# GROUND IMPROVEMENT

# &

# STABILIZATION

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# PREFACE

The ground at a construction site is not always suitable for supporting structures such as buildings, bridges, highways and dams. In order to overcome this problem several methods have been employed worldwide to improve engineering characteristics of soils. The methods can be categorized as mechanical stabilization, chemical stabilization, thermal and electrical stabilization, or inclusion of materials such as geosynthetics into the soil or inclusion of rock bolt into rock.  Surface protection also plays important role in preserving the soil characteristics against climate. The role of vegetation and tree on stabilizing soil, especially slope, has been studied. In the absence of tree, confinement system can be used to stabilize the topsoil against erosion.

This book contains eight chapters and each chapter presents the research done by staff of Geotechnical Engineering Division of the Faculty of Civil Engineering UTM over the past decade on the topic of Ground Improvement and Stabilization.

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# CHAPTER 6

# GEOSYNTHETICS REINFORCED RETAINING STRUCTURES

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## 6.1 Introduction

Geosynthetics are being used in Malaysia for a wide range of applicatios e.g. slope stabilization, construction of retaining structures, bridge abutment walls and embankments. As deformable material, geosynthetics have the effect of not only increasing the strength and ductility of soil, but also creating a more flexible structure. In the construction of geosynthetic reinforced soil structures, sucessive layers of free draining soil are compacted between sheets of reinforcement. This procedure results in a stable composite structure that can extend to significant height. Such structures can undergo fairly large deformation without catastrophic collapse and often without their serviceability be affected (Bell et al., 1983). From a mechanical standpoint, reinforcing soil provide the benefit of stiffening earthwork structures without increasing their mass.

Research showed that due to their drainage capability, geosynthetics can be used as reinforcement for cohesionless as well as cohesive soil. The drainage capability of geosynthetics in reinforced soil structures helps reducing the pore-pressure built up at the interface between soil and reinforcement, thus improving the interface shear strength (Fourie and Fabian, 1987). Further investigation (Fabian, 1990) confirmed that non-woven needle punched geotextile used as reinforcement can effectively drain the clay backfill, thus helps its consolidation and results in increase of undrained shear strength. Furthermore, because geotextiles are highly permeable and have drainage capability, when they are used as reinforcement of clayey soil, they can help accelerate the fill consolidation and increase its shear strength (Gofar, 1995).

Soil–geosynthetics interface plays an important role in the reinforced structures. Most laboratory studies dealt with stress transfer mechanism at the interface between the soil and the inclusions. Direct shear and pullout tests were performed for this purpose. If the reinforcement is in the form of a sheet that completely separates the soil above and below it, then the transfer mechanism is strictly friction. Hence the interface resistance can be readily determined by the direct shear test. However, if the reinforcement contains a large number of transverse elements such as geogrids, then the transfer mechanism would be the combination of friction and passive resistance and should be determined by pullout test. Pullout of reinforcing strip is a three dimensional phenomenon in which the soil dilatancy plays a major role.

Field studies were focused mostly on the magnitude and distribution of lateral stresses in the soil fill and the tensile stresses developed in the reinforcement. The magnitude and distribution of lateral force in the reinforcement are affected by factors such as the construction procedure and the stiffness of the foundation soil. Staged or combined incremental and immediate loading using strut is considered effective to reduce the post construction lateral deformation of the wall, especially on softer foundation, by allowing partial active yielding to take place during construction. The presence of water is not a problem, in reinforced soil structure since the reinforcement has high permeability and good drainage capability.

Since geosynthetics are made of polymeric material, creep is potential problem for their long term performance as reinforcement. However, research results suggest that problem might be less critical than anticipated because for geotextile, the soil confinement at the soil-fabric interface has the effect of increasing the tensile modulus of the inclusions.

## 6.2 Mechanism of Reinforcement

### *6.2.1 Strength Increase*

The basic mechanism of reinforced soil involves the generation of frictional forces at the soil-reinforcement interface. These forces induce an increase of confining pressure in the soil in the direction parallel to the reinforcement, thus restricting the lateral strains of the soil. This can be referred as an apparent anisiotropic cohesion of the composite material (Figure 6.1). As strain increases, the frictional resistance between soil and geosynthetics is mobilized, thus the soil stress deviator does not increase as much as it would without reinforcement and the apparent shear strength and axial stiffness of the soil are increased. The magnitude of strain required to develop the interface friction can be observed on the stress-strain curve when the stiffness of the reinforced soil begin to differ from the stiffness of unreinforced one.

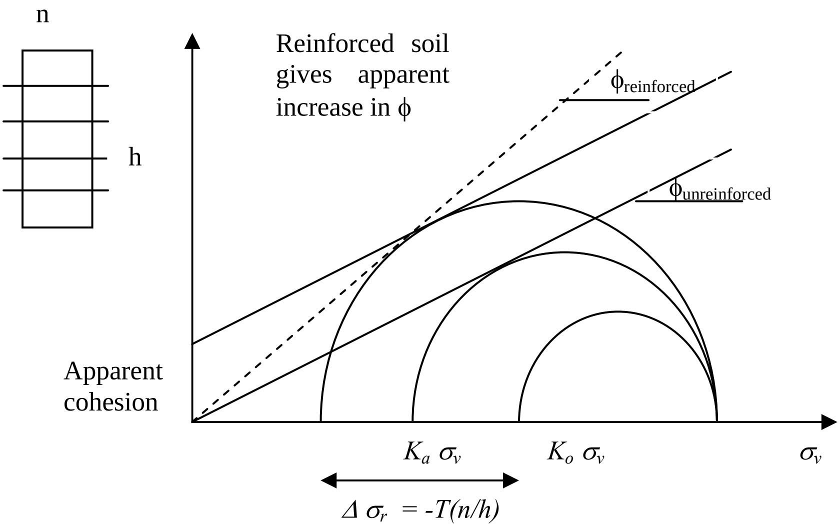



**Figure 6.1:** Basic mechanism of soil reinforcement and failure strength envelope (After Schosser and de Buhan, 1990)

The peak strength of the reinforced specimens was obtained for ratios of the minor to major boundary stresses less than the coefficient of earth pressure at active yielding state of unreinforced specimens (Figure 6.1). These observations were explained qualitatively using the concept of “enhanced confinement”. When tensile forces are induced in the inclusion by the soil deformation towards active yielding, the reinforcement tends to restrict this lateral (or radial) deformation of the specimen and its yielding. This results in apparent confining pressure at failure less than given by the active state of earth pressure. These effects were more pronounced when the number of reinforcement layers was increased. Investigations by Fabian (1990) and Ling and Tatsuoka (1993) have shown that at the same level of deformation, the strength ratio between geotextile reinforced wall and the unreinforced wall was about 1.8.

The response of geosynthetics reinforced structures was modeled by finite element formulations in which the reinforced soil system is represented either by a composite material (composite formulation) or by modeling individual component (discrete formulation). The discrete formulation requires more refined meshes be used since each component and interface must be presented, yet it provides an assessment of stress and strains of each element in the structure as well as the localized deformation near the edges of the reinforced soil mass. On the other hand, the composite formulation allows only the assessment of boundary deformation. Composite formulations are developed based on an extension of continuum concepts to a macro level of observation whereby the reinforced soil mass is treated as an equivalent anisotropic, homogeneous material.

### *6.2.2 Soil-Reinforcement Interaction*

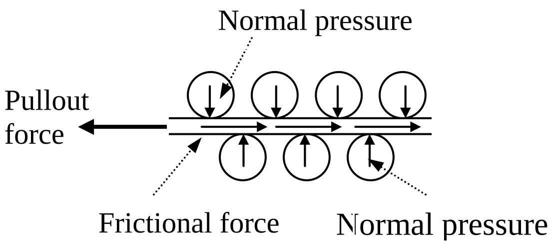
The interface mechanism in reinforced soil is influenced by the degree of irregularity that exist along the interface (geometric pattern and surface properties of the inclusions) as well as grain size and frictional characteristics of the fill material. It is also affected by the initial state of compaction of the soil fill and drainage capability of the fabric.

Depending on the type of geosynthetics inclusions, stresses are transferred from the soil to the reinforcement by the friction and/or passive resistance (Figure 6.2). Friction occurs when the stresses are transferred through shearing along the interface. Passive resistance exists when the stresses are transferred by bearing of the transverse inclusions elements against the soil. The frictional mechanism is dominant for linear and planar reinforcement such as steel strips, geotextiles and the longitudinal bars of geogrids. This mechanism is usually representated using an interface friction angle and adhesion. Passive resistance is the dominant mechanism for reinforcements containing transverse elements such as bar mats, grids and wire meshes.

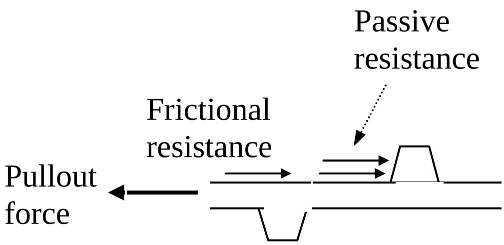
Direct shear tests are used to estimate the soil-reinforcement interface friction angle and cohesion (Collios et al., 1980, and Ingold, 1982). The frictional resistance is computed based on the average stress required to produce sliding of the soil against reinforcement under a given applied normal stress. Collios et al. (1980) introduced a concept of contact efficiency for the interface resistance developed between soil and reinforcement, defined as

$$E_c = (c_\alpha/c)\ 100\ \%$$
$$E_\phi = (\tan\delta/\tan\phi)\ 100\ \% \qquad (6.1)$$

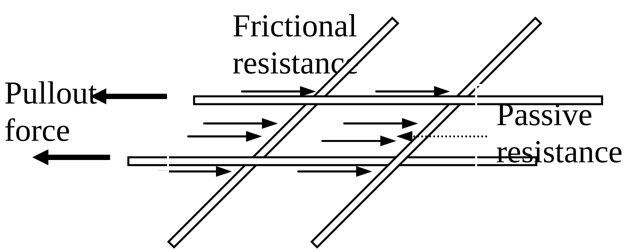
where $E_c$ and $E_\phi$ are the efficiency with respect to cohesion and friction respectively, $c_\alpha$ is the interface adhesion of soil to geosynthetics, $c$ is the cohesion of the soil, $\delta$ is the interface friction angle of soil to geosynthetics, and $\phi$ is the internal friction angle of the soil. The procedure of the direct shear test for geosynthtics reinforced soil is standardized in ASTM D5321-08.




(a) Pure friction (Direct shear test)




(b) Friction and passive resistance (pullout test)




(c) Friction and passive resistance (pullout test)

**Figure 6.2:** Stress transfer mechanism at the soil-reinforcement interface (after Christopher et al., 1989)

The anchorage resistance of soil reinforcing inclusions can also be modeled using pullout testing. The test is performed by pulling the geosynthetic inclusion at a controlled rate of displacement until a peak of the pullout resistance is reached. The pullout resistance is obviously dependent on the normal force applied to the surrounding soil that mobilizes shear forces on both sides of the inclusions. Measurement of deformations during the pullout test does not allow in general the determination of the tensile force distribution along the inclusion unless the displacement along the inclusion is also monitored. In this test, the, movement of the geosynthetic is due to its stretching and progressive mobilization of the interface strength.

It was observed (Kate et al., 1988) that under given normal stresses, the failure shear stress and friction coefficient of a sand-fabric interfaces obtained from pullout testing are less than those obtained from modified direct shear tests. On the other hand, the anchorage or pullout resistance of geogrids can far exceed their interface direct shear strength (Koerner, et al., 1989). This is because the pullout resistance of geogrids is the combination of the resistance to shearing along the top and bottom surfaces of longitudinal and transverse ribs and the passive resistance mobilized against the front face transverse ribs, where the soil can locally reach a passive yielding state. A detailed explanation of the interface mechanism for geogrid reinforced soil was proposed Jewell et al., (1984). The standard procedure for the pullout test is in ASTM D6706-01 (2007).

Interaction between the soil and the reinforcement in the finite element formulation is modeled using interface elements. According to Rowe and Ho (1988), any modeling of the interface behavior must consider the possible failure mechanisms as observed in the direct shear or pullout tests. If the reinforcement is in the form of a sheet that completely separates the soil above and below reinforcement, the interface resistance can be readily determined by direct shear test’s results (Rowe et al, 1985). In this case, provision for slip at the interface is the same irrespective of the mechanism of failure (i.e., direct shear or pullout). However if the reinforcement consists of geogrids (with large openings as compared to the grain size of the soil) or reinforcing strips, then special care is required to correctly model the failure mechanism. For strip reinforcement, independent movement of the soil above and below the plane of reinforcement can only occur during direct shear mode of failure.

### *6.2.3 Mobilization of Lateral Earth Pressure*

Fundamental to soil reinforcing in retaining structures is the role played by reinforcement with regard to soil yielding and mobilization of lateral earth pressures. Laboratory tests on the effect of reinforcement on lateral earth pressure were conducted using the triaxial apparatus (Holtz et al., 1982, and Wu, 1989) for a variety of tensile reinforcements ranging from flexible and extensible geosynthetics to stiffer metal inclusions. The specimen was subjected to axial compression perpendicular to the reinforcement layers. In addition, scale models were developed for unreinforced and reinforced sand-backfilled structures (McGown et al., 1988).

Field observations performed on geosynthetic reinforced structures mostly dealt with the magnitude and distribution of lateral earth pressures developed in the reinforced backfill and stress transferred from soil to geosynthetics. The confinement pressure resulted from lateral expansion of soil as a consequence of the yielding of the structure. The movement of the wall generates tension in geotextiles and the interaction between the soil and the geotextiles induces a redistribution of compressive stress in the soil mass and reduces the strain in the soil system.

The tensile stress in geosynthetics is obtained by measuring the strain developed along reinforcement layers and backcalculating them using the tensile modulus of the geosynthetics. Distribution of tensile strain showed its maximum at some distance from the front edge and decreases along the anchorage zone (e.g., Fannin & Hermann, 1990, and Balzer et. al., 1990). The locus of maximum tensile stress along a reinforcement layer is assumed to represent a critical failure surface.

The most prominent factor influencing the distribution of strain in polymeric reinforcement is the facing type (Bathurst et al, 1987). Their investigations suggested that wall with rigid facing panels may be subjected to additional vertical loads due to the settlement of the reinforced fill. This has been accounted for in the numerical study performed by Gofar (1994). Observation the influence of facing elements by Tatsuoka et al. (1986) and Gofar (1994) showed that the use of continuous rigid facing reduces deformation.

Construction procedures including fill placement and compaction force as well as connections between the reinforcing elements and the facing are important factors in geosynthetics reinforced walls (Richardson & Behr, 1988, Gofar, 1994). For extensible reinforcement, it is particularly important to utilize incremental construction methods to allow for the lateral strain to develop during construction, so that post-construction lateral stresses are minimized. Chou, et al. (1993) showed that much larger lateral wall displacements are developed under service load when the wall is built on soft clay foundation.

## 6.3 Design Approach

Optimum design of a reinforced wall structure should satisfy two conflicting requirements, i.e.: (1) geosynthetics have to develop high tensile strain to mobilize its confinement effect, and (2) the wall movement has to be limited to ensure satisfactory serviceability of the structure. The interaction between the tensile reinforcement and the soil fill depends on the strain developed within the structure. The reinforced fill must undergo sufficient deformation to mobilize its shear strength and the necessary interface interaction with the reinforcement. On the other hand, the serviceability criterion requires fairly low design tensile force adopted in order to maintain the deformation level.

The design analyses of reinforced soil structures are usually based on modified limit equilibrium approaches accrued from the analysis of conventional cantilever or gravity walls or unreinforced slopes. The methods are presented in a number of textbook (e.g Craig (2004), Gofar & Kassim (2005) and Koerner (2005)). Reinforced soil walls design requires the determination of the geometric and reinforcement characteristics to prevent external and internal failure. There are two criteria of stability: (1) the geometric and reinforcement requirements to prevent external failure, and (2) reinforcement resistance and length necessary to prevent internal or local failure.

The external stability analysis of the structure can be verified by considering a rigid gravity structure. As with classical unreinforced retaining structure, the external stability of reinforced soil structures is verified with respect to sliding on the base, overturning, bearing capacity failure of the foundation soil and deep-seated slope instability (rotation slip-surface of slip along a plane of weakness) (Figure 6.3). In such analysis, the reinforced soil structure is considered as a rigid body. The stability analysis for external failure can be referred to any textbook in Geotechnical Engineering. Because of the flexibility of the reinforced soil structure, the suggested factor of safety for external failure is lower than those used for reinforced concrete cantilever or gravity walls, i.e., 2.0 and 1.5 for overturning and sliding along the base respectively.

For internal stability, the local equilibrium for each soil layer around an element of reinforcement and the overall equilibrium of the wedges of reinforced soil are considered. The local stability analysis includes two possible modes of failure, i.e., breakage or excessive elongation and pullout of the reinforcement. Each mode of failure can be analyzed using the maximum tensile force developed at the intersection with the critical slip surface inside the reinforced fill. The length of reinforcement extending beyond this line is the available anchorage length that resists pullout. Hence, the internal stability analysis require a definition of an assumed critical slip surface within the reinforced mass (Figure 6.4).

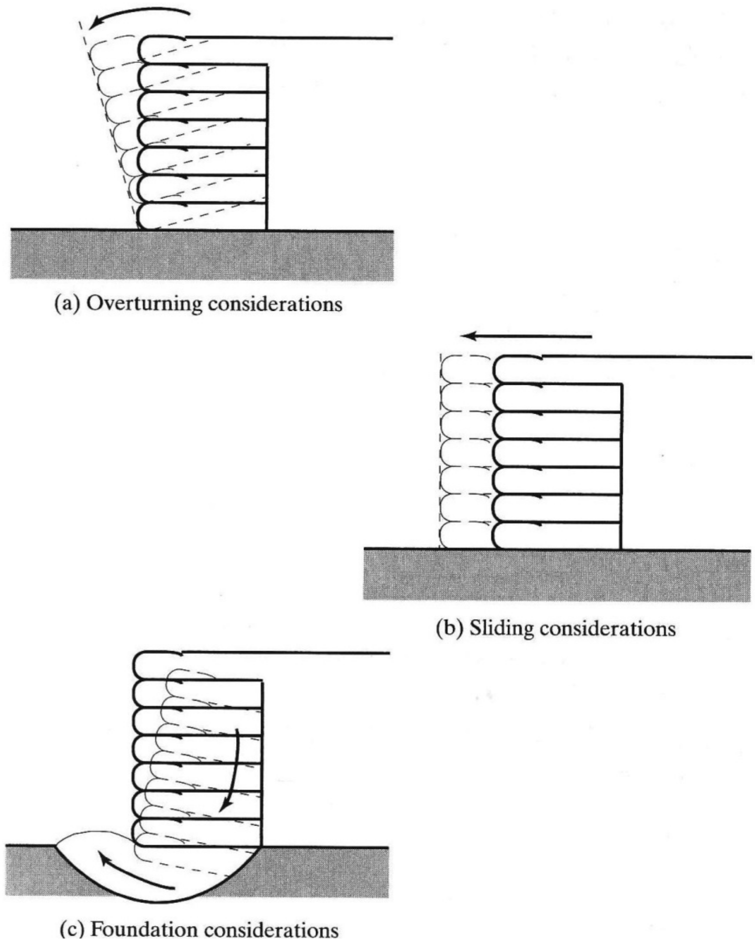



**Figure 6.3:** External failure mechanism in reinforced soil structures (after Koerner, 2005)

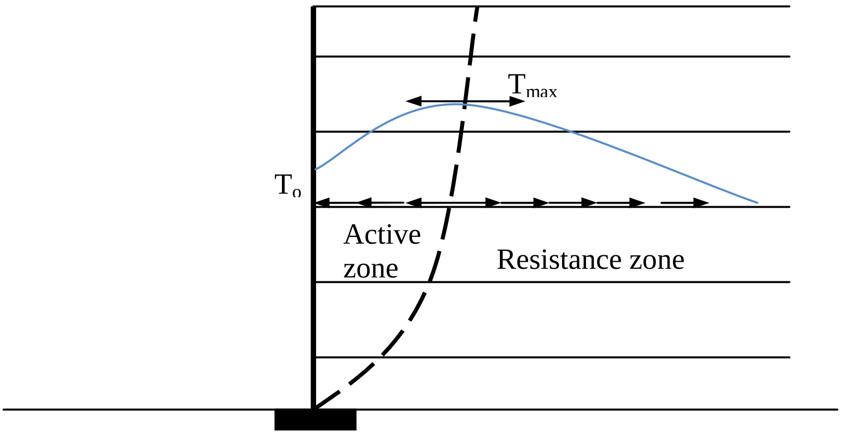



**Figure 6.4:** Location of critical slip plane or locust of maximum tensile force in reinforcement

In the current design procedure, different failure planes were used with respect to different types of reinforcement because the response of the reinforced structure depends on the extensibility of the reinforcing inclusions. There is no general form of slip plane which can be used for any types of reinforcement.

For inextensible reinforcement, a method was first introduced by Steward (1977, revised in 1982) known as US Forest Service Method. The method assumes that the wall is in ‘at rest’ condition ($K_o$ analysis) and will fail along Rankine failure plane.

For geotextile or extensible reinforcement, the failure mechanism of a structure reinforced with extensible inclusions resembles Coulomb's failure plane which starts at the toe and pas through a line inclined at the angle of ($45+\phi/2$) to the horizontal, where $\phi$ is the angle of internal friction of the fill material. (Figure 6.5a). The method, known as Tie Back wedge, was first introduced by Murray (1980). The movement of the wall is assumed to start at the top of the wall generating an active state stress throughout the reinforced wall.

Experiments performed at Laboratoire Central des ponts et Chaussees (LCPC), France (Schlosser & Long, 1974) for

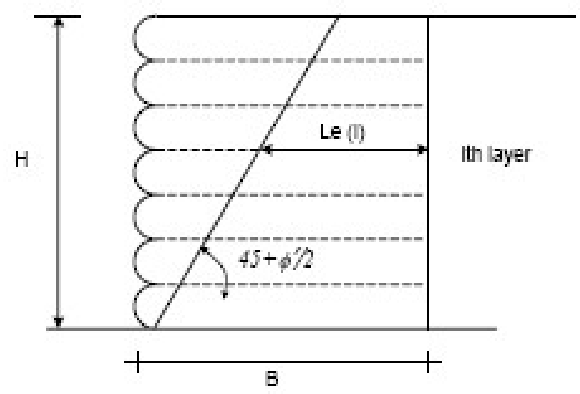
Reinforced Earth retaining structures on the basis of small scale models and prototype full scale structures showed that the locus of maximum tensile forces is essentially different from the classical Coulomb's failure plane. It is a curved surface that can be approximated by a bi-linear failure plane (Figure 6.5b). Back-calculated coefficients of lateral earth pressure varied from $K_o$ at the top of the walls to a value less than $K_a$ in the lower section of the walls. The observed shapes of the critical surfaces suggest that the movement of the wall start at the toe by rotation around the top. These findings led to the development of Coherent Gravity Method for inextensible reinforcement (Schlosser, 1978). Slight modification to this failure plane was proposed by Juran and Schlosser (1978) and the method was followed in the textbook by Craig (2005).

Reinforcing layers intersecting the potential failure surface are assumed to increase the resisting force or moment based on their tensile capacity and orientation. The tensile capacity of the reinforcing layers is taken as the minimum of its allowable pullout resistance behind the potential failure surface and its allowable design strength.

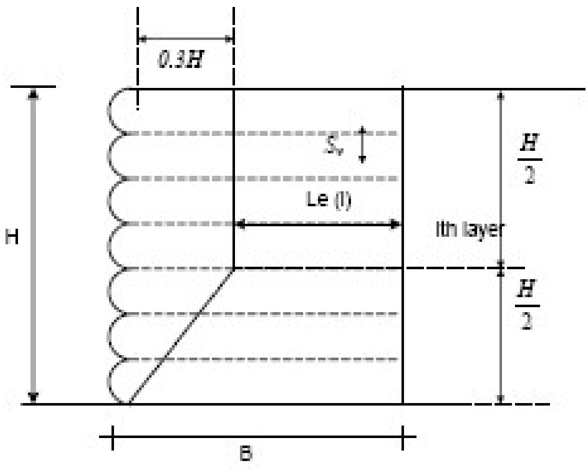
The internal stability analysis includes the computation of tensions developed in reinforcing layer. These forces should not exceed the tensile resistance of the reinforcement and the pullout anchorage capacity at the interface between the soil and the reinforcement. The allowable tensile resistance in the reinforcement ($T_{all}$) is

$$T_{all} = \frac{T_{ult}(CRF)}{F_D F_C F_S} \tag{6.2}$$

in which $T_{ult}$ is the ultimate strength of the reinforcement, $CRF$ is the creep reduction factor, and $F_D$, $F_C$, and $F_S$ are the reduction factors that account for chemically and/or biological durability, construction damage, and the uncertainty in the determination of the reinforcement strength.




a. Tie back wedge method




b. Coherent gravity method

**Figure 6.5:** Assumed failure planes for analysis of reinforced soil structure

The allowable pullout force ($P_a$) at the soil reinforcement interface should be less than the anchorage resistance:

$$P_a \leq \frac{P_r R_c}{FS_{po}} \tag{6.3}$$

where $P_r$ is the available pullout resistance for particular type of reinforcement, $R_c$ is the coverage ratio, and $FS_{po}$ is the prescribed factor of safety against pullout.

The pullout resistance of the reinforcement ($P_r$) is mainly a function of the type and the stiffness of the reinforcement and the interaction mechanism at the interface (Christopher et al., 1989). For linear reinforcement, the pullout resistance can be estimated as:

$$P_r = 2 f^* \alpha \sigma_v L_e \tag{6.4}$$

where $\sigma_v$ is the effective vertical stress, $L_e$ is the available anchorage length, and $a$ is the reinforcement effective unit.

The pullout resistance factor ($f^*$) is the combination of friction and passive bearing resistance at the soil-reinforcemeat interface,

$$f^* = f_q \alpha_\beta + K \mu^* \alpha_f \tag{6.5}$$

where $f_q$ is the bearing capacity factor for embedment, $\alpha_\beta$ and $\alpha_f$ are structural geometric factors for passive resistance and friction respectively, $K$ is the ratio of actual normal stress to the vertical stress, $\mu^*$ is the apparent friction coefficient, and $\alpha$ is the scale effect correction factor. The scale effect correction factor ($\alpha$) is defined as

$$\alpha = \frac{\tau_{av}}{\tau_p} = \frac{\tan \varphi_m}{\tan \varphi_{peak}} \tag{6.6}$$

where $\tau_{av}$ and $\tau_p$ are the average and ultimate interface lateral shear stresses mobilized along the reinforcement, while $\phi_m$ and $\phi_{peak}$ are the average interface friction angle and peak interface friction angle mobilized along the reinforcement. The summary of the pullout resistance for different type of reinforcing element for used in practice is summarized in Figure 6.6.

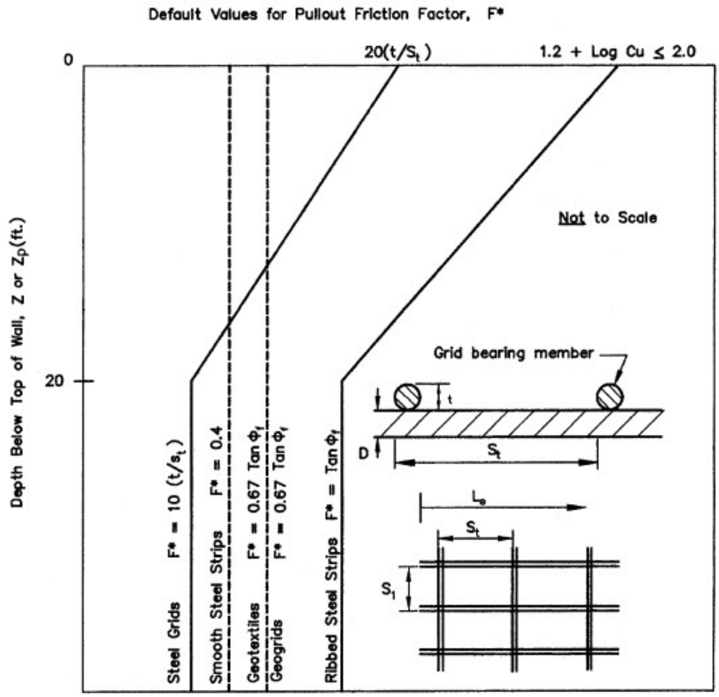



**Figure 6.6:** Pullout resistance factor for different types of reinforcement

The lift thickness ($S_v$) for geosynthetic walls varies with the strength of the inclusion and the maximum lateral earth pressure developed in the backfill,

$$S_v = \frac{T_u}{\sigma_h FS} = \frac{T_u}{\left(K\gamma_b zFS\right)} \tag{6.7}$$

where $\gamma_b$ is the unit weight of the back-fill, $z$ is the depth of the reinforcement layer, $FS$ is the prescribed factor of safety, and $T_u$ is the ultimate tensile strength. For Reinforced Earth walls, the lift thickness is governed by the size of the facing panel while for other types of reinforcement, the lift thickness can vary along the wall height.

## 6.4 Analytical and Empirical Analysis of Factors Affecting the Lateral Stress in Geosynthetic Reinforced Structure

Rational design procedures of reinforced soil structure are based on the fundamental understanding of the interaction between soil and geosynthetic reinforcement. As deformable material, geosynthetics respond differently from metal strips (Reinforced Earth) in their function as reinforcing elements. The deformability of geosynthetics can vary over six orders of magnitude. Theoretically the stiffness of reinforcing inclusions influences the stress state both locally (at the interface) and globally (reinforced mass a whole). Furthermore, construction procedure is known to have an effect on the development of lateral earth pressure. Therefore, a good approach for analyzing the reinforced soil structures should consider the deformability of the geosynthetics, interface mechanism and the construction procedure involved in the construction of the structure. As a result, the design analysis would allow for modulation relative to stiffness of the selected geotextiles and the level of acceptable deformation.

### *6.4.1 Effect of reinforcement stiffness on lateral earth pressure*

Christopher et al. (1989, 1993) proposed that the lateral earth pressure in the reinforced soil to vary with the anticipated movement of the wall. Therefore, the lateral earth pressure coefficient varies as a function of the global stiffness of the wall and the type of reinforcement as shown in Figure 6.7. The reinforced soil mass is assumed to approach an active yielding state at a depth of 6 m for all types of reinforcements except bar mats and welded wire in which the earth pressure coefficient varies from passive values to the 'at-rest' value at depth of 6 m. Wall reinforced with very flexible reinforcement such as geotextile and woven meshes will reach an active yielding condition along its height.

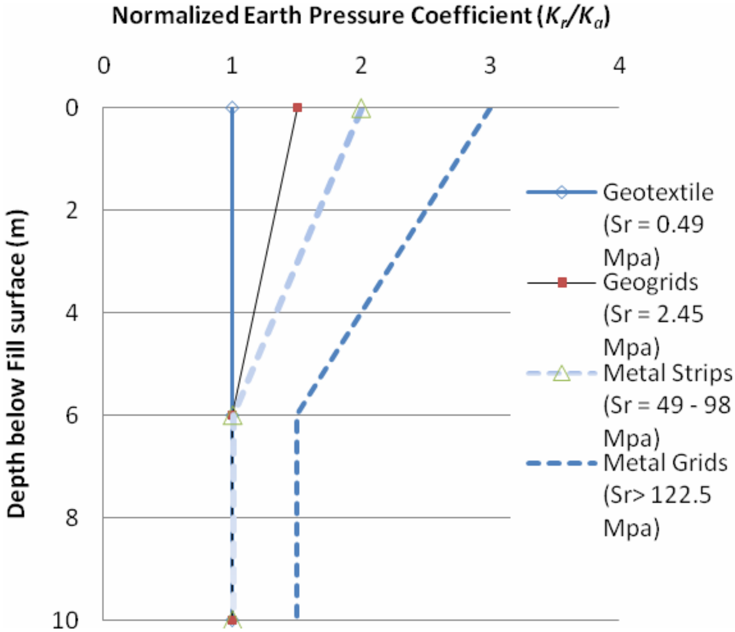



**Figure 6.7:** Relationship between $K/K_a$ and the stiffness ratio for the design of reinforced soil structures subjected to body force

*6.4.2 Combined Effect of stiffness of reinforcement and interface mechanism on lateral earth pressure*

As discussed in the previous section of this chapter, the benefits obtained from reinforcing soil originate in the generation of frictional forces at the soil-reinforcement interface and the tensile resistance of the reinforcement. The developed interaction increases the soil confining pressure, thus restricting the lateral strain in the soil and increasing its internal stability. Large lateral strain in the soil and strain in the inclusions are necessary for active yielding condition to develop.

With small strain, the soil remains close to the 'at-rest' condition. For a soil element subjected to a uniform boundary stress, this condition is illustrated by a Mohr circle shown in Figure 6.8a for unreinforced condition. In the reinforced soil wall, movement is restrained by the confinement at the soil-reinforcement interface. This effect can be represented by an additional increment of lateral stress acting on the soil, $\Delta\sigma_r$. Therefore, the coefficient of lateral stress can be computed as

$$K_r = \frac{\sigma_{H1}}{\sigma_V} = \frac{\sigma_h + \Delta\sigma_r}{\sigma_V} \tag{6.8}$$

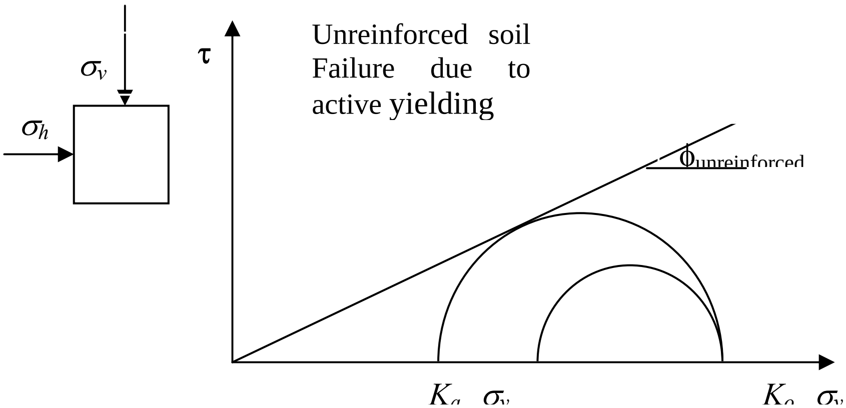
where $\sigma_v$ and $\sigma_h$ are the vertical and horizontal stresses for unreinforced case, $\sigma_{HI}$ is the horizontal stress for reinforced soil, and ,$\Delta\sigma_r$ is the horizontal stress increment due to confinement. This condition is illustrated in Figure 6.8b.

The confining pressure ($\Delta\sigma_r$) depends on the stiffness and the density of the reinforcing element as well as the stress transfer mechanism at the interface. The confining pressure for a perfectly adherent interface is

$$\Delta\sigma_r = -T\frac{n}{h} \tag{6.9}$$

where $T$ is the maximum tensile force in the reinforcement, and ($n/h$) is the density of the reinforcement (i.e., the number of inclusion by unit height of the soil).

a. Unreinforced soil




b. Reinforced soil

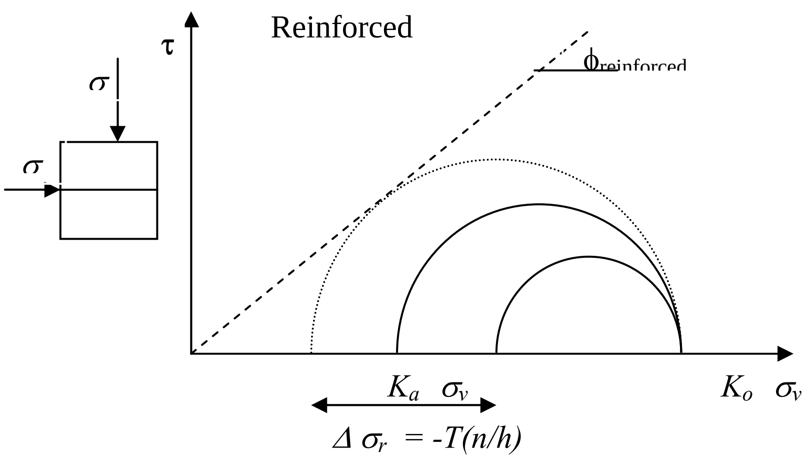



**Figure 6.8** Mohr circle for unreinforced and reinforced soil

The elastic solution for the relative improvement due to the enhanced confinement is given by equation 6.10 a for plane stress condition and equation 6.10b for plane strain condition (Bourdeau, 1991):

$$\frac{\Delta\sigma_r}{\sigma_v\left(K_o - K_a\right)} = \frac{1/E}{\dfrac{1}{E} + \dfrac{h}{nE_r}} \tag{6.10a}$$

$$\frac{\Delta\sigma_r}{\sigma_v\left(K_o - K_a\right)} = \frac{\left(1-\mu^2\right)/E}{\dfrac{\left(1-\mu^2\right)}{E} + \dfrac{h}{nE_r}} \tag{6.10b}$$

where $E$ is the Young's modulus of the soil, $E_r$, is the tensile modulus of the reinforcement, and $\mu$ is the Poisson's ratio of the soil. In terms of the ratio between, the coefficient of lateral stress in the reinforced soil ($K_r$) to that in the unreinforced soil at yielding ($K_a$), Equations 6.10 can be written as:

$$\frac{K_r}{K_a} = 1 + \frac{E_r \ \sin\phi}{E_r \ + \ S_v E} \tag{6.11a}$$

$$\frac{K_r}{K_a} = 1 + \frac{\left(1-\mu^2\right)\ \sin\phi \ E_r}{\left(1-\mu^2\right)E_r \ + \ S_v E} \tag{6.11b}$$

where $\phi$ is the soil friction angle, and $S_v$ is the spacing between two consecutive reinforcement layers ($h/n$).

Gofar (1994) performed finite element analysis for two different types of uncompacted soil and six different types of reinforcement (Table 6.1) to simplify the above equation.

**Table 6.1:** Stiffness of the reinforcing element considered in the study

| Type of reinforcing element | Tensile modulus (kN/m) |
|---|---|
| Non-woven Geotextile | 25 |
| Non woven Geotextile | 118 |
| Woven Geotextile | 500 |
| Geogrids | 2000 |
| Steel Strips | 4000 |
| Bar Mats | 8000 |

The effect of reinforcement density on the enhanced confining pressure was studied by varying the vertical distance between two reinforcement layers ($S_v$) from 0.4 to 1.2 m. Note that the confinement mechanism also depends on the stress transfer at the interface between the soil and the reinforcement, therefore study was made for different mechanism of interface mechanism. She arrives at an empirical Equation 6.12 for normalized coefficient of lateral earth pressure with the stiffness of reinforcement and apparent coefficient of friction.

$$\frac{K_r}{K_a} = 1.59\left[\sin\phi\left(\frac{E_r}{E_r + S_v E}\right)f\right]^{0.076} \qquad (6.12)$$

### *6.4.3 Effect of Construction Procedure and Compaction Effort*

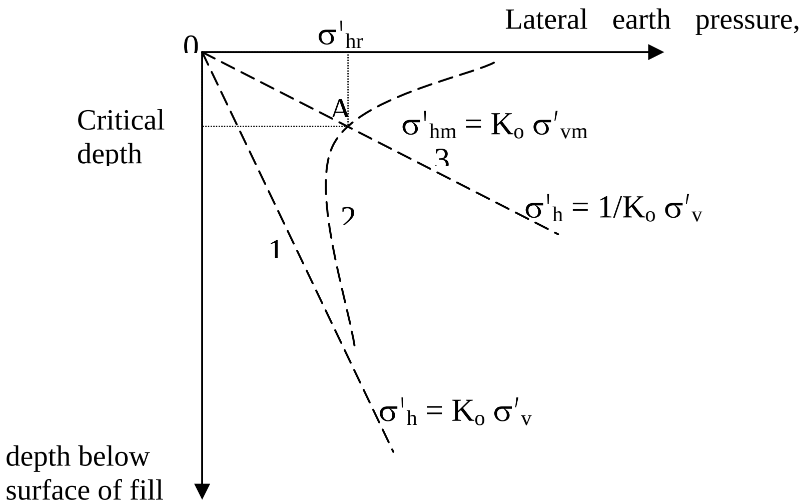
When a reinforced soil structure is being constructed, the soil fill is normally placed in successive lifts and compacted to optimize its strength and compressibility properties. However, initial distortion of the facing resulted by the fill compaction can generate unexpected tensile form within the not be considered herein.

During compaction, the vertical stresses are temporarily increased by an increment $\Delta\sigma_v'$. The largest stress increments occur at shallow depth immediately beneath the compaction equipment. Broms (1971) assumes that at shallow depth, after being compacted under $K_o$ condition, a soil element will follow an unloading path consecutive to the removing of the compaction effort at the slope of $1/K_o$ yielding to a final horizontal stress $(\sigma_{hf})$ (Figure 6.9a). At greater depth, where the change in stress beneath the compaction equipment is small, the soil will not reach the same unloading line. There, the final horizontal stress after compaction is assumed to remain equal to the stress when the compaction effort ($\sigma_{hc}$) was applied. The resulting pressure distribution is shown in Figure 6.9a by curve 2 below a critical depth $Z_{cr}$ at which the maximum pressure ($\sigma_{hrm}$) occurs and by line 3 above this level. This approach was extended by Ingold (1979) to the case of smooth walls that yields during compaction by substituting $K_a$ for $K_o$.

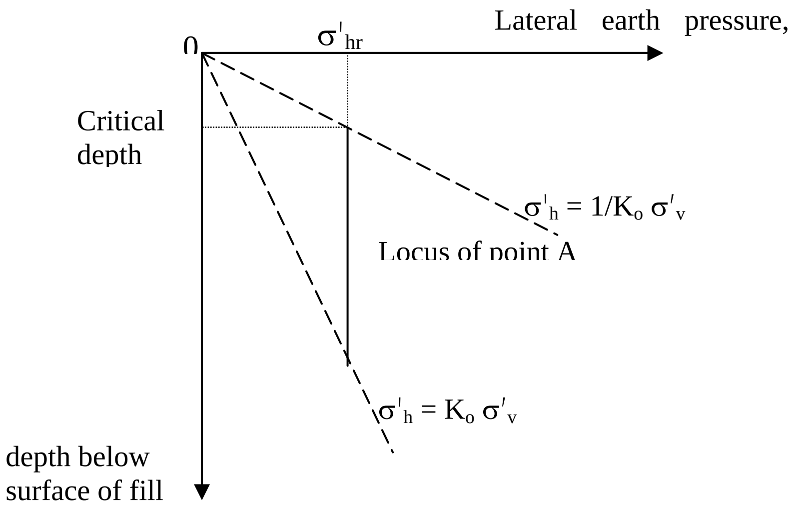
The result of finite element analysis (Gofar, 1994) shows that: the lateral earth pressure coefficient in compacted fill is strongly related to the compaction force and the applied pressure at that point. This result agrees well with the available analytical solution. The empirical relationship between the ratio of the lateral earth pressure in compacted soil to the lateral earth pressure coefficient at initial stage can be written as

$$\frac{K_c}{K} = 1 + 1.1\left(\frac{\Delta\sigma_{hc}}{q + \sigma_{ovb}}\right) \tag{6.13}$$

where $K_c$ is the lateral stress coefficient after compaction, $K$ is the initial coefficient of lateral stress for the uncompacted case, $\sigma_{hc}$ is the compaction force which varies with depth ($z$), $q$ is the applied load, and $\sigma_{ovb}$ is the overburden pressure which also varies with depth ($z$).




b. Theoretical




a. Simplified distribution

**Figure 6.9:** Stress distribution in granular soil during compaction (after Clayton et al. 1993)

Equation 6.13 shows that the effect of compaction is generally limited to shallow depth, and the effect becomes less sensitive as the surface applied load is increased. In the following development, it will be assumed that the results represented in Equation 6.13 obtained for unreinforced soil can also be extended to the reinforced soil.

### *6.4.4 Summary*

Table 6.2 summarize the coefficient of lateral earth pressure to be used for the analysis of unreinforced and reinforced soil structures based on the numerical study performed by Gofar (1994). The coefficient of lateral earth pressure proposed herein could be used in combination of the current design procedure when the effect of the stiffness of reinforcement, interface mechanism and compaction are to be considered.

**Table 6.2:** Summary of formulation lateral earth pressure coefficient for soil structures

| Case | $K$ | Equation/comment |
|---|---|---|
| Unreinforced wall | | |
| Uncompacted | $K_a$ | at yielding |
| Compacted | $K_{ac}$ | $K_{ac} = K_a\left[1+1.1\left(\frac{\Delta\sigma_{hc}}{q+\sigma_{ovb}}\right)\right]$ |
| Reinforced Wall | | |
| Uncompacted | $K_r$ | $K_r = 1.59\,K_a\left[f\sin\phi\left(\frac{E_r}{E_r+S_vE}\right)\right]^{0.076}$ |
| Compacted | $K_{rc}$ | $K_{rc} = K_r\left[1+1.1\left(\frac{\Delta\sigma_{hc}}{q+\sigma_{ovb}}\right)\right]$ |

The use of the equations with the current design procedure was validated with data obtained from full scale model test wall performed by Karchafi and Dysli (1993) and the comparison of the predicted value and actual behavior could be referred to Gofar (1994).

## 6.5 Conclusions

Most design methods for reinforced soil structure were developed empirically and specifically for a certain type of reinforcement. The fundamental differences in these design approaches are the assumptions of the lateral earth pressure and the load transfer mechanism at the soil-reinforcement interface, both are related to the increase of confining pressure resulting from reinforcement.

The variation of lateral earth pressure coefficient for reinforced wall depends on the deformability and the density of the reinforcing element and the friction at the soil reinforcement interface. In this case, the coefficient of lateral stress is larger than $K_a$ due to the enhancement of confining pressure. Note that the enhancement of confining pressure at the soil-reinforcement interface is the basic mechanism of soil reinforcement.

The variation of lateral earth pressure coefficient in unreinforced and reinforced wall is also depends on fill compaction. Construction procedures, including soil placement and compaction, reduce the concentration of vertical pressure below the loaded surface. However, initial distortion of the facing can results from these compaction efforts. This distortion generates higher forces transferred to the reinforcement.

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# CHAPTER 7

# Reinforcement Mechanisms of Rock Bolt

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## 7.0 Introduction

Failure in rock mass is implied as the incapability of a rock mass to support its own weight. Installation of stabilising methods ensures the inherent strength of the rock is maintained before excessive failure takes place. The effectiveness of any stabilising method usually depends on the type of instability in rock and the stabilising mechanisms of the selected method. Various methods are currently available for stabilising unstable rock and usually more than one method of stabilisation are adopted to achieve the required stability.

Methods for rock stabilisation are divided into two types namely, support systems and reinforcement systems (Windsor and Thompson, 1993). The former includes shotcrete and wire-mesh in which, stabilising elements are installed on the rock surface. The latter comprises reinforcing elements installed in the rock that includes rock bolts and dowels. A rock bolt is a steel bar, which is inserted into a hole drilled in the rock. Despite of its many varieties, all rock bolts have in common the following elements: a steel bar (shank), an anchoring device (resin or grout) at one end, and a tensioning device (bearing plate and nut) at the other, as shown in Figure 7.1 (Brady and Brown, 1985). The reinforcement bar helps to mobilise the inherent strength of the rock mass by modifying its internal strength and deformation characteristics. However, the effectiveness of the bolt in mobilising the inherent