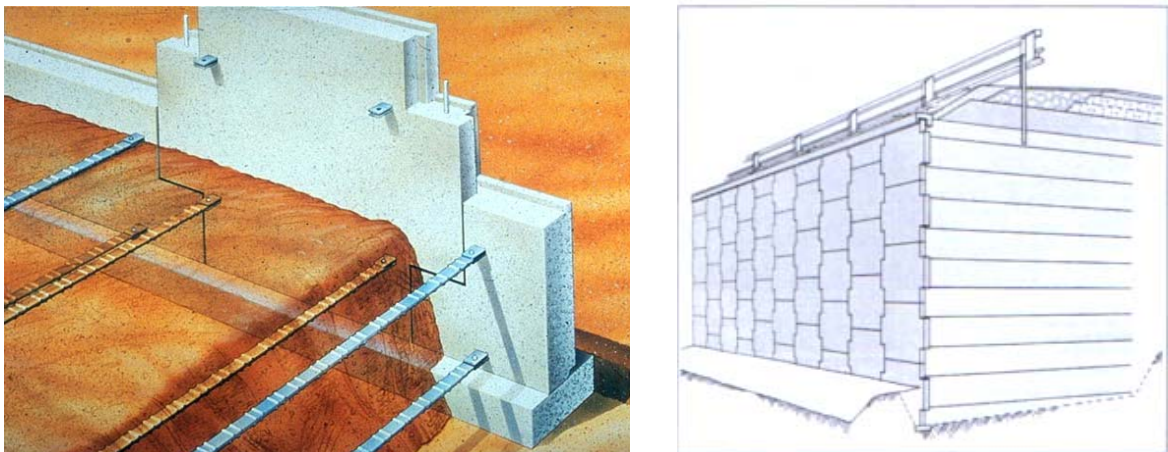


Figure 1.2 Cut-away view of a typical Reinforced Earth wall

1.2. Historical Development

Tensile inclusions have been used in soils for several thousand years. Large religious towers, called Ziggurats, were built by the Babylonians between 5,000 and 2,500 years ago. These structures, of which an example is shown on Figure 1.3, had walls faced with clay bricks in an asphalt mortar with blocks of sun-dried brick behind. Layers of reed matting were laid as horizontal reinforcing sheets in the mud. Additional reinforcement was included in some Ziggurats in the form of ropes about 2 inches (50 mm) in diameter

placed perpendicular to the wall and regularly spaced in the horizontal and vertical directions (Mallowan,1963).

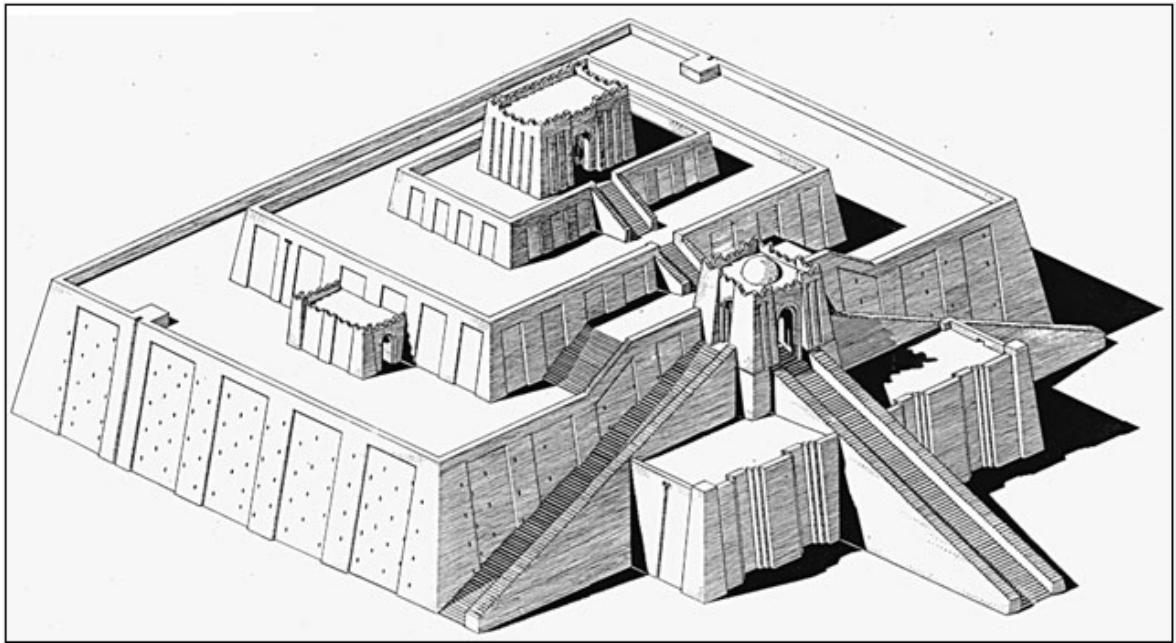


Figure 1.3 Ziggurat of Ur in Mesopotamia, about 2500 B.C. (Reinforcement is considered a major factor in its longevity.)

Many primitive peoples used sticks and branches for reinforcement of mud dwellings. Corduroy roads strengthened with logs placed transversely over soft subgrades were widely used for construction of roads across very weak ground in colonial America. In 1822, C.W. Pasley reported tests on model walls, 4 ft 3 in high, made of soil reinforced by horizontal layers of sacking material. The reinforcement was not connected to the wall face. The wall stability was reported to increase by 12 percent.

The development of the reinforced earth technique was marked by the following realizations:

- (a) The French architect and inventor Henri Vidal pioneered the development of modern earth reinforcement techniques; the system he developed, known as Reinforced Earth in United States, was patented in 1966 as *Terre Armee* in French and Reinforced Earth in English.

- (b) The first retaining wall was built in Pragneres France (1965).
- (c) The first group of structures was constructed on the Roquebrune-Menton highway (1968-1969). Ten retaining walls on unstable slopes totalizing a facing area of 5500m².
- (d) The first wall supporting important concentrated walls at its upper surface (traveling gantry cranes) was built at the Dunkerque port (1970).
- (e) The first highway bridge abutment (14 m high) was built in Thionvile (1972).
- (f) Stabilization of highway slopes was accomplished in France (1974) and in California (1977). Stabilization of railway slopes was accomplished for French Railroad Administration (1973), retaining structures were constructed (Stocker et al, 1979; Shen et al 1981; Cartier and Gigan, 1983; Guillou 1983). Applications for tunneling (Louis, 1979) and other civil and industrial projects (Louis, 1981) were realized.
- (g) The fundamental researches on the mechanism and the design of the reinforced earth, including, essentially, 15 full-scale experiments, were realized from 1967 to 1978 by the “Laboratoire Central des Ponts et Chaussees” in Paris.
- (h) Since 1972, the “Laboratoire Central des Ponts et Chaussees” and the “Reinforced Earth Company” have undertaken jointly the studies on the durability of the reinforcements and on the phenomenon of corrosion of metals buried in the backfill soil. Since then, an entire experience was acquired in this field due to the laboratory tests, to the experiences in the corrosion box, to full-scale experiments and to observations on actual structures constructed since 1968.

Two stages marked the technological development of the reinforced earth are:

- (a) The invention of the facing with concrete panels in 1971. Presently most of the structures are realized with this type of facing.
- (b) The development and the fabrication in 1975 of ribbed reinforcement strips for high adherence. These strips, 5 mm thick, made of ordinary mild galvanized steel enable a large improvement of the soil-reinforcement friction.

Some applications of the technique are shown in Figure 1.4. Structures were constructed in 32 countries and there presently several specifications issued by state institutes on this technique (Germany, United States). The local company Reinforced Earth A.Ş. in Turkey has been established in 1988.



Figure 1.4 Applications of Reinforced Earth as Retaining Walls and Tiered Walls

2. EARTH REINFORCEMENT STRUCTURES

2.1. Types of Structures

Earth reinforcement structures (MSE system) can be described by the reinforcement geometry, stress transfer mechanism, reinforcement material, extensibility of the reinforcement material, and the type of facing and connections.

Three types of reinforcement geometry can be considered:

- **Linear unidirectional:** Strips, including smooth or ribbed steel strips, or coated geosynthetic strips over a load-carrying fiber.
- **Composite unidirectional:** Grids or bar mats characterized by grid spacing greater than 150 mm.
- **Planar bidirectional:** Continuous sheets of geosynthetics, welded wire mesh, and woven wire mesh. The mesh is characterized by element spacing of less than 150 mm.

Distinction can be made between the characteristics of metallic and nonmetallic reinforcements:

- **Metallic reinforcements:** Typically of mild steel. The steel is usually galvanized or may be epoxy coated (Figure 2.1).
- **Nonmetallic reinforcements:** Generally polymeric materials consisting of polypropylene, polyethylene, or polyester (Figure 2.2).

There are two classes of extensibility:

- **Inextensible:** The deformation of the reinforcement at failure is much less than the deformability of the soil.
- **Extensible:** The deformation of the reinforcement at failure is comparable to or even greater than the deformability of the soil.

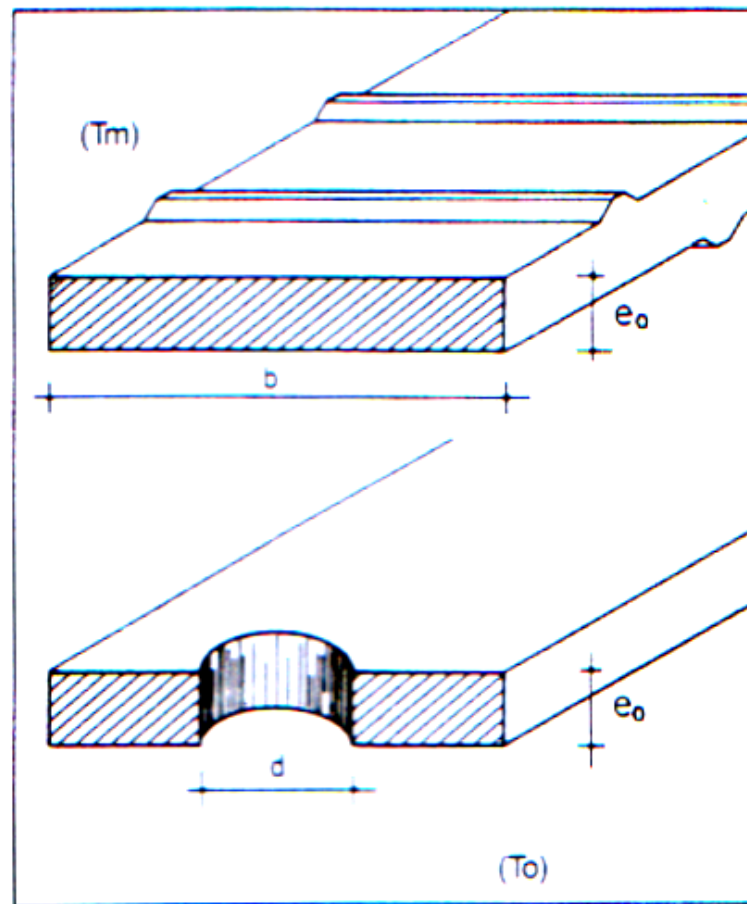


Figure 2.1 Linear unidirectional metallic strip cross-sections

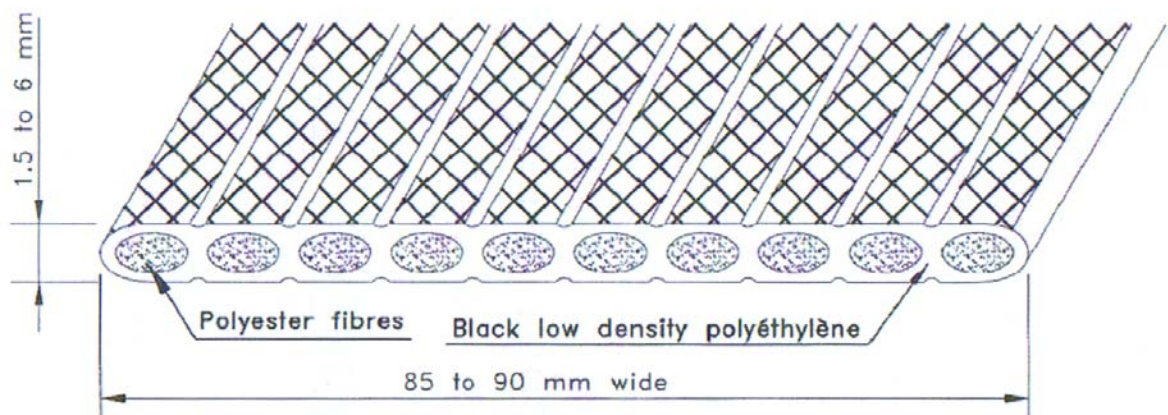


Figure 2.2 Linear unidirectional nonmetallic strip cross-section

An earth reinforcement system has three basic components (Figure 2.3) which are:

- Reinforcements,
- General backfill,
- Facing elements.

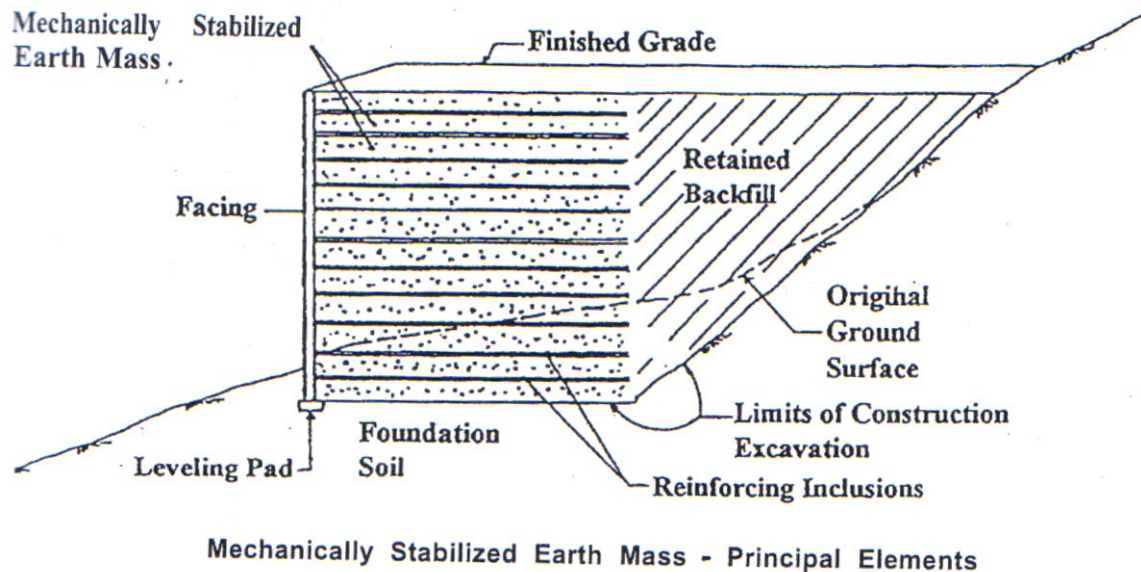


Figure 2.3 Generic cross-section of an Earth Reinforcement System

For some of the earth reinforcement systems a fourth element can also be described as the connection parts. In many of systems these parts are used and checked in the design calculations. Two main mechanisms of stress transfer between the reinforcement and the soil can be outlined as:

- friction at the interface of the reinforcement and soil,
- And passive soil bearing resistance on reinforcement surfaces oriented normal to the direction of relative movement between soil and the reinforcement.

Strip, rod and sheet reinforcements transfer stress to the soil predominantly by friction, while deformed rod and grid reinforcements transfer stress to the ground mainly through passive resistance, or both passive resistance and frictional stress transfer. The ribbed reinforcement's main idea is to use the passive soil bearing resistance in addition to the friction at the interface of reinforcement and soil.

2.1.1. Strip Reinforcement

With strip reinforcement methods, a coherent reinforced soil material is created by the interaction of longitudinal, linear reinforcing strips and the soil backfill. The strips, either metal or plastic, are normally placed in horizontal planes between successive compaction layers of soil backfill.

Reinforced Earth is a strip reinforcement system which uses prefabricated galvanized steel strip, either ribbed or smooth (Figure 2.4). Facing panels fastened to the strips usually consist of either precast concrete panels or prefabricated metal elements. Backfill soil should meet specific geotechnical and durability criteria.

Plastic strips have been introduced in an effort to avoid the problem of corrosion in adverse environments. However, all aspects of their durability are not yet fully known. Currently, the only commercially available nonmetallic strips are the Paraweb strip, in which the fibers are made of high tenacity polyester or polyaramid given added strength by extrusion through dies or by drawing and Fibratain. The strips are fastened to wall facings, typically consisting of precast concrete panels. Soil backfill is generally granular ranging in size from sand to gravel.

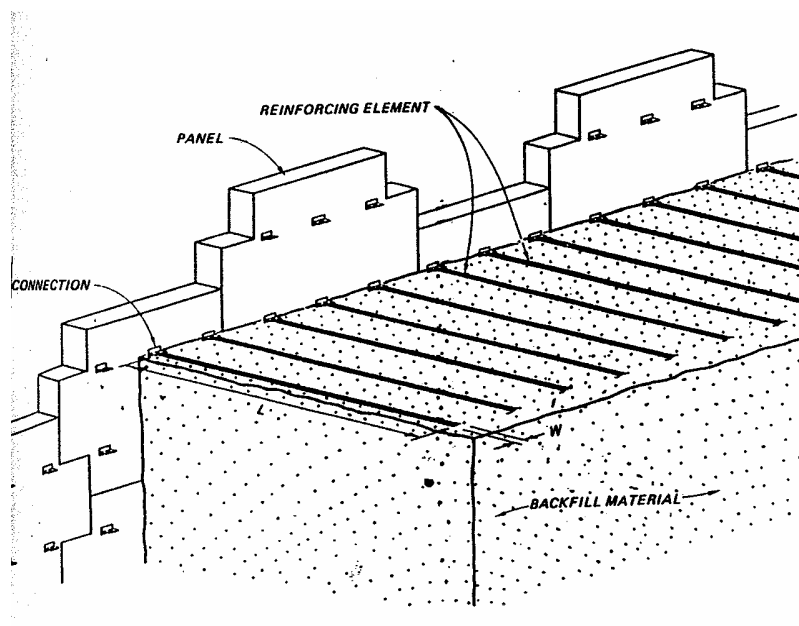


Figure 2.4 Schematic Diagram of Reinforced Earth Wall

2.1.2. Grid Reinforcement

Grid reinforcement systems consist of metallic or polymeric tensile resisting elements arranged in rectangular grids placed in horizontal planes in the backfill to resist outward movement of the reinforced soil mass. Grid transfers stress to the soil, through passive soil resistance, on transverse members of the grid and through friction between the soil and horizontal surfaces of the grid.

The Welded Wire Wall (Figure 2.5) and Reinforced Soil Embankment (RSE) (Figure 2.6) systems employ standard welded wire mesh grid reinforcements within the backfill to constitute reinforced soil structure. The two systems differ, however not the facing arrangements. In the Welded Wire Wall, the end of each mesh layer is bent upwards and attached to the mesh above. Backing meshes may be added behind the outer facing to reduce the mesh operating size for retention of backfill soil. The Reinforcing Soil Embankment couples the reinforcing mesh with precast concrete facing panels. The wire meshes used are the same types that have been used extensively for reinforcement of concrete slabs.

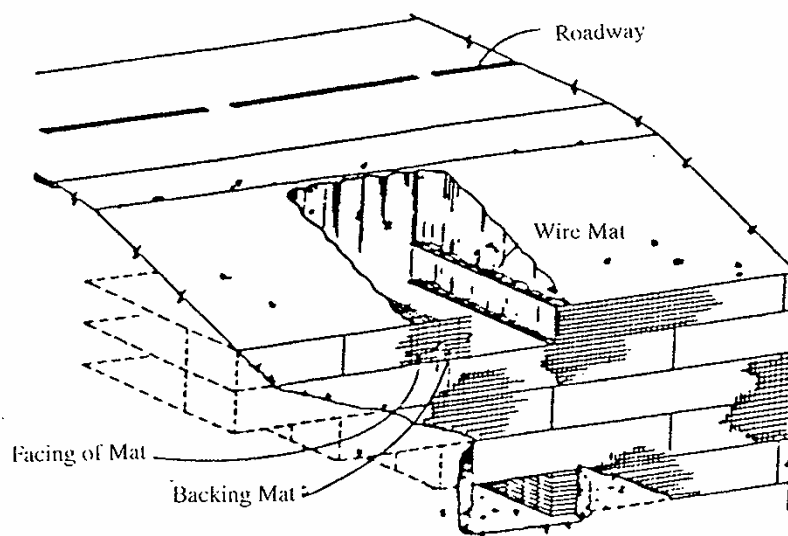


Figure 2.5 Schematic Diagram of a Welded Wire Wall

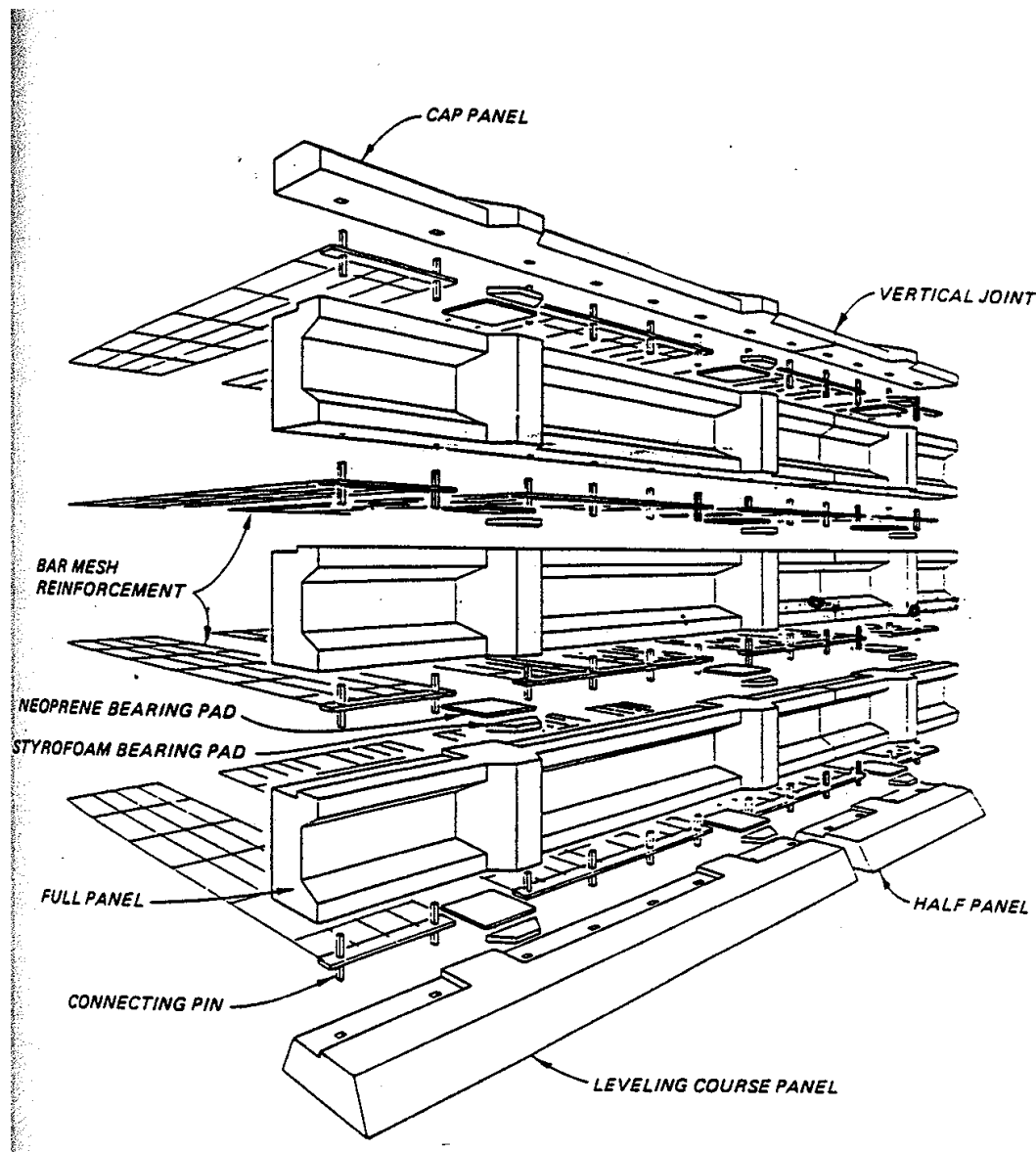


Figure 2.6 Schematic Diagram of a Reinforced Soil Embankment

Grid reinforcements made of stable polymer materials provide good resistance to deterioration in adverse soil and groundwater environments. Tensar Geogrids (Figure 2.7) are strength polymer grid reinforcements manufactured from the density polyethylene 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.

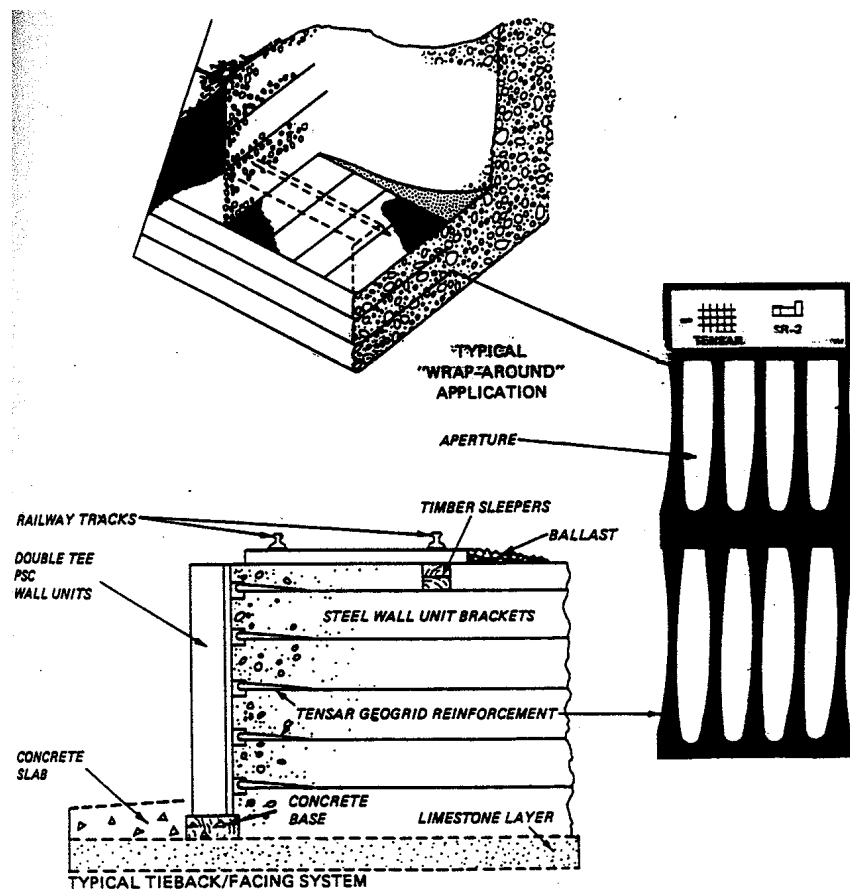


Figure 2.7 Schematic Diagram of a MSE Wall with Geogrid Reinforcements

2.1.3. Sheet Reinforcement

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 (Figure 2.8). The majority of geotextile fabrics used in soil reinforcement are made of either polyester or polypropylene fibers. Woven, nonwoven, needle-punched, nonwoven heat-bonded, and resin-bonded fabrics are available as well as materials made by other processes. The backfill material typically consists of granular soil ranging from silty sand to gravel. Facing elements are commonly constructed by wrapping the geotextile around the exposed soil at the face and covering the exposed fabric with gunite, asphalt emulsion, or concrete. Alternatively, structural elements such as concrete panels or gabions can be used. Connection between

the geotextile sheet and structural wall elements can be provided by casting the geotextile into the concrete, friction, nailing, overlapping or other bonding methods.

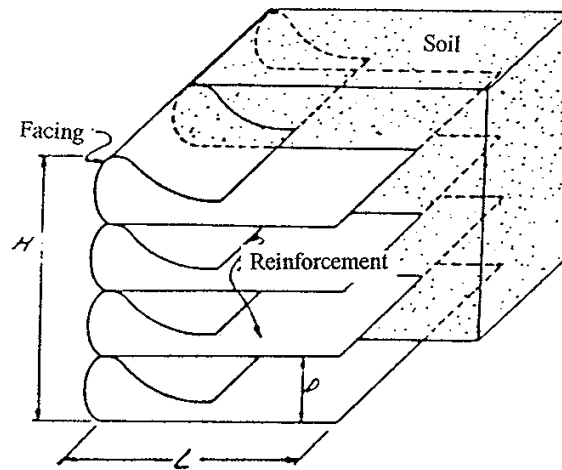


Figure 2.8 Schematic Diagram of a Reinforced Soil Wall with Geotextile Sheet Reinforcements

2.1.4. Rod Reinforcement

Anchored Earth employs slender steel rod reinforcements bend at one end to form anchors (Figure 2.9). Soil-to-reinforcement stress transfer is assumed to be primarily through passive resistance, which implies that the system operates similarly to tied-back retaining structures. As such, anchored earth is perhaps not truly a reinforced soil system. Nonetheless, friction should also be developed along the length of linear rod. Therefore, although this friction is not currently allowed for in design, the system may behave in some respects as a reinforced soil. As currently envisioned, the rods will be attached to concrete panel facings. Anchored Earth is undergoing continuing research and has not yet been extensively used.

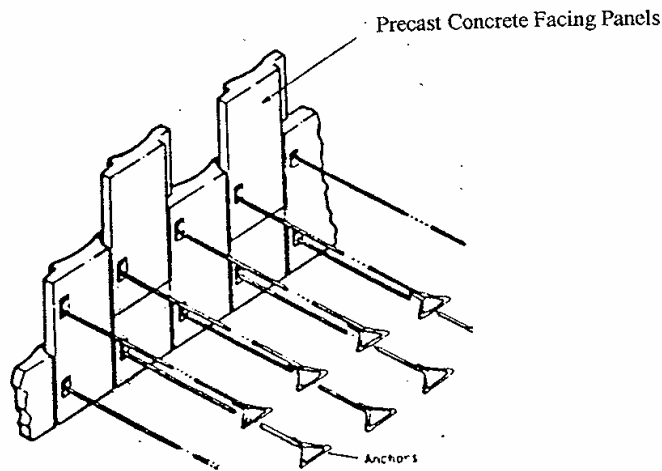


Figure 2.9 Schematic Diagram of an Anchored Earth Retaining Wall

2.1.5. Fiber Reinforcement

A composite construction material with improved mechanical properties can be created by the inclusion of tensile resistant strands (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 development stages. Materials being investigated for possible use include natural fibers (reeds and other plants), synthetic fibers (geotextile threads), and metallic fibers (small diameter metal threads).

2.1.6. Cellular Reinforcement Systems

Cellular reinforcements may be used at the base of embankments and retaining walls to significantly increase the bearing capacity of underlying weak soils and, hence, the stability of the embankments. Early laboratory investigations and theoretical analyses of such systems were performed at University of California, Berkeley, by Rea and Mitchell. Field tests of these “grid cells” were performed at the U.S. Army Engineering Waterways Experiment Station by Webster and Alford. More recently the systems have been further developed and are now commercially available under trade names such as GEOWEB.

2.2. Strip Reinforced Earth Structures

2.2.1. General Applications of Strip Reinforced Earth Structures

Strip reinforced soil structures may be cost-effective alternatives for all applications where reinforced concrete or gravity type walls have traditionally been used to retain soil. Retaining structures are used not only for bridge abutments and wing walls but also for slope stabilization and to minimize right of way for embankments. For many years, retaining structures were almost exclusively made of reinforced concrete and were designed as gravity or cantilever walls which are essentially rigid structures and cannot accommodate significant differential settlements unless founded on deep foundations. With increasing height of soil to be retained and poor subsoil conditions, the cost of reinforced concrete retaining walls increases rapidly.

Mechanically Stabilized Earth Walls (MSEW) is cost-effective soil-retaining structures that can tolerate much larger settlements than reinforced concrete walls. By placing tensile reinforcing elements in the soil, the strength of the soil can be improved significantly such that the vertical face of the soil/reinforcement system is essentially self supporting. Use of a facing system to prevent soil raveling between the reinforcing elements allows very steep slopes and vertical walls to be constructed safely. In some cases, the reinforcements can also withstand bending from shear stresses, providing additional stability to the system.

Strip reinforced soil walls offer significant technical advantage over conventional reinforced concrete retaining structures at sites with poor foundation conditions. In such cases, the reduced cost of reinforced soil versus conventional construction, plus the elimination of costs for foundation improvements, such as piles and pile caps, that may be required for support of conventional structures have resulted in cost savings of greater than 50 per cent on completed projects. In situations where a steep reinforced slope can replace a conventional wall, cost savings can be 70 per cent or more. Some additional successful uses of reinforced soil include (Figure 2.10):

- (a) Temporary reinforced soil structures, which have been especially cost effective for temporary detours necessary for major highway reconstruction projects.
- (b) Reinforced soil dikes, which have been used for containment structures for water and waste impoundments around oil and liquid natural gas storage tanks. (The use of reinforced soil containment dikes is not only economical but it can also result in savings of land, because a vertical face can be used, and reduce construction time).
- (c) Dams and sea walls and to increase the height of existing dams.

Reinforcement of earth embankments allows use of steeper slopes. The reinforcement also gives resistance to surface erosion as well as to seismic shock. Horizontal layers of reinforcements at the face of a slope also permit heavy compaction equipment to operate close to the edge, thus improving compaction and decreasing the tendency for surface sloughing.

2.2.2. Construction Materials

Three basic components of strip reinforced earth wall systems are namely:

- reinforcements,
- soil backfill,
- facing elements and,
- connection parts.

Detailed structural and physical information about those main components are introduced below.

2.2.2.1. Soil Backfill in the Wall. Most strip reinforced earth walls systems have used cohesionless soil backfill but fill in the wall can be either completely frictional or completely cohesive frictional material. Fill consisting of alternate layers of frictional material and cohesive material should not be used for the present. The use of soft chalk, unburnt colliery shale and unsuitable material is not permitted. Pulverised-fuel ash (PFA) can be used as fill material but requires special provisions.

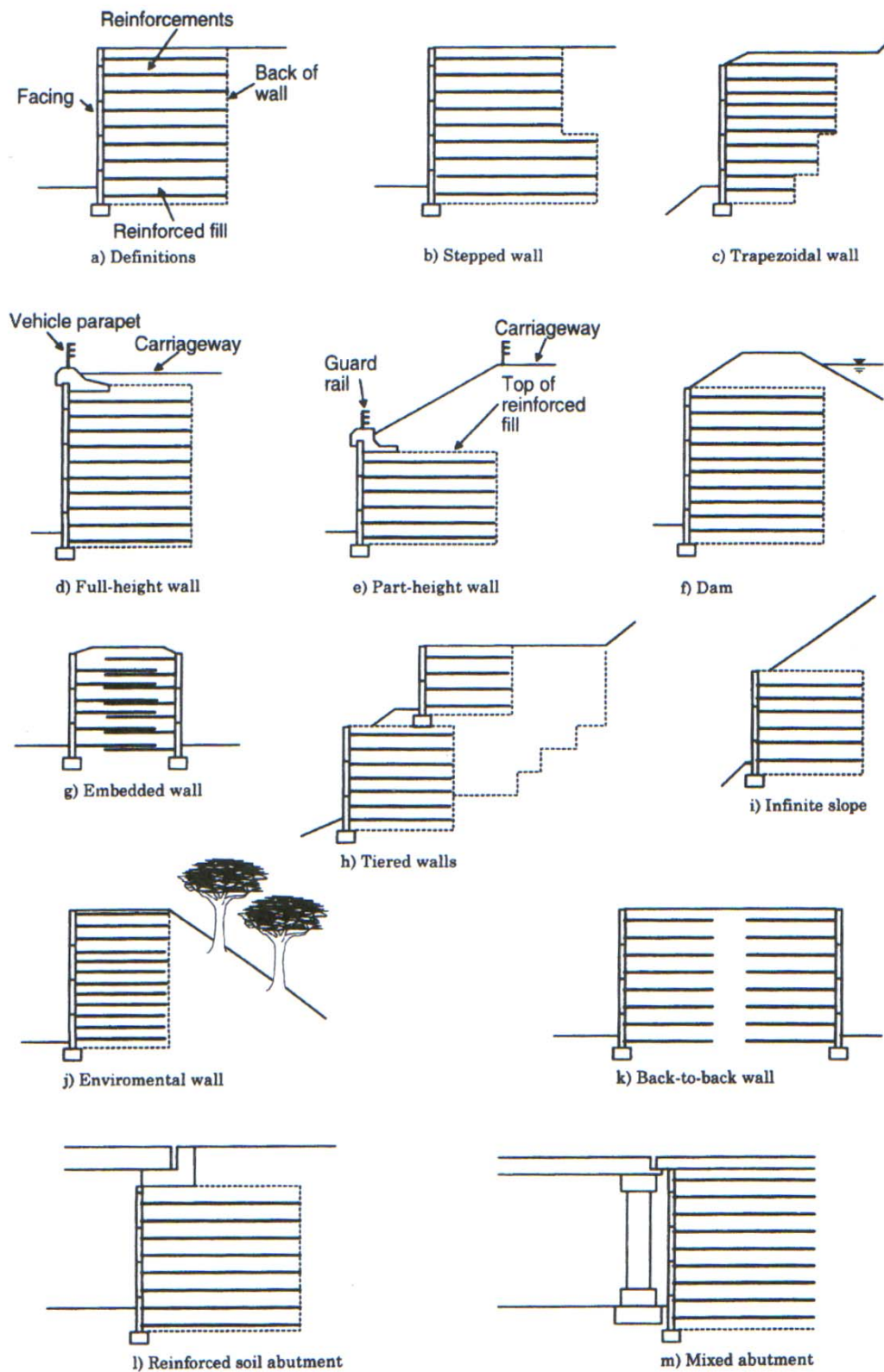


Figure 2.10 General Applications of Reinforced Earth Walls

The advantages of cohesionless soil backfill are that it is stable (will not creep), free-draining, not susceptible to frost, and relatively noncorrosive to reinforcement. The main disadvantage where cohesionless soil has to be imported is cost. The main advantage of cohesive soils is availability and hence lower cost. The disadvantages are long-term durability problems (corrosion and/or frost) and distortion of the structure (due to creep of the soil backfill).

The choice of fill and reinforcing element material to be adopted for the design will depend upon many factors including a through knowledge of the soils which are available on or near the site. In parallel, the site investigation should include for additional soil tests to be carried out for reinforced earth structures in order to assist in the selection of fill material. The following soil properties will need to be considered:

- density,
- grading,
- uniformity coefficient,
- pH value,
- chloride ion content,
- total SO₃ content,
- resistivity,
- redox potential,
- angle of internal friction,
- coefficient of friction between the fill and the reinforcing elements.

Additionally for cohesive frictional fill:

- cohesion,
- adhesion between the fill and the reinforcing elements,
- liquid limit,
- plasticity index,
- consolidation parameters.

The quality of the backfill material used in reinforced earth, whatever its origin may be natural or industrial and must be conform to well determined criteria, as following:

(a) Mechanical Criteria:

For the reinforcements of high adherence, the internal friction angle of the saturated material measured under the conditions of rapid shearing must be superior or equal to 25° . For smooth reinforcements, the angle of soil- reinforcement friction measured under the same conditions must be superior or equal to 22° .

For practical reasons this criteria of friction is generally replaced by criteria concerning the particle size distribution (grading) of the backfill material. The predominant factor is the percentages in weight of particles smaller than $80\ \mu\text{m}$ and of particles smaller than $15\ \mu\text{m}$. This practical reasoning is schematised in Table 2.1.

(b) Criteria of Setting up and Execution:

According to this criteria the largest dimension of the elements should not exceed 250 mm, taking into account the small thickness of the layers (0.33 or 0.375 mm). It is also necessary to limit the moisture content of materials which are sensible to water according to the Recommendation for Roads Earthworks (R.T.R.) in order to avoid the difficulties during the compaction. According to the soils classification given in the “R.T.R.” document, three categories of backfill soils can be distinguished as:

- soils directly utilizable in Reinforced Earth,
- soils directly utilizable in their natural state,
- soils which are utilizable on condition that the mechanical criteria is satisfied.

These categories are presented in Table 2.2, which constitutes a guide for the choice of the backfill material. This table shows that the following materials are not to be used in Reinforced Earth:

- The classes of soils sensible to water which are too humid, marked by the index ‘h’ in the Table 2.2,
- The classes of soils A_3 and A_4 concerning soils which are essentially clayey. For these soils the mechanical criteria is generally not satisfied.

- The classes of soil C₃ and D₃ concerning the materials comprising elements larger than 250mm. For these soils criteria of setting up and execution is not satisfied.
- The use of materials belonging to class F, and particularly of industrial wastes. These materials should be an object of a special study.

Table 2.1 Mechanical Criteria for the Choice of the Backfill Material

< 80 μm	$\leq 15 \%$	Satisfying Mechanical Criteria				
	> 15 %	< 15 μm	$\leq 10 \%$	Satisfying Mechanical Criteria		
			10 % to 20 %	H.A. Reinforcements	Internal friction angle $\geq 25 \%$	Satisfying Mechanical Criteria
					Internal friction angle $< 25 \%$	Inadequate Material
				Smooth Reinforcements	Internal friction angle $\geq 22 \%$	Satisfying Mechanical Criteria
					Internal friction angle $< 22 \%$	Inadequate Material
			$\geq 20 \%$	Inadequate Material		

(c) Chemical and Electrochemical Criteria:

Related to the durability of the reinforcements the resistivity of the backfill material should be measured first determined on the saturated material after one hour of soil-water contact under 20⁰. This resistivity value must be superior to

- 1000 ohm-centimeter (Ωcm) for structures outside water,
- 3000 ohm-centimeter (Ωcm) for structures in soft water.

The activity in hydrogen ions of the soil (measured by extracting the the water from the soil-water mixture) should be in the range of 5 to 10 measured due to its hazardous effects on the reinforcements and facing panels.

Table 2.2 Guide for the Choice of the Backfill Material

Classes of Soil Distinguished in the Classification RTR		Soil Utilizable in Reinforced Earth	Soil Requiring a Verification of a Mechanical Criterium	Soil Inadequate in its Natural State
<u>Soils of class A</u> D ≤ 50mm passing 80 μm > 35 %	A _{1m} ; A _{1s} A _{2m} ; A _{2s}		×	
	A _{1h} ; A _{2s} A ₃ ; A ₄			×
<u>Soils of class B</u> D < 50mm passing 80 μm 5 % to % 35	B ₁ ; B ₃ B _{2m} ; B _{2s} B _{4m} ; B _{4s}	×		
	B _{5m} ; B _{5s} B _{6m} ; B _{6s}		×	
	B _{2h} ; B _{4h} B _{5h} ; B _{6h}			×
<u>Soils of class C</u> 50mm passing 80 μm > 5 %	C _{2m} ; C _{2s}	×		
	C _{1m} ; C _{1s}		×	
	C ₃ ; C _{2h} ; C _{1h}			×
<u>Soils of class D</u> passing 80 μm < 5 %	D ₁ ; D ₂ ; D ₃	×		
	D ₄			×
<u>Soils of class E</u> Evolutive rocks	C _{ra} ; C _{rb} ; E ₂	×		
	E ₃		×	
	C _{rc} ; C _{rd}			×

The content of soluble salts is in principle determined only for natural backfill materials, the resistance of which is in the range of 1000 Ωcm to 5000 Ωcm and for backfill materials of industrial origin. For the purpose the concentration of chlorine $[\text{Cl}^-]$ and sulphate $[\text{SO}_4^{2-}]$ is measured. The values of concentrations should respect the following conditions :

- Structures outside water : $[\text{Cl}^-] \leq 200 \text{ mg/kg}$ and $[\text{SO}_4^{2-}] \leq 1000 \text{ mg/kg}$,
- Structures in soft water : $[\text{Cl}^-] \leq 100 \text{ mg/kg}$ and $[\text{SO}_4^{2-}] \leq 500 \text{ mg/kg}$.

The backfill material should not contain organic material. However, in doubtful cases, for submerged structures, it is possible to verify that the organic content determined through some chemical tests should not exceed the authorized limit of 100 p.p.m.

2.2.2.2. Reinforcing Elements. The reinforcements used for the strip reinforced earth walls should have the following characteristics:

- (a) They should provide a high apparent friction coefficient with the backfill material.
- (b) They should have a high tensile strength, a failure mode which is not brittle and very limited susceptibility to creep.
- (c) They should be flexible enough to conform with the deformability of the reinforced earth material in order to enable easy construction.
- (d) They should have a high durability.
- (e) They should be cost-effective.

Strip reinforcements in the market are manufactured from tensile steel or polymeric materials.

Presently, ordinary mild galvanized steel is the most frequently used. The reinforcement strips are generally linear bands, a few millimeters thick, and a few centimeters wide. The general reinforcement characteristics for the two most common types of reinforced earth walls are given below:

- (a) For metallic facings, the reinforcement strips are cut of the same sheet which is used for the fabrication of the facing elements. They are made of mild galvanized steel and are generally 4-5mm thick. They have a width of 40, 50 or 60 mm.
- (b) For concrete panel facings, the reinforcement strips are made of mild galvanized steel. They have a cross-section of 40 by 5 mm, or 60 by 5 mm, and their surface is ribbed in order to improve the apparent soil reinforcement friction. These ribbed strips are called Highly Adherent Reinforcements. The dimensions and the spacings of the ribs are designed in order to maximize the apparent friction coefficient used in the Limit State design calculations (Figure 2.11). These ribs in fact, contribute to pull out resistance by utilizing the passive resistance of the soil (Figure 2.12).

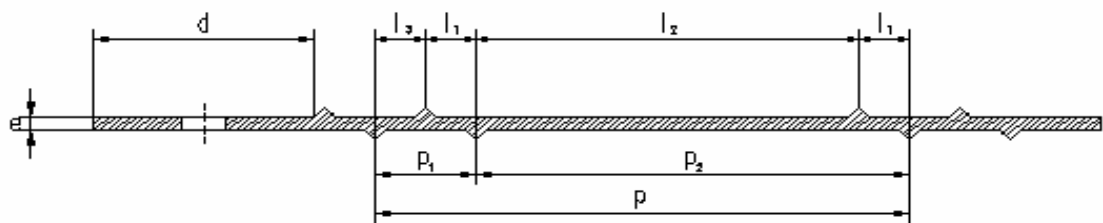


Figure 2.11 Arrangements of Ribs Reinforced Strips

As seen in Figure 2.11 the required values of the given distances are:

Width (b): 40 mm $\pm$ 1.5 mm

Thickness (e): 5 mm – 0.2 mm, + 0.5 mm

Number of ribs per side 12 $\pm$ 2

143 mm < p < 200 mm

$p_1 > 25$ mm $p_2 > 105$ mm $d > 83$ mm

$l_1, l_3 > 10$ mm $l_2 > 95$ mm

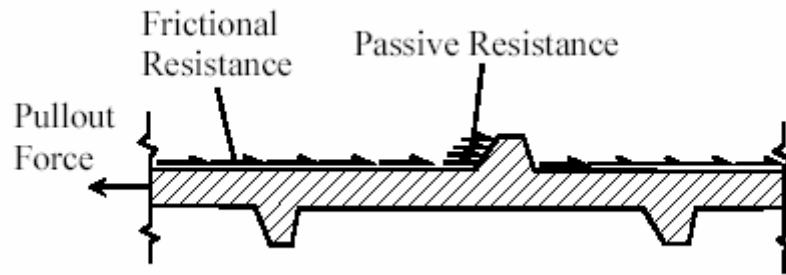


Figure 2.12 Working Mechanism of High Adherence Reinforced Strips

A summary of the required standards for the high adherence galvanised coated steel strips are given below:

For steel grade:

EN 10025-S355JR or equivalent grade.

The chemical component of steel:

Silicon : $0.15\% < \text{Si} < 0.35\%$

Carbon: $\text{C} < 0.26\%$

Manganese: $0.50\% < \text{Mn} < 1.60\%$

Phosphorus: $\text{P} < 0.04\%$

Sulphur: $\text{S} < 0.05\%$

The mechanical requirements:

Rupture strength $R_m > 510 \text{ Mpa}$

Yield Point $R_{eH} > 355 \text{ Mpa}$

Coating thickness:

min $70\mu\text{m}$

The metallic reinforcing strips which are buried in the backfill material are the most sensitive of the different reinforced earth elements. This degradation results from the electrochemical corrosion of the metal in contact with the soil. In general, the most corrosive soils contain large concentrations of soluble salts, especially in the form of sulfates, chlorides and bicarbonates, and they may have very acidic or highly alkaline pH

values. Clayey and silty soils (characterized by fine texture, high water holding capacity and consequently, by poor aeration and poor drainage) are also prone to being potentially more corrosive than soils of a coarse nature such as sands and gravels, where there is a free circulation of air and where corrosion approaches the atmospheric type. Additionally, buried metals can corrode significantly by differential aeration or bacterial action. However, such behavior is mostly associated with fine-grained soils.

The Reinforced Earth Company and the Laboratoire des Ponts et Chausees (LCPC) in France have conducted extensive laboratory testing to determine the effect of chlorides and sulfates on the corrosion rate of buried galvanized strips. The results indicate that chlorides in concentrations up to 200 part per million and sulfates up to 1000 parts per million have no significant effect.

On 1996 Segrestin P. and Bastick M. made a valuable addition to the literature with their work on the behaviors of extensible and inextensible reinforcements, namely the polymer and geotextile reinforcements versus steel usage in the reinforced earth walls. They concluded that, the inextensible metal reinforcements work on their entire length. The friction which is mobilized along the strip is almost uniform, but smaller than the limiting shear stress which can be mobilized all along the reinforcement. Conversely, extensible reinforcements practically make use of only the minimum adherence length which is strictly necessary. The friction which mobilized along this length is equal to the limiting stress. The safety factor materializes by the extra length which remains available, but which is not actually activated.

2.2.2.3. Facing Panels. In early Reinforced Earth walls the basic facing elements were metallic half-cylinders of a semi-elliptic section, which were flexible and stable with respect to the thrust exerted by the backfill soil. Since 1971, plane cross-shaped concrete panes have been used more often than the metallic facing. This second type of facing enables the building of walls that can easily be curved in plan, which are well adapted to retaining structures in urban areas.

The metallic facings are still use in structures where difficult access and/or difficult handling require light facing.

Today, the facing may consist of one of the following materials with the advanced technology in material science:

- reinforced concrete (either in situ, or precast units),
- galvanized carbon steel panels
- stainless steel panels,
- proprietary material,
- metallic facing.

(a) Facing with Metallic Panels:

The metal facing elements and the reinforcement strips are fabricated from a galvanized steel sheet. A facing element has a length of 33.3 cm (a distance corresponding to the spacing between two levels of reinforcement) and a thickness of 3 mm. The connection between two elements is made with the help of an overlapping joint which is simply adjusted on the internal face (Figure 2.13). It prevents the soil from sloughing away and ensures, in the transverse direction, the deformability of the facing by the sliding of the elements on the overlapping joint.

(b) Facing with Concrete Panels:

The standard precast concrete facing elements cross shaped, wit overall dimensions of 1.50 m by 1.50 m and is made of either unreinforced or reinforced concrete (Figure 2.14). Thickness of the panels varies from 14 to 18 cm, and weight varies from 1.1 to 1.65 tons.

Each panel normally provides connections for four reinforcing strips. The connections are made by tie-strips which are embedded in the concrete and made of the same metal as the reinforcing strips. Vertical assembly-alignment pins provide correct alignment between the panels but are specifically designed to allow horizontal deformation. Compressible material is placed in horizontal joints between panels and allows some vertical deformation. Each panel contains lifting anchors to facilitate handling and placing.

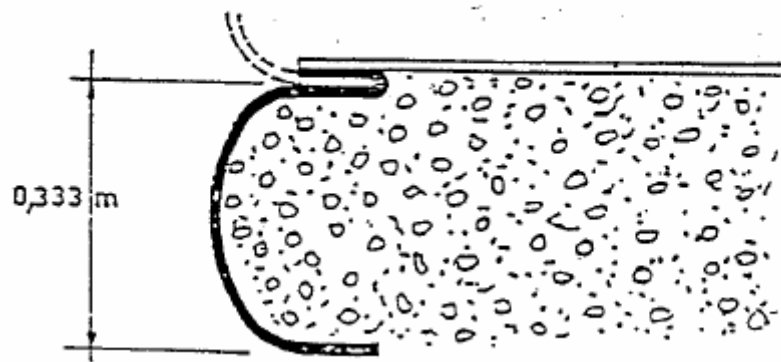


Figure 2.13 Sketch of Metallic Facing Element



Figure 2.14 Reinforced Earth Panels

The panels are generally prefabricated in molds, thus enabling good uniformity and quality control. In addition to the standard panels which can be used to obtain desired overall geometry. These special panels include:

- half-panels, 0.75 m high, which are used at the base and at the top of the wall,
- special panels, of which height varies by steps of 20 cm, and which are used to give the upper line of the facing the desired shape,
- angle elements which enable changes in the direction of the facing.

2.2.2.4. Subsoil. A thorough knowledge of the soil should be gained from the site investigation in order to check the external stability of the reinforced earth structure and its associated bearing capacity/settlement relationship at various locations along the site of the proposed structure.

2.2.2.5. Retained Fill. Wherever possible embankments retained by reinforced earth structures should not be made of material containing soluble salts which affect the durability of reinforcing elements. Materials such as crushed slag, pulverised fuel ash and unburnt colliery shale should be avoided unless additional drainage facilities are incorporated in the structure or it can be shown that these materials would not present a durability hazard.

2.2.2.6. Connection Parts. Connection parts used in steel reinforced earth walls can be seen in Figure 2.15. These parts are not essentially a part of every reinforced earth system. Some geotextile walls do not use connection parts but wrap over the layers to cover it duty. The main aim of the connections is to provide a reliable connection between the reinforcing strips and panels. As seen in Figure 2.15 due to the bolt hole in the reinforcement there exist a cross section loss in the reinforcement itself. Since the connection parts namely “tie strips” has two arms on both the top and below the strip, if same grade steel is used in the manufacture process it is obvious that the connections will not be critical in design. But as said before the diminished cross section is considered in design calculations. To determine the necessary stresses usually empirical formulas supported by previous experimental studies are used. It is accepted that in the connections

the axial force on reinforcements are maximum %85 percent of the maximum axial force calculated on the strip.



Figure 2.15 Connection Parts in Steel Strips

2.2.3. Phases of Construction of Reinforced Earth Components

2.2.3.1. Setting Leveling Pads. Leveling pad is the nonstructural foundation of reinforced earth structures, its only purpose is to control the elevations of the panels. Leveling pad is generally formed 15cm depth by 35cm wide in order to ensure the panels alignment. It must be correctly leveled in order to ensure an appropriate alignment for the first row of panels and to facilitate the setting up of the whole facing.

2.2.3.2. Setting Facing Elements. The facing panels should not be damaged during precasting, transporting or erection. If any damage does occur, a decision to reject the considered element must be made rapidly before the erection of the panel. The replacement of a panel which backfill has already been placed, is a time consuming operation which requires a complete dismantling of a part of the structure.

The stability of the facing during the backfilling operation is ensured for the first row of panels by temporary struts placed on the external side of the wall, and for successive levels by temporarily securing facing panels by wooden wedges and screw clamps. In a structure where there is a risk of fine materials migrating through the joints between facing panels under the action of water the vertical joints are sealed by a filter fabric applied against the inside of the concrete. Horizontal joints are sealed because of cork placed between layers of panels.

The tolerance between three successive panels measured using a 15 foot long straight edge, placed in any direction against at least 2 panels, should not exceed 1 inch according to Reinforced Earth Company specifications.

For the cross shaped reinforced earth precast panels, the half panels are used in the initiation of the precast panels as seen in Figure 2.16. The numbers in the figure represents the placement order if the direction of construction is as given. It is also seen that these half panels are placed over the leveling pad with 150cm spacing then the full panels are placed.

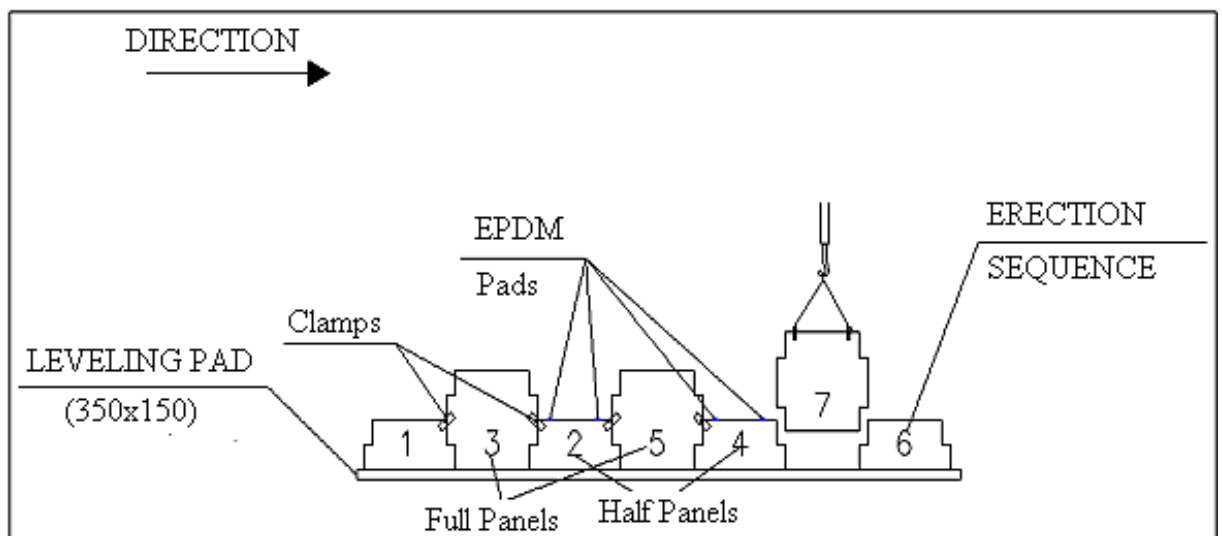


Figure 2.16 Precast Panels Erection Sequence

2.2.3.3. Placement of Reinforcements. The reinforcements should be laid flat on the compacted embankment and fixed to the tie-strips. Their number, corresponding to the number of tie-strips, is easily controlled. Before backfilling, all the reinforcements must be bolted to the tie-strips, and corrosive protection, if required should be applied.

It may be necessary to lower the reinforcements in order to provide either the necessary spacing for the top layer of a road base, or to provide enough overburden to develop the required pullout resistance.

2.2.3.4. Placement and Compaction of Backfill Soil. Placement of the backfill material on a layer of reinforcement should begin at the center of the first reinforcement reached by the equipment. Equipment should not cross directly over placed reinforcements. Care should be taken to ensure that the reinforcing strips are properly aligned after dumping the backfill. Backfill layer thickness should average 12 inches in the case of metallic facing and 15 inches in the case of concrete facing. Proper compaction of the backfill soil is required to minimize subsequent settlements and to insure good soil-reinforcement stress transfer.

Each fill layer must be leveled after compaction to ensure that all the reinforcements are in contact with the soil over their entire bottom surface. This may require some manual filling and tamping, particularly near the connection of the reinforcements to the facing and in zones of difficult access.

If backfilling is done with materials sensitive to water, the contractor must take measures to prevent any ponding of rainwater or flow through or over the facing. Besides, backfilling in front of the embedded part of the lower row of facing panels is usually done before the structure reaches a height of 10 ft.

Compaction control should be done after the compaction of the layers. The offered relative compaction value is 95% of the lab compaction value.

2.2.3.5. Construction Equipment. Construction of Reinforced Earth structures in addition to earthwork equipment, the following are required:

- (a) Small vibrating compactor compact the zone situated up to a distance of 3 to 5 ft from the facing (to avoid damage to facing panels by heavy compaction equipment).
- (b) Lifting equipment, about 2 tons capacity, for transport of the panels from the storage area and for their set up.

2.2.4. Principle of Soil-Reinforcement Interaction

In Strip Reinforced Earth, the mechanism of soil to reinforcement stress transfer is mainly friction between the soil and reinforcement surfaces when smooth reinforcement strips are used. On the other hand, when ribbed strips are used, stress transfer is also developed by passive resistance on the ribs as illustrated on Figure 2.12.

Knowledge of the stress transfer in Reinforced Earth has been gained from many pullout tests on reinforcements located either in actual structures or in reduced scale models. Although this type of test is not entirely representative of true conditions, it does give results which are sufficiently precise for deduction of factors influencing soil-reinforcement stress transfer.

In pullout tests, the reinforcements are extracted from the reinforced soil mass, and the pullout force displacement curve is recorded. Because of soil dilatancy which develops in the vicinity of the reinforcements, the normal stress exerted on the surface is actually unknown. The pullout tests give only an apparent friction coefficient μ , which is defined by the ratio

$$\mu = \frac{\tau}{\sigma_v} = \frac{T}{(2bL\sigma_v)} \quad (2.1)$$

where:

τ = the average shear stress along the reinforcement

σ_v = the overburden stress

T = the applied pullout force

b = the width of the reinforcement

L = the length of the reinforcement

This apparent friction factor contains the contributions of surface shear and passive resistance to total stress transfer.

In dense granular soils, the values of μ are usually significantly greater than the values obtained from direct shear tests. This is mainly because dense granular soil in the vicinity of the reinforcements tends to increase its volume, i.e., dilate, during shear. This positive volume change is restrained by the surrounding soil. This confining effect results in an increase of the normal stresses exerted on the reinforcements and, consequently, in a high value of apparent friction coefficient. Furthermore, when ribbed strips are used, local passive failure zones are likely to develop against the faces of the ribs, thus adding to the pullout resistance.

Available information on the factors effecting the value of the apparent friction coefficient μ has been reviewed and summarized by Schlosser and Elias (1978), McKittrick (1978), Mitchell and Schlosser (1979). The data provide a clear indication that peak and residual values of μ are functions of:

- the nature of the soil (grading and angularity of the grains),
- the friction characteristics of the soil,
- the soil density,
- the effective overburden stress,
- the geometrical factors and surface roughness of the reinforcements,
- the rigidity of the reinforcements,
- the amount of fines in the backfill – this factor being a most critical one.

2.2.4.1. Influence of Reinforcement Surface Characteristics. All the pullout tests performed on smooth reinforcements and on ribbed reinforcements (also referred to as highly adherent or H.A.) have shown that the curves of μ as a function of displacement are of the form Figure 2.17.

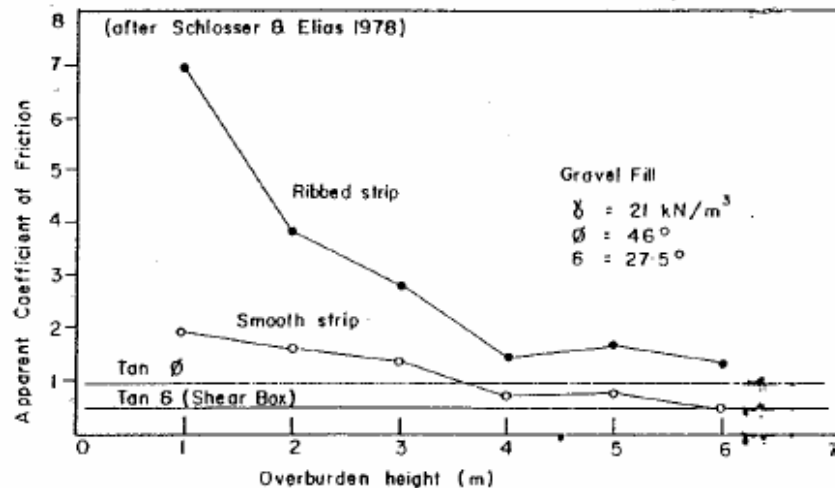


Figure 2.17 Influence of Reinforcement Surface on the Apparent Friction Coefficient

In the case of smooth reinforcement, the curve has a very noticeable peak which is obtained at a small displacement, and the residual value of μ is approximately half of the peak value. In the case of ribbed reinforcements, the values of μ at the maximum of the curve and at the residual value are only slightly different, and the maximum is obtained at comparatively large displacements.

The shapes of these curves imply that the approximate values of μ used in the design of Reinforced Earth structures should be the maximum value for ribbed reinforcements and the residual value for smooth reinforcements.

2.2.4.2. Influence of the Density of the Embankment. The laboratory studies on reduced scale models have shown that when the embankment is in a loose state, the apparent friction coefficient is always close or equal to the real friction coefficient. On the contrary, if the embankment is in a dense state, and this is always the case of actual structures even if they are only slightly compacted, the apparent friction coefficient can be highly superior to the real friction coefficient. These results can be explained by the phenomenon of

dilatancy. Under high densities, the shear stresses which develop in the immediate vicinity of the reinforcements have the tendency to increase locally the volume of the soil. This expansion is restrained by the low compressibility of the surrounding soil zones. Consequently there is an increase of the normal stress exerted on the faces of the reinforcement and therefore the value of μ is superior to the value of the real friction coefficient.

2.2.4.3. Influence of Overburden Stress. Pullout test on reinforcements located in actual structures, as well as the laboratory studies using dense sands, have shown that the value of the apparent friction coefficient decreases when the vertical overburden stress increases (Figure 2.17). Under high overburden stress the coefficient μ approaches

- the value of $\tan\phi$, where ϕ is the internal friction angle of the soil, for the ribbed reinforcements which also mobilize soil-to-soil shearing,
- the value of $\tan\delta$, where δ is the soil to reinforcement surface friction angle, for smooth reinforcements.

The dilatancy effect which develops in dense granular soil is a very important contribution to the pullout resistance of ribbed reinforcements. To demonstrate this, shear tests were done under constant volume conditions in densely compacted sand (Guilloux et al., 1979). These effects represented an extreme case and are expected to give the maximum increase of the normal stress which can be obtained when dilatancy is prevented. The results showed a very large increase of the normal stress, and the corresponding values of the apparent friction coefficient, μ , calculated as the ratio between the average applied shear stress and the initially applied normal stress, were very high. As it can be seen in Figure 2.17, the value of μ obtained in the constant volume tests exceeded those measured in the laboratory models and in full-scale embankments where some volume increases could develop by as significant margin.

2.2.5. Advantages and Disadvantages

Reinforced soil structures have many advantages compared to conventional reinforced concrete and gravity walls. These include:

- (a) Simple and rapid construction which does not require large equipment.
- (b) Does not require experienced craftsmen with special skills for construction.
- (c) Requires little site preparation.
- (d) Flexible nature of the structure enables earthquake resistance.
- (e) Needs little space in front of the structure for construction operations.
- (f) Reduced right-of-way acquisition by constructing or excavating steeper slopes.
- (g) Does not need rigid, unyielding foundation support, because reinforced structures are tolerant to deformations.

The relatively small quantities of manufactured materials required, rapid construction, and in addition, competition among the developers of different proprietary systems has resulted in a cost reduction relative to traditional types of retaining walls. Reinforced systems are likely to be more economical than other wall systems for walls higher than about 4.6 m or where special foundations would be required for a conventional wall.

One of the greatest advantages of reinforced soil structures is their flexibility and capability to absorb deformations due to poor subsoil conditions in the foundations. Also, based on observations in seismically active zones, reinforced soil structures have demonstrated a higher resistance to seismic loading than rigid concrete structures. Besides, precast concrete facing element for stabilized soil structures can be made with various shapes and textures (with little extra cost) for aesthetic considerations. Masonry units, timber and gabions can also be utilized with advantage.

A few general disadvantages may be associated with reinforced soil structures. Reinforced soil walls:

- (a) Requires a relatively large space behind the wall face to obtain enough wall width for internal and external stability.

- (b) Requires granular fill at the present time for many of the reinforcement soil systems. (At sites where there is a lack of granular soils, the cost of importing suitable fill material may render the system uneconomical).

Corrosion of steel reinforcing elements, deterioration of certain types of exposed facing elements such as fabrics or plastics by ultra violet rays, and degradation of plastic reinforcement in the ground must be addressed in each project by means of suitable designed criteria.

2.3. Principles of Reinforced Earth Walls

2.3.1. Basic Concepts

If the surface of a semi-infinite mass of cohesionless soil at rest is horizontal, at depth h below the surface vertical overburden stress and corresponding lateral stress are given as:

$$\sigma_v = \gamma h \quad (2.2)$$

$$\sigma_h = K_0 \gamma h \quad (2.3)$$

where γ is unit weight of the soil, h is depth and K_0 is coefficient of earth pressure at rest. According to Jacky (1944), for both normally consolidated clays and compacted soils coefficient of earth pressure at rest is given as

$$K_0 \approx 1 - \sin \phi \quad (2.4)$$

and if the soil is allowed to expand laterally, the lateral stress, reduces to a limiting (or failure) value of

$$\sigma_h = K_a \gamma h \quad (2.5)$$

and coefficient of active earth pressure K_a in Eq. 2.5 is given as:

$$K_a = \frac{(1 - \sin \phi)}{(1 + \sin \phi)} = \tan^2(45^\circ - \phi/2) \quad (2.6)$$

On the other hand, if the soil is compressed laterally, the lateral stress increases to limiting value of

$$\sigma_h = k_p \gamma h \quad (2.7)$$

and coefficient of passive earth pressure in the Eq. 2.7 is given as:

$$K_p = \frac{(1 + \sin \phi)}{(1 - \sin \phi)} = \tan^2(45^\circ + \phi/2) \quad (2.8)$$

2.3.2. The Work of Henry Vidal

Since the introduction of Reinforced Earth technology, laboratory scale models have proven useful in gaining an understanding of the behavior of full-scale structures. Henri Vidal, the inventor of Reinforced Earth, constructed and studied sand and paper models in the early 1960s. From 1962 to 1982, with the operation and financial support of the Reinforced Earth Group, independent laboratories around the world conducted more than a dozen scale-model research projects.

In his early work Henry Vidal (1966, 1969) accurately identified and explained the fundamental mechanism of Reinforced Earth. It was pointed out that unreinforced soil obeys the Mohr-Coulomb failure criterion which for a cohesionless soil may be simply defined by two linear failure envelopes inclined that $+\phi$ and $-\phi$ to the normal stress axis. If such a soil loaded by a vertical principal stress σ_1' then for the soil not to fail there must also be a lateral confining stress σ_3' acting on the soil. The minimum value of σ_3' consistent with stability is $K_a \sigma_1'$. This limiting condition is represented by the Mohr stress circle shown in solid line in Figure 2.18.

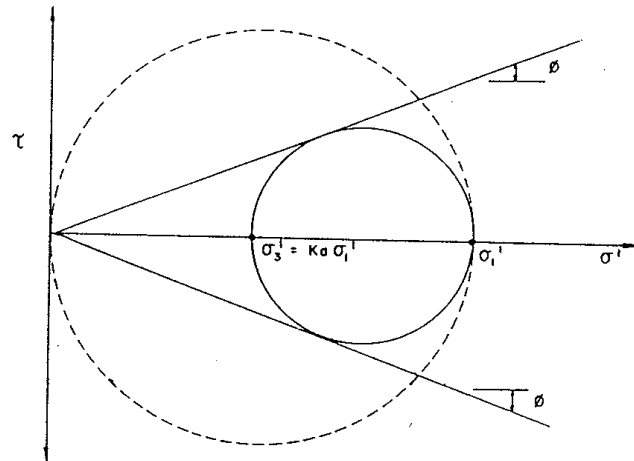


Figure 2.18 Failure in Unreinforced Soil

Now the same soil mass is reconsidered with horizontal reinforcement strips. Consider a layer between two adjacent reinforcement strips (Figure 2.19). If enough friction is developed, the top and bottom of the layer will be attached to the reinforcements. If the strips are close enough then the whole soil layer will be more or less constrained and the maximum strain that it can experience in the direction of the reinforcements will be order of the strain in the reinforcements. For this case frictional stress transfer between soil and reinforcement is illustrated schematically in Figure 2.20. Generally all reinforcement material available have a Young's modulus greater than that for the soil so that the resulting strains in the soil will be so small that the soil is essentially at rest and the lateral pressure within it can be assumed to be equal to the passive earth pressure $K_0 \gamma h$.

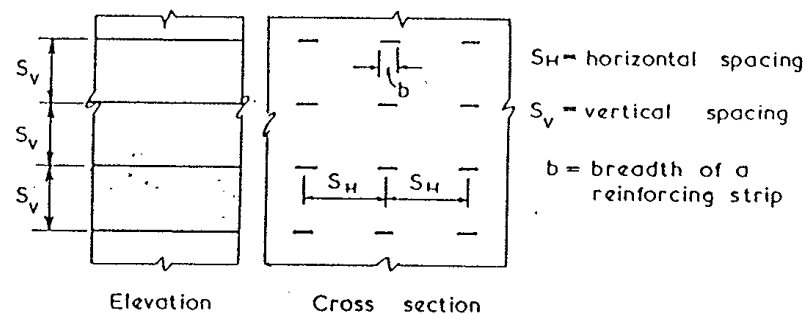


Figure 2.19 Arrangement of Reinforced Strips

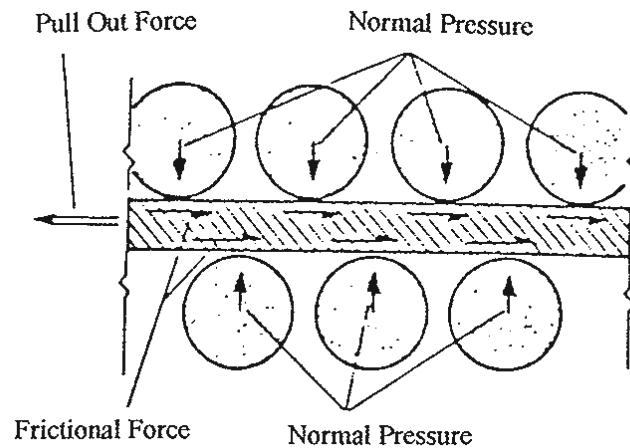


Figure 2.20 Frictional Transfer Between Soil and Reinforcement

As the action of loading the element of reinforced soil induces a tensile force in the reinforcement so there is a compressive lateral stress generated in the soil. This induced confining stress $\Delta\sigma_3'$ is analogous to an externally applied confining pressure σ_3' and provided that $\Delta\sigma_3' > K_a\sigma_1'$ there is no failure in the soil. This concept may be more readily understood by analyzing Figure 2.21. The left-hand stress circle represents an unreinforced soil under the action of confining stress σ_3' . Failure occurs under a major principle stress σ_1' . If the same soil were reinforced then during the process of loading the confining pressure increases to $\sigma_3' + \Delta\sigma_3'$ and failure occurs at a much higher stress level of σ_1' . Theoretically, soil failure can never occur provided it is laterally confined as the stress circle is always within the strength envelope, no matter what the value of σ_1' . Failure can only occur if the reinforcement breaks or pulls out of the soil. Failure ultimately occurs by bond that is slippage between the soil and reinforcement or by tensile failure of the reinforcement.

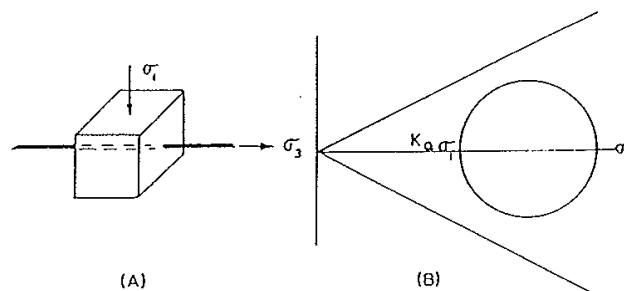


Figure 2.21 Lateral Confinement Concept

3. CASE STUDIES: BEHAVIOR OF REINFORCED EARTH STRUCTURES DURING PREVIOUS EARTHQUAKES

3.1. Italy, May 6, 1976

On May 6, 1976, the region of the Frioul in northern Italy was hit by an earthquake of 6.4 Richter magnitude. Its epicenter was located near the town of Gemona (Figure 3.1) of which 50% was destroyed. Major damage was recorded within a radius of 40km.

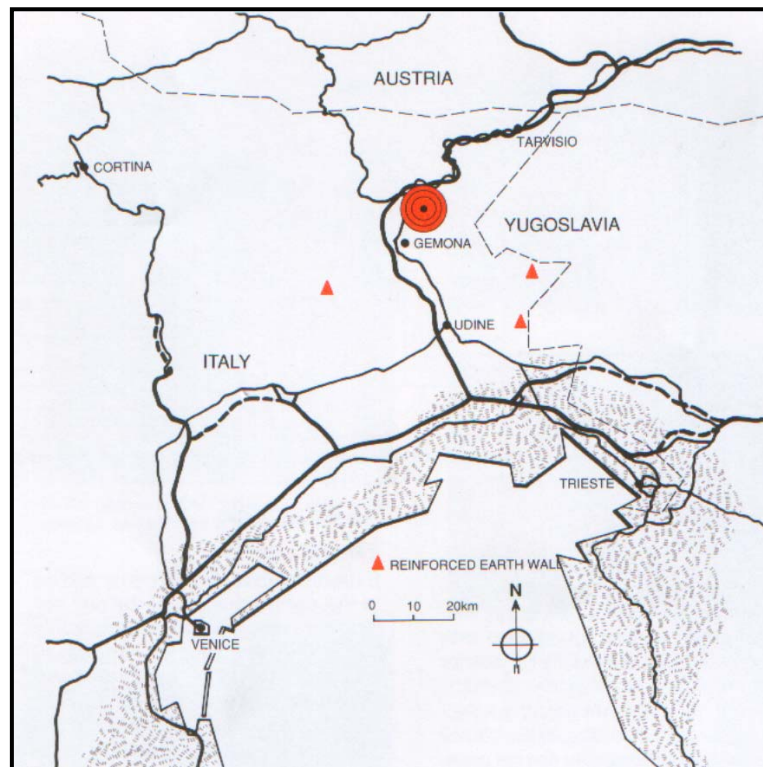


Figure 3.1 Reinforced Earth Structure Locations, Italy

Three Reinforced Earth structures with metallic facing elements of heights ranging from 4 to 6m had been constructed in the region one to two years earlier at distances of 25 to 40km from Gemona. Reinforcements were relatively short and proportioned only for static loading conditions with minimum factors of safety for adherence. After the earthquake no structure showed signs of damage or movement (Figures 3.2, 3, 4).



Figure 3.2 House ruins in Gemona

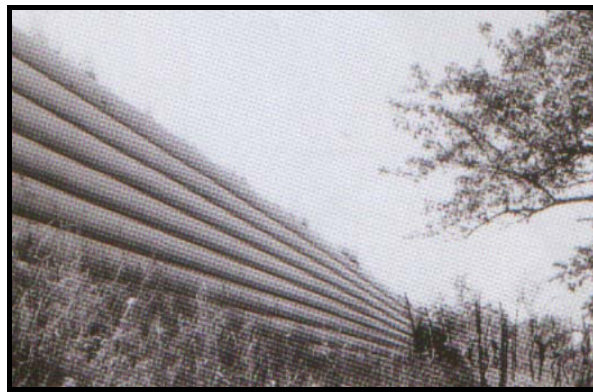


Figure 3.3 Dolegna del Collio structure with metal facing, near Gorizia

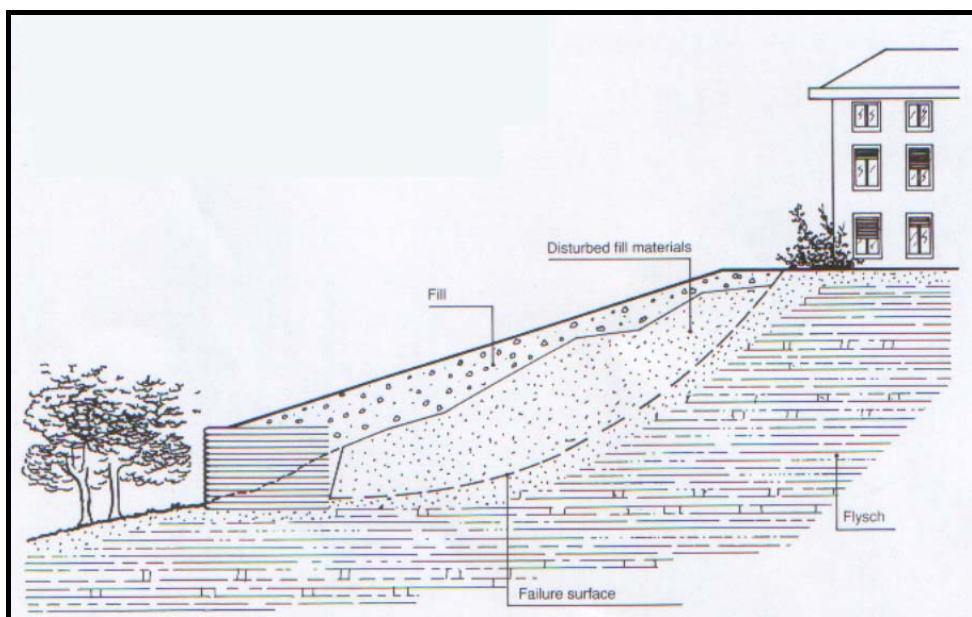


Figure 3.4 The structure at Dolegna del Collio was constructed to be used as a toe buttress for a small landslide

3.2. Japan, May 25, 1983

On May 25, 1983, an earthquake registering 7.7 on the Richter scale, with its epicenter located at sea, shook the entire northern section of the island of Honshu, particularly the districts of Akita and Aomori (Figure 3.5) where the maximum ground acceleration varied between 0.12g and 0.27g. The harbor installations of Akita, in particular, suffered great damage. On the peninsula of Oga, where 20 persons died, many roads and homes were severely damaged due to liquefaction of the alluvial silty sands (Figure 3.6).

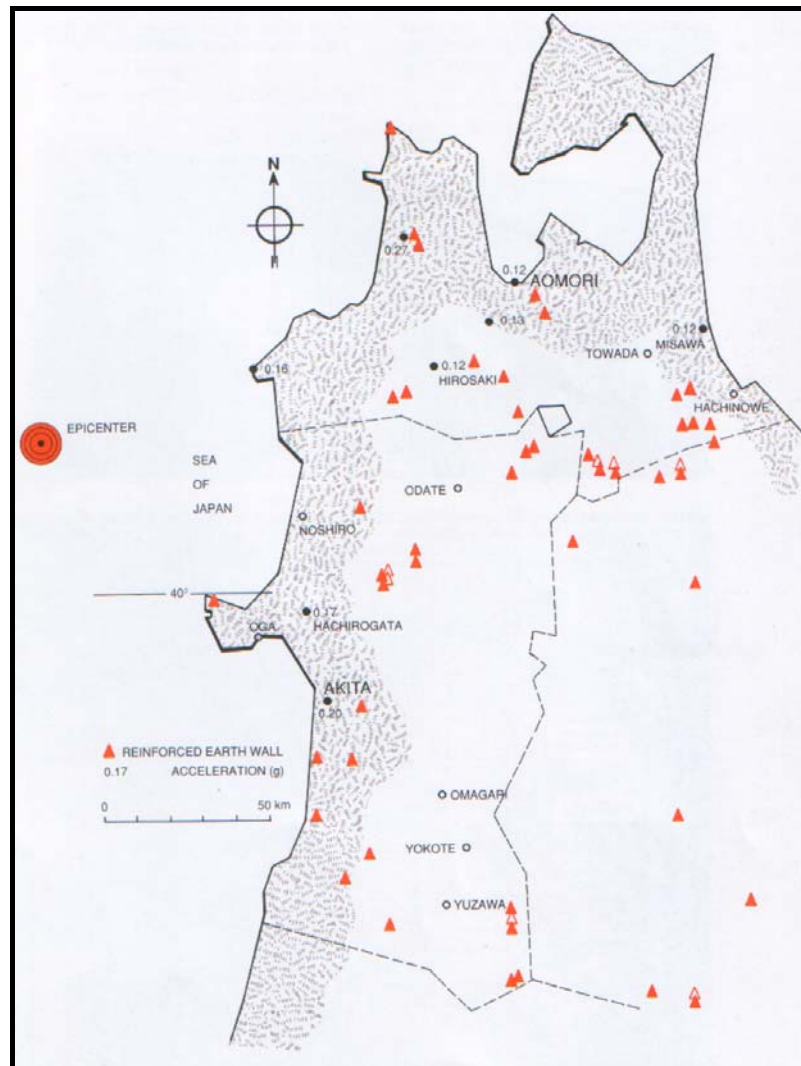


Figure 3.5 Reinforced Earth Structure Locations, Japan

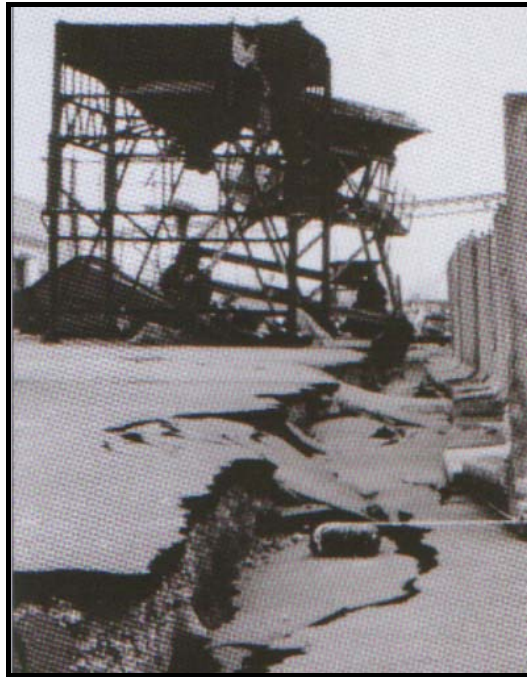


Figure 3.6 Severely damaged traffic and storage area at Akita harbor

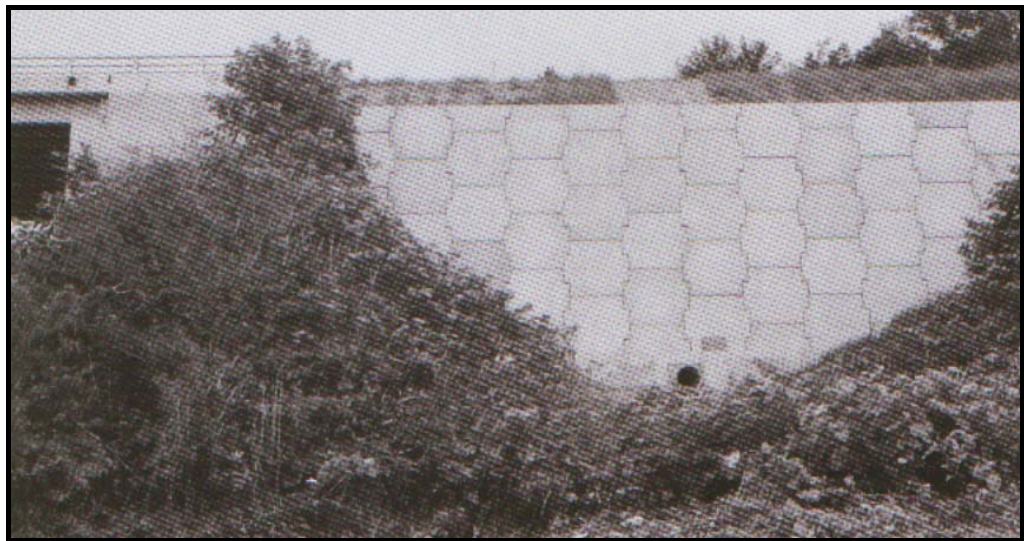


Figure 3.7 Reinforced Earth wall supporting the road from Kitaura to Kurosawa, on the peninsula of Oga.

Forty-nine Reinforced Earth structures of varying heights were visited in the region during the weeks that followed. One of them was located at the tip of the peninsula of Oga, at the point nearest the epicenter. They were all found to be in perfect condition, even the one that had settled by a few centimeters in relation to nearby structures supported on piles (Figure 3.7).

3.3. Belgium, November 8, 1983

A quite unexpected earthquake occurred at Liege on November 8, 1983 (Figure 3.8). Its magnitude was only 5.0 on the Richter scale, but the epicenter was located in a populated area and at shallow depth (3 to 5km). It, therefore, had extremely destructive effects, fortunately in a zone that was not very extensive (Figure 3.9).

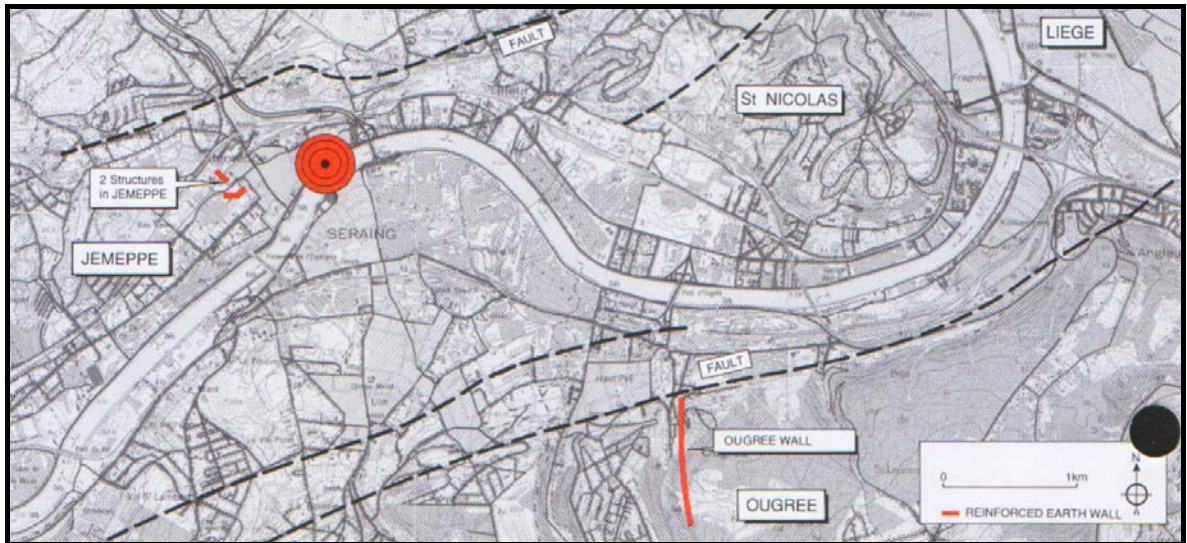


Figure 3.8 Reinforced Earth Structure Locations, Belgium



Figure 3.9 In the quarter of Saint Nicolas, a third of the homes was destroyed or damaged.

Two small walls of Reinforced Earth were located less than 1 km from the epicenter, in the region of Jemeppe where ground accelerations reached 0.15 to 0.2g. No deformation was observed, based on comparisons to observations made only a few days earlier. Opposite one of these walls, the roof of a garage moved 3cm (1.2in.) (Figure 3.10).



Figure 3.10 Views of one of the walls at Jemeppe and the neighboring garage

At Ougree, 3.5km from the epicenter, a large structure built of Reinforced Earth, (1.500m long and up to 17m high including three abutments) runs along a hillside. This structure also showed no damage or deformation, while many houses below were damaged.

3.4. Mexico, September 19, 1985

The earthquake that ravaged the center of Mexico City on September 19, 1985 had its origin 430km from the city, on the Pacific coast (Figure 3.11). The particular nature of the geology on which the city is built resulted in a significant amplification of the ground acceleration. A part of the city is built on a former lake filled 30 to 40m deep with clayey mud with a water content of roughly 400%. In this area the ground motion was 50 times greater than on the neighboring hillsides where damage was insignificant (Figure 3.12).

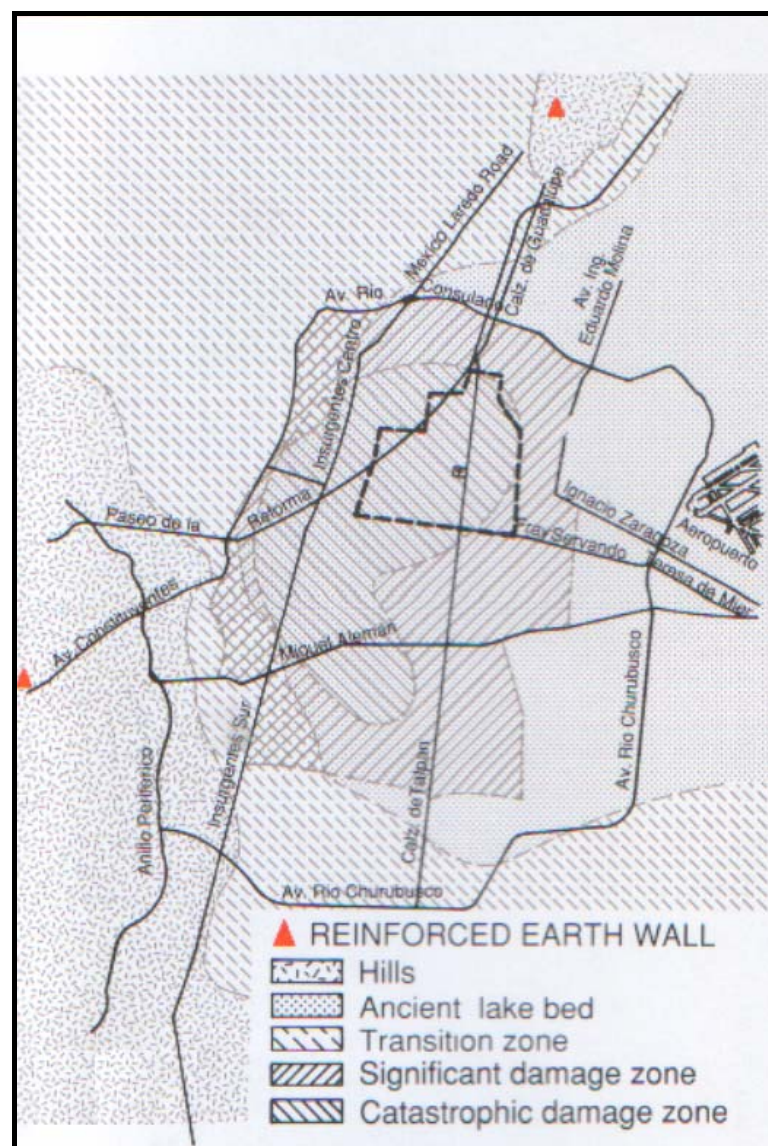


Figure 3.11 Reinforced Earth Structure Locations, Mexico

The bearing capacity of the soil deposits of the former lake bed is so limited that no embankment construction could be carried out. Thus, there are no walls constructed of

Reinforced Earth in this area. The two closest structures, located in the hills, performed in an excellent fashion (Figure 3.13).



Figure 3.12 Collapsed building in downtown Mexico City

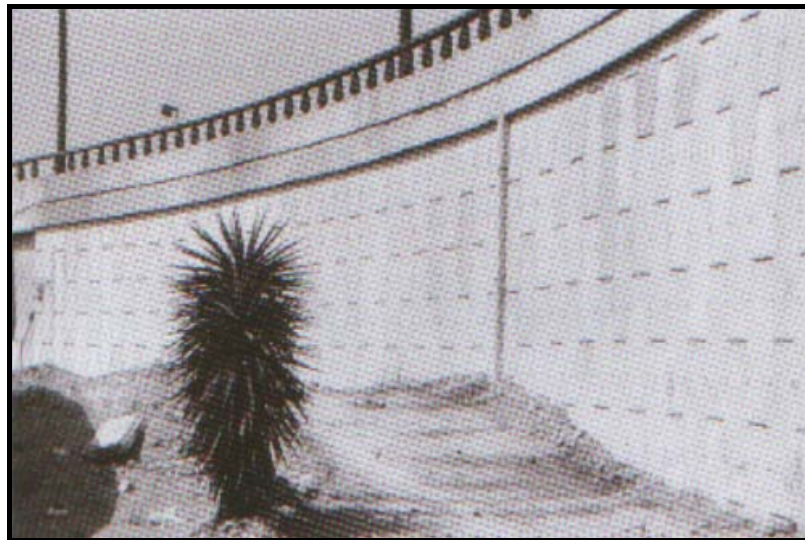


Figure 3.13 Reinforced Earth wall outside Mexico City

3.5. New Zealand, March 2, 1987

On March 2, 1987, an earthquake measuring 6.3 on the Richter scale, preceded a few minutes earlier by a weaker tremor, hit the Bay of Plenty north of New Zealand. Near Edgecumbe, a five foot wide crevasse was seen opening and running at the surface of the plain, forming a rise of 6.5 feet (Figure 3.14). In town the homes and buildings suffered great damage; silos and electric transformers were overturned (Figure 3.15).

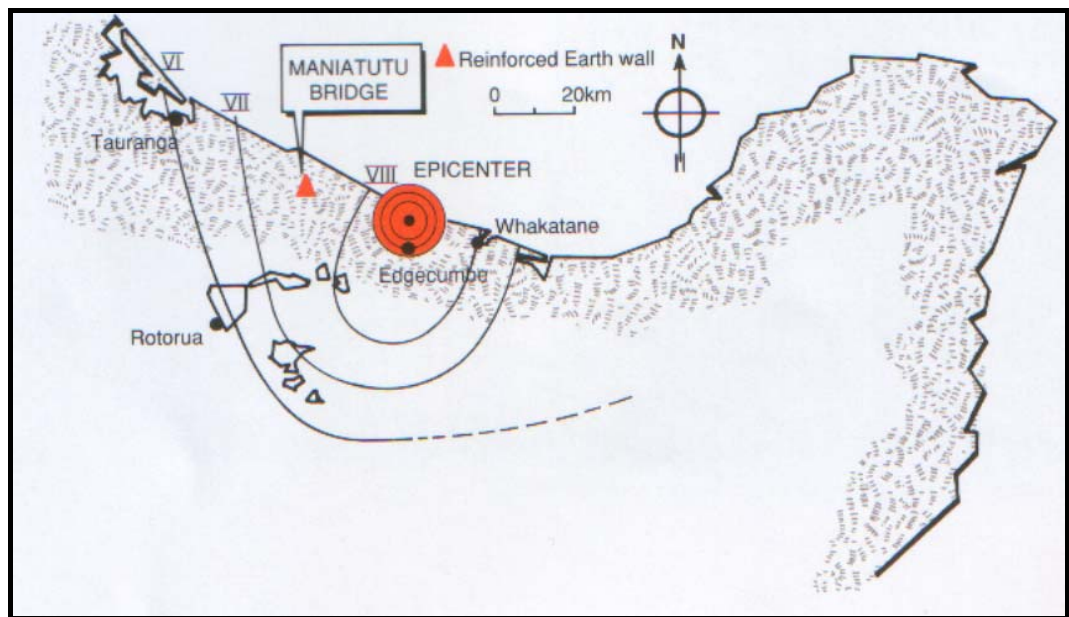


Figure 3.14 Reinforced Earth Structure Locations, New Zealand



Figure 3.15 Overturned milk silo near Edgecumbe

In Maniatutu, 30km from the epicenter, the construction of a bridge on abutments built of Reinforced Earth was just being completed. The prefabricated elements of the deck, already placed but not yet secured, began "dancing" on their supports causing the workers to flee. The backfilling of this Reinforced Earth abutment had not reached its final level and there was only limited adherence available to support the structure. There was still no noticeable deformation of the abutments. They were maintained in that condition, completed, and opened to traffic the following year (Figure 3.16).



Figure 3.16 Maniatutu Bridge

3.6. San Francisco, October 17, 1989

The earthquake of October 17, 1989, in California, was caused by movement along a section of the San Andreas Fault, near Mount Loma Prieta, 95km south of San Francisco. Measuring 7.1 on the Richter scale, it originated at a depth of 15km and caused major damage up to 110km away. The maximum horizontal accelerations recorded at ground level were considerable. They were around 0.5 or 0.6g in the region of the epicenter where they remained greater than 0.3g for about ten seconds, accompanied by vertical accelerations of the same order. In Oakland and San Francisco, accelerations ranged from 0.15 to 0.25g.

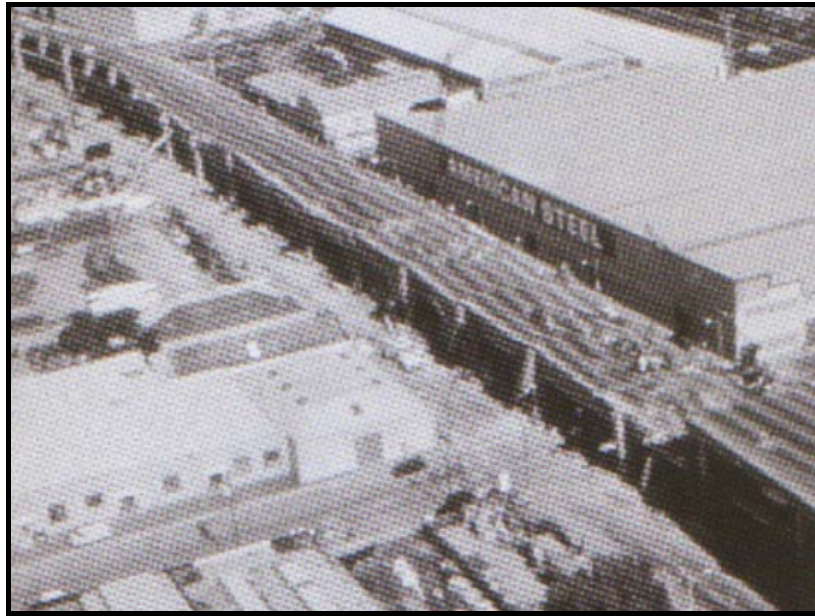


Figure 3.17 Aerial view of damaged viaducts on I-880

In the entire region affected by this earthquake, 20 walls built of Reinforced Earth were identified and inspected. These were distributed among nine sites and represented a total facing area of $20,500 \text{ m}^2$ (220,000 sq. ft.)- None of them suffered a severe damage (Figures 3.18, 19).

One of the structures is located in Watsonville, 11km from the epicenter. It forms a wall of the basement of a shopping center, the superstructures of which were severely damaged. In the same region, the Struve Slough bridge collapsed after having oscillated on its piers.

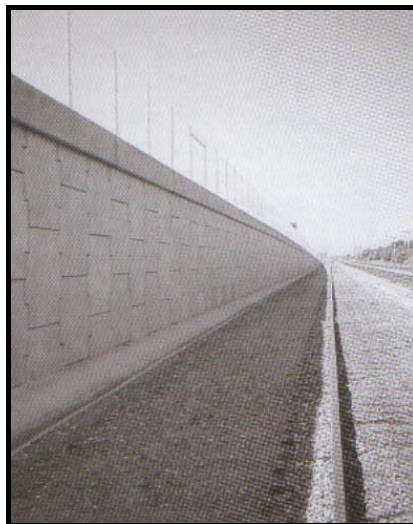


Figure 3.18 Richmond, California

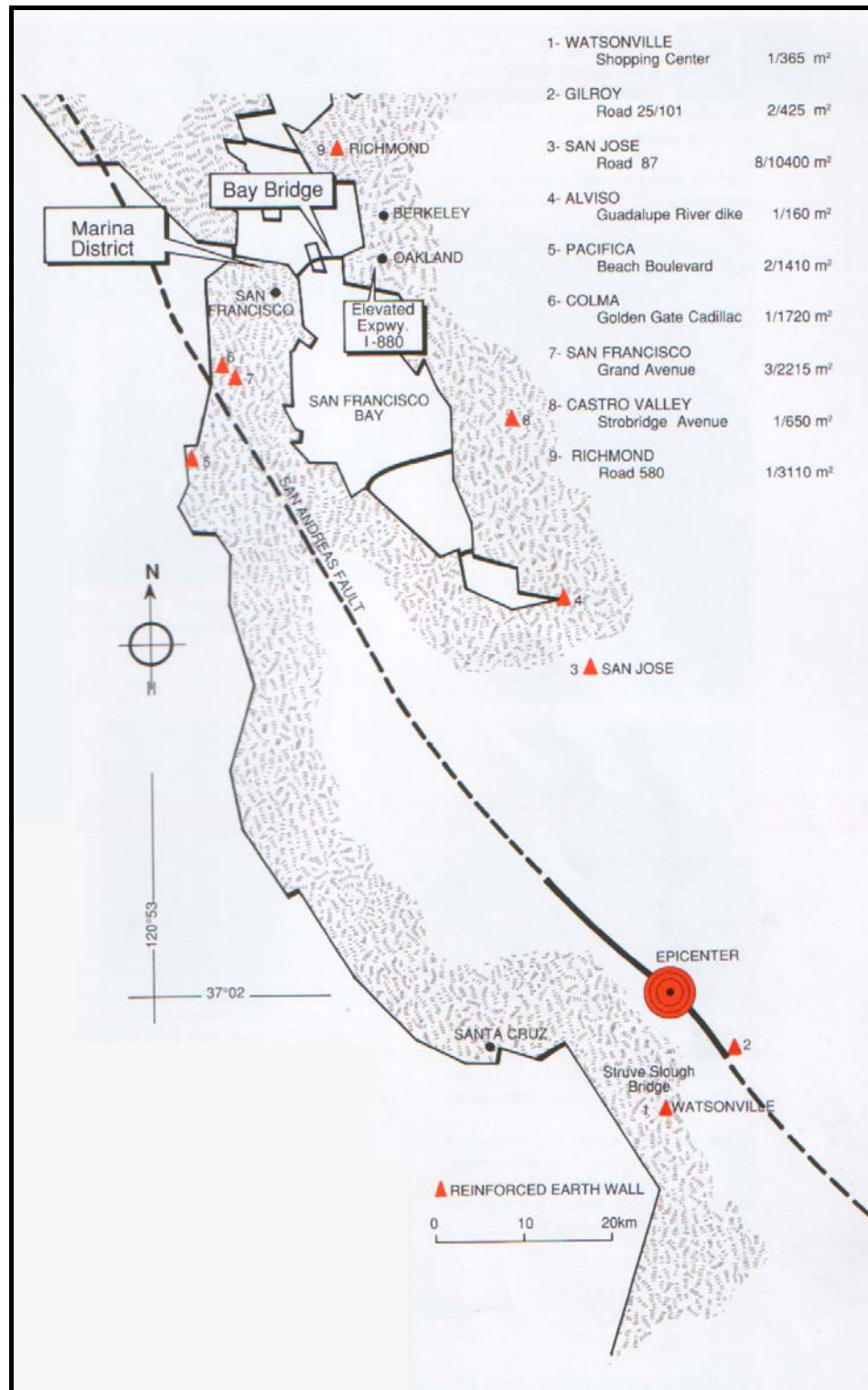


Figure 3.19 Reinforced Earth Structure Locations, San Francisco



Figure 3.20 Grand Avenue over the Caltrain Line at the South San Francisco station

Another wall is located at Richmond, 10km from the viaduct of Route 880 (The Nimitz Freeway) which suffered a dramatic collapse of the upper deck onto the lower deck (Figure 3.17). This wall is also founded on the soft mud of the bay, which was responsible for most of the cases of destruction in that area.

Everywhere the excellent performance of Reinforced Earth structures confirmed the stability of this unique material, as well as the conservatism of the design procedures employed by The Reinforced Earth Company in proportioning their structures (Figure 3.20).

3.7. Northridge, January 17, 1994

The 6.7 Richter magnitude Northridge Earthquake which occurred on January 17, 1994, caused severe structural damage to buildings and freeways. The earthquake occurred in the densely populated San Fernando Valley in Northridge, California, 20 miles northwest of downtown Los Angeles. The Northridge earthquake generated the highest vertical ground accelerations ever recorded in California. Ground accelerations were recorded as high as 1.82 g horizontal and 1.18 g vertical 3 miles from the epicenter. The extensive structural damage included the collapse of 6 major freeway bridges, 18 parking buildings with garages and carports on the first floor.

There are 23 Reinforced Earth structures within the affected area of the earthquake. The structures include 21 Reinforced Earth walls supporting the Los Angeles Metrolink Rail Line, CalTrans mountain highways, freeways and on/off ramps, city streets in San Pedro and Los Angeles, Chevron refinery oil storage tanks, a housing development in Los Angeles and 2 Reinforced Earth bridge abutments in Corona. The distance of the Reinforced Earth structures from the quake epicenter range from 8 to 52 miles. Three Reinforced Earth structures are located less than 9 miles from the epicenter. In this area, ground accelerations ranged from 0.46g to 0.66g horizontal and from 0.18g to 0.29g vertical. Regardless of wall locations, all structures have performed as intended and are structurally sound and intact. Only one wall, Lyons Avenue shown, located 10 miles from the epicenter in Valencia, suffered superficial damage to the bottom facing panels.

3.8. Japan, January 17, 1995

The Kobe Earthquake was a terrible tragedy that destroyed over 150,000 structures (houses, buildings, bridges, elevated roads, port works and utility services).

Of the 120 Reinforced Earth structures inspected after the earthquake, 70% were over 5 m high and 15% were over 10 m high. The structures were designed using estimated ground accelerations of 0.15g to 0.2g. The actual ground acceleration was 0.27g. Ground movement was evident above or next to 22 of the structures, with 10 walls showing minor cracking of isolated concrete panels and 3 walls exhibiting significant lateral movement (Tatsuoka 1995 and Kobayashi, 1997). Deformations recorded in walls at Awaji Island and Hosiga-oka Park varied between 4 mm to 113 mm (displacement relative to bottom of the panel at midheight and top of walls). All of the walls remained functional after the earthquake.

3.9. Turkey, August 17, 1999

One bridge and ramp structure was surveyed at Arifiye (Figure 3.21), almost immediately adjacent to the epicenter (Segrestin 2000 and Asheim & Mander, 2000). Although the bridge itself collapsed, the Reinforced Earth ramp walls sustained only nominal damage and remained stable (Figures 3.22, 23, 24). Shear deformations from differential settlements propagated upward through the panels, separating some by as much as 75 mm. The Reinforced Earth walls were designed for a ground acceleration of 0.10 g. This resulted in only a minor increase in the amount of reinforcing strips compared to static design. Yet the actual ground acceleration was measured at 0.4 g. It is interesting to note that if the full effect of ground acceleration were considered in design under current practice, then at least 40% more reinforcement would have been added. The fact that the increased reinforcement did not prove to be needed is a good indication of the safety of the technology and conservative nature of current design principles.

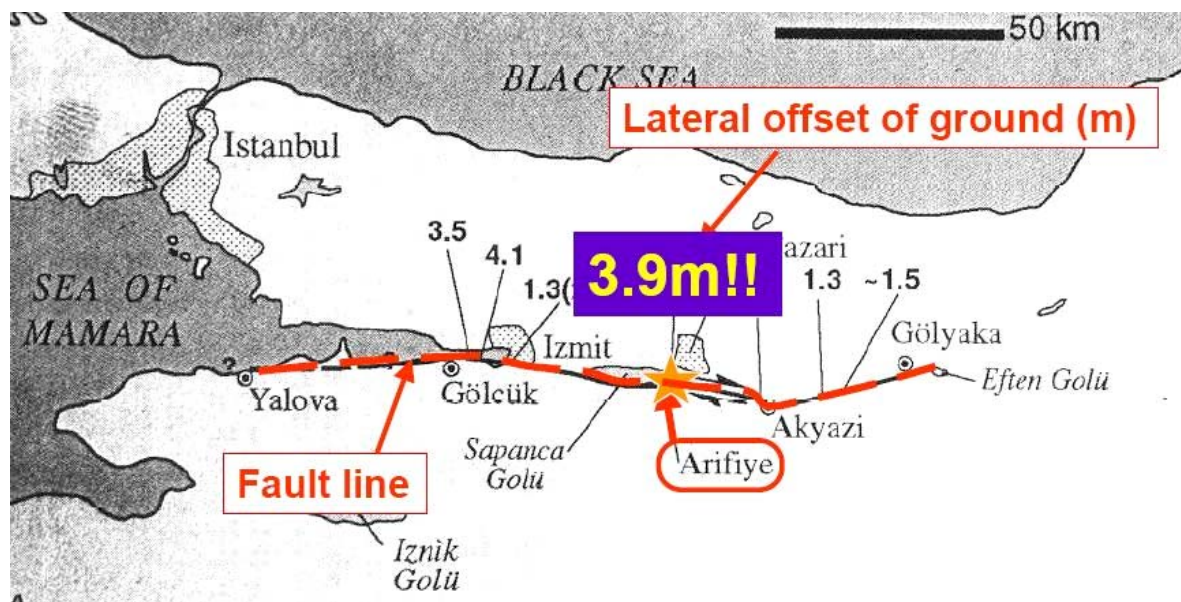


Figure 3.21 Reinforced Earth Structure Location, Arifiye



Figure 3.22 Arifiye Reinforced Earth ramp wall

The performance of Reinforced Earth structures in seismic events can be attributed to many factors. Reinforced Earth is flexible, allowing significant differential movement to occur within the reinforced mass without risking the integrity of the wall or its supporting structure. The Reinforced Earth granular backfill serves as an excellent damping medium. Panel joints are open allowing pore pressures (if any) to dissipate. The strength and flexibility inherent in Reinforced Earth structures allows the structures to withstand seismic events.



Figure 3.23 Reinforced Earth Ramp Walls and collapsed bridge, Arifiye

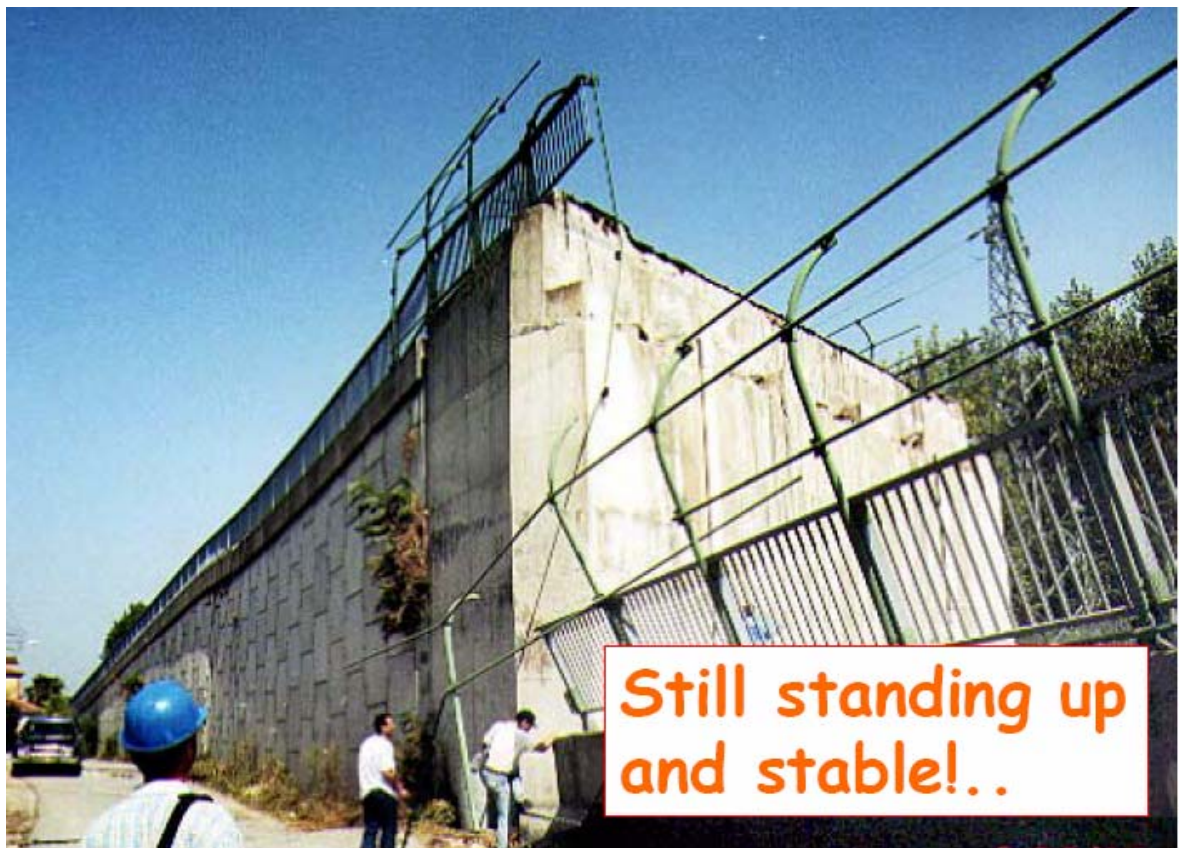


Figure 3.24 Reinforced Earth Ramp Walls, Arifiye (1999)

4. METHOD OF APPROACH

4.1 Design of Strip Reinforced Earth Walls

4.1.1. Introduction and Basis

The functioning of the reinforced earth, composite material, is essentially based on the existence of friction between the earth and reinforcements. Experimental and theoretical research have showed that the mechanism is complex and that it corresponds to the behaviour of an imaginary coherent material with an anisotropic cohesion proportional to the tensile resistance of the reinforcements.

Design method for general type of strip reinforced earth structures contains general design guidelines and a unified evaluation method common to all reinforced wall systems. The method is based on current experience, which is limited to:

- soil walls having a near-vertical face (face inclination of 70° to 90°),
- structures up to 100 ft (30 m) in height for inextensible steel reinforcement,
- structures up to 50 ft (15 m) height for extensible polymer reinforcement,
- vertical reinforcement spacings ranging from 0.5 ft to 3 ft (150 mm to 910 mm),
- granular backfill,
- segmented and flexible facing systems,
- structures with adequate drainage to eliminate hydrostatic water pressure.

4.1.2. Reinforcements

The horizontal reinforcements in a reinforced soil wall act by restrained lateral displacement of the reinforced fill. The extensibility of the reinforcements compared to the deformability of the fill is an essential feature of the behaviour of the wall, as it controls the state of horizontal stress in the reinforced soil mass. Inextensible reinforcement creates a relatively unyielding mass such that the state of horizontal stress approaches an at-rest,

K_o , condition while with extensible reinforcement, the fill can yield laterally so that an active state, K_a , condition can be reached throughout the reinforced soil mass. Reinforcements constructed with linear metal elements (such as metal strips) are inextensible and those reinforcements can rupture before the soil reaches a failure state. Other materials such as geogrids, wire woven at oblique angles, and woven geotextiles are in between truly extensible and inextensible materials. However, as these materials can deform substantially before failure, the soil reaches a failure state before reinforcement ruptures, they are generally assumed as extensible for design purposes.

4.1.3. Reinforcement Tension

The variation of the tensile forces along the reinforcement and the location of the maximum force has been established both experimentally, through instrumented models and full-scale structures, and theoretically, using numerical analysis. Strain gauge measurements show the variations in tensile forces along reinforcing strips, or (at a minimum) the averages of these variations (Figure 4.1). From these curves, it is possible - to locate the point of maximum stress at each level of reinforcing strips. By connecting these points, one can derive the line of maximum tensile force in the structure.

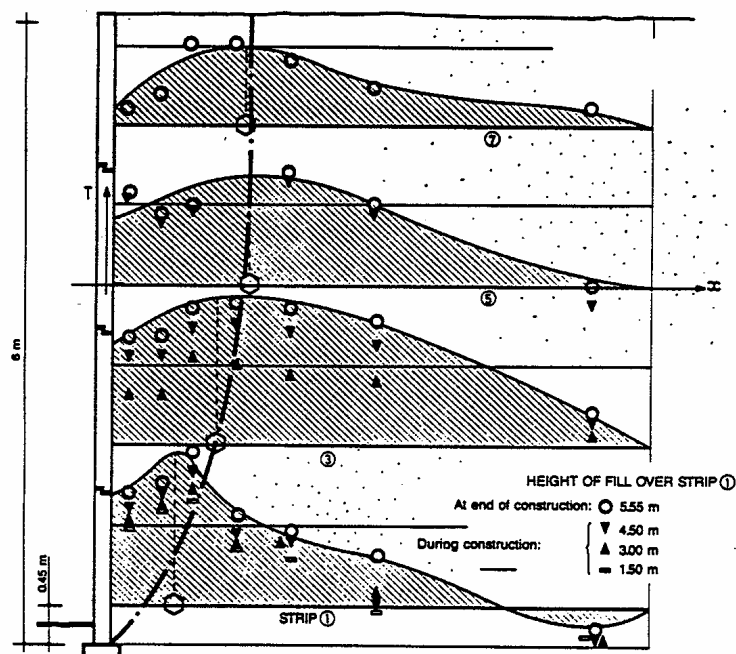


Figure 4.1. Line of Maximum Stress at Each Reinforcement Layer After Strain Gauge Measurements

A graph showing variation in maximum tensile force as a function of depth can also be derived from these measurements (Figure 4.2). In all projects, it has been observed that stress is not entirely proportional to depth. Stresses are higher at the top of the wall and lower at the base.

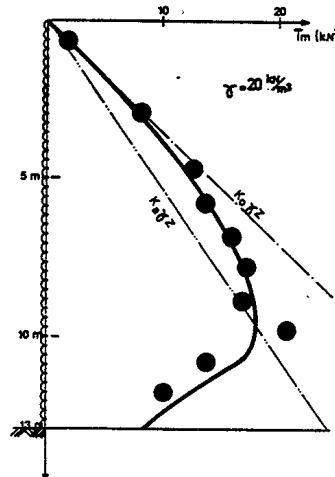


Figure 4.2 Maximum Tensile Force as a Function of Depth [11]

Tensile stresses within the reinforcing strips are at a maximum at a certain distance behind the facing. In order to create a maximum force at that location, the shear stresses exerted by the frictional fill material on the reinforcement must be in opposite directions on the two sides of the peak force. The line joining the point of maximum tensile force separates the active zone in which the reinforcing strips retain the fill, from the passive zone, in which the friction of the fill retains the reinforcing strips. Those mentioned zones can also be outlined as:

- an active zone where the shear stresses exerted by the soil on the reinforcements are directed towards the exterior of the wall,
- a resistant zone where these stresses are directed towards the interior of the wall.

The location of the maximum tensile force line is influenced by the extensibility of the reinforcement as well as the overall stiffness of the facing. Figure 4.3 show the limiting locations of the maximum tensile forces line in wall with extensible and inextensible reinforcements.

All data confirms that when metallic reinforcing strips are used, the line separating the two zones begins at the toe of the structure and follows a nearly vertical path to a point less than $0.3H$ from the facing at the top of the structure. This is true regardless of the structure's dimensions (up to $L/H=0.4$) even for the structures with a trapezoidal section Figure 4.4.

For each reinforcement an adherence length L_a is defined which is the length of the portion of the reinforcement situated in the resistant zone. In particular this line is not coinciding with the straight line which starts at the lower edge of the facing and is inclined at an angle of $\pi/4 + \phi/2$ as shown in Figure 4.3.

With inextensible reinforcements, the maximum tensile forces line can be modelled by a bilinear failure surface which is vertical in the upper part of the wall. The state of stress is assumed to be at rest at the top and decreases to the active state in the lower part of the wall.

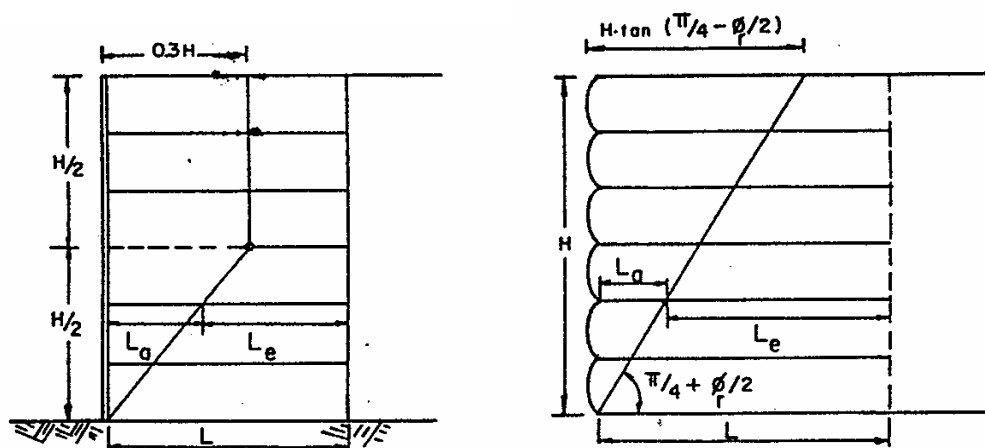


Figure 4.3 Tensile Forces in the Inextensible & Extensible Reinforcements and Schematic Maximum Tensile Force Line

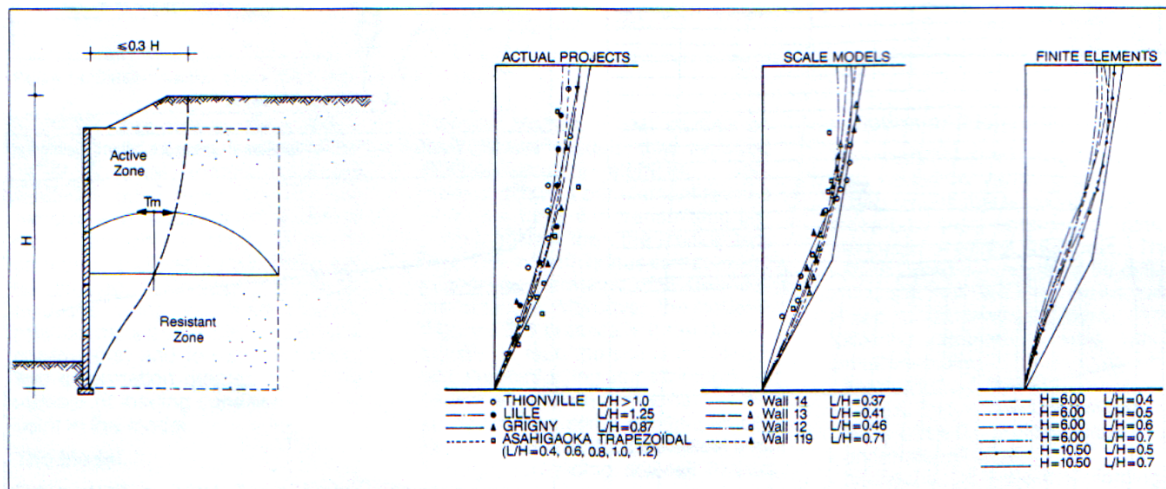


Figure 4.4 Active Zone; Measurements Taken on Actual Projects, Line Obtained Using Scale Model, Results of Finite Element Analysis

With extensible reinforcements, the maximum tensile forces line coincides with the Coulomb or Rankine active failure plane, and the stresses in the fill correspond to the active earth pressure condition. The location of the maximum tensile forces line may also be affected by the external factors such as the shape of the structure and surcharge conditions.

4.1.4. Modes of Failure of a Reinforced Soil Wall

The current design procedures for reinforced earth retaining structures consider the internal and external stability analyses separately.

4.1.4.1. Internal Stability. For internal stability, two failure mechanisms are considered:

- failure by breakage of the reinforcing strips – designed for by assuring that reinforcement cross section is adequate,
- failure by pullout of the reinforcing strips – designed for by assuring that reinforcement surface area and length are adequate.

Each mode of failure can be analyzed using the maximum tensile force line. This line is assumed to be the most critical potential slip surface. The length of reinforcement extending beyond this line will thus be available pullout length.

4.1.4.2. External Stability. For external stability, the Reinforced Earth structure is considered to behave as a gravity structure, and four classical failure mechanisms are analyzed as shown in Figure 4.5. Those mechanisms are:

- sliding of the structure on its base,
- bearing capacity failure of the foundation soil,
- overturning of the structure,
- deep seated stability failure (rotational slip-surface or slip along a plane of weakness).

Due to the flexibility and satisfactory field performance of reinforced soil walls, the adopted values for the factors of safety for external failure are lower than those used for reinforced concrete cantilever or gravity walls. For example, the factor of safety for overall bearing capacity is 2.0 rather than the conventional value of about 3.0, which is used for more rigid structures.

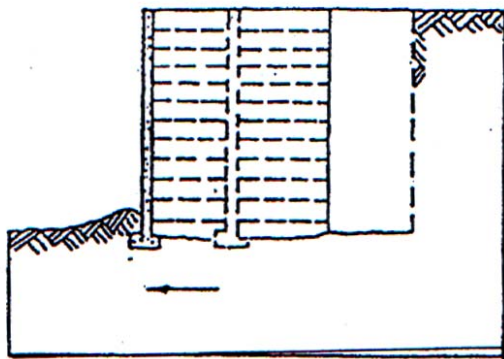
4.1.5. Loading and Boundary Conditions

Reinforced Earth structures can be designed to support different types of boundary loadings. The most common conditions as illustrated in Figure 4.6 include static loads such as:

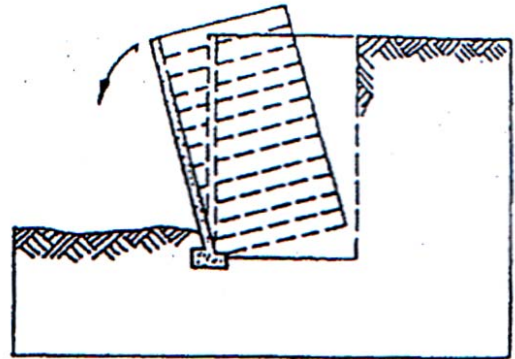
- earth thrust,
- vertical and horizontal concentrated line (or point) loads on bridge abutments,
- cyclic traffic loads that are represented by an equivalent uniformly distributed surcharge,
- earth slopes and highway embankments on walls,
- water pressures on reinforced earth dams and channels.

and dynamic loads such as:

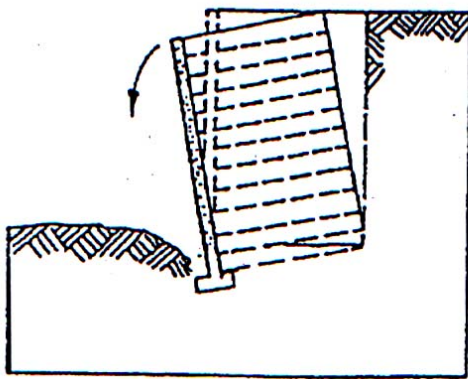
- vertical and horizontal vibrations induced by railway traffic.
- seismic loadings.



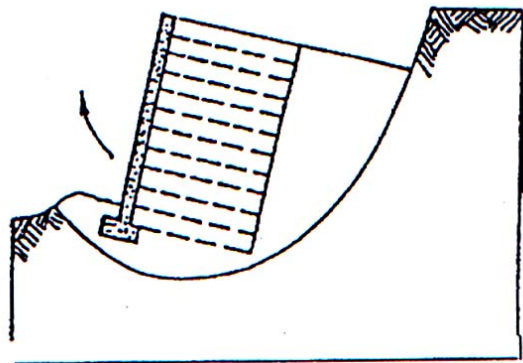
(a) Sliding



(b) Overturning (eccentricity)



(c) Bearing capacity



(d) Deep seated stability (Rotational)

Figure 4.5 Potential External Failure Mechanisms of a Reinforced Earth Soil Wall

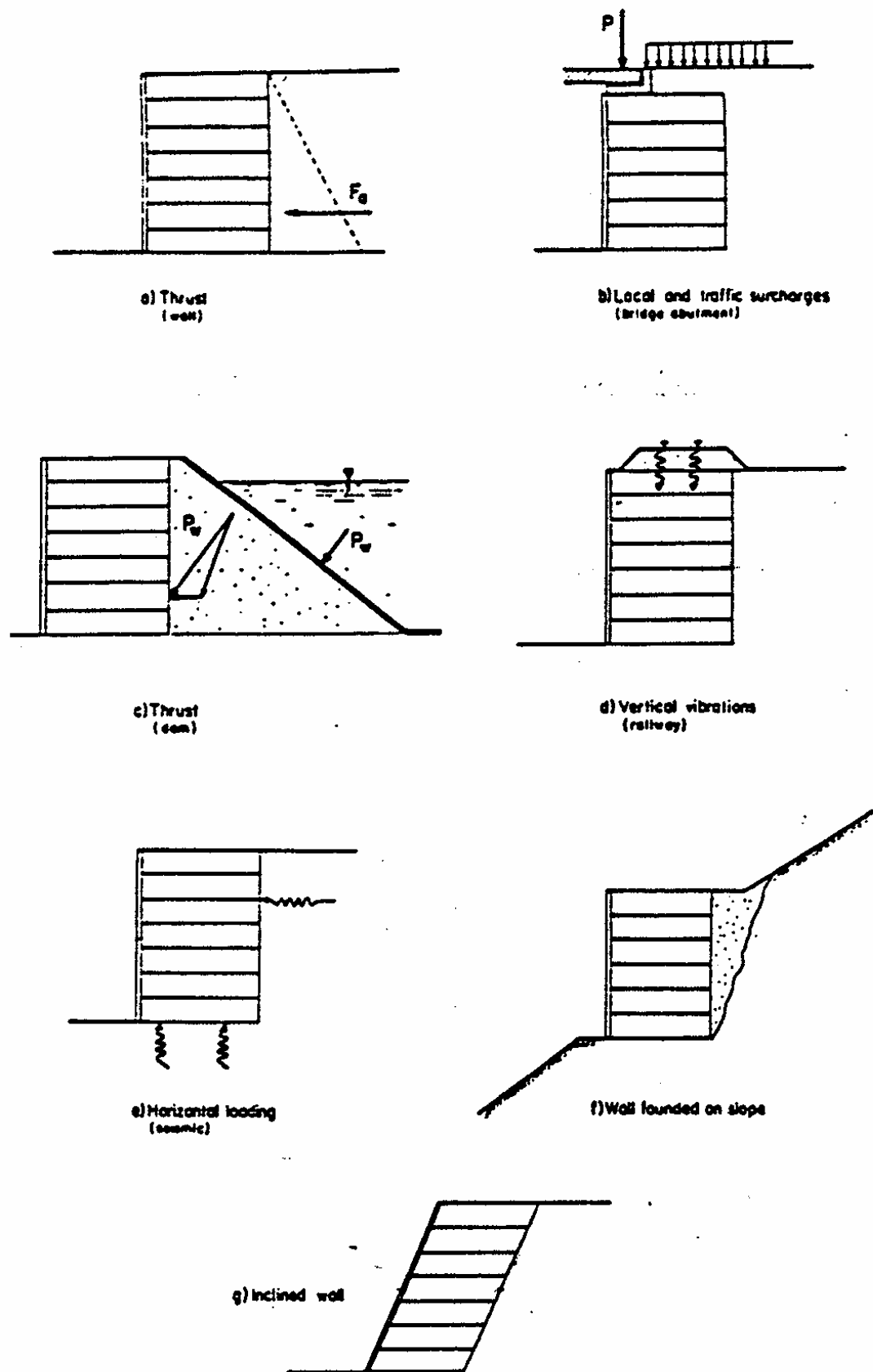


Figure 4.6 Different Loading and Boundary Conditions

4.1.6. Actions due to Water

The actions due to water, or more generally related to water, can result in multiple effects among which the principal ones, concerning the reinforced earth structures are:

- the phenomenon of bouyancy corresponding to the thrust of Archimede,
- the hydrostatic pressure,
- seepage forces,
- the variation of the coeficient of soil-reinforcement friction.

In the design, only the hydrostatic pressure and the eventual seepage forces are to be included in the combinations as actions. The other effects modify the characteristics of the material.

In a general manner, the actions due to water concern structures in aquatic sites (riverbanks or sea borders). The water level is variable (marling, rising of water) and can be different in the interior and in the exterior of the reinforced earth wall. For each particular site, two couples of values will be defined for the water level. The first couple, corresponding to what is called “The characteristic levels” is defined in the case of structures at river borders, by the highest water level (H.W.) and by the lowest water level (L.W.) determined on centennial basis.

The other couple is defined, always in the case of structures on river borders, by the decennial H.W. and the decennial L.W. In the case of structures in maritime sites, the definition of the level is analogous to the specified above for the structures on borders of water streams.

In this case, in the characteristic situation, no variable action except the one related to water should be taken into account in the combinations. However, a difference ΔH between the water level outside and inside the structure is to be considered and it is assumed to be equal to the maximum observed lowering of the water level for the considered river, unless particular justifications are considered for highly permeable material. This difference of level is variable between the characteristic levels and is

considered in such a way that the most unfavourable effect is obtained. This condition is shown in the Figure 4.7.

It is then necessary to consider a difference between the water level inside and outside the reinforced earth wall which is arbitrarily fixed to $\Delta H/2$. The water level can then be simply varied between the associated levels in such a way that the most unfavourable effect is obtained.

4.1.7. Design Steps

Except for special cases of walls either founded on slopes or those having sloped face, Reinforced Earth retaining structures normally have rectangular cross-sections, reinforcement strips are of the same length for the full height of the wall, and the wall face is vertical Figure 4.8.

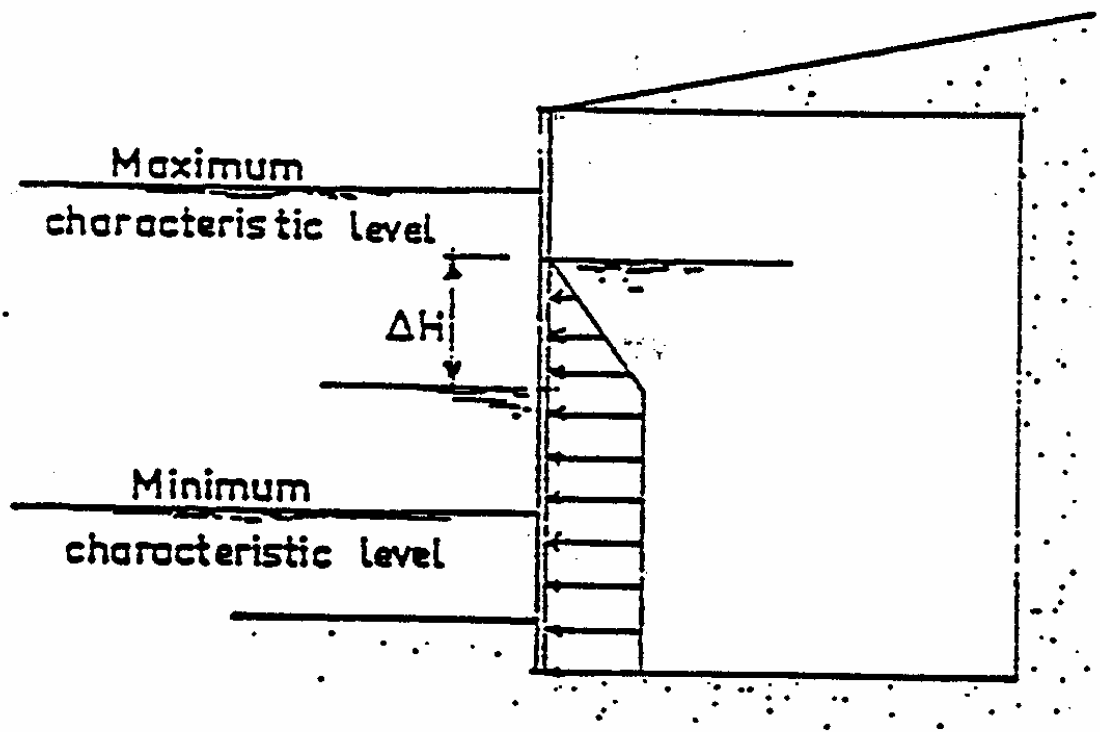


Figure 4.7 Actions Due to Water

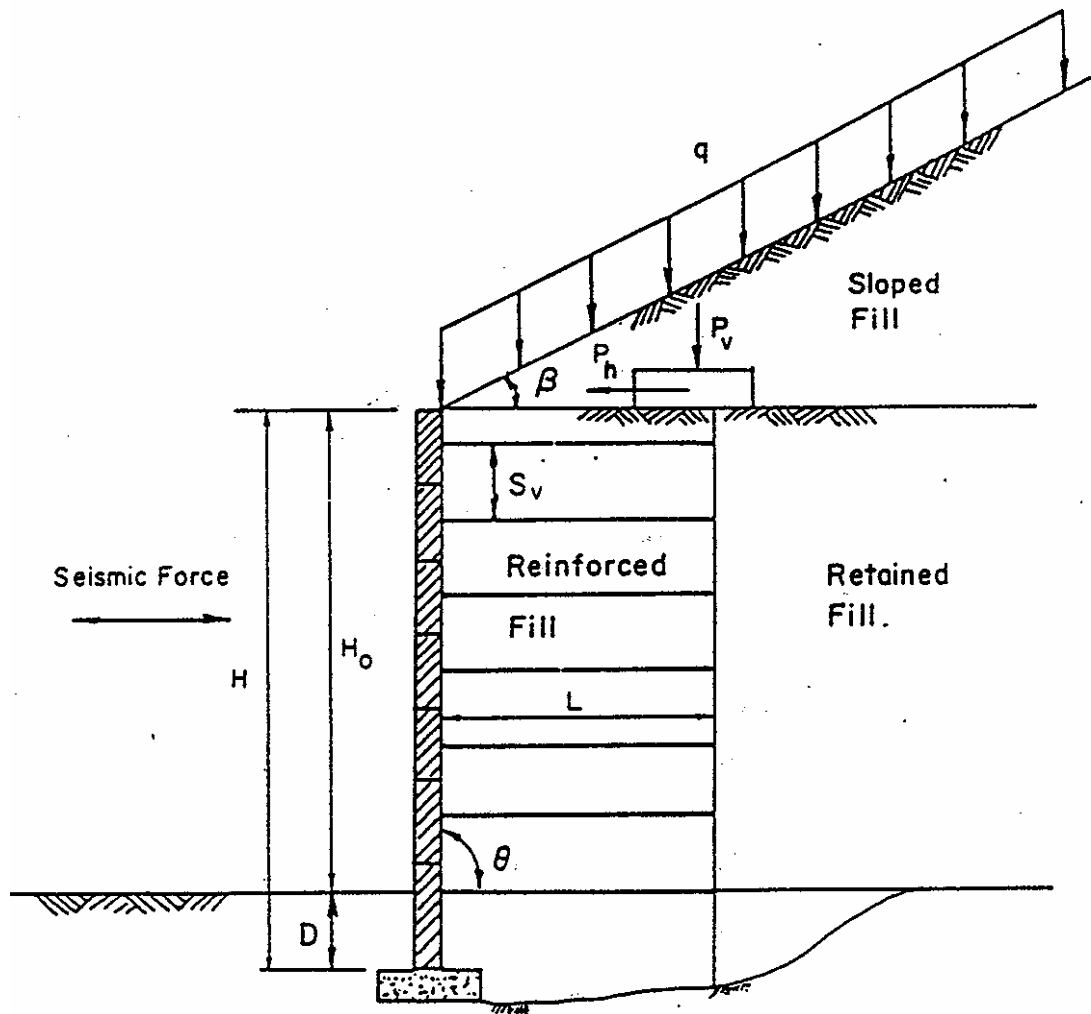


Figure 4.8 Geometric and Loading Characteristics of a Reinforced Soil Wall

4.1.7.1. Wall Embedment Depth. An embedment depth, D , is usually required for reinforced earth structures to avoid bearing failure of the foundation soil. Embedment is also required because of risk of local failure in the vicinity of the facing, depth of frost and risk of scour or erosion in the vicinity of the facing. Minimum embedment depth D at the front of the wall (Figure 4.8) recommended by AASHTO-AGC-ARTBA Task force 27 is given in Table 4.1. Larger values of embedment depth may be required, depending on depth of frost penetration, shrinkage and swelling of foundation soils, seismic activity, and scour. Minimum in any case can be taken as 1.5 ft (0.46 m).

Table 4.1 Minimum Embedment Depth D at the Front of the Wall

Slope in Front of Wall	Minimum D to Top of Levelling Pad
Horizontal (walls)	H/20
Horizontal (abutments)	H/10
3H:1V	H/10
2H:1V	H/7
3H:2V	H/5

4.1.7.2 Vertical Spacing Requirements. A predetermined vertical spacing of the reinforcement is required for further evaluating the required reinforcement strength. The spacing requirements can be given, as would be the case in a review of specific designs provided by others, or determined from fabrication and construction requirements including type of facing, facing connection spacings, and lift thickness required for fill placement.

For wrapped faced walls with sheet type reinforcement, the vertical spacing should be a multiple of the compacted lift thickness required for the fill (typically 20 to 30 cm). For spacings greater than 0.61 m, intermediate layers that extend a minimum of 0.90 to 1.2 m into the backfill are recommended to prevent excessive bulging of the face between the layers. For convenience, an initial uniform spacing of 30 to 61 cm could be selected. Following the internal stability analysis, alternative spacings can easily be evaluated by analysing the reinforcement strength requirements at different wall levels and modifying the spacing accordingly or by changing the strength of the reinforcement to match the spacing requirements.

4.1.7.3. Reinforcement Length. Determination of the reinforcement length L is an iterative process, taking into account external stability and internal pullout resistance. For the first trial section to be analyzed, width of the reinforced soil wall and therefore the length of reinforcements are assumed by using the below criteria:

$$L = 0.5(H_0 + D) = 0.5H \geq 1.83m \quad (4.1)$$

Traditionally the minimum length of reinforcement is empirically limited to $0.7H$. On the other hand, current research indicates that walls on firm foundations which meet all external stability requirements can be safely constructed using lengths as short as $0.5H$.

4.1.7.4. Lateral Earth Pressures and Vertical Stresses. The lateral earth pressure at the back of the reinforced soil wall due to the retained fill increases linearly from the top as it is illustrated in Figure 4.9. For relatively stiff, inextensible reinforced systems, the lateral thrust (or pressure) has been found to be inclined downward relative to the horizontal by an inclination angle λ . The inclination of the lateral pressure has not been confirmed for extensible reinforcement. For a wall with a horizontal surface, the inclination angle λ of the earth pressure relative to the horizontal is taken as:

$$\lambda = \left(1.2 - \frac{L}{H} \right) \phi_b \quad (4.2)$$

when reinforcements are inextensible

$$\lambda = 0 \quad (4.3)$$

when reinforcements are extensible, then P_a , the thrust at the back of the wall, is equal to

$$P_a = 0.5 K_{ab(\lambda, \beta)} \gamma H^2 \quad (4.4)$$

plus any influence from any surcharge loads acting on the retained backfill, with

$$K_{ab} = \tan^2(45^\circ - \phi_b / 2) \quad (4.5)$$

if $\lambda = 0$ and $\beta = 0$, where K_{ab} is based on Coulomb's lateral earth pressure coefficient and it is a function of ϕ_b and λ , if $\lambda \neq 0$.

In case of a wall retaining an infinite slope inclined at the angle β , λ can be taken as

$$\lambda_b = \phi_b [1 - (1 - \beta / \phi_b)(L / H - 0.2)] \quad (4.6)$$

for inextensible reinforcement, and

$$\lambda_\beta = \beta \quad (4.7)$$

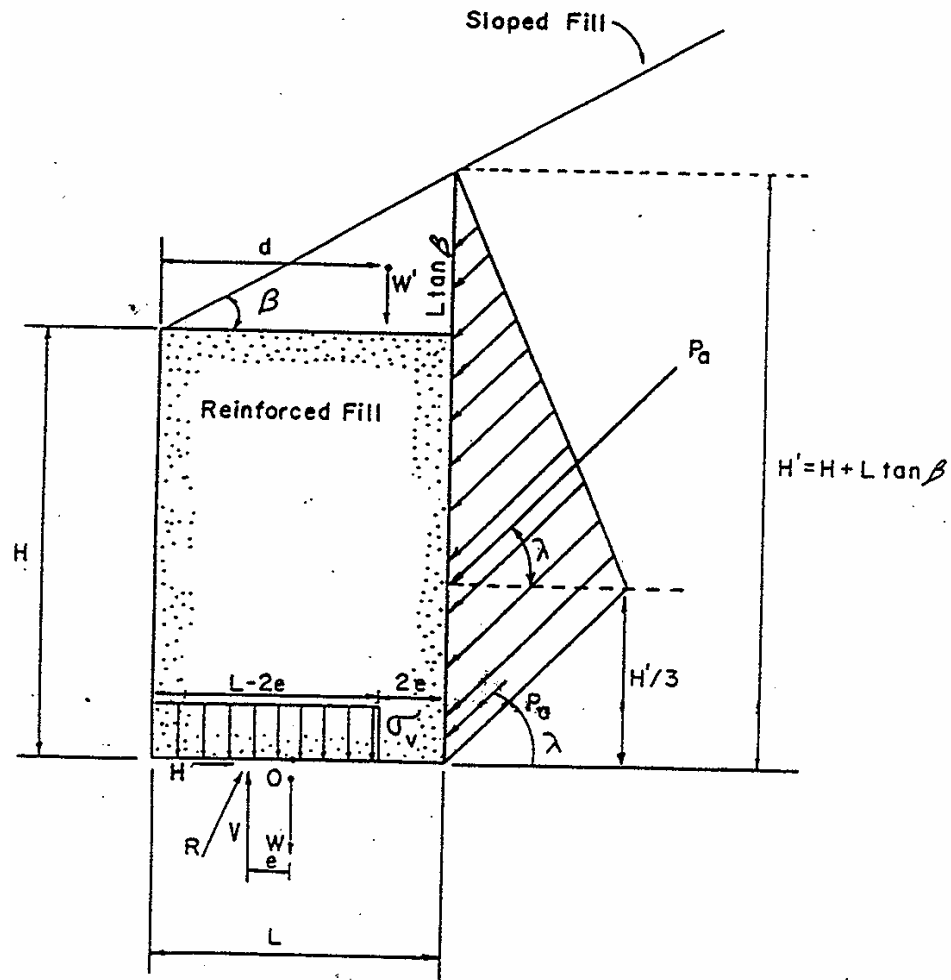


Figure 4.9 Magnitude of Vertical Stress with Increasing Depth

for extensible reinforcement and $K_a(\phi, \lambda, \beta)$ is the active earth pressure coefficient, calculated from the following equation:

$$K_a = \left[\frac{(\sin(\theta - \phi_b) / \sin \theta)}{(\sqrt{\sin(\theta + \lambda) + \sqrt{\sin(\phi_b + \lambda) \sin(\phi_b - \beta) / \sin(\theta - \beta)}})} \right]^2 \quad (4.8)$$

Figure 4.9 also shows the vertical stresses at the base of the wall defined by H' . It should be noted that the weight of any wall facing is neglected in the calculations. So, preliminary calculation steps can be summarised as:

(A) Determination of λ .

(B) Calculation of thrust behind the wall [16,31] as

$$P_a = 0.5K_{(\phi, \lambda, \beta)} \gamma_b H'^2 \quad (4.9)$$

(C) Calculation of eccentricity, e , of the resulting force on the base by considering moment equilibrium of the mass of the the reinforced soil selection as $\Sigma M_0=0$. V force in Figure 4.8 must equal the sum of the vertical forces on the reinforced fill and this condition yields to an eccentricity of

$$e = \frac{[P_a \cos \lambda H' / 3 - P_a \sin \lambda L / 2 - W'(d - L / 2)]}{[\gamma_r HL + W' + P_a \sin \lambda]} \quad (4.10)$$

(D) Calculation of the equivalent uniform vertical stress on the base, σ_v as

$$\sigma_v = \frac{(\gamma_r HLW' + P_a \sin \lambda)}{(L - 2e)} \quad (4.11)$$

and this approach, proposed originally by Meyerhof (Figure 4.10), assumes that eccentric loading results in a uniform redistribution of pressure over a reduced area at the base of the wall. This area is defined by a width equal to the wall width less twice the eccentricity as shown in Figure 4.9.

(E) Adding the influence of surcharge and concentrated loads to σ_v .

4.1.7.5. External Stability

(a) Sliding Along the Base:

For the considered system to be resistant sliding along the base it is required that:

$$FS_{\text{sliding}} = (\Sigma \text{ horizontal resisting forces}) / (\Sigma \text{ horizontal sliding forces}) \geq 1.5 \quad (4.12)$$

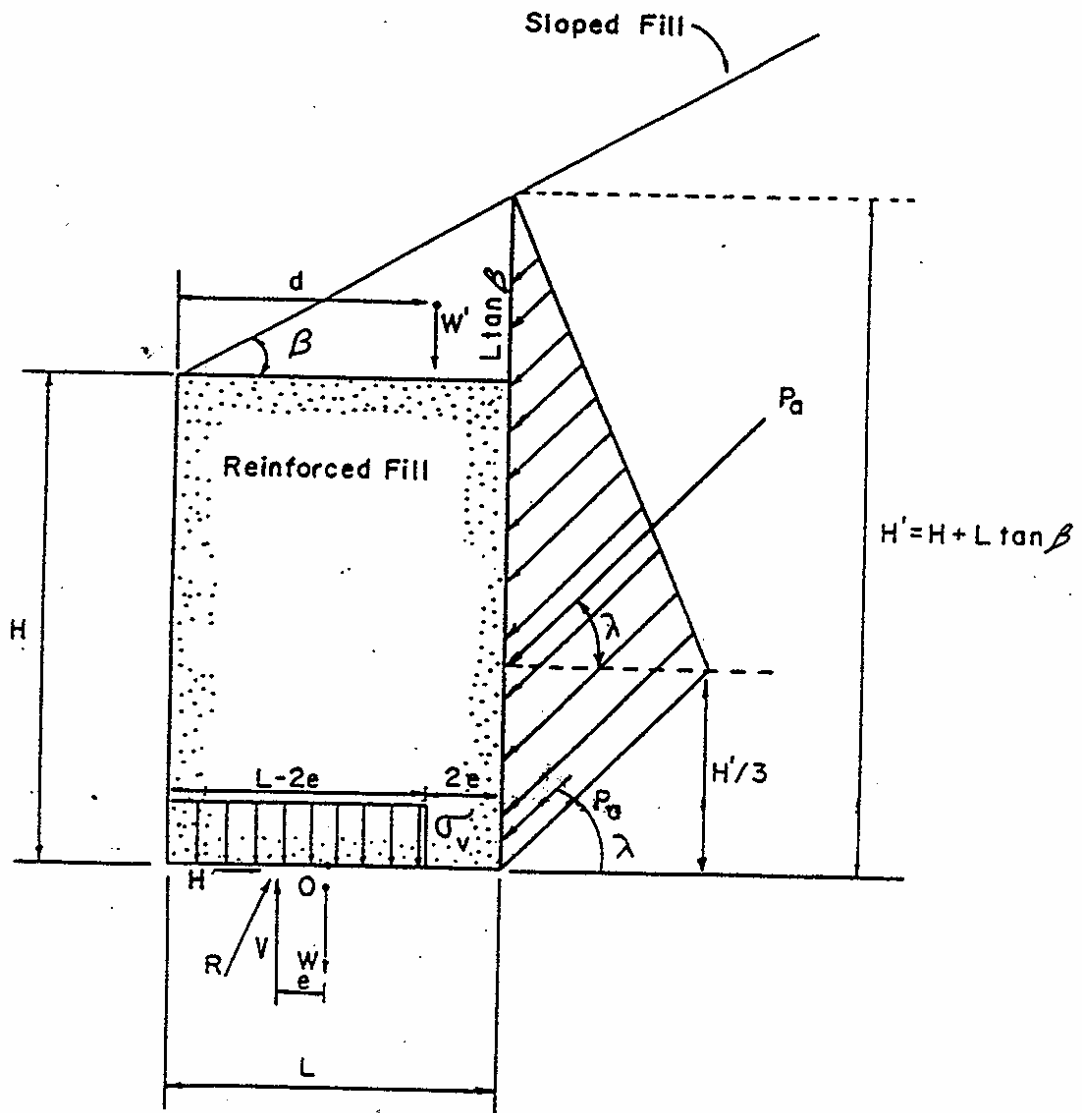


Figure 4.10 Meyerhof Formula and Pressure at the Base

Factor of safety against sliding for a reinforced soil wall with extensible reinforcement, $\lambda = 0$, retaining a horizontal backfill ($\beta = 0$), and supporting a uniform surcharge load can be calculated as:

$$FS_{\text{sliding}} = \frac{(V_q + W)\mu}{(P_b + P_q)} \quad (4.13)$$

In this formula, reaction at the base is calculated by the equation:

$$V_q = \gamma_s h_s L \quad (4.14)$$

and weight of the structure can be calculated as:

$$W = \gamma_r HL \quad (4.15)$$

and the effect of the permanent surcharge loads as:

$$P_q = K_{a,b} \gamma_s h_s H \quad (4.16)$$

and the effect of the backfill lateral thrust as:

$$P_b = 0.5 K_{a,b} \gamma_b H^2 \quad (4.17)$$

It should be noted that any passive resistance at the toe due to embedment is ignored due to the potential for that soil to be removed through natural or man made processes (e.g. erosion, utility installation, etc.). The shear strength of the facing system is also conservatively neglected. In addition, the resisting force is the lesser of the shear resistance along the base or of a weak layer near the base of the reinforced soil wall and the sliding force is the horizontal component of the thrust on the vertical plane at the back of the wall (Figure 4.11).

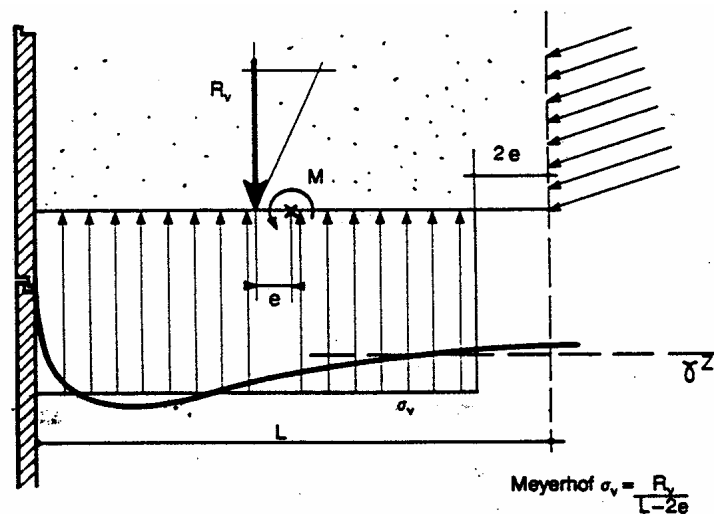


Figure 4.11 External Stability of a Reinforced Earth Wall

Calculation steps for a general type of Reinforced Earth wall with a sloping surcharge can be outlined as:

(A) Calculation of thrust behind the wall as:

$$P_a = K_{(\phi, \lambda, \beta)} (0.5 \gamma_b H'^2 + qH') \quad (4.18)$$

and active height as:

$$H' = H + L \tan \beta \quad (4.19)$$

(B) Calculation of active sliding force as:

$$P_b = P_a \cos \lambda_\beta \quad (4.20)$$

(C) Choice of the minimum ϕ for three possibilities of sliding modes of sliding along the foundation soil, if its shear strength (c, ϕ_f) is smaller than that of the backfill material, sliding along the reinforced backfill (ϕ_r) and for sheet type reinforcement, sliding along the weaker of the upper and lower soil-reinforcement interfaces (ρ). The soil-reinforcement friction angle ρ , should preferably be measured by means of interface direct shear tests. Alternatively, it might be assumed on the basis of $F^* \alpha$ values used for pullout resistance determinations.

(D) Calculation of the resisting force per unit length of wall as:

$$P_R = (W + V_q + P_a \sin \lambda_\beta) \mu \quad (4.21)$$

$$\mu = \min(\tan \phi_f, \tan \phi_r, \tan \rho) \quad (4.22)$$

and the effect of external loadings on the reinforced mass which increases sliding resistance should only be included if the loadings are permanent. For example, live load traffic surcharges should be excluded.

(E) Calculation of the factor of safety with respect to sliding and checking it if it is greater than the required value of $F.S.=1.5$. If $F.S. \geq 1.5$ considered system is decided to be safe against sliding failure. If not, reinforcement length, L can be increased, slope angle β , can be decreased, and calculations are repeated.

(b) Overturning:

Owing to the flexibility of reinforced soil structures, it is unlikely that a block overturning failure could occur. Nonetheless, an adequate factor of safety against this classical failure mode will limit excessive outward tilting and distortion of a suitably designed wall. Overturning stability is analysed by considering rotation of the wall about its toe. It is required that:

$$FS_{\text{overturning}} = \Sigma \text{resisting moments} / \Sigma \text{driving moments} \geq 2.0 \quad (4.23)$$

The resisting moments result from the weight of the reinforced fill, the vertical component of the thrust, and the surcharge applied on the reinforced fill (dead load only). The driving moments result from the horizontal component of the thrust exerted by the retained fill on the reinforced fill and the surcharge applied on the retained fill (dead load and live load).

Figure 4.12 illustrates the calculation of the external overturning stability of a reinforced soil wall with extensible reinforcement, $\lambda=0$, retaining a horizontal backfill with a uniform surcharge load. As in the case of sliding stability, the beneficial effect of embedment is neglected. Formula for the calculation of factor of safety against overturning of this wall can be given as

$$FS_{\text{overturning}} = \frac{(V_q + W)(L/2)}{(P_b(H/3) + P_q(H/2))} \quad (4.24)$$

and this calculated value should be checked that it is greater than the required value. If not, reinforcement length, L should be increased and eccentricity, e of the resulting force at the base of the wall should be calculated and checked that eccentricity does not exceed $L/6$. If $e > L/6$, reinforcement length should be increased.

(c) Bearing Capacity Failure:

To prevent bearing capacity failure, it is required that the vertical stress at the base calculated with the Meyerhof distribution does not exceed the allowable bearing capacity of the foundation soil, determined considering a safety factor of 2.0 with respect to the ultimate bearing capacity.

$$\sigma_v \leq q_{all} = \frac{q_{ult}}{FS_{bc}} = \frac{q_{ult}}{2} \quad (4.29)$$

For a reinforced earth wall with extensible reinforcement ($\lambda=0$) and retaining a horizontal backfill, factor of safety against bearing capacity failure can be calculated firstly by evaluating the eccentricity at the base of the wall as:

$$e = \frac{\left(P_b \frac{H}{3} + P_q \frac{H}{2} \right)}{(V_q + W)} \quad (4.30)$$

and finally calculating the equivalent vertical stress at the base and comparing this value with the allowable bearing capacity of the foundation soil as:

$$\sigma_{vmax} = \frac{(V_q + W)}{(L - 2e)} \quad (4.31)$$

$$q_{all} = \frac{q_{ult}}{2} \geq \frac{\sigma_{vmax}}{F.S.} \quad (4.32)$$

and Figure 4.13 illustrates the calculation of the bearing capacity of a wall with extensible reinforcement ($\lambda=0$) retaining a level backfill and supporting a uniform surcharge.

Calculation steps for a general type of reinforced earth wall ($\lambda \neq 0$) with a sloping backfill ($\beta \neq 0$) can be outlined as:

(A) Calculation of the eccentricity e of the resulting force at the base of the wall by using Eq. 4.10.

(B) Calculation of the vertical stress σ_v at the base assuming Meyerhof distribution.

$$\sigma_v = \frac{(V_q + W + P_a \sin \lambda)}{(L - 2e)} \quad (4.33)$$

(C) Determination of the ultimate bearing capacity q_{ult} using classical soil mechanics methods.

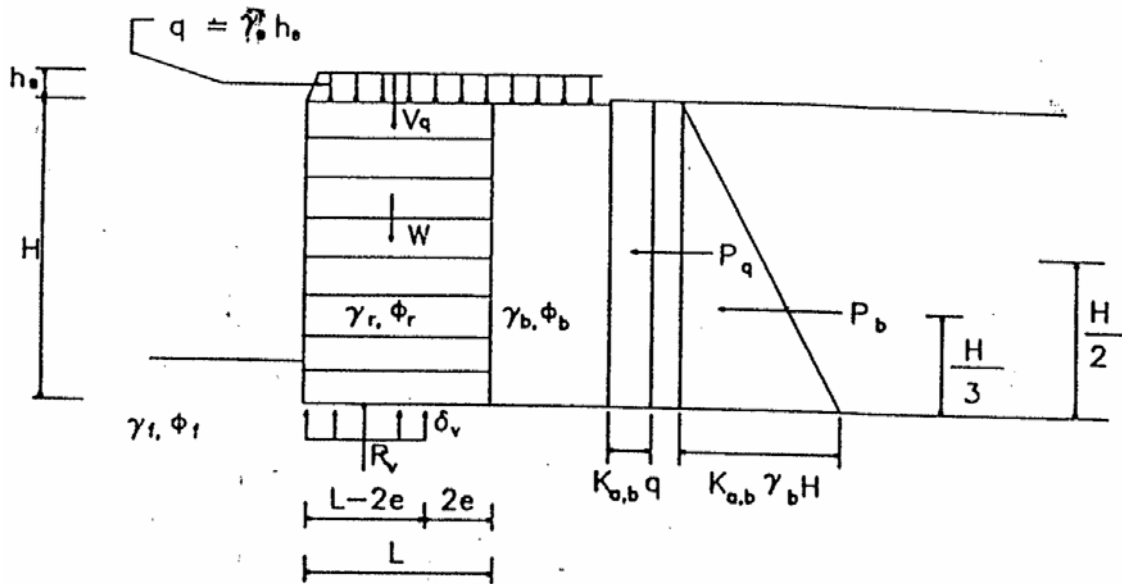


Figure 4.13 Bearing Capacity for External Stability of a Reinforced Earth Wall

$$q_{ult} = c_f N_c + 0.5(L - 2e)\gamma_f N_\gamma \quad (4.34)$$

in this formula N_c and N_γ are dimensionless bearing capacity coefficients and can be obtained from most soil mechanics textbooks.

(D) Checking that Eq. 4.32 holds. If not, σ_v can be decreased and q_{ult} increased by lengthening the reinforcements. If adequate support conditions can not be achieved or lengthening reinforcements significantly increases costs, precompression improvement of the foundation soil is needed dynamic compaction, soil replacement, jet grouting, stone columns etc.

(d) Overall Stability:

Overall stability is determined using rotational or wedge analysis, as appropriate, which can be performed using a classical slope stability analysis report. In the stability analysis calculations, the reinforced soil wall is considered as a rigid body and only failure surfaces completely outside a reinforced mass are considered. For simple structures with rectangular geometry, relatively uniform reinforcement, spacing and a near vertical face, compound failures passing both through the unreinforced and reinforced zones will not be generally be critical. After all analyses, if the minimum factor of safety is less than the required value, reinforcement length is increased or foundation soil is improved.

(e) Seismic Loading:

During an earthquake the retained backfill exerts a dynamic horizontal thrust, P_{AE} , on the reinforced soil wall, in addition to the static thrust. Besides, the reinforced soil mass is subjected to a horizontal inertia force

$$P_{IR} = Ma_m \quad (4.35)$$

where M is the mass of the reinforced wall section and a_m is the maximum horizontal acceleration in the reinforced soil wall. The reinforced soil mass is reduced to account for only the effective reinforcement length.

Dynamic horizontal force P_{AE} can be evaluated by the pseudo-static, Mononabe-Okabe analysis as shown in Figure 4.14 and added to the static forces acting on the wall (weight, surcharge and static thrust). After assigning the forces acting on the wall, the dynamic stability with respect to external stability is then evaluated. In practice, allowable minimum dynamic safety factors are assumed as 75 percent of the static safety factors.

The seismic external stability evaluation can be evaluated according to the following steps:

- (A) A peak horizontal ground acceleration αg is selected based on the design earthquake where α is the maximum ground acceleration coefficient.
- (B) The maximum acceleration a_m developed in the wall is determined according to the Eq. 6.36 where α_m is the maximum wall acceleration coefficient at centroid.

$$\alpha_m = (1.45 - \alpha)\alpha \quad (4.36)$$

$$a_m = \alpha_m g \quad (4.37)$$

- (C) Horizontal inertia force P_{IR} and seismic thrust P_{AE} are calculated as

$$P_{IR} = 0.5\alpha_m \gamma_r H^2 \quad (4.38)$$

$$P_{AE} = 0.375\alpha_m \gamma_b H^2 \quad (4.39)$$

those calculated forces are added to the static forces P_b , P_q acting on the structure as seismic thrust P_{AE} and 60 percent of P_{IR} . The reduced P_{IR} is used in the further calculations since these two forces are unlikely to peak simultaneously after the calculation of new acting forces.

for this study indicates that the maximum tensile force is primarily related to the stiffness of the reinforced soil mass which is controlled by the extensibility and density of reinforcement. Based on this research, a conservative relationship between the global reinforced soil stiffness and the horizontal stress was developed as shown in Figure 4.15. This figure was prepared by back analysis of the lateral stress ratio K from available field data.

Calculation steps can be outlined as for determining the tensile forces in reinforcements can be outlined as:

- (A) Calculation of concentrated surcharge loads $\Delta\sigma_v$ and $\Delta\sigma_h$. $\Delta\sigma_v$ is the increment of vertical stress due to concentrated vertical loads using a 2V:1H pyramidal distribution. This additional vertical stress increment can be calculated as

$$\Delta\sigma_v = \frac{P_v}{D} \quad (4.40)$$

for strip loading

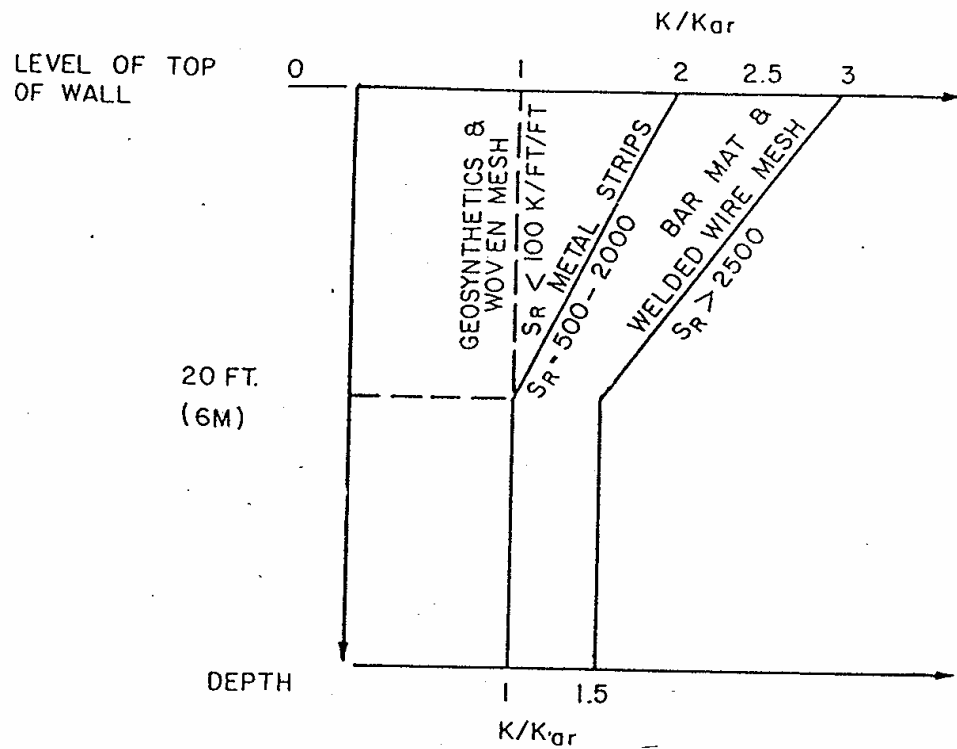


Figure 4.15 Variation of the Stress Ratio K with Depth in a Reinforced Earth Wall

$$\Delta\sigma_v = \frac{P_v'}{D(L+Z)} \quad (4.41)$$

for isolated footing load

$$\Delta\sigma_v = \frac{P_v'}{D^2} \quad (4.42)$$

for point load, where D is effective width of applied load with depth and L is the length of footing (Figure 4.16). In addition, $\Delta\sigma_h$ is the increment of horizontal stress due to horizontal concentrated surcharges as shown in Figure 4.17, if any, and calculated as

$$\Delta\sigma_h = 2F_H \left(\frac{z}{l_1} \right) \quad (4.43)$$

and l_1 in Eq. 4.43 can be determined from the equation below

$$l_1 = \frac{L_b}{\tan(45^\circ + \phi/2)} \quad (4.44)$$

where L_b is the effective width of the applied load at the point of application and F_H is the applied horizontal load.

- (B) Calculation of horizontal stresses σ_h along the potential failure line at each reinforcement layer from the weight of the retained fill plus, if present, uniform surcharge loads q .

$$\sigma_h = K(\gamma_r Z + q + \Delta\sigma_v) + \Delta\sigma_h \quad (4.45)$$

where $K = K(Z)$ is based on ϕ is shown in Figure 4.15. K is based on the stiffness of the reinforced section which is defined by the global reinforcement stiffness factor S_R .

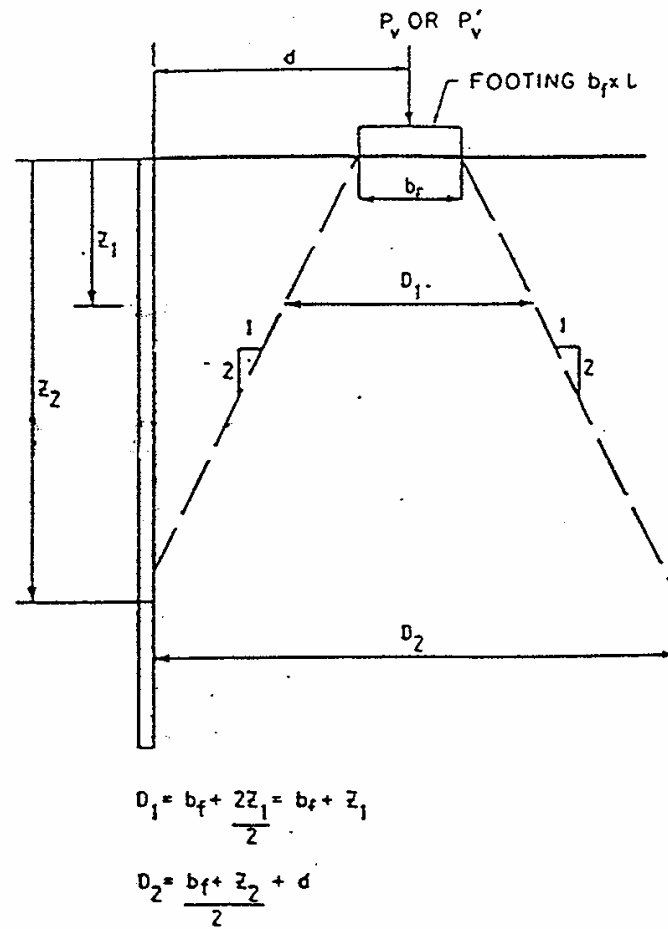


Figure 4.16 Schematic Illustration of Concentrated Load Dispersed for Vertical Loads

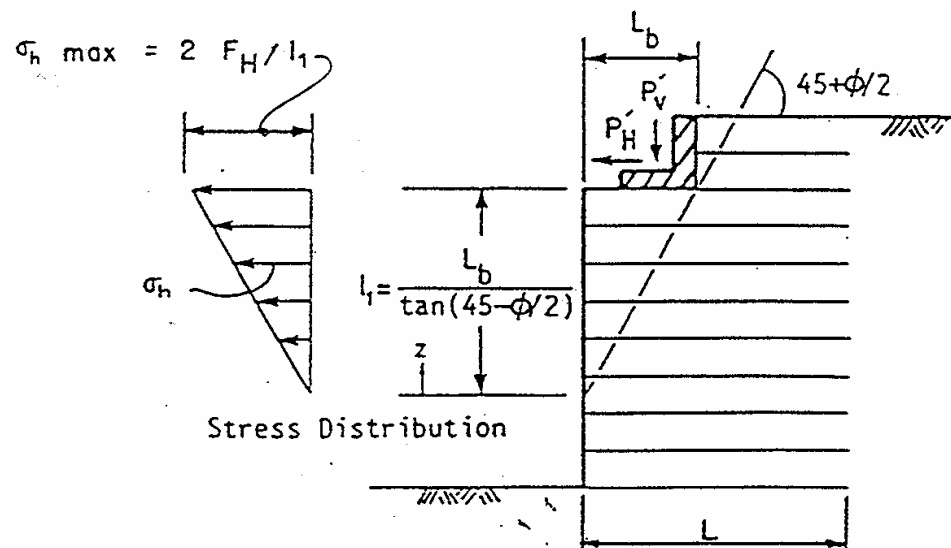


Figure 4.17 Schematic Illustration of Concentrated Load Dispersed for Horizontal Loads

$$S_R = \frac{(EA')}{\left(\frac{H}{n}\right)} \quad (4.46)$$

where A' is the average area of the reinforcement per unit width of wall which can be calculated as

$$A' = \frac{(bt)}{S_h} = R_c t \quad (4.47)$$

where R_c is defined as coverage ratio. In Figure 4.15, K_{ar} is the active lateral earth pressure coefficient in the reinforced soil wall which is expressed as

$$K_{ar} = \tan^2(45^\circ - \phi_r / 2) \quad (4.48)$$

for walls with horizontal surface and

$$K_{ar} = \cos \beta \left[\frac{\left(\cos \beta - \sqrt{(\cos^2 \beta - \cos^2 \phi_r)} \right)}{\left(\cos \beta + \sqrt{(\cos^2 \beta - \cos^2 \phi_r)} \right)} \right] \quad (4.49)$$

for sloped surface at angle β . For sloping soil surfaces above the reinforced soil wall section, either the actual surcharge can be replaced by a uniform surcharge equal to

$$q = 0.5 \gamma h_s \quad (4.50)$$

where h_s is the height of the slope at the back of the wall or by calculation of K based on the slope angle β . For preliminary calculations when the reinforcement type is unknown, $S_R=1,000$ k/ft/ft can be assumed for inextensible reinforcement and $S_R=50$ k/ft/ft can be assumed for extensible reinforcement. The final design should always be checked based on the stiffness factor for the actual reinforcement and spacing to be used.

(C) Calculation of the maximum tension $T_{\max}$ per unit length along the wall in each reinforcement layer is calculated by using the equation below

$$T_{\max} = S_v \sigma_h \quad (4.51)$$

and calculation of $T_{\max}$ allows the determination of reinforcement size at each number n of discrete reinforcements (metal strips, bar mats, geogrids, etc.) per unit width of wall face or the tensile capacity required of sheet type reinforcement (welded wire mesh, geosynthetic) to be used.

(b) Internal Stability with respect to Breakage:

Stability with respect to breakage of the reinforcements requires that:

$$T_{\max} \leq T_a R_c \quad (4.52)$$

where T_a is the allowable tension force per unit width of the reinforcement and R_c is the coverage ratio defined as:

$$R_c = \frac{b}{S_h} \quad (4.53)$$

where b is the gross width of the reinforcing element, and S_H is the center-to-center horizontal spacing between reinforcements.

At the connection of the reinforcements with the facing, it should also be checked that the tensile force T_0 determined as indicated in Figure 4.18 is not greater than the allowable tensile strength of the connection.

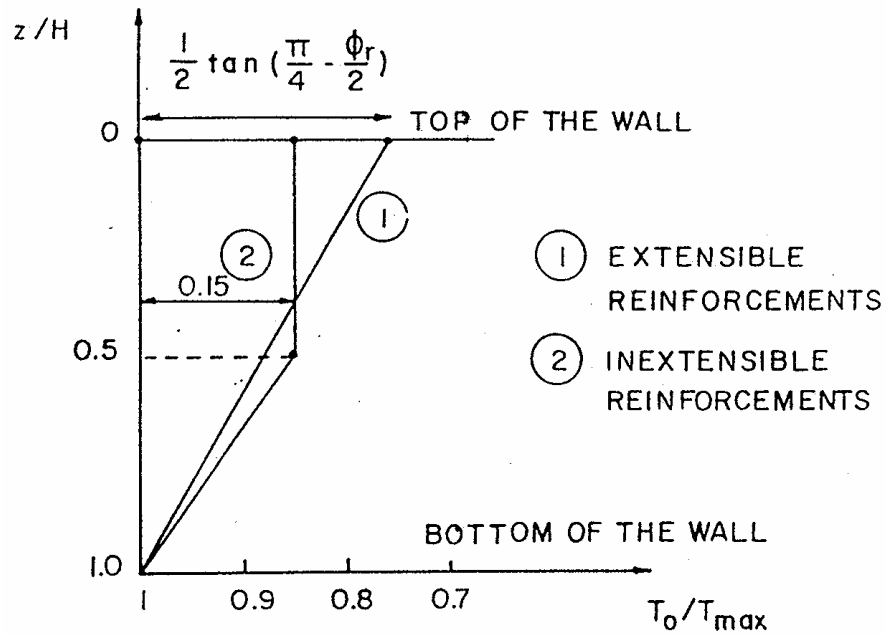


Figure 4.18 Determination of the Tensile Force T_0 in the Reinforcements at the Connection with the Facing

(c) Internal Stability with respect to Pullout Failure:

Stability with respect to pullout of the reinforcements requires that the following criteria be satisfied:

$$T_{\max} \leq \left(\frac{1}{FS_{PO}} \right) P_r R_c \quad (4.54)$$

$$T_{\max} \leq \left(\frac{1}{FS_{PO}} \right) F^* \alpha \gamma_r z' L_e C R_c \quad (4.55)$$

where FS_{PO} is the safety factor against pullout, P_r is the available pullout resistance for a particular type of reinforcement, $C = 2$ for strip, grid, and sheet type reinforcement and π for circular bar reinforcements, F^* is the pullout resistance factor, α is the scale effect correction factor, $\gamma_r z'$ is the overburden pressure, including distributed surcharges and L_e is the length of embedment in the resisting zone. Therefore the required embedment length in the resisting zone can be determined as:

$$L_e \geq \left(\frac{1.5}{T_{\max}} \right) F^* \alpha \gamma_r z' L_e C R_c \quad (4.56)$$

If this criterion is not satisfied for all reinforcement layers, the reinforcement length has to be increased and/or reinforcement with a greater pullout resistance per unit width must be used.

In the case of a reinforced soil wall with a sloping surcharge, the overburden pressure varies from the distance from the face, and the maximum pullout resistance, P_r , can be calculated according to the equation below:

$$P_r = \int_{L-L_e}^L [CF^* \gamma_r z'(x) dx] \quad (4.57)$$

and solution of this Eq. 4.57 gives:

$$P_r = CF^* \gamma_r Z_{ave} L_e \quad (4.58)$$

in which Z_{ave} is the distance from the ground surface to the midpoint of the bar in the resisting zone. The total length of reinforcement, L_t , required for internal stability is then determined as:

$$L_t = L_a + L_e \quad (4.59)$$

In this formula, L_a is obtained from Figure 4.3 for simple structures not supporting concentrated external loads such as bridge abutments. For the total height of a reinforced soil wall with extensible reinforcement.

$$L_a = (H - Z) \tan(45^\circ - \phi'_r / 2) \quad (4.60)$$

In this equation Z is defined as depth to the considered reinforcement level. For a wall with inextensible reinforcement from the base up to $H/2$, L_a can be taken as:

$$L_a = 0.6(H - Z) \quad (4.61)$$

and for the upper half of the wall:

$$L_a = 0.3H \quad (4.62)$$

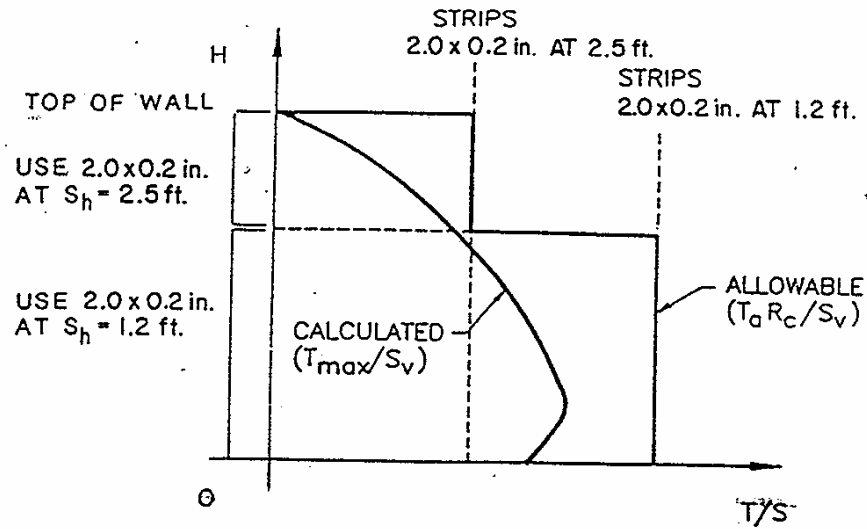
In practice, the majority of reinforced soil walls constructed to date have used $0.7H$ as a minimum reinforcement length requirement. Research including monitoring of structures has indicated that shorter lengths can be used provided internal and external stability and for those cases a through evaluation of the fill, backfill, and foundation properties be performed prior to acceptance of the design.

(d) Strength and Spacing Variations:

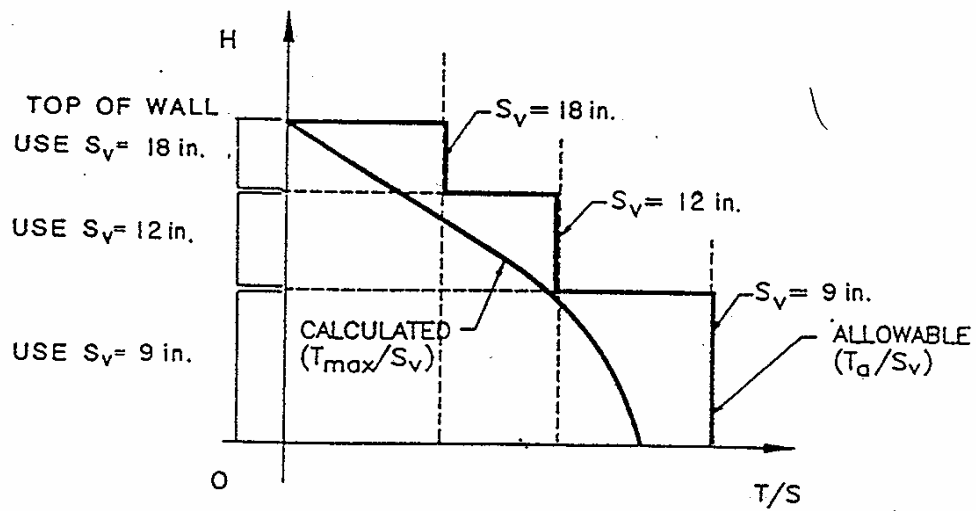
Use of a constant reinforcement density and spacing for the full height of the wall usually gives more reinforcement near the top of the wall than is required for the stability. Therefore a more economical design may be possible by varying the reinforcement density with depth.

In the case of reinforcements consisting of strips, grids or mats, which are used with precast concrete facing panels, the vertical spacing is maintained constant and the reinforcement density is increased with depth by increasing the number and/or the size of the reinforcements. The spacing plots in Figure 4.19, provide a simple method to visualise the effects of changing reinforcement density. The vertical axis represents elevation within the wall and the horizontal axis can be thought of as horizontal stress to be restrained by the reinforcement. On the other hand, vertical lines on the plot represent maximum horizontal stresses permitted to be carried by a specific reinforcement density.

Finally, it is obvious that the reinforcement density is a function of cross-sectional area of reinforcing elements, allowable reinforcement material stress, and horizontal and vertical reinforcement spacing.



A) LINEAR REINFORCEMENT DENSITY VERSUS HEIGHT:
INEXTENSIBLE REINFORCEMENT



B) PLANAR REINFORCEMENT DENSITY VERSUS HEIGHT:
EXTENSIBLE REINFORCEMENT

Figure 4.19 Examples of Determination of Equal Reinforcement Density Zones

(e) Internal Stability with Respect to Seismic Loading:

A seismic loading induces an internal inertial force P_I acting horizontally on the active zone in addition to the existing static forces. This force will lead to incremental dynamic increase in the maximum tensile forces in the reinforcements. It is assumed that the location and slope of the maximum tensile force line does not change during the seismic loading. This assumption is conservative especially relative to reinforcement rupture and considered acceptable relative to pullout resistance.

Calculation steps for internal stability analyses with respect to seismic loading can be summarised in steps as:

(A) Maximum acceleration $\alpha_m g$ in the wall and force $P_I(z)$ acting on the reinforced soil mass above level z is calculated by the equations below:

$$\alpha_m = (1.45 - \alpha)\alpha \quad (4.63)$$

$$P_I(z) = M(z)\alpha_m g \quad (4.64)$$

Total horizontal stress in the reinforced fill and consequently the first component T_{m1} of the maximum tensile force is calculated firstly by determining horizontal stress σ_h which is calculated using K coefficient:

$$\sigma_h = K\sigma_v + \Delta\sigma_h = K(\gamma_r H + q + \Delta\sigma_v) + \Delta\sigma_h \quad (4.65)$$

and finally maximum tensile force component T_{m1} (Figure 4.20 and Figure 4.21) is calculated in each reinforcement as below:

$$T_{m1} = S_v \sigma_h \quad (4.66)$$

(B) The dynamic increment T_{m2} (Figure 4.20 and Figure 4.21) directly induced by the inertia force P_I in the reinforcements is calculated. This is accomplished by distributing P_I in the different reinforcements proportionally to their “resistant area”.

$$T_{m2} = \frac{P_I(R_c L_e)i}{\left(\sum (R_c L_e)i\right)} \quad (4.67)$$

which is the resistant area of the reinforcement at level i divided by the sum of the resistant area for all reinforcement levels. Finally, total maximum tensile force becomes:

$$T_{\max} = T_{m1} + T_{m2} \quad (4.68)$$

For checking the stability with respect to breakage and to pullout of the reinforcement, seismic safety factors of only 75 percent of the minimum allowable static safety factors values are used. This leads to:

$$T_{\max} \leq \frac{1}{0.75} T_a R_c \quad (4.69)$$

for breakage failure.

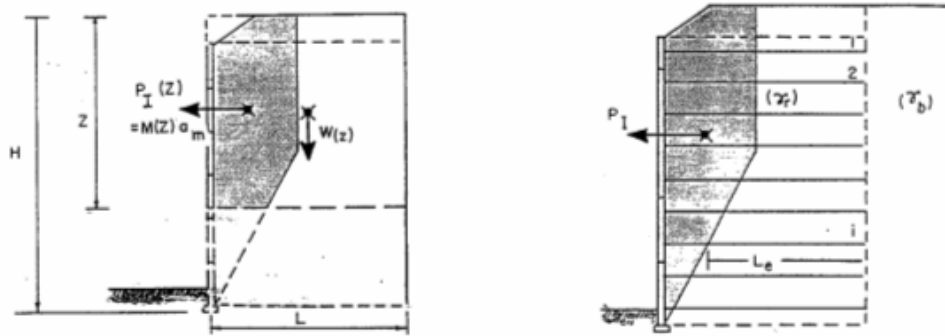


Figure 4.20 Internal Seismic Stability of a Reinforced Earth Wall

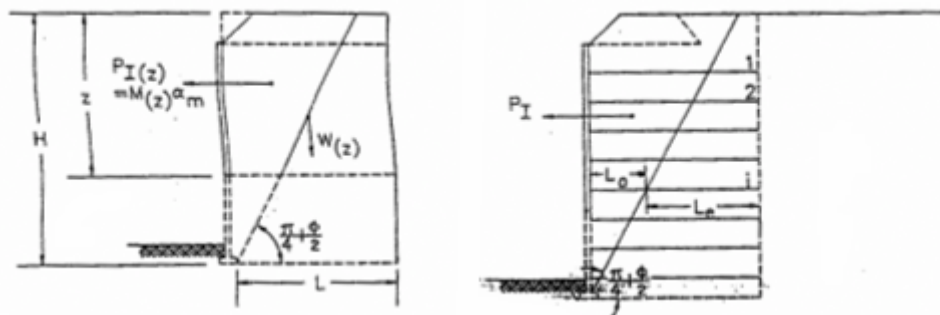


Figure 4.21 Internal Seismic Stability of a Reinforced Earth Wall

$$T_{\max} \leq \frac{P_r R_c}{0.75 F S_{PO}} \quad (4.70)$$

$$T_{\max} \leq \frac{2F_d^* \alpha}{1.125} \gamma z' L_e R_c \quad (4.71)$$

for pullout failure and

$$F_d^* (\text{dynamic}) = 0.8 F^* \quad (4.72)$$

This design method with respect to seismic loading was developed for inextensible strip reinforcements. The extensibility of the reinforcements affects the overall stiffness of the reinforced soil mass. As extensible reinforcement reduced the overall stiffness, it is expected to have an influence on the design diagram of the lateral earth pressure induced by the seismic loading. As the overall stiffness decreases, damping should increase. Therefore, the inextensible reinforcement analysis should be conservative for extensible reinforcement.

4.1.8. Settlements

External stability design of reinforced earth walls also requires a calculation of the anticipated total and differential settlements caused by the superimposed loads of the structure. The deformability of the reinforced soil material is relatively large. Therefore the admissible settlements of those structures are limited only by the longitudinal deformability of the facing and by the purpose of the structure (retaining of an embankment, supporting a road or a concentrated load). Two cases for the considered settlements should be distinguished.

- (A) If important differential settlements develop during the construction (for example in the case of construction by successive stages on compressible soils) the joints at the upper part of the wall risk to be either closed or too opened.
- (B) Important total longitudinal differential settlements (about 5 percent) can cause disorders in the facing (cracks or breakages of the panels or cracks in the metallic

elements), which can eventually be detrimental to the long-term behaviour of the structures because of the local outflow of the backfill material.

The admissible differential settlement for the standard facing is function of the height of the wall. A structure of 15m height can sustain without any damage a longitudinal differential settlement of:

- 1 percent for the facing with concrete panels,
- 2 percent for the metallic facing.

Depending on the amount of settlement and the time required for it to occur, there are several methods of adapting a reinforced soil retaining wall to the site conditions or of accelerating the consolidation of the site, should be required. Rapid settlements can generally be accommodated during the construction process by final adjustments to the dimensions of the top course of facing panels. Slower settlements, if large in magnitude, must often be accelerated to allow for corrective measures during construction. Methods for accomplishing this include temporary surcharging of the structure foundation, installing vertical drains, and other traditional methods (Figure 4.22).

When differential settlement along the facing is expected to exceed the one-to-two per cent which can be accepted by the facing system without risk of damage, the wall face can be given additional “degrees of freedom” by installing vertical slip joints.

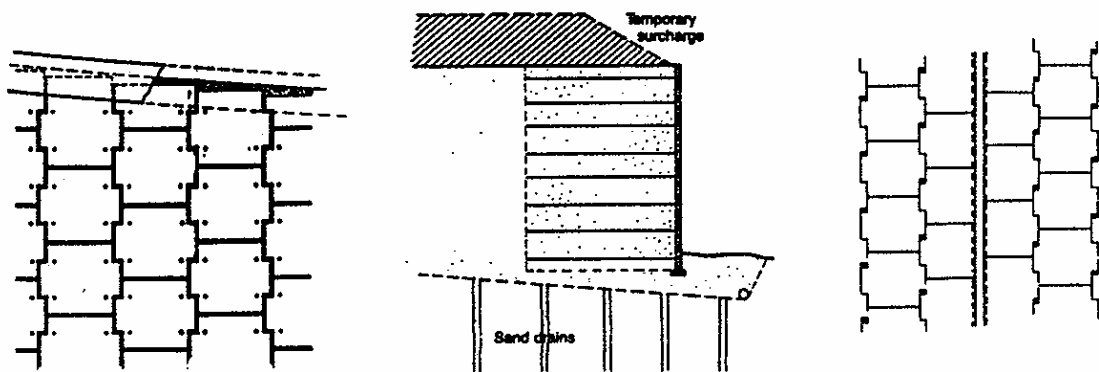


Figure 4.22 Adaptation to Settlements, Coping, Preloading, Vertical Joints

4.1.9. Soil Improvement

With respect to failure of the foundation soil the reinforced earth structure behaves like an earth embankment. Whenever the foundation soil is very poor, it is probably necessary to improve its bearing capacity (Figure 4.23). The techniques of construction and improvement must be studied and defined with the soil engineer. Among these techniques the following ones can be considered:

- substitution of the poor layers by an adequate soil eventually treated or stabilised,
- preloading,
- dynamic consolidation,
- setting stone columns,
- compaction grouting (jet grouting),
- execution vertical drains to accelerate the consolidation.

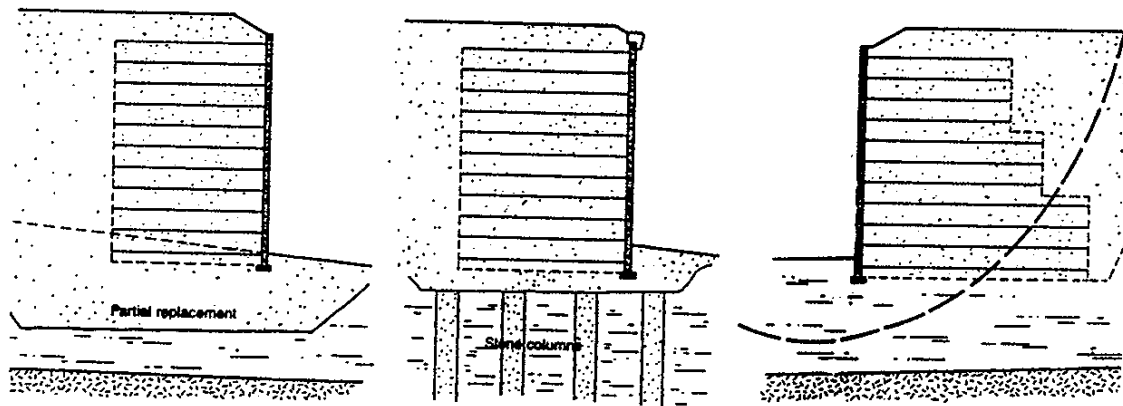


Figure 4.23 Improvement of the Foundation System

4.2. Principles of The Numerical Methods Used

4.2.1. Assumptions in Limit State Calculations

In practice the design of Reinforced Earth walls are done with limit state calculations which proved to be safe over 30 years of time.

One basic assumption is made in the soil reinforcement interaction by a certain friction coefficient is applied to the overburden pressure in order to calculate the pull out resistance of the soil.

Another assumption is made in determining the active zone of the soil, as mentioned earlier depending on the experimental studies researchers provided valuable documentation on the active zone and resisting zone boundaries for both inextensible and extensible reinforcements. In Figure 4.24 a sample illustration can be seen.

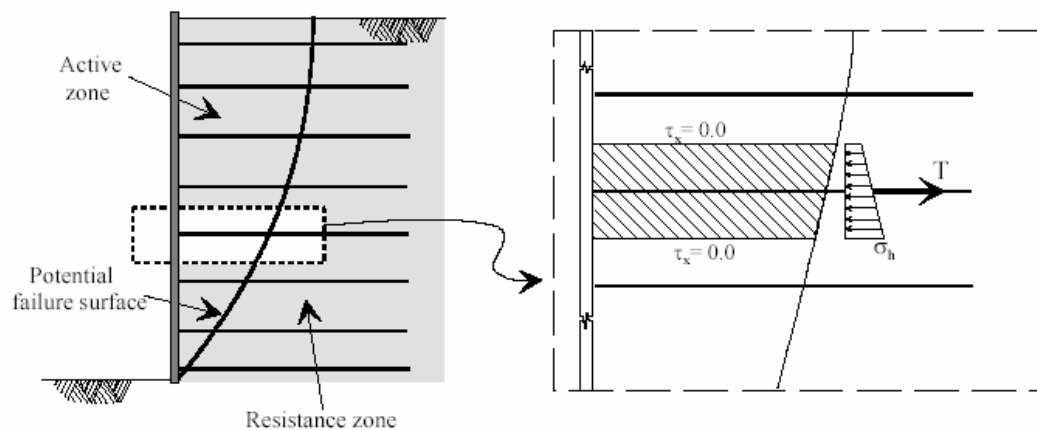


Figure 4.24 Active and Resisting Zones in Reinforced Earth Structures

The most important aspect that the limit state designs do not calculate is the deformation of the structure as a whole and the parts of it ie. reinforcements. Since full scale tests have already been done. Limits of the usual structures are known. But with the changing loading conditions, subsoil conditions, with increasing demand for reinforced earth structures to carry higher loads than usual and to achieve more heights than usual a tool for simulating the structures economically is needed.

4.2.2. Introduction to VALDEZ Software

VALDEZ is the software used to design Reinforced Earth walls that is produced by Reinforced Earth Company. For the case to be discussed the results of the VALDEZ design will be tested in the numerical analysis and the deformations of the structure will be checked.

The presentation of the VALDEZ software is given in Appendix A.

4.2.3. Finite Element Method

Finite Element Method (FEM) was first developed in 1943 by R. Courant, who utilized the Ritz method of numerical analysis and minimization of variational calculus to obtain approximate solutions to vibration systems. Shortly thereafter, a paper published in 1956 by M. J. Turner, R. W. Clough, H. C. Martin, and L. J. Topp established a broader definition of numerical analysis. The paper centered on the "stiffness and deflection of complex structures".

By the early 70's, FEM was limited to expensive mainframe computers generally owned by the aeronautics, automotive, defense, and nuclear industries. Since the rapid decline in the cost of computers and the phenomenal increase in computing power, FEM has been developed to an incredible precision. Present day supercomputers are now able to produce accurate results for all kinds of parameters.

The development of the finite element method in engineering mechanics is often based on an energy principle, e.g., the virtual work principle or the minimum total potential energy principle, which provides a general, intuitive and physical basis that has a great appeal to engineers.

Mathematically, the finite element method (FEM) is used for finding approximate solution of partial differential equations (PDE) as well as of integral equations such as the heat transport equation. The solution approach is based either on eliminating the differential equation completely (steady state problems), or rendering the PDE into an

equivalent ordinary differential equation, which is then solved using standard techniques such as finite differences, etc.

In solving partial differential equations, the primary challenge is to create an equation which approximates the equation to be studied, but which is numerically stable, meaning that errors in the input data and intermediate calculations do not accumulate and cause the resulting output to be meaningless. There are many ways of doing this, all with advantages and disadvantages. The Finite Element Method is a good choice for solving partial differential equations over complex domains (like cars and oil pipelines) or when the desired precision varies over the entire domain. For instance, in simulating the weather pattern on Earth, it is more important to have accurate predictions over land than over the wide-open sea, a demand that is achievable using the finite element method.

Finite element method relies on the fact that, the total potential energy of an elastic body is defined as the sum of total strain energy and the work potential. Which can be illustrated as:

$$\pi = u + v \quad (4.73)$$

and if;

$$\frac{\delta U}{\delta d} + \frac{\delta W}{\delta d} = 0 \quad (4.74)$$

the system is in equilibrium.

The total strain energy is:

$$U = \int \frac{1}{2} \{\varepsilon\}^T \{\sigma\} dV \quad (4.75)$$

where, $\{\varepsilon\}$ is strain vector and $\{\sigma\}$ is stress vector.

Assuming that a load P is directed along the x axis in a 2 dimensional case, we have;

$$\sigma_x = P/A \quad (4.76)$$

where, A is the cross sectional area and from the Hooke's law:

$$\varepsilon_x = \sigma_x / E \quad (4.77)$$

Assuming the material to be isotropic we have:

$$\{\varepsilon\} = \begin{bmatrix} 1/E & \mu/E & 0 \\ \mu/E & 1/E & 0 \\ 0 & 0 & 1/G \end{bmatrix} \{\sigma\} \quad (4.78)$$

and knowing the basic equation of elasticity:

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} = \frac{E}{1-\mu^2} \begin{bmatrix} 1 & \mu & 0 \\ \mu & 1 & 0 \\ 0 & 0 & \frac{1-\mu}{2} \end{bmatrix} \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} \quad (4.79)$$

The general stress equation can be formed as:

$$\{\sigma\} = [D](\{\varepsilon\}_{tot} - \{\varepsilon\}_0) + \{\sigma\}_0 - \alpha \Delta T \{D\}_T \quad (4.80)$$

in this general equation the initial stress values and heat changes are taken into account.

The most important assumption in the finite element analyses are the shape functions, that is, in every small finite element strains are considered to be changed by this function. Since small element sizes are chosen to analyze the whole model, the error resulting from this assumption is ignorable.

In a usual rectangular finite element in 2 dimensions; there are 4 nodes and 8 possible displacement directions. Knowing this for the whole element 2 displacements can be written as:

$$\begin{aligned} u &= N_1 d_1 + N_2 d_2 + N_3 d_3 + N_4 d_4 + N_5 d_5 + N_6 d_6 + N_7 d_7 + N_8 d_8 \\ v &= N'_1 d_1 + N'_2 d_2 + N'_3 d_3 + N'_4 d_4 + N'_5 d_5 + N'_6 d_6 + N'_7 d_7 + N'_8 d_8 \end{aligned} \quad (4.81)$$

Applying the shape functions final equation forms:

$$[m]\{\ddot{d}\} + [c]\{\dot{d}\} + [k]\{d\} = \{P\} - [\{f\}_x + \{f\}_s + \{f\}_{\varepsilon_0} + \{f\}_{\sigma_0} + \{f\}_T] + \{P\}_{known} \quad (4.82)$$

Equation 4.82 is the most general form of the finite element equations in which, the energy changes due to acceleration, damping, stiffness and direct external loads, body forces, boundary loads, initial strains, initial stresses, stresses formed due to temperature change and forces due to known displacements.

4.2.4. Introduction to PLAXIS Software

PLAXIS is a finite element package intended for the two dimensional analysis of deformation and stability in geotechnical engineering. Geotechnical applications require advanced constitutive models for the simulation of the non-linear, time dependent and anisotropic behavior of soils and/or rock. In addition since soil is a multi face material, special procedures are required to deal with hydrostatic and non-hydrostatic pore pressures in the soil. Although the modeling of the soil is an important issue, many projects require modeling structures and interaction of the structures with soil.

Strip reinforced soil structures may be cost-effective alternatives for all applications where reinforced concrete or gravity type walls have traditionally been used to retain soil. Retaining structures are used not only for bridge abutments and wing walls but also for slope stabilization and to minimize right of way for embankments. For many years, retaining structures were almost exclusively made of reinforced concrete and were designed as gravity or cantilever walls which are essentially rigid structures and cannot accommodate significant differential settlements unless founded on deep foundations. With increasing height of soil to be retained and poor subsoil conditions, the cost of reinforced concrete retaining walls increases rapidly.

4.2.5. Comparison of the Used Methods

VALDEZ is a limit state program that utilizes both load factors and material factors on the different parts of the calculation. In addition the program is dividing the calculated safety factors by the required safety factors so that the design engineer does not have to keep in mind all the safety factors for different conditions (i.e. sliding or overturning). In this manner it is different from the numerical analysis, in this kind of analysis it is not possible to analyze the structure for overturning or sliding part by part. Instead these methods analyses the structure as a whole and gives the deformation values as output. The design engineer should check if the deformations are in the applicable limits or not. Because of the general demand for a factor of safety in designs, different ways of interpreting a safety factor are considered in the numerical analysis methods. This is done by reducing the ϕ (internal friction angle) and c (cohesion) values given in the input for soils up to failure or increasing the specific density or load up to failure. Then the program gives the ratio as a safety factor, but especially in specific designs like reinforced earth walls, it should be clear that this numerical factor of safety is not the counterpart of the any safety factors calculated in limit state design but it is for giving an idea about the whole structure.

5. CASE STUDY DAVUTPAŞA INTERCHANGE REINFORCED EARTH WALLS

5.1. Introduction

In the content of the Construction of Davutpaşa Interchange and Connection Roads, Reinforced Earth Walls (Figure 5.1) were designed and built by Reinforced Earth Company, Turkey.

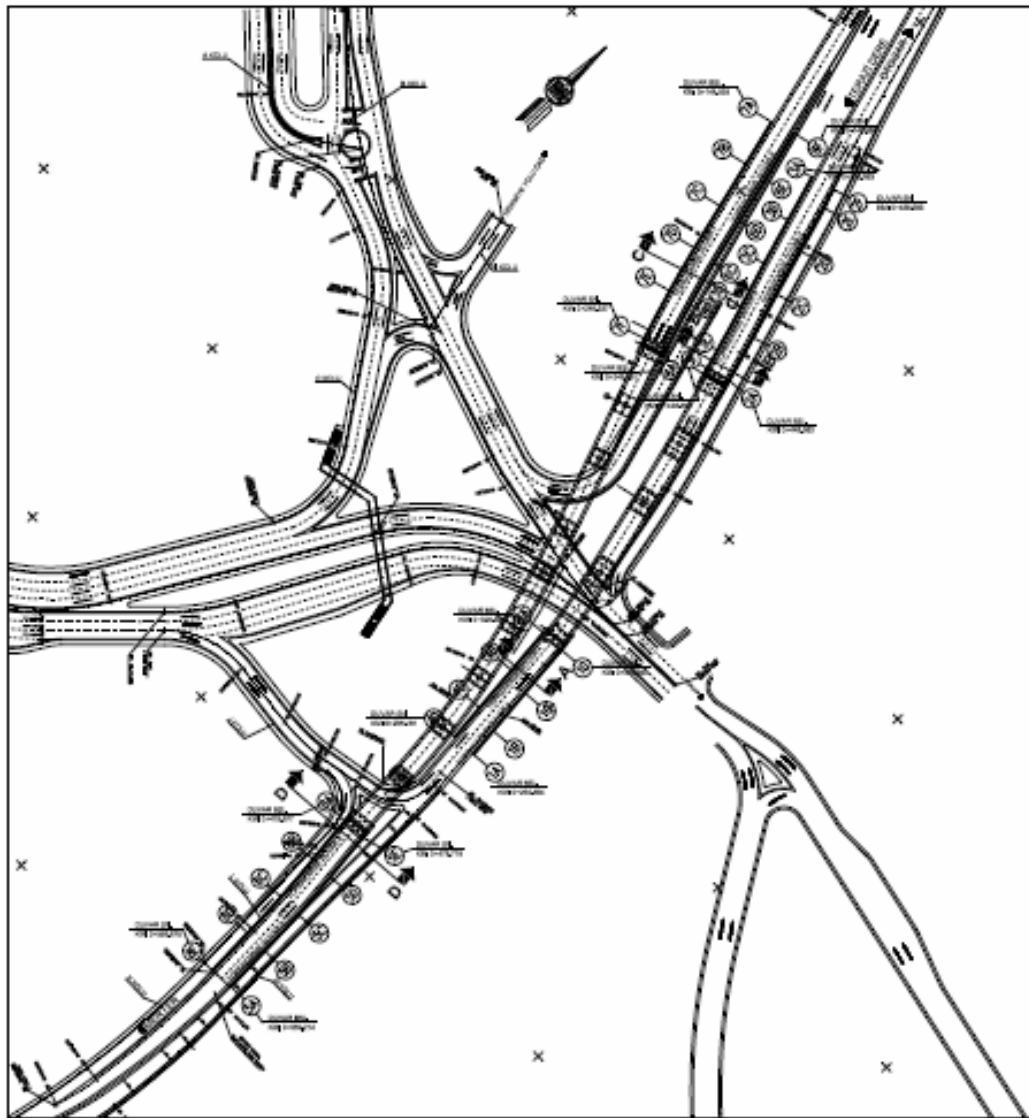


Figure 5.1 Plan View of the Davutpaşa Interchange

5.2. Subsoil Investigations

In the region there was a borehole to investigate the subsoil, in the borehole (Figure 7.2) SPT values found were quite high ($N > 50$) after 8.00m and the average SPT values are about ($N = 35$) in between 1.50-4.50m. Regarding these values the soil parameters for the soil layers have been assigned. Mohr Coulomb model is used to model the soil layers.

To apply numerical methods to soil layers some additional parameters are required to be able make additional calculations. These parameters are not defined for limit state calculations. This parameters were “Young’s Modulus” (E), and “Poisson’s Ratio” (ν), “Shear Modulus” (G), and “Bulk Modulus” (K). Obviously knowing two of the four parameters will be enough because of the formulas given:

$$G = \frac{E}{2(1 + 2\nu)} \quad (5.1)$$

$$K = \frac{E}{3(1 - 2\nu)} \quad (5.2)$$

The estimated parameters for the soil layers are given in Table 5.1, these parameters are deduced from the SPT N values, in accord to the study of Kulhawn and Mayne, 1990.

Table 5.1 Estimated Soil Properties

Soil type	ϕ	c (kPa)	E (MPa)	ν
Middle stiff, occasionally gravelly, clayey sand	30	-	80	0.3
Middle stiff occasionally blocky rare gravelly silty sand	30	-	50	0.3
Reinforced Earth Fill	36	-	60	0.35

ISVEREN CLIENT İBB		SONDAJ LOGU / BOREHOLE LOG										SONDAJ BOREHOLE		SK-1					
SAYFA/PAGES : 1 / 1												SONDOR DRILLER		B. TORUN					
PROJE ADI/PROJECT NAME : DAVUTPAŞA KAVŞAĞI ve BAĞLANTI YOLLARI İNŞAATI																			
SONDAJ YERİ/BORING LOCATION : İSTANBUL																			
KILOMETRE/KILOMETER :						MUH. BOR. DER./CASING DEPTH (m) :													
SONDAJ DERİNLİĞİ/BOREHOLE DEPTH (m) : 20.00						BAS. BIT. TARİHİ/START-FINISH DATE: 25/10/2005 - 28/10/2005													
SONDAJ KOTU/BOREHOLE ELEVATION (m) : 51.10						KOORDİNAT/COORDINATE (N-S) y:													
YERALTI SUYU/GROUNDWATER (m) :						KOORDİNAT/COORDINATE (E-W) x:													
Sondaj Derinliği Boring Depth (m)	Numune Cinsi Sample Type	Manevra Boyu Run (m)	Standart Penetrasyon Deneyi Standard Penetration Test				JEOTEKNİK TANIMLAMA GEOTECHNICAL DESCRIPTION					Profil / Profile	Dayanım / Strength	Ayrışma / Weathering	SCR, %	Karet % / Core Recovery	ROD, %	Sirkülasyon Su Kaybı Circulation Water Loss, %	
			DARBE SAYISI No. of BLOWS		GRAFIK GRAPH														
			0-15 cm	15-30 cm	30-45 cm	N	10	20	30	40	50								
1	SPT-1	1.50	10	17	18	35													
2		1.95																	
3	SPT-2	3.00	13	14	16	30													
4		3.45																	
5	SPT-3	4.50	21	15	21	36													
6		4.95																	
7	SPT-4	6.00	23	18	20	38													
8		6.45																	
9	SPT-5	7.50	16	20	17	37													
10		7.95																	
11	SPT-6	9.00	25	34	35	69													
12		9.45																	
13	KAROT-1	10.50													21	40	21		
14	KAROT-2	12.00													18	32	15		
15	KAROT-3	13.50													25	25	23		
16	KAROT-4	15.00													22	54			
17	KAROT-5	16.50													25	43			
18	KAROT-6	18.00													23	44	18		
19	KAROT-7	19.50													17	46			
20	KAROT-8														17	25	6		

Figure 5.2 Borehole log of Davutpaşa Interchange

5.3. Earthquake Potential Evaluation of the Region

The seismic risk map of İstanbul is given in Figure 5.3. The region that the interchange will be built is highly seismic and the maximum expected horizontal acceleration was $a_0/g=0.4$.

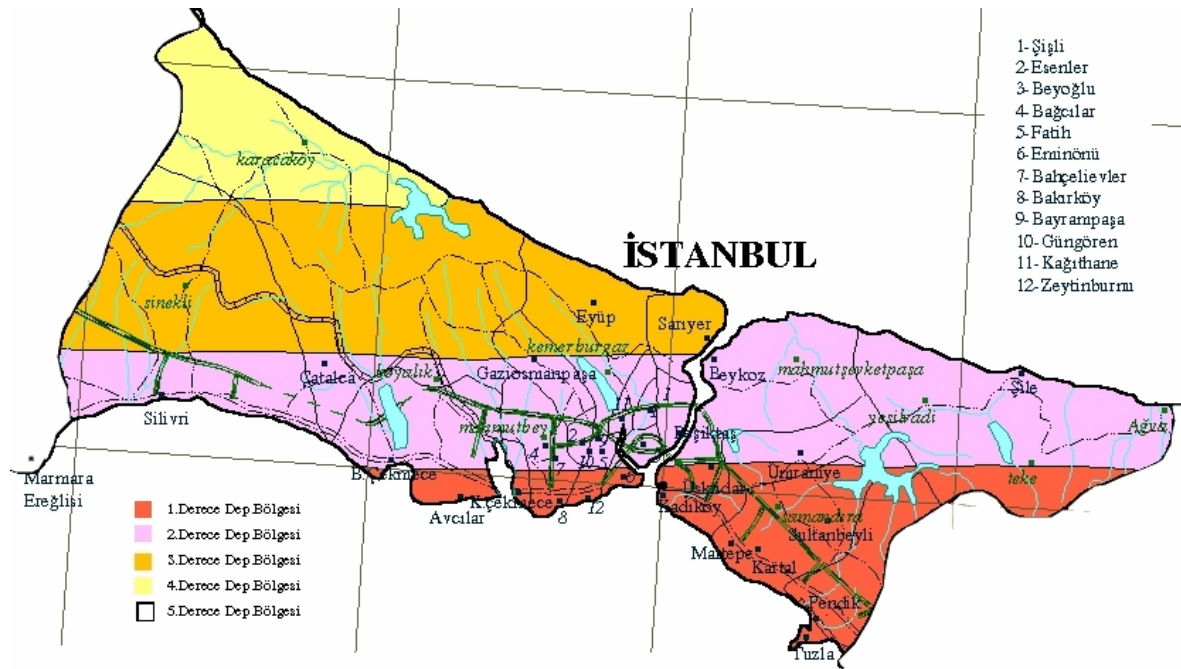


Figure 5.3 Seismic Risk Map of İstanbul

5.4. Design with Limit State Analysis

The limit state analysis has been performed by the VALDEZ program. This program is designed specifically to design reinforced earth walls. In Figure 5.4 help screen of the software can be seen. 3 type of soil can be defined in the analysis the reinforced earth backfill, the general backfill and the foundation soil (Figure 5.5).

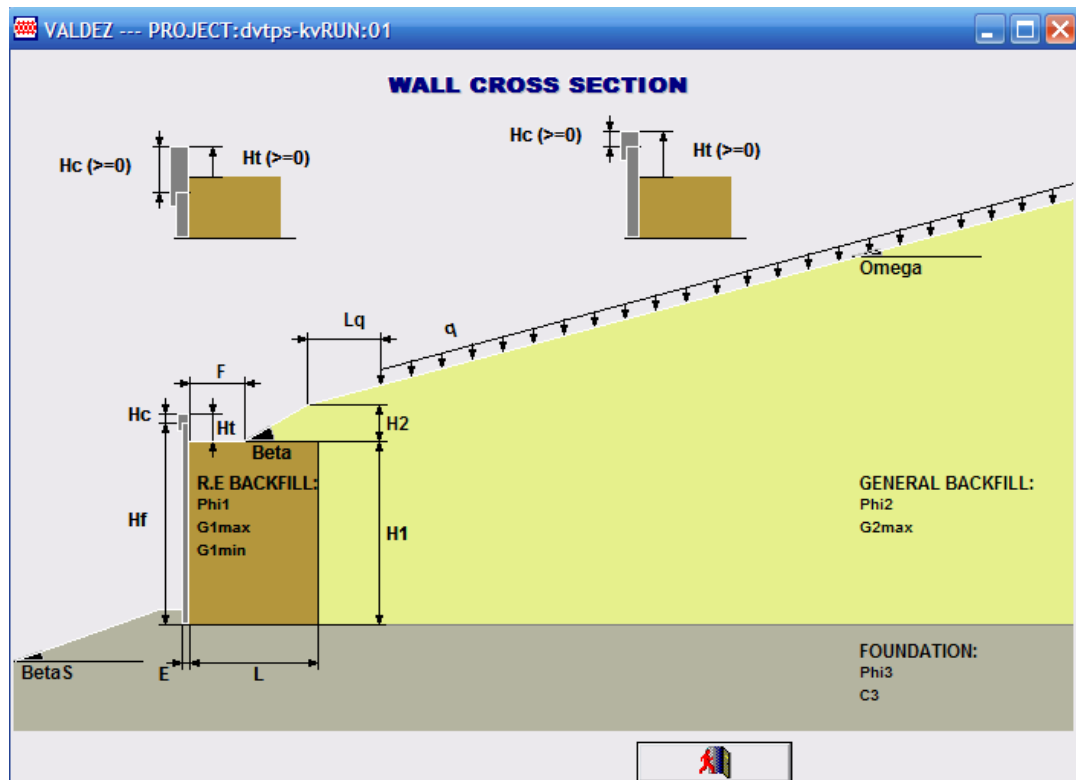


Figure 5.4 VALDEZ Data Explanation

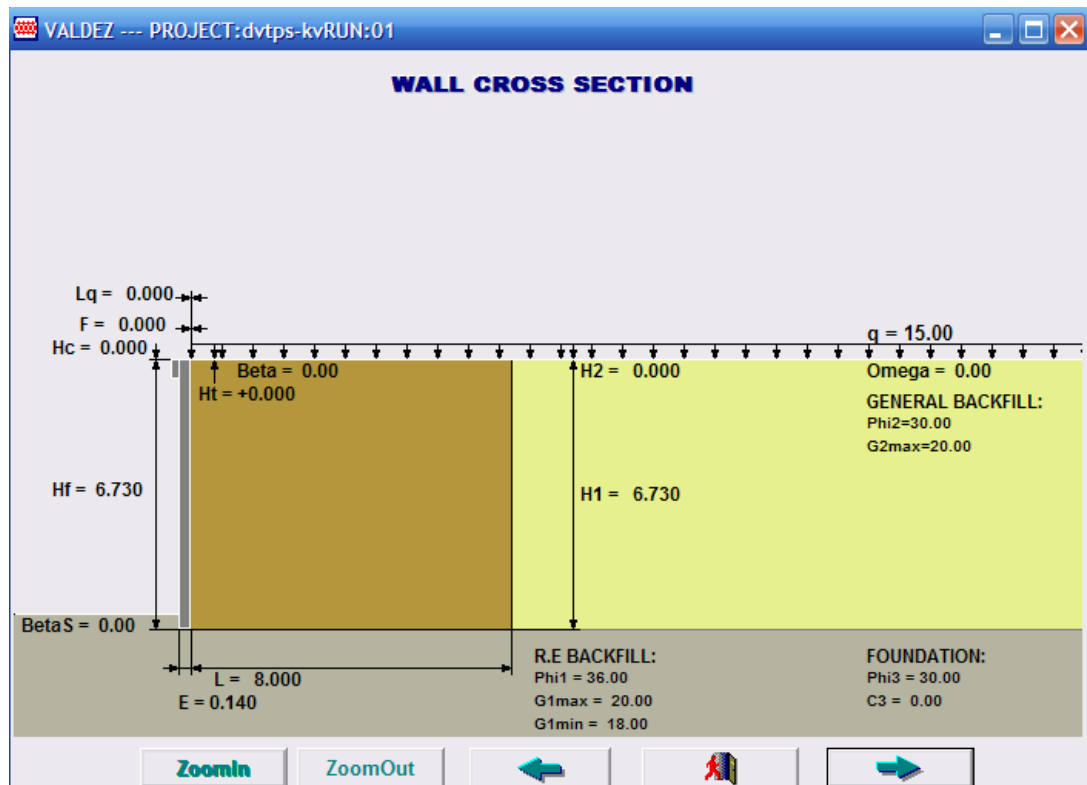


Figure 5.5 VALDEZ Davutpaşa Interchange RE Wall Design

The detailed design output is given in Appendix B. The important part will be emphasized following which is the density of the strips.

5.4.1. Density of Strips

8m long strips are used to establish the external stability to the reinforced earth wall and necessary adherence length to the steel strips. Density of the calculated strips are given in Table 5.2 as they will be used again in the data of the numerical analysis. In a 6.73m height wall 9 rows of strips have been used.

VALDEZ is calculating the number of strips required for 3m. To indicate this densities of strips are left as fractions.

Table 5.2 Strip Density of the Davutpaşa Reinforced Earth Wall

Row Number	Strip Depth (m)	# of Strips/1.00m
1	0.365	4/3
2	1.115	4/3
3	1.865	4/3
4	2.615	4/3
5	3.365	4/3
6	4.115	4/3
7	4.865	4/3
8	5.615	5/3
9	6.365	5/3

5.5. Analysis with PLAXIS Software

PLAXIS does not use a code to define the stages, a mesh system must be pregenerated -PLAXIS generates its own mesh- related to the geometry defined. Self generating mesh makes PLAXIS much practical, but choosing a specific point and look for the stresses and stains at that specific is not impossible but very hard in PLAXIS.

In addition PLAXIS makes use of higher order elements which makes possible to use larger elements without giving out the precision.

PLAXIS is seemed to be the combination of four modes “Input”, “Calculations”, “Output”, “Curves”. Input part is used to define the system, since undefined parts -soil layers, strips etc.- cannot be added later on. The complete system must be defined in the Input part. Figure 5.6 shows the general view of the software.

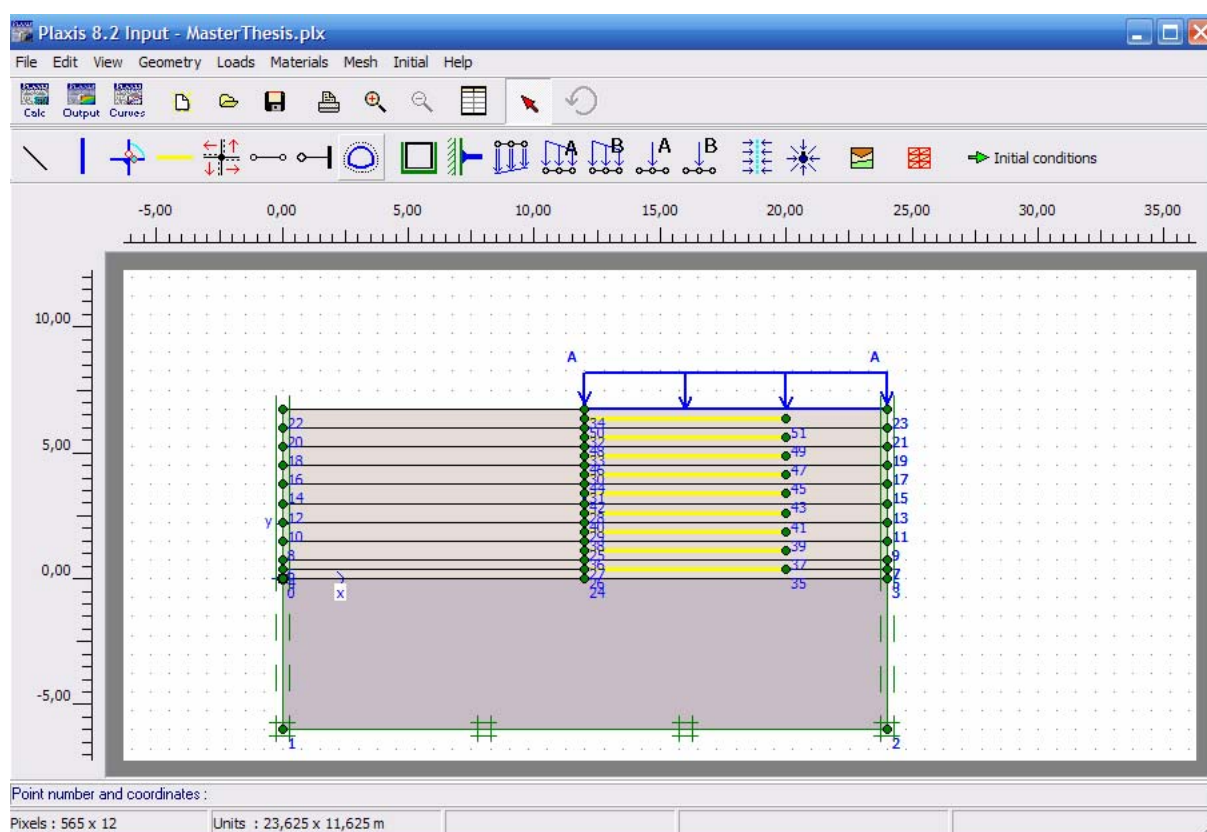


Figure 5.6 General View of the PLAXIS INPUT

As seen in the Figure 5.6 the model boundaries and the soil properties are the same as VALDEZ. After deactivating the soil layers that are not present in the initial stage, the initial stresses are generated by K_0 method. PLAXIS in this stage does not deform the system but forms the initial stresses.

After generating initial stresses program passes to Calculation mode.

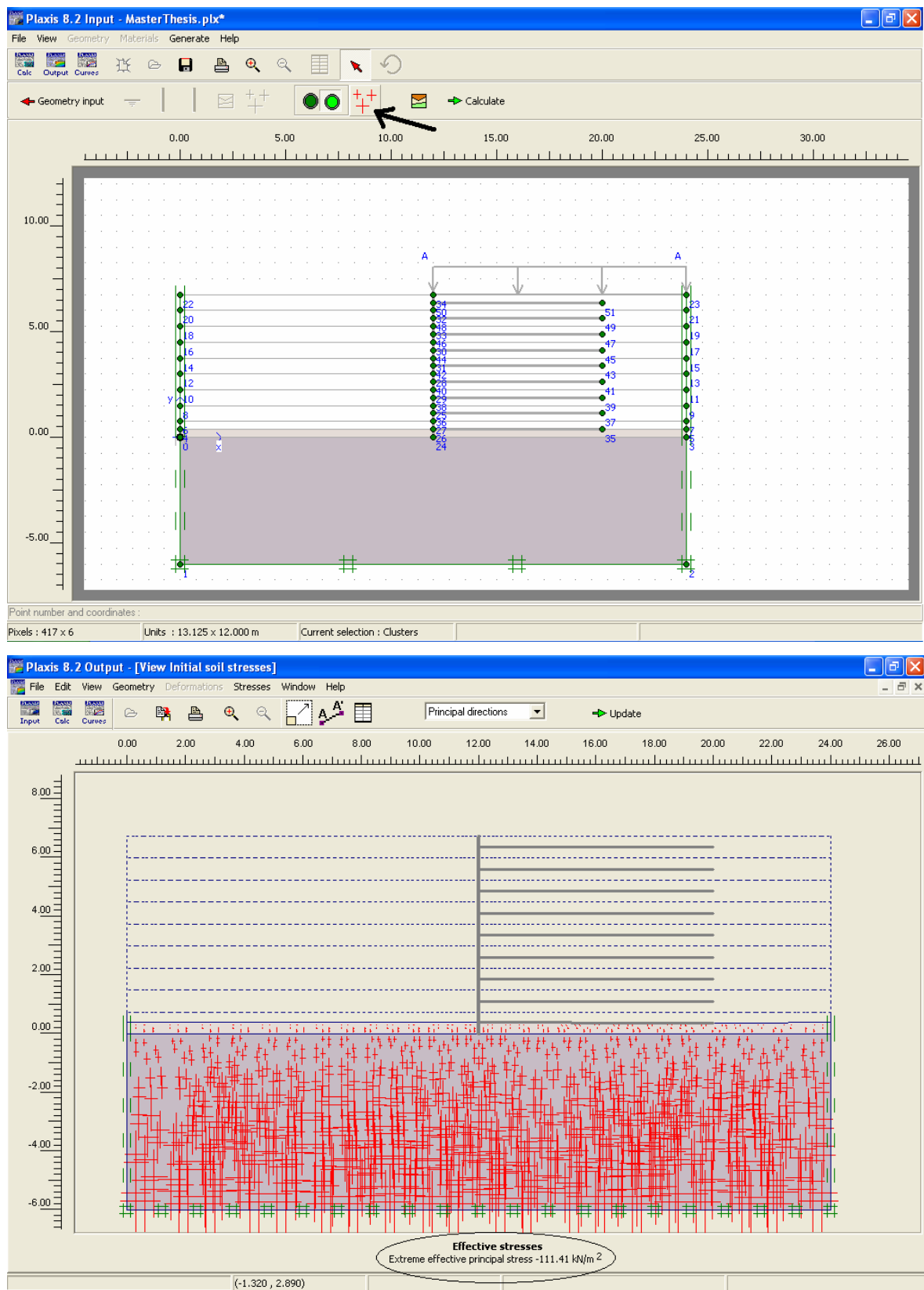


Figure 5.7 Initial Stress Generation and Initial Stresses

In the calculation part the stages of fill are defined as shown in the Figures 5.8, 5.9, 5.10, 5.11. As seen in Figure 5.8 the first layer of reinforced earth fill and steel strips are placed.

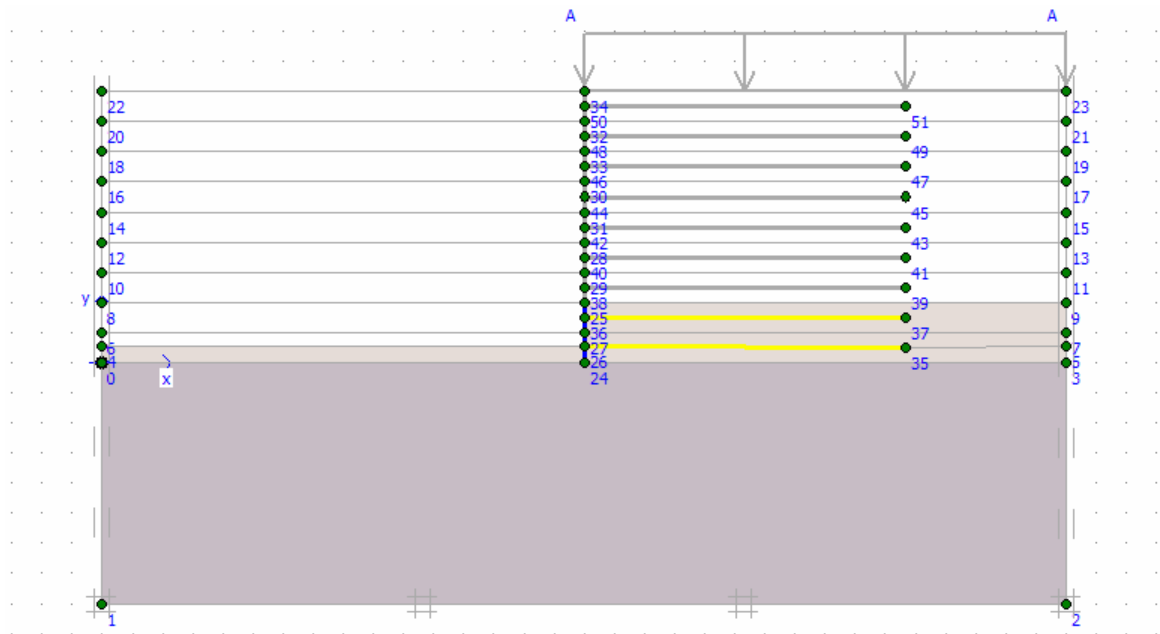


Figure 5.8 Activating First Layer of the Fill and Strips

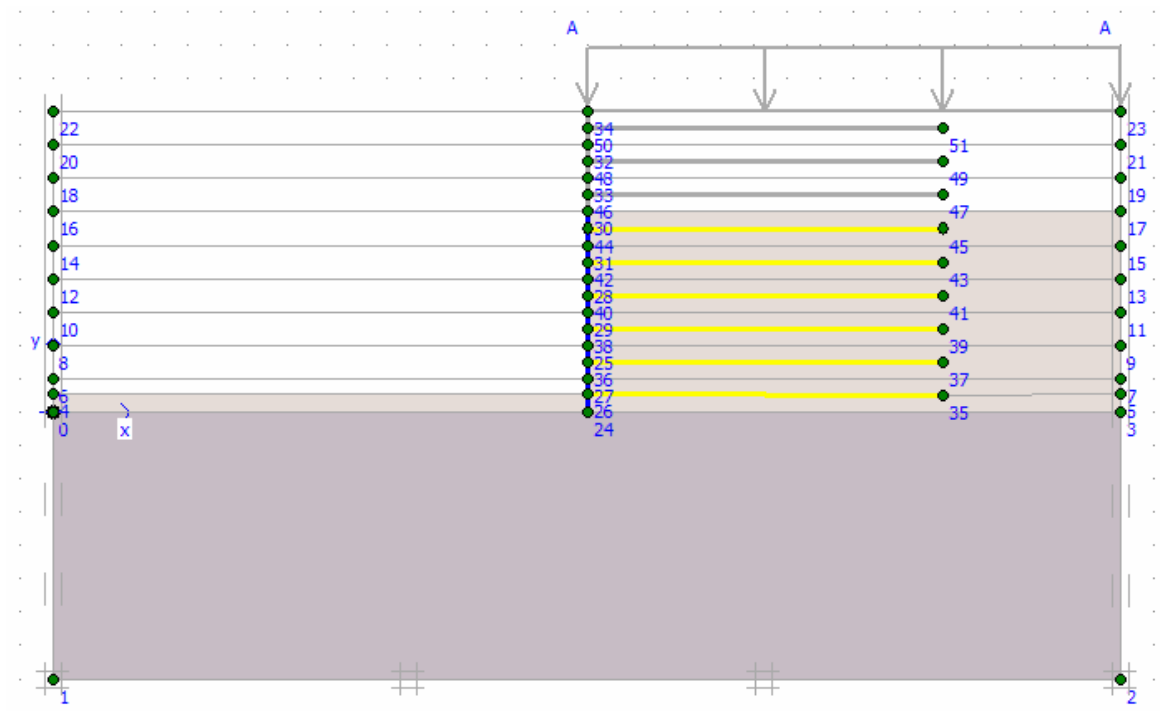


Figure 5.9 Activating more Soil Layers and the Strips

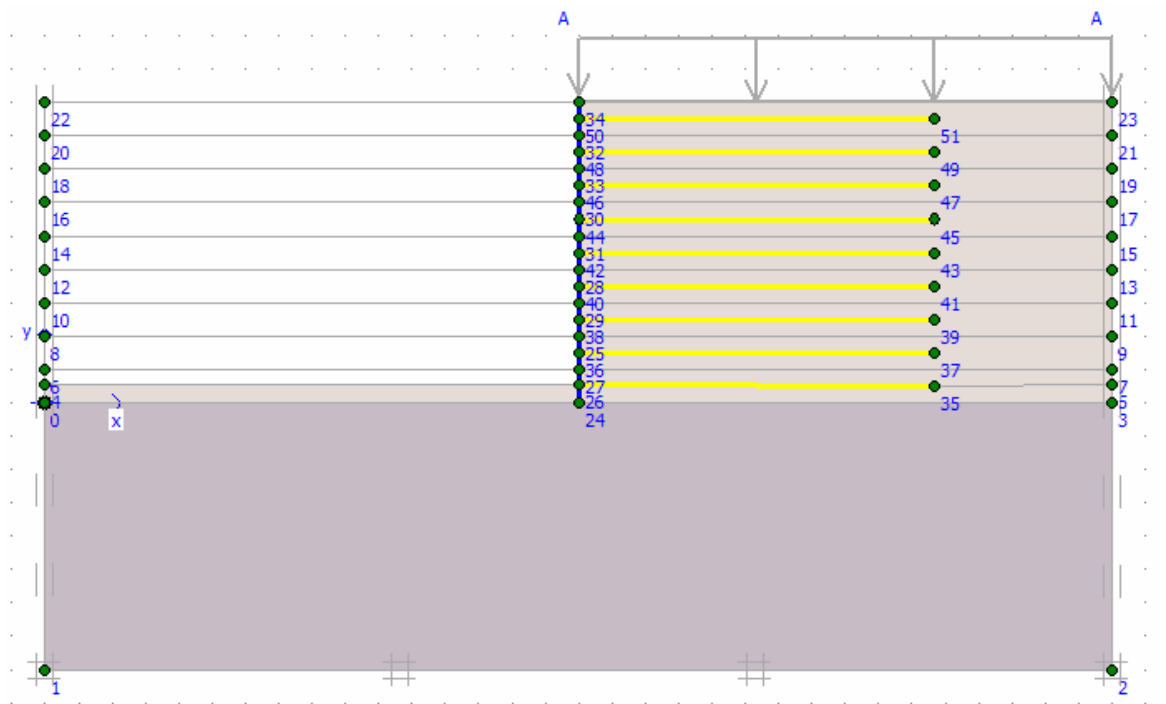


Figure 5.10 Activating all Soil Layers and the Strips

The figures shows the construction stages clearly and visually. The acceleration application to the system is done by changing the coefficient of the predefined acceleration from 0 to 1 (Figure 5.11).

Results are summarized in Table 5.3, and given in the Figures 5.12, 5.13, 5.14, 5.15, 5.16.

Table 5.3 Maximum Deformations Calculated by PLAXIS

	Without Earthquake	With Earthquake
X Displacement (cm)	0.40	1.48
Y Displacement (cm)	1.78	3.36

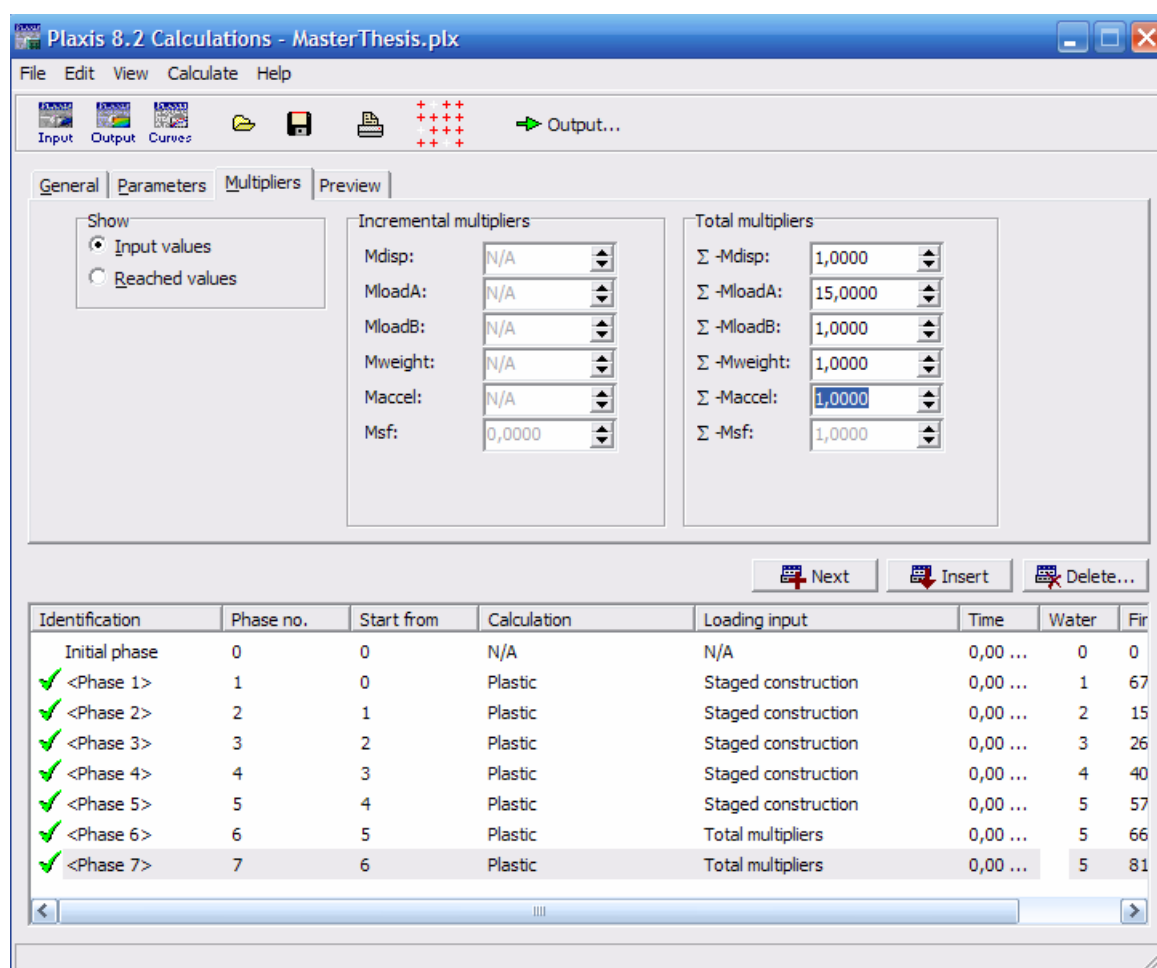


Figure 5.11 Activating the Earthquake Acceleration

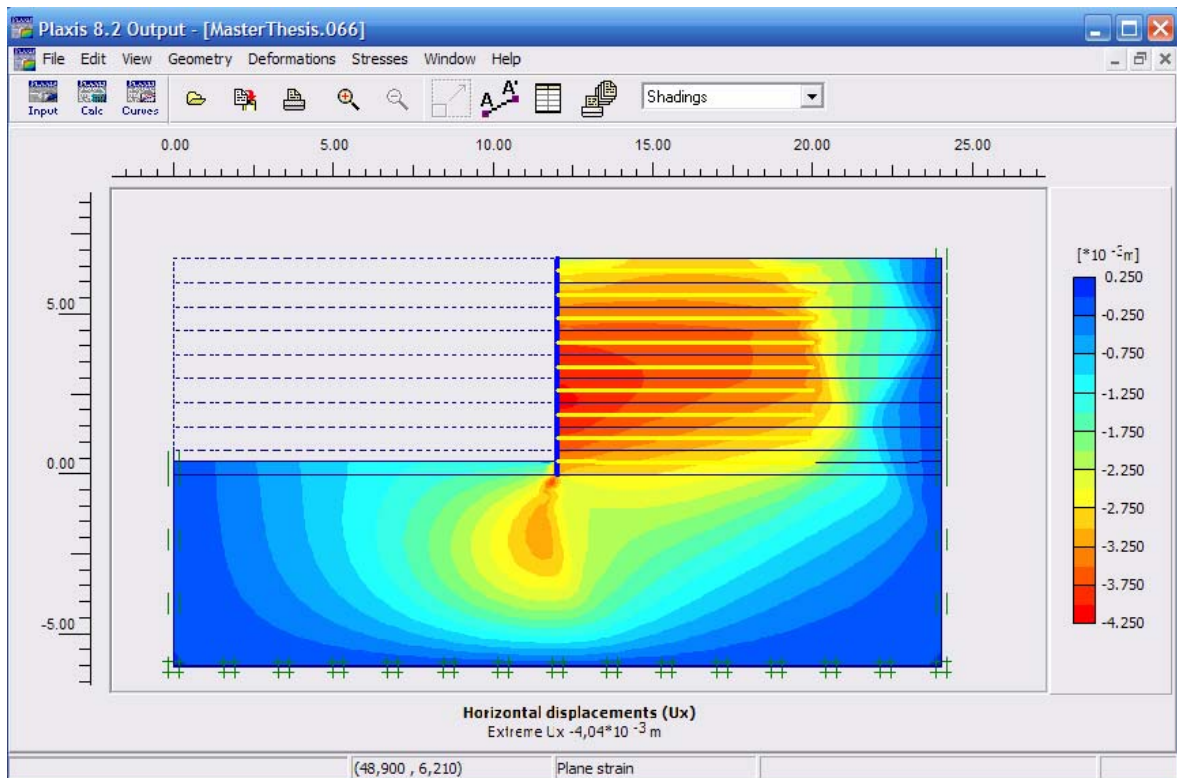


Figure 5.12 X displacement (Without Earthquake)

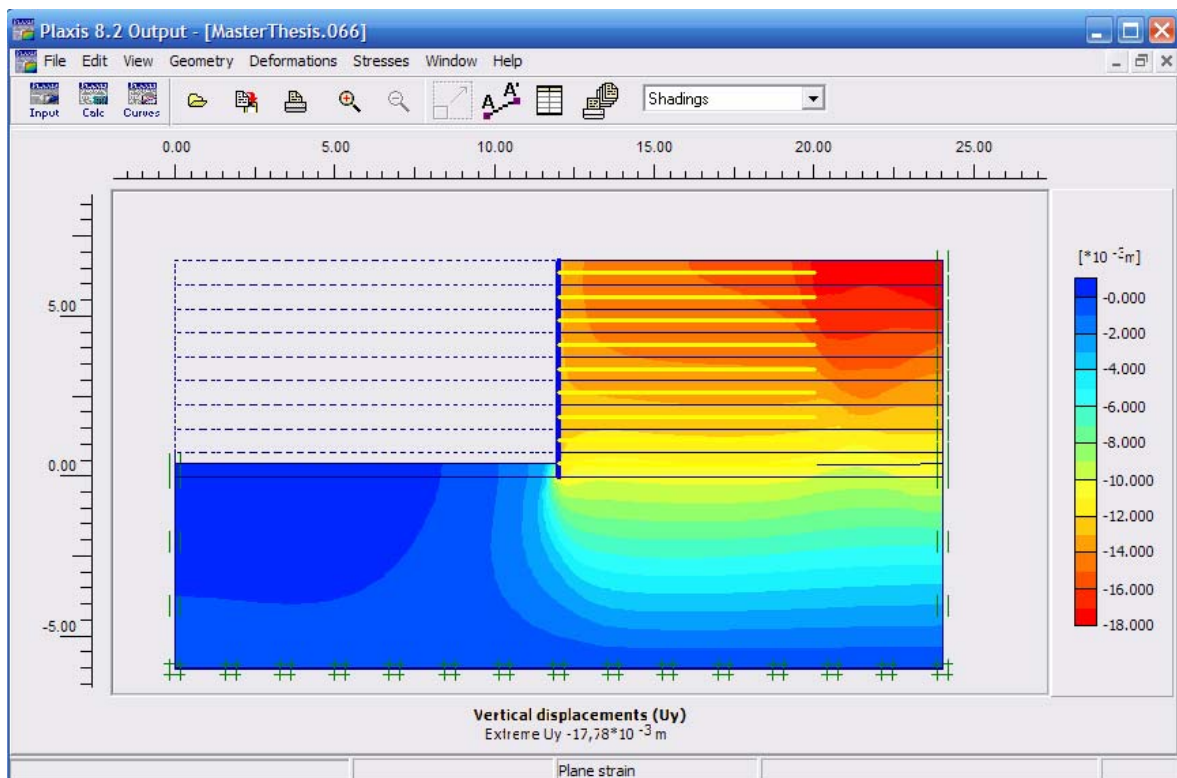


Figure 5.13 Y displacement (Without Earthquake)

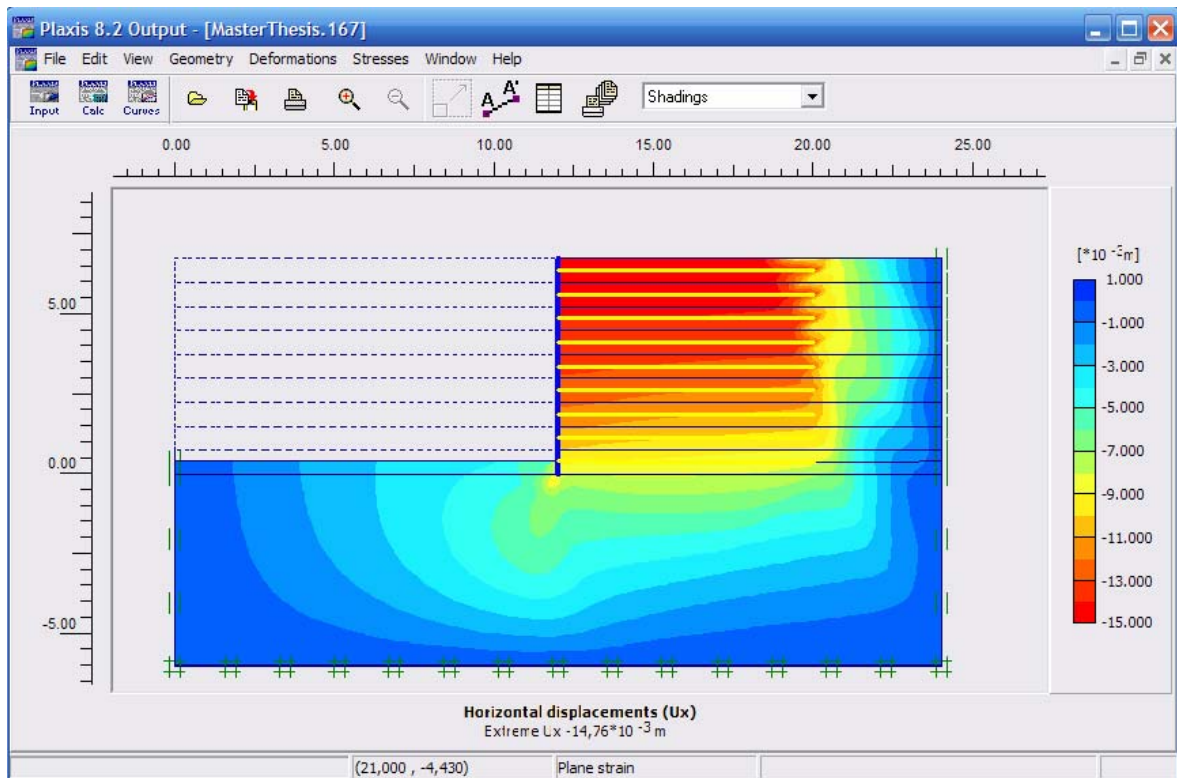


Figure 5.14 X displacement (With Earthquake)

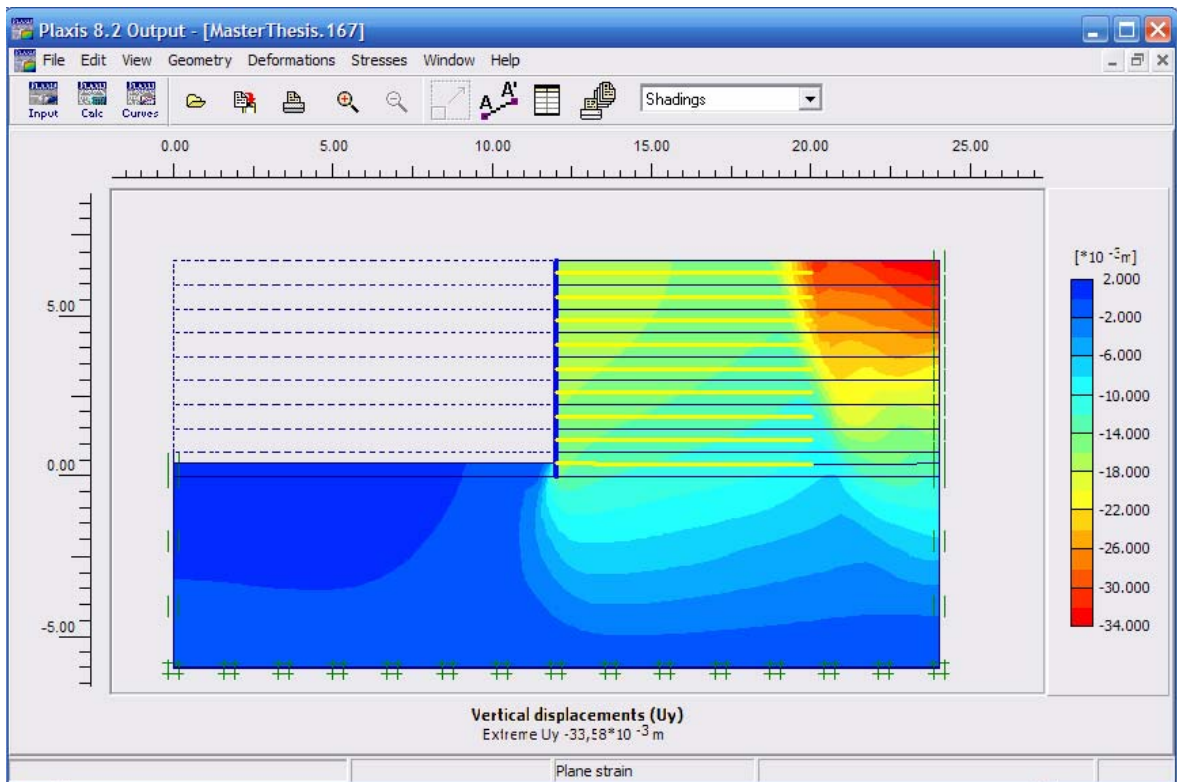


Figure 5.15 Y displacement (With Earthquake)

6. CONCLUSION

The purpose of this study is to present a literature review and evaluation of types and general concepts of all earth reinforcement structures, especially reinforced earth retaining walls with metallic strip reinforcements, their applications, design methods, components, behavior under seismic loads. Case studies including behavior of structures during previous earthquakes and an example of applied project are investigated. In addition introducing numerical analysis to Reinforced Earth walls and estimating the displacements formed in the structures.

A study on the seismic response of reinforced soil retaining walls was carried out with both limit state and finite element analysis. Two different softwares were used, VALDEZ and PLAXIS respectively.

VALDEZ utilizes both load factors and material factors on the different parts of the calculation. It analyzes the structure as a whole and gives the results for overturning or sliding part by part. PLAXIS not only estimate the displacements but also analyze the structural parts in detail as well. The axial force on the reinforcements can be seen. In the same manner the moments, shears can be observed for each precast concrete panel layer and for each step of construction.

Design calculations were carried out for a 6.73m high Reinforced Earth wall, under seismic loading with maximum horizontal acceleration ($a_0/g = 0.4$). By using VALDEZ software which uses limit state approach; solutions including strip length, reinforcement density were determined. Internal and external stability analysis were evaluated. These length and densities were used for 1m width of wall in PLAXIS software. Horizontal and vertical displacements, axial forces on the reinforcements were calculated. The deformations and forces were in the applicable limits (Figure 6.1).

The connection points were critical location for reinforcement and facing element (precast concrete panel) interaction.

As coinciding with site investigations after some recent earthquakes, finite element simulations of reinforced soil retaining walls exposed an intact behavior under seismic events. These structures have a strong stability if they were designed well with respect to current design methods.

Reinforced Earth structures have proven to be safe and flexible in the presence of seismic events throughout the world. Current design codes apply a very conservative approach, especially in the determination of external stability. A plastic deformation approach be taken instead in the design of Reinforced Earth structures; whereby consideration for seismic design will be based on wall height, ground acceleration, and allowable deformation. The wall height will determine how much reliance needs to be paid to seismic design, with lower height walls being less restrictive than moderate to tall walls.



Figure 6.1 Davutpaşa Interchange Reinforced Earth Walls

APPENDIX A: PRESENTATION OF VALDEZ PROGRAM

PRÉSENTATION OF THE "VALDEZ" CALCULATION

CALCULATION METHOD FOR THE DESIGN

OF REINFORCED EARTH RETAINING STRUCTURES WITH VERTICAL FACING

TAISOFT PROGRAM : VALDEZ

Method, notations, results and output of the program

The calculation is carried out according to the AFNOR standard NF P 94-220-0/1 :
Renforcement des sols-Ouvrages en sols rapportés renforcés par armatures ou nappes peu extensibles et souples-Dimensionnement
Backfilled structures with inextensible and flexible strips or sheets-Design

All references to a standard are to the NF P 94-220-0/1 standard.

Note : Letters A to G correspond to the chapters of this document, whereas numbers 1 to 2.3.2. refer to the paragraphs of the program output.

A. IDENTIFICATION OF THE PROJECT

The first page presents all the necessary elements to identify the project and the calculation.

B. CALCULATION HYPOTHESES

1. GENERAL DATA

1.1 STRUCTURE CLASS

The service life and the project site define the steel sacrificial thickness to corrosion.
(c.f. §5.1.1, §5.1.2 and §6.5 of the standard and the following table, taken from the AFNOR standard NF A 05-252).

Minimum sacrificial thicknesses (in millimetres)								
Metal	Service life							
	5 years		30 years		70 years		100 years	
	S	G.S.	S	G.S.	S	G.S.	S	G.S.
Site								
Dry.....	0.5	0	1.5	0.5	3	1.0	4	1.5
Submerged..	0.5	0	2.0	1.0	4	1.5	5	2.0

S. = Black steel,

G.S. = Hot dipped galvanised steel, 500g/m².

The safety level, when indicated to be « high », corresponds to sensitive structures as defined in §5.1.3 of the standard.

1.2 R.E. STRIPS

The materials used are either black steel or steel with a surface treatment such as galvanisation or other.

For each type of strip that comes into the construction of the structure, the geometrical and mechanical characteristics are defined :

width, b ,

thickness, e_0 ,

sacrificial thickness, e_s , defined earlier,

the grade of the steel, σ_r .

The allowable tensile strengths, T_r and T_{r0} are given, for one strip, for its full section and at the connection :

$$T_r = \sigma_r \cdot (b - b_t) \cdot (e_0 - e_t - e_s) / \gamma_{mt}$$

(all dimensions are given in mm)

where b_t and e_t are the width and the thickness tolerances (given in the last pages of the output-Complementary Data).

For the definition of γ_{mt} , see further, chapter D.

T_{r0} is computed in a similar way, taking into account the connection.

1.3 FACING

The type of facing is specified here.

Standard panels are cruciform-shaped 14cm thick concrete panels.

1.4 SEISMIC DATA

Maximal horizontal acceleration, a_0/g :

Let a_0 , the maximal horizontal acceleration at ground level on the project site. The horizontal design acceleration, a_d , in the Reinforced Earth structure and the backfill is given by :

$$\frac{a_d}{g} = \frac{a_0}{g} \times \left(1.45 - \frac{a_0}{g} \right)$$

a_n , the reference maximal horizontal nominal acceleration is given in the rules of the Association Française du génie Para-Sismique (AFPS) as a function of the soil category (S_1 to S_3) and the seismic zone (from 0 to III). A relationship, specific to Reinforced Earth structures, in accordance with the AFPS recommendations, has been established as follows :

$$\begin{aligned} a_0 &= a_n \quad S_1 \text{ site, compact soil} \\ a_0 &= 0.9a_n \quad S_2 \text{ site, medium soil} \\ a_0 &= 0.8a_n \quad S_3 \text{ site, poor soil} \end{aligned}$$

The reduction factor of live loads, ψ , is equal to 0, in accordance with §7.3.1 and §8.3.1 of the standard.

The dynamic variation factor, (ϵ), is equal to either :

- 0 (vertical acceleration is not taken into account)
- 1 (vertical acceleration is taken into account)

C. SECTION CHARACTERISTICS

2. SECTION (followed by the section number)

When several sections are designed during the same run of the program, the following points are repeated for each one.

2.1 SECTION DATA

2.1.1 GEOMETRY

The dimensions necessary for the definition of the section of the structure as well as the applied surcharge are presented here.

Please refer to fig.1 for the notations.

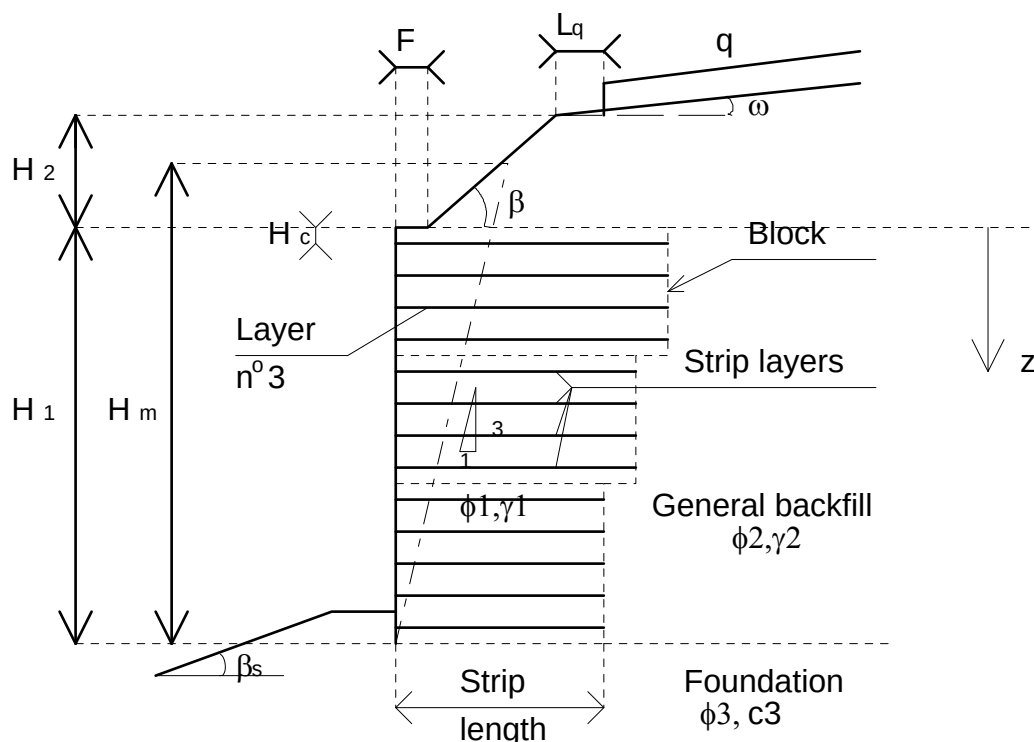


Fig. 1 - Cross-section of a wall (trapezoidal section)

In the case of a trapezoidal section, the length and number of strips are given for each block numbered from top to bottom.

In the case of a rectangular section, which corresponds to a trapezoidal section with only one block, only length of the strips is given.

2.1.2 SOIL PROPERTIES

The properties of the soils taken into consideration for the calculation are given hereafter:

R.E. BACKFILL (soil 1) : Maximum and minimum densities (γ_1) and internal angle of friction (ϕ_1).

GENERAL BACKFILL (soil 2) : Density (γ_2) and internal angle of friction (ϕ_2).

FOUNDATION (soil 3) : Internal angle of friction (ϕ_3) and cohesion (C_3).

Refer to fig.1 for notations.

2.1.3 EARTH PRESSURE

The earth pressure diagram applied at the back of the wall depends on the geometry of the embankment above and the surcharge as well as the soil properties.

The earth pressure is inclined to the horizontal with an angle, δ , whose value depends on the flexibility of the structure. This value is computed in accordance with the standard, annex F.

The static earth pressure coefficients, K_{2x} and K_{2y} , are also computed in accordance with the standard, annex F.

In the case of a seismic design, the two dynamic earth pressure coefficients, K_{aex} and K_{aey} , are computed following Mononobe-Okabe's formulae, in accordance with the AFPS recommendations.

D. LOAD CASES AND LEVEL OF SAFETY

The Load Factors that need to be taken into account for each load case are shown in the following table, in accordance with the standard (§7.3.1 and §8.3.1).

Load case	Structure's selfweight γ_{F1G}	Backfill thrust γ_{F1G}	Surcharge above R.E. γ_{F1q}	Surcharge behind R.E. γ_{F1q}	Density of R.E. backfill
Fund. 1 (A)	1.20	1.20	1.33	1.33	Max
2 (B)	1.00	1.20	0	1.33	Min
3 *	1.00	1.00	0	0	Max
Accid. 1s	1.00	1.00	0	0	Min
2s	1.00	1.00	0	0	Max

* Case 3 is considered in §10.3.1 of the standard, for the computation of settlements.

Case 1s corresponds to case S of the standard, for external stability, and case 2s corresponds to case S of the standard, for internal stability.

To each of these cases, corresponds two sub-cases :

- . +dW : vertical acceleration directed upwards,
- . -dW : vertical acceleration directed downwards.

Partial safety factors and method factors (which take into account uncertainties on the computation model), in accordance with the standard (§7.3 and §8.3), are shown in the following table :

	Static (Fundamental)			Seismic (Accidental)		
	Partial safety factors		Method factors (γ_{F3})	Partial safety factors		Method factors (γ_{F3})
	(ordinary structures)	(sensitive structures)		(ordinary structures)	(sensitive structures)	
EXTERNAL STABILITY						
. Base sliding			1.000			1.000
Friction	$\gamma_{mq} : 1.20$	1.20		1.10	1.10	
Cohesion	$\gamma_{mc} : 1.65$	1.65		1.50	1.50	
. Overturning*	$\gamma_{mr} : 1.50$	1.50	1.000	1.50	1.50	1.000
. Bearing capacity	$\gamma_{mq} : 1.50$	1.50	1.125	1.50	1.50	1.000
INTERNAL STABILITY						
. Tension	$\gamma_{mt} : 1.50$	1.65	1.125	1.50	1.65	1.000
. Adherence	$\gamma_{mf} : 1.20$	1.30	1.125	0.96**	1.04**	1.000

* This criterion is not considered in the standard.

** The standard requires the value of the partial safety factor on adherence for seismic calculation to be 1.20 for ordinary structures and 1.30 for sensitive structures. The Valdez program has already taken into account a vertical stress reduction factor of 0.8 in the seismic calculations. It is therefore necessary to introduce in the data a partial safety factor of 0.96 ($0.80 \times 1.20 = 0.96$) and 1.04 ($0.80 \times 1.30 = 1.04$).

E. EXTERNAL STABILITY

The checking of the external stability, as required by §7 of the norm, is not included in the usual Design Package of the Terre Armée company.

The results of the Valdez calculation only constitute a part of the elements necessary to the checking; the design body in charge of this checking must use these elements together with the results of a geotechnical report.

2.2.1. LIMIT STATE FOR EXTERNAL STABILITY

For each load case considered, the program calculates :

- R_v and R_h , the vertical and horizontal resultant forces (in kN/m), at the base of the structure,
- $q_{ref.}$, the reference pressure on the base (in kPa),
when $H_m \geq L$, the Meyerhof pressure, equal to $R_v \cdot \gamma_{F3} / 2 \cdot x$
when $H_m \leq L$, the pressure is calculated at a distance of $H_m/4$ of the facing
- $2 \cdot x$, the width over which this pressure is applied.

The minimum embedment depth is given for normal subsoil.

The body responsible for the checking of external stability has to check the two following criteria:

a. Bearing capacity

The bearing capacity is checked when :

$$q_{ref.} \leq \frac{q_{fu}}{\gamma_{mq}}$$

where q_{fu} is the actual value of the bearing capacity which must take into account the soil characteristics and the inclination of the resultant force of which the tangent has a value of R_h/R_v .

b. Sliding on the base

The sliding on the base is checked when :

$$R_v \times \frac{\tan \phi}{\gamma_{m\phi}} + \frac{c}{\gamma_{mc}} \times L \geq \gamma_{F3} \times R_h$$

where ϕ and c are the angle of internal friction and the cohesion of either the reinforced fill or the the subsoil.

2.2.2 SLIDING ON THE BASE-OVERTURNING (indicative only)

The program gives, indicatively, the following elements :

*overdesign factors, Γ , regarding base sliding is given by :

$$\Gamma = \frac{R_v \times \frac{\tan \phi}{\gamma_{m\phi}} + \frac{c}{\gamma_{mc}} \times L}{\gamma_{F3} \times R_h}$$

This overdesign factor must be greater than one.

* the minimum values of the internal angle of friction for a purely frictional fill and the cohesion for a purely cohesive fill of a Foundation soil/Reinforced Earth fill contact in order to satisfy the minimum overall safety factors.

*the overdesign factor regarding overturning, given for information only as it is not considered in the standard, is the ratio of the stabilising and overturning moments.

F. INTERNAL STABILITY

The calculation width depends on the facing system.

2.3.1 STRIP RUPTURE-TENSILE LOAD AT FACING : *Overdesign factors*

For each layer of strips, the calculation values are detailed here.

. Columns 1 and 2 indicate the reference number and the depth z (in m), of the considered layer of strips.(c.f. fig.1)

. Column 3 - S_V , the vertical spacing of the layers.

. Column 4 - K , the earth pressure coefficient, calculated according to the following figure (c.f. §8.2.2.1 of the standard)

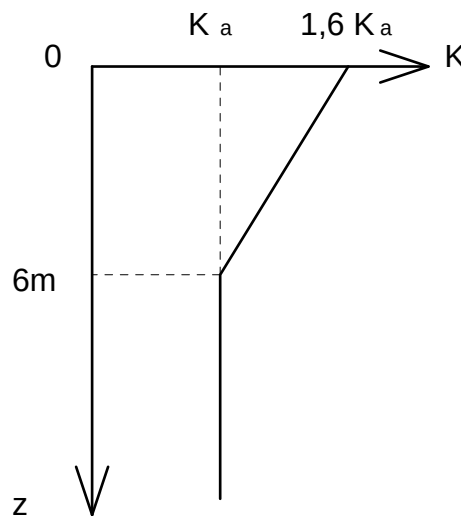


Fig. 2

. Column 5 : type, the type of strips.

. Column 6 : Num, the number of strips in the layer considered, per calculation width (3m for walls erected with standard panels).

. Column 7 : case, the load case considered.

. Column 8 : σ_h , the horizontal stress :

$$\sigma_h = K \cdot \sigma_v$$

where K is given in column 4 and σ_v is the vertical stress at the depth of the considered layer, computed from Meyerhof's formulae.

- . Column 9 : $T_{\max}$, the maximum tension in one strip (c.f. §8.2.2 of the standard)

$$T_m = \gamma_{F3} \cdot \sigma_h \cdot S_v / n$$

where n = number of strips per metre run and γ_{F3} is the method factor for tension checking.

- . Column 10 : T_o , the tension in one strip at the facing.

$$T_o = \alpha_i \cdot T_m$$

where α_i is given in §8.2.3 of the standard.

- . Column 11 : T_r/T_m ; the overdiseign factor regarding maximum tension in the strips.

- . Column 12 : T_{ro}/T_o , the overdiseign factor regarding tension in the strips at the facing.

- . Column 13 : the type of facing in use.

When panels are considered, the program indicates which type should be used :

UR = unreinforced panels

R = reinforced panels

RS = 18 cm thick special panels

2.3.2 **ADHERENCE** : *Overdiseign factors*

- . Columns 1 and 2 indicate the reference number and the depth z (in m), of the considered layer of strips.(c.f. fig.1)

- . Column 3 : μ^* , the friction coefficient calculated in accordance with §6.3 of the standard,.

- . Column 4 : L , the length (in m) of the strips of the considered layer.

- . Column 5 : L_a , the adherence length (in m) of the strips of the considered layer (c.f. §3.1 of the standard)

- . Column 6 : h_a , the mean depth of the adherence length (defined in §6.3 of the standard).

- . Column 7 : type, the type of strips.

- . Column 8 : Num, the number of strips in the layer considered, per calculation width.

- . Column 9 : case, the load case considered.

- . Column 10 : $T_{\max}$, the maximum tension in one strip.

- . Colonne 11 : T_f , the strip pull-out capacity, partial safety factors included.

$$T_f = r_f / (N \times \gamma_{mf})$$

where r_f is given in §8.2.4 of the standard.

- . Column 12 : T_f/T_m , the overdiseign factor regarding strip pull-out, where :

Are then detailed :

- . the type of strips in use,

- . for each type of strip in use, the quantity of strips required for a width of facing equal to the calculation width and the weight of strips per square metre of facing.

G. COMPLEMENTARY DATA

1. STANDARDISATION FILE

The Valdez program seeks data for the calculation in a standardisation file which is in accordance with the various rules applicable in the indicated country (e.g. : NF P 94-220-0/1 standard in France).

2. CALCULATION METHOD

The calculation method is that in use in the concerned country (e.g. : Limit state in France, Working loads in the U.S.A.)

3. FACTORS

The load, partial security and method factors are presented here according to the tables in chapter D of this present document.

4. STRIPS

For each type of strip which comes into use, some complementary data such as tolerances and dimensions at the connection are detailed here.

5. FACING

In this chapter, the vertical spacing of the layers and the flexibility of the facing are given (c.f. §8.2.3 of the standard).

APPENDIX B: VALDEZ CALCULATION OF DAVUTPAŞA INTERCAHANGE REINFORCED EARTH WALLS

```

*****
*                               *
*   Reinforced Earth : Program Valdez   *
*                               *
*                               v2.8      *
*****

```

Job number : dvtps-kv
=====

Structure : DAVUTPAŞA INTERCHANGE AND
=====
CONNECTION ROAD CONSTRUCTION

REINFORCED EARTH WALLS

Designed by : T. K.
=====

```
*****
*      1 .      GENERAL DATA      *
*****
```

```
1 . 1  STRUCTURE CLASS :
-----
```

Service life : 0 years

Site : No water

```
1 . 2  R.E. STRIPS :
-----
```

protection : Galvanized

type 1: HA 40X4

```
-----
grade          : 510.0 MPa      Allowable tensile strength :
width          b : 40.0 mm
thickness      eo : 4.0 mm      full section Tr : 36.65 kN
sacrif. thick. es : 1.0 mm      connection Tro : 24.28 kN
```

```
1 . 3  FACING: ESCAMA ESTANDAR
-----
```

```
1 . 4  SEISMIC DATA :
-----
```

Maximal horizontal acceleration a_0/g : 0.40
Reduction factor of live loads : 0.00
Dynamic variation factor : 1

```
*****
*      2 . SECTION 01      *
*****
```

```
2 . 1  SECTION DATA :
```

```
2 . 1 . 1  GEOMETRY :
-----
```

```
R.E MASS
Facing height  Hf : 6.730 m      R.E.backfill height  H1 : 6.730 m
Coping height  Hc : 0.000 m      Free board           Ht : 0.000 m
Slope height   H2 : 0.00 m
Slope angle    beta : 0.00 deg.   Terrace angle omega   : 0.00 deg.
Setback        F : 0.00 m         Angle at toe Beta_s   : 0.000
```

Strip Length : 8.00 m

SURCHARGES

Load q : 15.00 kPa Position Lq : 0.00 m
Type : Road traffic

2 . 1 . 2 SOIL PROPERTIES :

```

-----
R.E. BACKFILL
Density      gamma1 maxi : 20.00 kN/m3      Friction    phi1          : 36.00 deg.
              gamma1 mini : 18.00 kN/m3

GENERAL BACKFILL
Density      gamma2      : 20.00 kN/m3      Friction    phi2          : 30.00 deg.

FOUNDATION
Cohesion     C3          : 0.00 kPa        Friction    phi3          : 30.00 deg.

```

2 . 1 . 3 EARTH PRESSURE :

Inclination of earth pressure at back of R.E. mass delta = 4.84 deg.

Earth pressure coefficients:
k2x = 0.000 k2y = 0.319 (Static)
kaex = 0.000 kaey = 0.482 (Dynamic)

2 . 2 EXTERNAL STABILITY

2 . 2 . 1 EXTERNAL STABILITY :

case	Rv kN/m	Rh kN/m	M kNm/m	qref kPa	2.x m
1	1006.61	242.71	3581.97	149.41	7.12
2	1679.65	242.71	6333.84	231.46	7.54
3	1107.83	144.10	4235.24	149.78	7.65
1s +dW	1101.81	418.45	3242.24	197.47	5.89
-dW	894.73	418.45	2399.44	172.03	5.36
2s +dW	1222.88	441.46	3657.57	216.21	5.98
-dW	992.79	441.46	2721.12	187.95	5.48

Minimum embedment depth = 0.40 m

2 . 2 . 2 SLIDING ON THE BASE - OVERTURNING :

SLIDING ON THE BASE					OVERTURNING
Overdesign factor		minimal value			Overdesign factor
case	slip in R.E.	slip in found.	phi(°)	Cohesion(kPa)	
1	2.51	2.00	16.14	-	4.66
			-	49.20	
2	4.19	3.33	9.84	-	7.72
			-	49.20	
1s	1.29	1.03	29.30	-	1.91
			-	77.11	

2 . 3 INTERNAL STABILITY

Calculation width : 3.00 m

2 . 3 . 1 STRIP RUPTURE - TENSILE LOAD AT FACING : Overdesign factors

layer	z m	deltah m	k	strip		case	sigmah kPa	Tmax kN	To kN	Tr/Tm	Tro/To	
				type	Num.							
@ 1	0.365	0.740	.403	1	4	1	2.65	1.47	1.25	24.88	19.39	UR4 RC
@						2	13.04	7.24	6.15	5.06	3.95	
@						1s	2.65	6.05	5.15	6.05	4.72	
@						2s	2.94	6.73	5.72	5.45	4.25	
@ 2	1.115	0.750	.384	1	4	1	7.78	4.38	3.72	8.37	6.53	UR4 RC
@						2	20.27	11.40	9.69	3.21	2.50	
@						1s	7.72	9.02	7.67	4.06	3.17	
@						2s	8.58	10.03	8.52	3.66	2.85	
@ 3	1.865	0.750	.365	1	4	1	12.50	7.03	5.97	5.21	4.06	UR4 RC
@						2	26.82	15.09	12.83	2.43	1.89	
@						1s	12.32	11.71	9.95	3.13	2.44	
@						2s	13.68	13.00	11.05	2.82	2.20	
@ 4	2.615	0.750	.346	1	4	1	16.82	9.46	8.04	3.87	3.02	UR4 RC
@						2	32.73	18.41	15.65	1.99	1.55	
@						1s	16.48	14.14	12.02	2.59	2.02	
@						2s	18.29	15.70	13.34	2.33	1.82	
@ 5	3.365	0.750	.327	1	4	1	20.78	11.69	9.94	3.14	2.44	UR4 RC
@						2	38.01	21.38	18.17	1.71	1.34	
@						1s	20.20	16.33	13.88	2.24	1.75	
@						2s	22.40	18.12	15.40	2.02	1.58	
@ 6	4.115	0.750	.308	1	4	1	24.39	13.72	11.72	2.67	2.07	UR4 RC
@						2	42.67	24.00	20.50	1.53	1.18	
@						1s	23.50	18.30	15.63	2.00	1.55	
@						2s	26.03	20.29	17.33	1.81	1.40	
@ 7	4.865	0.750	.288	1	4	1	27.65	15.55	13.94	2.36	1.74	UR4 RC
@						2	46.71	26.28	23.55	1.39	1.03	
@						1s	26.39	20.21	18.11	1.81	1.34	
@						2s	29.20	22.38	20.06	1.64	1.21	
@ 8	5.615	0.750	.269	1	5	1	30.58	13.76	12.91	2.66	1.88	UR4 RC
@						2	50.14	22.56	21.16	1.62	1.15	
@						1s	28.87	18.64	17.48	1.97	1.39	
@						2s	31.89	20.63	19.35	1.78	1.25	
@ 9	6.365	0.740	.260	1	5	1	34.39	15.27	14.96	2.40	1.62	UR4 RC
@						2	54.90	24.37	23.88	1.50	1.02	
@						1s	32.09	20.18	19.77	1.82	1.23	
@						2s	35.38	22.30	21.85	1.64	1.11	

2 . 3 . 2 ADHERENCE : Overdesign factor

layer	z m	f*	L	La	Za	type	Num.	case	Tmax kN	Tf kN	Tf/Tm
! 1	0.665*	1.453	8.00	6.04	0.365	1	4	1	1.47	5.99	4.07
!								2	7.24	20.26	2.80
!								1s	6.05	6.47	1.07
!								2s	6.73	7.19	1.07
! 2	1.115	1.356	8.00	6.17	1.115	1	4	1	4.38	9.57	2.19
!								2	11.40	25.10	2.20
!								1s	9.02	10.34	1.15
!								2s	10.03	11.49	1.15
! 3	1.865	1.260	8.00	6.29	1.865	1	4	1	7.03	15.17	2.16
!								2	15.09	32.93	2.18
!								1s	11.71	16.39	1.40
!								2s	13.00	18.21	1.40
! 4	2.615	1.163	8.00	6.42	2.615	1	4	1	9.46	20.03	2.12
!								2	18.41	39.63	2.15
!								1s	14.14	21.64	1.53
!								2s	15.70	24.04	1.53
! 5	3.365	1.066	8.00	6.54	3.365	1	4	1	11.69	24.10	2.06
!								2	21.38	45.10	2.11
!								1s	16.33	26.02	1.59
!								2s	18.12	28.92	1.60
! 6	4.115	0.970	8.00	6.69	4.115	1	4	1	13.72	27.41	2.00
!								2	24.00	49.45	2.06
!								1s	18.30	29.61	1.62
!								2s	20.29	32.90	1.62
! 7	4.865	0.873	8.00	7.07	4.865	1	4	1	15.55	30.81	1.98
!								2	26.28	54.13	2.06
!								1s	20.21	33.28	1.65
!								2s	22.38	36.97	1.65
! 8	5.615	0.776	8.00	7.44	5.615	1	5	1	13.76	33.30	2.42
!								2	22.56	57.36	2.54
!								1s	18.64	35.97	1.93
!								2s	20.63	39.96	1.94
! 9	6.365	0.727	8.00	7.82	6.365	1	5	1	15.27	37.12	2.43
!								2	24.37	62.96	2.58
!								1s	20.18	40.08	1.99
!								2s	22.30	44.54	2.00

Strips type 1 : HA 40X4

Strips type 1 : 304.0 meters for 3.0 m width of wall

Reinforcing strips weight for one square meter of facing: 21.53 kg

 * COMPLEMENTARY DATA *

1 STANDARDISATION FILE : TURK40X5

2 CALCULATION METHOD : Limit state

3 FACTORS :

load cases	LFW	Load factors			R.E density
		LFp	LFq1	LFq2	
1	1.00	1.35	0.00	1.50	1
2	1.35	1.35	1.50	1.50	2
3	1.00	1.00	0.00	0.00	2
1s	1.00	1.00	0.00	1.00	1
2s	1.00	1.00	1.00	1.00	2

R.E density : 1 = min - 2 = max

Safety factors	FSg	FSgc	FSr	FSc	FSt	FSf
Static	1.20	1.65	1.50	1.50	1.50	1.35
Seismic	1.20	1.50	1.50	1.50	1.50	1.00
Method factors						
Static	1.000		1.000	1.000	1.000	1.000
Seismic	1.000		1.000	1.000	1.000	1.000

4 STRIPS :

Strip type 1

Width tolerance : 1.50 mm Thickness tolerance : 0.20 mm
 Weight : 1.43 kg/lm
 Hole diameter : 12.70 mm
 -> tolerance : 0.30 mm

5 FACING :

Vertical strip spacing : 0.750 mm
 bottom height : 0.365 mm
 Facing flexibility : 2
 Flexibility : 1 = rigid (ex: full height facing)
 2 = discrete (ex :std panels)
 3 = flexible (ex :steel facing)

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