

FORENSIC STUDY ON GEOSYNTHETIC REINFORCED SOIL WALLS WITH  
CORNER TURNS

By

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## **ABSTRACT**

### **FORENSIC STUDY ON GEOSYNTHETIC REINFORCED SOIL WALLS WITH CORNER TURNS**

Geosynthetic Reinforced Soil (GRS) involve the use of reinforcement inclusions to enhance the engineering properties of the soil mass. In doing so, the backfill soil is internally retained, which dispenses the primary need of conventional concrete elements, such as those adopted in a gravity retaining wall system. There were discrepancies exhibited on the facings of a GRS structure when there are sharp turns at the corner edges. This stemmed the need to examine the failure mechanisms associated with the GRS structure in order to develop an understanding of what drives and resists the mechanisms. In doing so, the soil and geosynthetic properties standalone were investigated, along with the soil-geosynthetic interaction, compaction effects and the facing contribution. A numerical analysis model was carried out using a numerical software TNO Diana. The compression and tension effects on the modular blocks used as facing elements in such walls were examined, and the patterns had corroborated well with an empirical prototype model, which indicates that the numerical analysis is suitable for inspecting the stress/strain values manifested as a design procedure.

## ÖZET

### **KÖŞE DÖNÜŞLÜ GEOSENTETİK DONATILI ZEMİN DUVARLAR ÜZERİNE TEKNİK ÇALIŞMA**

Zemin kütesinin mühendislik özelliklerine göre, duvar kesitlerinden oluşan yatay basıncı almak üzere geosentetik donatılar zeminlerde kullanılmaktadır. Bu durumda zemin içinde oluşan gerilmeler donatılar yardımı ile taşınır ve bu durumda geleneksel ağırlık tipi istinat duvarlarında olduğu gibi beton sistemler kullanılmasına gerek kalmaz. Fakat, bir geosentetik donatılı zemin yapısının köşelerinde keskin dönüşler olduğunda yapının yüzeylerindeki bloklarda istenilmeyen bazı durumlar gözlemlenebilmektedir. Bu mekanizmaya hangi kuvvetlerin yol açtığı ve karşı koyduğunun anlaşılmasına yönelik bir fikir elde edebilmek için köşe içeren geosentetik donatılı zemin yapılarının göçme mekanizmalarının araştırılmasına gereksinim duyulmuştur. Bu amaçla hem zemin ve geosentetik özellikleri hem de zemin-geosentetik ilişkisi, sıkıştırma etkileri ve duvar yüzeyinin katkısı araştırılmıştır. TNO Diana olarak bilinen bir numerik program kullanılarak bir numerik analiz modeli oluşturulmuştur. Modüler blok duvarların ön cephesini teşkil eden bloklar üzerindeki basınç ve çekme etkileri araştırılmış ve bir deneysel prototip model ile bu numerik analiz sonuçları doğrulanabilmektedir. Bu da numerik analizde belirtilen basınç/deplasman değerlerini araştırmada uygun bir tasarım prosedürü olduğunu göstermektedir.

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## LIST OF SYMBOLS

|             |                                                                                                      |
|-------------|------------------------------------------------------------------------------------------------------|
| 2D          | 2-Dimensional                                                                                        |
| 3D          | 3-Dimensional                                                                                        |
| $a_{si}$    | Structural cross-sectional area parallel to structure face of the $i$ th reinforcement layer element |
| $a'_{bc}$   | Adhesion coefficient relating soil cohesion to soil/reinforcement bond                               |
| B           | Facing Block Equivalent Depth                                                                        |
| $C_c$       | Coefficient of Curvature                                                                             |
| $C_u$       | Coefficient of Uniformity                                                                            |
| $c'$        | Cohesion                                                                                             |
| D/H         | Reinforcement Distribution Factor                                                                    |
| $D_{min}$   | Minimum Soil Diameter from Sieve Analysis                                                            |
| $D_{max}$   | Maximum Soil Diameter from Sieve Analysis                                                            |
| $D_{50}$    | Average Soil Diameter from Sieve Analysis                                                            |
| $E'_c$      | Confined Elastic Modulus                                                                             |
| $e_{cr}$    | Critical Void Ratio                                                                                  |
| $e_{field}$ | Field Void Ratio                                                                                     |
| $e_{min}$   | Minimum Void Ratio                                                                                   |
| $e_{max}$   | Maximum Void Ratio                                                                                   |
| f           | Coefficient denoting Interaction Mechanism Mobilised                                                 |
| $f_b$       | Coefficient Defining Bonding Interaction                                                             |
| $f_{ds}$    | Coefficient Defining Direct Sliding Mechanism                                                        |
| $FS_{bond}$ | Factor of Safety Against Bond Failure                                                                |
| G           | Shear Modulus                                                                                        |
| $G_i$       | Initial Shear Modulus                                                                                |

|              |                                                                                     |
|--------------|-------------------------------------------------------------------------------------|
| $G_s$        | Specific Weight                                                                     |
| $H_c$        | Critical Height based on Passive Pressure at Failure                                |
| $H'_c$       | Critical Height to Allow for Compaction                                             |
| $h_i$        | Reinforced soil height above the $i$ th reinforcement element                       |
| $I_D$        | Relative Density                                                                    |
| $I_R$        | Empirical Relative Density Index                                                    |
| $K$          | Earth Pressure Coefficient                                                          |
| $K$          | Unfactored Strength for Limit Equilibrium Mechanism                                 |
| $K_{2D}$     | Unfactored Strength for 2D Limit Equilibrium Mechanism                              |
| $K_{3D}$     | Unfactored Strength for 3D Limit Equilibrium Mechanism                              |
| $K_1$        | Coefficient of Earth Pressure based on Mohr Coulomb Failure Criteria                |
| $K_{1,\phi}$ | Frictional component of Limiting Coefficient of Lateral Earth Pressure              |
| $K_A$        | Active Earth Pressure Coefficient                                                   |
| $K_{AH}$     | Active Earth Pressure Coefficient based on Coulomb's Earth Pressure Theory          |
| $K_{aw}$     | Constant Active Earth Pressure Coefficient                                          |
| $K_m$        | Corresponding Lateral Coefficient                                                   |
| $K_o$        | At Rest Earth Pressure Coefficient                                                  |
| $K_{op}$     | At rest coefficient of earth pressure for Normally Consolidated soil at Lower Limit |
| $K_p$        | Passive Earth Pressure Coefficient                                                  |
| $K_{ps}$     | Coefficient of Passive Earth Pressure During Passive Shearing                       |
| $K_R$        | Ordinate Depicting 3D to 2D Failure Difference                                      |
| $L$          | Reinforcement Length                                                                |
| $L_a$        | Active Length                                                                       |

|              |                                                                               |
|--------------|-------------------------------------------------------------------------------|
| $L_e$        | Passive Length                                                                |
| $L_{ej}$     | Length of reinforcement in the resistant zone outside potential failure wedge |
| $L_i$        | Length at Right Angles to Structure Face of $i$ th Reinforcement              |
| $L_{min,E}$  | Minimum Reinforcement Length for External Failure                             |
| $L_{min,P}$  | Minimum Reinforcement Length for Internal Failure (Pullout)                   |
| $L/H$        | Feasible Length to Height Failure Ratio                                       |
| $l_c$        | 3D Failure Volume Half Length                                                 |
| $l_T$        | Acting Length of Resultant Tensile Force                                      |
| $M_1$        | End Cap Moment Equation 1                                                     |
| $M_{1s}$     | End Cap Moment Equation 2                                                     |
| $M_2$        | Central Cylinder Moment Equation 1                                            |
| $M_{2s}$     | Central Cylinder Moment Equation 2                                            |
| $m_v$        | Coefficient of Volume Compressibility                                         |
| $n$          | Number of Blocks Between Spacing                                              |
| $N_c$        | Bearing Capacity Factor for Cohesion                                          |
| $N_q$        | Bearing Capacity Factor for Surcharge Load                                    |
| $p$          | Load Intensity from Compaction                                                |
| $p'$         | Effective Pressure                                                            |
| $p_a$        | Atmospheric Pressure                                                          |
| $P_{at}$     | Basic permissible axial tensile stress in reinforcement                       |
| $q$          | Surcharge Load                                                                |
| $\Delta P_s$ | Change in Reinforcement Stiffness                                             |
| $R$          | Radius of Log Slip Surface Failure Plane                                      |
| $S_v$        | Reinforcement Spacing                                                         |
| $s'$         | Mean Effective Stress                                                         |
| $t$          | Current Shear Stress                                                          |

|                  |                                                                                 |
|------------------|---------------------------------------------------------------------------------|
| $t_i$            | Initial Shear Stress                                                            |
| $t_p$            | Peak Shear Stress Soil can Sustain                                              |
| $t/s'$           | Stress Ratio                                                                    |
| $T_{ad}$         | Maximum Adherence in Reinforcement                                              |
| $T_{d^\circ}$    | Reinforcement Temperature Conditions                                            |
| $T_{Dj}$         | Design strength of the reinforcements at jth level in wall                      |
| $T_{fi}$         | Horizontal shear applied to contact area of width b at top of structure         |
| $T_{hi}$         | Resultant tension from the reinforced fill height above element layer           |
| $T_i$            | Total maximum tensile force resisted by the ith layer of reinforcement          |
| $T_{mi}$         | Tension from Bending Moment caused by external loading acting on wall           |
| $T_{max}$        | Maximum Tensile Force on Reinforcement/m                                        |
| $\Sigma T_{max}$ | Summation of Maximum Tensile Forces on Reinforcements                           |
| $T_{si}$         | Vertical loading applied to a strip contact area of width b at top of structure |
| $t_d$            | Specific Loading Period                                                         |
| $S_v$            | Vertical Spacing Between Reinforcements                                         |
| $W$              | Width of Reinforcement                                                          |
| $V$              | Normalised Volume Potential Sliding Mass                                        |
| $z$              | Height Under Investigation                                                      |
| $\tau_f$         | Soil Shear Strength                                                             |
| $\mu$            | Coefficient of friction between fill and reinforcing element                    |
| $\phi'$          | Friction Angle                                                                  |
| $\phi'_\mu$      | Component of Constant Volume Friction Angle                                     |

|                                  |                                                                                  |
|----------------------------------|----------------------------------------------------------------------------------|
| $\phi_{cv}$                      | Constant Volume Friction Angle                                                   |
| $\phi_d$                         | Design Friction Angle                                                            |
| $\phi_M$                         | Mobilised Friction Angle                                                         |
| $\phi'_o$                        | At-Rest Friction Angle                                                           |
| $\phi_P$                         | Peak Friction Angle                                                              |
| $\phi'_{p\text{- plane strain}}$ | Plane Strain Friction Angle                                                      |
| $\phi'_{r\&d}$                   | Friction Angle from Particle Rearrangement and Damage                            |
| $\phi'_{p\text{- Triaxial}}$     | Triaxial Compression Friction Angle                                              |
| $\psi$                           | Dilatancy Angle                                                                  |
| $\psi_{max}$                     | Maximum Dilatancy Angle                                                          |
| $\Theta$                         | Failure Plane                                                                    |
| $\Theta_{opt}$                   | Optimum Orientation for Reinforcement                                            |
| $\alpha^*$                       | Coefficient Defining Apparent Memory Loss in Soil from Shearing During Unloading |
| $\epsilon'_{1/3}$                | Strain Rate in Major/Minor Principal Direction                                   |
| $\epsilon'^p_{1/3}$              | Plastic Strain Rate in Major/Minor Principal Direction                           |
| $\epsilon_R$                     | Reinforcement Strain                                                             |
| $\epsilon_v$                     | Volumetric Strain                                                                |
| $\epsilon_x$                     | Soil Strain in x Direction                                                       |
| $(\epsilon_z)$                   | Strain Amount                                                                    |
| $\rho$                           | Bulk Density                                                                     |
| $\rho_{dry}$                     | Dry Density                                                                      |
| $\rho_{max}$                     | Maximum Density                                                                  |
| $T$                              | Specific Volume on Critical State Line                                           |
| $\nu$                            | Poisson's Ratio                                                                  |
| $\nu'$                           | Specific Volume                                                                  |
| $\nu_\lambda$                    | Specific Volume under Atmospheric Pressure                                       |

|                       |                                                                 |
|-----------------------|-----------------------------------------------------------------|
| $w$                   | Water Content                                                   |
| $\sigma_h$            | Horizontal Pressure                                             |
| $\sigma'_{h,CMUP}$    | Lateral Earth Pressure based on Current Minimum Unloading Point |
| $\sigma'_{hm}$        | Maximum Compaction Induced Earth Pressure on Back of Wall       |
| $\sigma'_{h,MPLP}$    | Laeral Earth Pressure based on Maximum Past Loading Point       |
| $\sigma_n$            | Effective Normal Stresses                                       |
| $\sigma_v$            | Vetical Pressure                                                |
| $\sigma_1/\sigma_2$   | Stress Ratio                                                    |
| $\gamma$              | Tensile Strain                                                  |
| $\gamma_b$            | Bulk Unit Weight                                                |
| $\gamma_{max}$        | Maximum Unit Weight of Soil                                     |
| $\gamma_w$            | Water Unit Weight                                               |
| $\delta$              | Peak Wall-Soil Interface Friction Angle                         |
| $\delta_{bb}$         | Block-Block Interface Friction Angle                            |
| $\delta_{gb}$         | Geosynthetic-Block Interface Friction Angle                     |
| $\delta_{sb}$         | Soil-Block Interface Friction Angle                             |
| $\omega$              | Wall Facing Batter from Vertical                                |
| $\frac{dp}{d\beta}$   | 3D Mechanism Slip Surface Differential Equation                 |
| $\rho, \alpha, \beta$ | Polar Coordinate System                                         |

## LIST OF ACRONYMS/ABBREVIATIONS

|         |                                                                                                                     |
|---------|---------------------------------------------------------------------------------------------------------------------|
| 2D      | 2-Dimensional                                                                                                       |
| 3D      | 3-Dimensional                                                                                                       |
| AS 4678 | Australian Standard AS 4678-2002 Earth Retaining Structures                                                         |
| BS8006  | British Standard Code of Practice for Strengthened Reinforced<br>Soils and Other Fills                              |
| CMUP    | Current Minimum Unloading Point                                                                                     |
| CSL     | Critical State Line                                                                                                 |
| CVR     | Critical Void Ratio Line                                                                                            |
| EBGEO   | Recommendations for Design and Analysis of Earth Structures<br>Using Geosynthetic Reinforcements                    |
| FHWA 1  | Federal Highway Association Design and Construction of<br>Mechanically Stabilized Earth Walls and Reinforced Slopes |
| FOS     | Factor of Safety                                                                                                    |
| GRS     | Geosynthetic Reinforced Soil                                                                                        |
| kPa     | Kilopascals                                                                                                         |
| LE      | Limit Equilibrium                                                                                                   |
| MPLP    | Maximum Past Loading Point                                                                                          |
| MSE     | Mechanically Stabilised Earth Wall                                                                                  |
| ms      | Microstrains                                                                                                        |
| NCMA    | National Concrete Masonry Association                                                                               |
| OCR     | Overconsolidation Ratio                                                                                             |
| RMUP    | Recent Maximum Loading Point                                                                                        |

## 1. INTRODUCTION

### 1.1. Research Motivation

Construction projects commonly require a soil mass to be retained at steep angles (including right angles) that would otherwise not be attained under the soil's own self weight. Soil at a higher grade will have a tendency to move down to a lower grade to reach its natural, stable configuration (Ranjan and Rao, 2005). Soil at a higher grade will have a tendency to move down to a lower grade to reach its natural, stable configuration (Ranjan and Rao, 2005). Such occurrences would require a supporting system to uphold the steeply battered backfill into place. Examples include:

- Excavating a working platform onto a sloping ground for housing or highway construction;
- Side supports for temporary excavation structures;
- Providing abutment for bridge decks;
- Fill and Cut retentions; and
- Right-of-way provisions for an embankment for a pavement platform

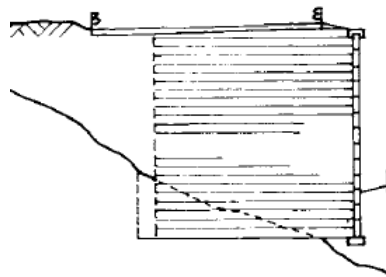


Figure 1.1. Excavating a working platform onto sloping ground from (Jones, 1996).

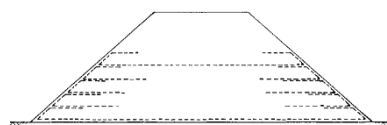


Figure 1.2. Fill retentions for right of way provisions from (Jones, 1996).

The soil's shear strength would inherently enable the soil to have a certain limiting capacity of steepness that can be sustained from within. Any further steepness beyond such limit would cause a more pronounced force to be exerted upon the soil, known as earth pressure (Ranjan and Rao, 2005). As the earlier mentioned construction projects would require the soil to be retained at angles greater than its self-angle of repose, employing retaining systems would become necessary.

Such retaining system is classified depending on whether the battered backfill is being supported externally by a means of an outer structure (such as a gravity wall self-weight) , or internally, by employing reinforcement inclusions within the soil mass, which are extended beyond the potential failure surface to mobilize the stabilizing forces (South Carolina Department of Transportation, 2010).

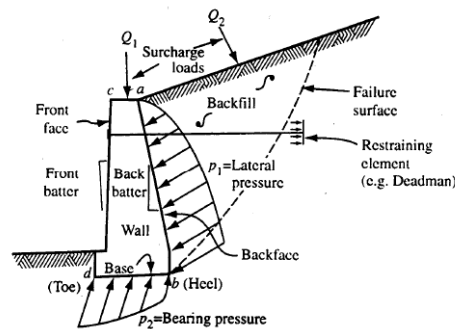


Figure 1.3. Externally supported retaining wall structure from (South Carolina Department of Transportation, 2010).

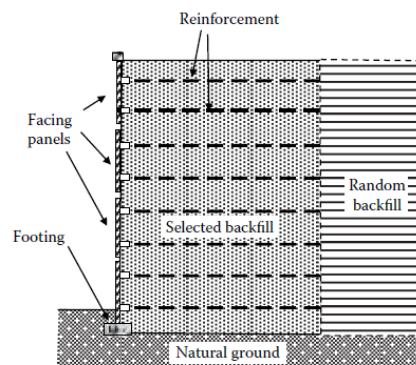


Figure 1.4. Internally supported geosynthetic reinforced soil (GRS) wall configuration from Clayton *et al.* (2013).

The following research is focused on an existing internally stabilizing reinforcement structure known as Geosynthetic Reinforced Soil (GRS) wall with segmental block facings. It was put under scrutiny in a previous assessment by Dr Murat Hamderi and Dr Erol Güler (Hamderi and Guler, 2015). It was made up of a height of 13 metres with geotextile reinforcement lengths ranging between 8 to 9 metres. These reinforcements exhibit tensile strengths of 40 kN/m for the top wall section and 80 kN/m for the lower section. The reinforcement is spaced at 0.2 metres for the upper and lower sections and with spacing variations between 0.2 to 0.4 metres for the middle portion. The wall is underlined by a weak foundation soil.

In attribution to its sharp corner, Hamderi and Guler (2015) recognised the presence of cracks and block facing separation in the upper and lower portion of the its corner within the first of year of service. Such problems associated with the segmental walls were found to be commonplace, and may be as a consequence of international standards such as Federal Highway Administration (FHWA) and the National Concrete Masonry Association (NCMA) adopting a 2-dimensional (2D) plane strain analysis approach on what is inherently a 3-dimensional (3D) problem.



Figure 1.5. Discrepancies observed in a typical GRS wall structure with corner turns from Hamderi and Guler (2015).

Finally, the sheer advantages associated with the internally stabilised wall technology over the traditional retaining systems further motivates the need to address the problems and find means of enhancing the GRS system behaviour, in such that the the GRS wall systems are more economical, less prone to settlement and have less invariance in strengths.

## **1.2. Research Objectives**

The primary objective of this research is to develop a sound understanding on the behaviour of the GRS systems in its interactions between the soil, reinforcement and the facing. By deciphering all the common failure mechanisms given in the design standards, a platform to understand the factors driving such failure and the mechanisms to resist them will be established. The causes of such wall discrepancies and the measures to alleviate these cracks will be sought for in the research.

## **1.3. Research Methodology Overview**

The research methodology is split into three main phases.

The first phase from Chapter 2 to 8 will involve establishing the general driving and resisting mechanisms to failure. A postulation of modes of Instability from various International Standards will then be presented. This follows the soil and its engineering Parameters: important soil parameters involved in GRS behaviour. Subsequently the impacts of compaction on construction induced methods on the GRS behaviour will be assed. An analysis of the reinforcement inclusions: Spacing, Length, Elasticity and other Properties affecting its interaction with the soil will also be carried out. The facing and connections in terms of the contribution to resisting mechanisms will be carried out. And finally, an assessment of the GRS Wall System Stability in a3-Dimensional approach will be carried out.

The second phase will be a base prototype model preparation followed by a numerical study conducted. A numerical analysis using the TNO Diana package has been carried out to model a 3D retaining structure and investigating how various parameters play a role on the soil-geosynthetic behaviour.

Conclusions from the literature and the experimental/numerical work are then drawn.

## 2. GRS SYSTEM: FUNDAMENTALS, DRIVING AND RESISTING MECHANISMS

The main elements of the GRS segmental wall system is the reinforced soil mass and a wall face of stacked facing units known as segments. The schematic below shows the different components of the GRS structure:

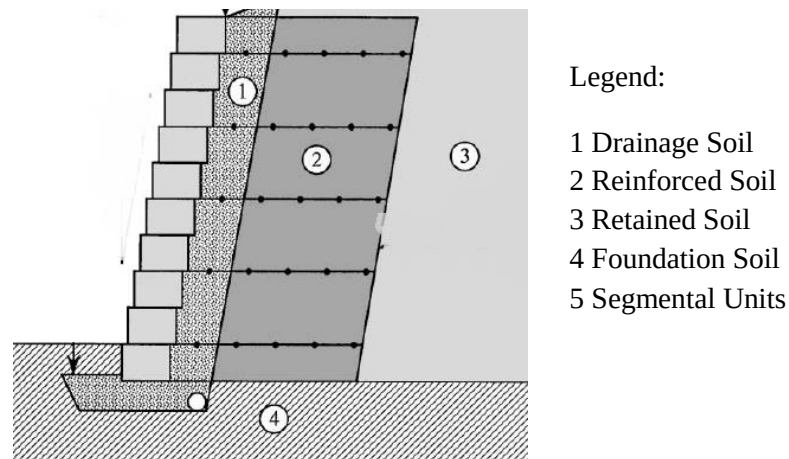


Figure 2.1. GRS Segment Layout from (Koerner and Soong, 2001).

The reinforced zone comprises the soil within the reinforcement inclusion span. Such reinforcement inclusions are placed horizontally to unify the composite reinforced soil mass. In turn, the system will have an enhanced resistance against the destabilising forces generated by the retained soil and any external surcharge load.

Although the GRS segmental wall system strongly resembles the Mechanically Stabilised Earth (MSE) system, the design concepts incorporated however distinguishes one from the other. Wu and Payer point out that the reinforcement inclusions in the GRS serve to enhance the engineering properties of the soil mass whilst the MSE utilises the reinforcements as ‘quasi-tiebacks’ in facilitating to stabilise a potential failure wedge formed between the critical slip plane and wall facing to prevent reaching failure.

Thus the reinforced soil mass is to a degree (but not entirely) analogous to reinforced concrete in that the reinforcement improves the mechanical properties by compensating the soil’s deficiency in tensile resistance. Such enhancement stems from the interaction between the reinforcement and the soil by through stress transfer mechanisms between one another.

The segmental units contribute to facing stability (Bathurst *et al.*, 2006; Ehrlich and Mirmoradi, 2013), though there are also conflicting arguments posed whether or not the segmental units have a primary structural role in wall stability (Jonathan T Wu and Payeur, 2015). Further elaboration on facing stability is provided in chapter 7.

The following chapter explains thoroughly the overall mechanism of the GRS structure by first examining the driving mechanisms acting upon the soil, the resisting mechanisms to uphold the soil and the fundamental concepts pertinent to each.

## **2.1. Driving Mechanism: Earth Movement and Earth Pressure**

To understand the fundamentals of the retaining structure operation, it is imperative first explain the earth pressure concept and how it is inextricably linked as a driving mechanism to wall movement.

A retaining wall, with all its complexity and form, can be simplified for the purpose of explanation as a slender rigid wall supporting a relatively less stiff backfill soil. Once the wall is subjected to changing forces being applied, it will undergo movement until it reaches equilibrium. Such equilibrium indicates that the result of forces being applied by the wall to support the soil are being matched by equal and opposite forces acting upon the wall (Clayton *et al.*, 2013)

Karl Terzaghi (Clayton *et al.*, 2013; Ranjan and Rao, 2005) conducted a series of large scale model retaining walls which consisted of a box of dry sand being hinged at its base, with the wall being allowed to rotate about its base. As soon as the sand is poured behind the wall, pressure will be exerted onto the wall before it is able to move. The soil is said to be at an at-rest condition, where the rigid wall is unyielding and there are no displacements and deformations taking place. (Ranjan and Rao, 2005)

Terzaghi used the concept of an earth pressure coefficient,  $K$ , as a ratio between the horizontal and vertical effective stress below the soil surface.  $K_0$  or  $K_{at-rest}$  corresponds to the horizontal and vertical earth pressures at rest (Clayton *et al.*, 2013; Terzaghi *et al.*, 1996).

As the wall under the at-rest pressure is allowed to move away from the backfill, the horizontal stress state that was once confined is now allowed to release and

decrease, hence causing earth pressure coefficient  $K$  to decrease. The backfilled sand moves along with the wall, with its inherent frictional shear strength fully mobilized. The stress state is now the active earth pressure state, a minimum value in which any further displacements beyond the active state will not lead to a further decrease in pressure (Bowles, 1997).

On the other hand, a wall movement towards the backfill would instigate a greater lateral earth pressure exerted on the backfill soil, causing the earth pressure coefficient to rise. The stress state reaches a maximum limiting value, known as passive earth pressure state, where in which the full shear strength of the soil mobilised has caused it to fail.

The driving mechanisms in both the active and passive cases come from a failure wedge defined by a critical surface developing. The active case possesses minimum driving weight for the mechanism as oppose to the passive wedge where the soil volume within the critical surface is substantially larger (Bowles, 1997).

This substantial difference in the size of the wedge comes from resisting mechanism to attain equilibrium, or more specifically, plastic equilibrium. In the active case, the wall moves away enabling flexibility for the earlier confined backfill soil mass to expand. The soil's shearing resistance will be utilized to oppose such movement, preventing the soil from breaking away from the rest of its mass as it moves downwards and outwards towards the wall (Ranjan and Rao, 2005). The passive case has the soil's shearing resistance utilized to oppose compression of the soil mass as the wall is moved towards the backfill. (Ranjan and Rao, 2005) The induced shear stress that is being counteracted by the shearing resistance in both cases is a component of the amount of weight generated from the failed wedge. The failed wedge for the passive case needs to be larger because a greater weight is needed to produce enough driving shear stresses to overcome the shearing resistance counteracting the compressing forces within the soil. This greater weight is reflected in the larger failure wedge volume needed (Clayton *et al.*, 2013).

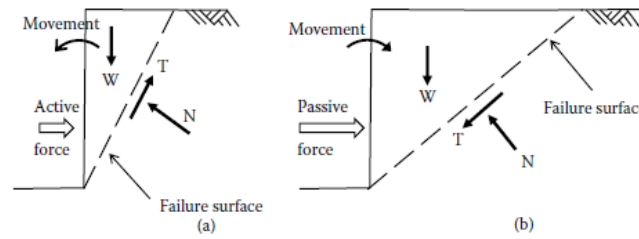


Figure 2.2. Wall Movement Active and Passive from (Clayton *et al.*, 2013).

Terzaghi culminated the results for the wall movement relative to the earth pressures needed to reach plastic equilibrium under Figure 2.3 below. The passive case can be seen to require greater movement, thus a greater driving wedge volume needed to resist the shear resistance of the soil to counteract its induced movement.

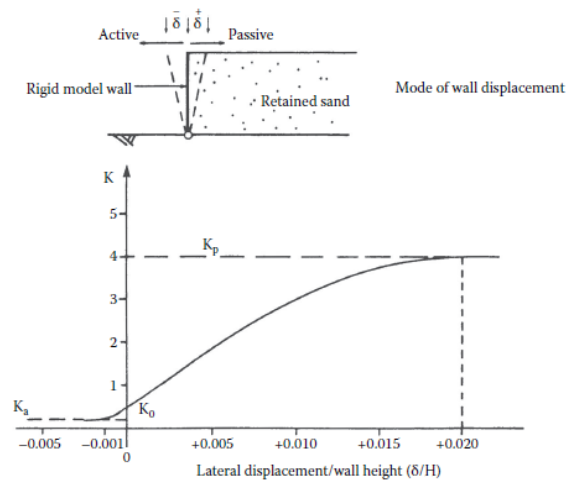


Figure 2.3. Culmination of Terzaghi's lateral displacements/wall heights results to corresponding Earth Pressure Coefficients from (Clayton *et al.*, 2013).

### 2.1.1. Soil Properties, Ground Movement and Earth Pressure State

The retaining wall problem has so far understood as a stability calculation type of problem (Terzaghi *et al.*, 1996), where the magnitudes of the resisting and driving forces are analysed to ensure there is enough resistance capacity to endure the driving demands.

Revisiting the resisting mechanism of the backfill itself, although its weight primarily contributes to be a driving force, the backfill concurrently exhibit strength properties enabling it to resist its own driving forces. Such inherent resistance capacity in this case will be referred to as its shear strength.

The full mobilisation of the shear strength from the driving mechanisms causes the soil body to be on the verge of failure, and is said to have reached a limiting condition known as a plastic equilibrium state (Terzaghi *et al.*, 1996). This was characterized with its earth pressure states as active and passive earth pressure.

Rankine delineated the plastic equilibrium stress states for the ratio of vertical to horizontal stresses under active ( $K_a$ ) and passive ( $K_p$ ) conditions as:

$$K_{a,p} = \tan^2(45^\circ - \frac{\phi}{2}) \quad (2.1)$$

A trace back to the Mohr-Coulomb model in representing the soil strength would aid in understanding the plastic equilibrium relationship Rankine adopted.

The stress state of the soil under shear and normal stress combinations is represented by a Mohr Circle, with its dimensions defined by principal stresses  $\sigma_1$  and  $\sigma_3$  on a shear stress against normal stress curve. Once the Mohr Circle substantiates to a size where it intersects with the failure envelope, the soil is said to have reached failure i.e. a plastic equilibrium.

Such a failure envelope is simplistically represented by a straight line as:

$$\tau_f = c' + \sigma' \tan \phi' \quad (2.2)$$

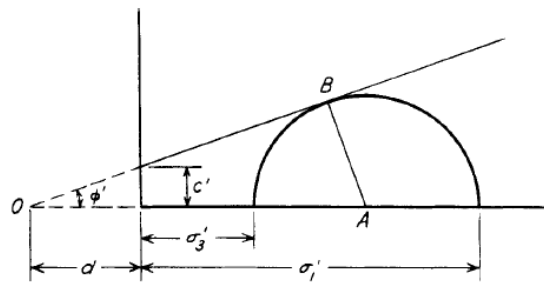


Figure 2.4. Mohr Circle with Mohr-Coulomb failure plane from (Terzaghi *et al.*, 1996).

$c'$  corresponds as the cohesion intercept and  $\phi'$  being the angle of shearing resistance. Further information on how cohesion and friction provides shear strength is elaborated in Chapter 4. The theoretical angle between the minor principal plane and the failure plane is  $\Theta$ , where from geometrics,  $2\Theta$  is equal to  $90^\circ + \phi'$ , making  $\Theta$  in turn equal to  $45^\circ + \phi'/2$

The sine of friction angle is trigonometrically related to the cohesion intercept and the principal stresses as:

$$\sin \phi' = \frac{\frac{1}{2}(\sigma'_1 - \sigma'_3)}{c' \cot \phi' + \frac{1}{2}(\sigma'_1 - \sigma'_3)} \quad (2.3)$$

Rearranging and applying the further trigonometric relations produces the Mohr Coulomb failure criterion equation which relates the material properties  $c'$  and  $\phi'$  with the principal stresses at failure:

$$\begin{aligned} (\sigma'_1 - \sigma'_3) &= (\sigma'_1 + \sigma'_3) \sin \phi' + 2c' \cos \phi' \\ \sigma'_1 &= \sigma'_3 \tan^2(45^\circ + \frac{\phi'}{2}) + 2c' \tan^2(45^\circ + \frac{\phi'}{2}) \end{aligned} \quad (2.4)$$

Considering first a cohesionless soil (which eliminates the second component of the equation, the ratio between the major and minor principal stresses must not exceed the value:

$$\frac{\sigma'_1}{\sigma'_3} = \tan^2(45^\circ + \frac{\phi'}{2}) \quad (2.5)$$

The allocation of the major and minor principal stresses is independent of the application direction i.e. vertical and horizontal axes, which in turn implies two forms of earth pressure coefficients for the ratio of horizontal to vertical stress: Active Earth Pressure and Passive Earth Pressures

For a Passive case where the horizontal stress is much greater than the vertical stress (thereby respectively allocated to be the major and minor principal stresses), the  $K_p$  ratio of the horizontal to minor principal stress takes the form above. The opposite is to be said for the active case, where the horizontal stress has been decreased to be much lower than the vertical stress, consequently leading to the reciprocal of  $K_p$  relationship to be adopted.

In the subsequent page are the equations for  $K_p$  and  $K_a$ :

$$K_p = \frac{\sigma'_h}{\sigma'_v} = N_\phi = \tan^2(45^\circ + \frac{\phi'}{2}) \quad (2.6)$$

$$K_a = \frac{1}{K_p} = \tan^2(45^\circ - \frac{\phi'}{2}) \quad (2.7)$$

In the passive case, the major principal stress equalling to the horizontal pressure and the minor equalling to the vertical earth pressure brings about a passive Rankine state, with two sets of failure planes each inclined at  $\Theta = 45^\circ + \phi'/2$  to the vertical (major principal plane direction) (Knappett and Craig, 2012; Terzaghi *et al.*, 1996). The active case has the minor principal stresses at the horizontal, the shear planes rise at an angle of  $45 + \phi/2$  with the horizontal, and forms a trace pattern of shear planes on a vertical section parallel to the direction of stretching, known as the shear pattern, as shown in Figure 2.5 below.

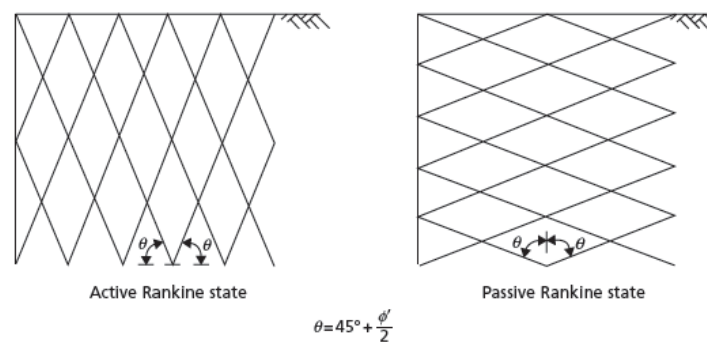


Figure 2.5. Rankine's Active and Passive shear plane patterns from (Craig and Knappett, 2012).

The two limit plastic equilibrium states, active and passive, are now defined based on soil strength and principal stress parameters, as a manifestation of failure occurring from wall movement. The next point of interest is to define the spectrum of the feasible wall movement ranges between the two plastic equilibrium states. This correlates to how much shear strength can be mobilised until failure occurs. These intermediate ranges are known as states of elastic equilibrium, with an important state, known as the at rest condition, being the initial stress state of the soil mass. The Mohr Coulomb model below shows the intermediate ranges before reaching the active and passive Rankine states.

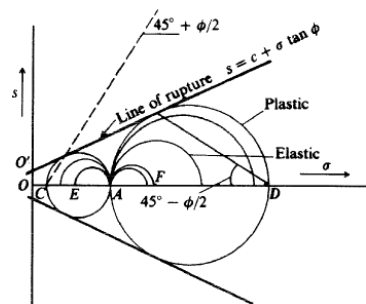


Figure 2.6. Active and Passive Mohr Circles with failure envelope relationships from (Terzaghi *et al.*, 1996).

The stress conditions for a soil at rest are represented by the coefficient of earth pressure at rest,  $K_0$ , which depicts the ratio of horizontal to vertical effective stresses as the soil does not exhibit any lateral yield. The  $K_0$  conditions theoretically represent the immediate placement of the backfill before any form of lateral yielding occurs.

It is worth mentioning that the vertical effective stress depends on the soil's bulk unit weight ( $\gamma_{\text{bulk}}$ ) which will be investigated separately in terms of its effect on the reinforced soil wall behaviour under chapter 5.

The  $K_0$  conditions had been quantified theoretically for normally consolidated granular material based on the soil's effective friction angle by Jacky (1944) as:

$$K_0 = 1 - \sin \phi' \quad (2.8)$$

Mayne and Kulhawy had also developed an equation for  $K_0$  to take into account the stress history as

$$K_0 = (1 - \sin \phi'). OCR^{\sin \phi'} \quad (2.9)$$

A relationship between  $K_0$  and Poisson's ratio can also be derived from the horizontal strains pertaining at a soil depth  $z$ .

The soil's strain in the  $x$  direction is given as:

$$\epsilon_x = \frac{1}{E} (\sigma'_x - \nu (\sigma'_z + \sigma'_x)) \quad (2.10)$$

By considering that the earth pressure at rest is associated with zero lateral strain, or the lateral earth pressure at rest is:

$$\Delta \sigma_h = \Delta \sigma_v \frac{\nu}{(1-\nu)} \quad (2.11)$$

Although the ground parameters are paramount in determining the earth pressures, the designer must also consider the movement types the wall can accommodate and the deflections that can be sustained as a result of kinematic constraints and serviceability characteristics. Such factors would cause the limiting modes of plastic equilibrium from new restrictions to be re-evaluated (Frank *et al.*, 2013)

Kinematic constraints can take the form of the connection types used, which parts of the walls have restraint movements (for instance it is hypothesized that a 3

dimensional, non-plane-strain wall may exhibit unique movements at the edges as oppose to a plane-strain modelled longitudinal retaining wall.

## 2.2. Resisting Mechanism: Upholding the Backfill Soil

So far, the soil self-weight standalone was observed to initiate wall movement, especially when the backfill was battered at a vertically steep angle. Any extra surcharges imposed or inertial forces would further exacerbate the effects, which would further entail the need for a robust retaining system to uphold the backfill soil.

Expanding from the simple model of a rigid slender wall retaining a backfill soil, various retaining wall systems are available to uphold the backfill soil. One of them is a gravity retaining wall, which is an external support system where the majority of the resisting stability is obtained from the wall's self-weight (Manceau *et al.*, 2012).

Alternatively, the more holistic retaining system known as a Reinforced Earth Wall, was developed by Henri Vidal which resists the driving mechanisms internally, dispensing the need of an external wall to retain the soil altogether (Wesley, 2010). The resisting mechanism comes from directly enhancing the shear and tensile capacities of the soil mass itself with the use of reinforcement inclusions (Manceau *et al.*, 2012).

The reinforced soil acts as a coherent gravity block acting on the foundation soil (as shown in Figure 2.7), which is able to resist any external design mechanisms such as sliding, bearing capacity and overturning (see section on failure mechanisms). Internally, the reinforcement counteracts the development of earth pressures acting on the face that cause strains manifesting lateral wall movement, as shown in Figure 2.8.

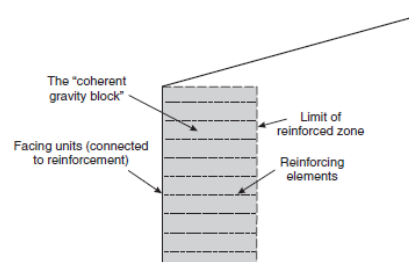


Figure 2.7. GRS structure revisited from (Wesley, 2010).

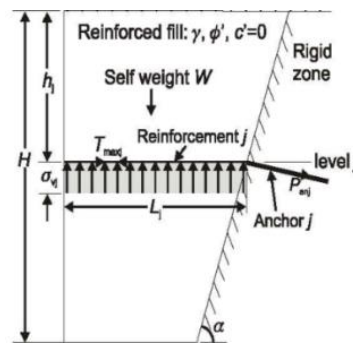


Figure 2.8. Internal reinforcement mechanism from (Lawson and Choi, 2010).

Manceau *et al.* (2012) reinforce the idea of equilibrium earlier explained regarding wall movement in that the soil's self-weight (and any further surcharge imposed) acts as a vertical load with an axial compression imposed upon the soil. The soil corresponds with a lateral tensile strain, causing the soil to expand. Such tensile strain is limited by the shear strength of the soil. (Refer to Chapter 4 for the angle of shearing resistance). Once reinforcement is introduced, the soil's shearing resistance is enhanced, as the newly synergised reinforced soil shear resistance is the sum of the mobilised shearing of the soil and the mobilised tensile force that the reinforcement provides. In turn, further resistance to the axial compression as well as to the overall lateral deformations evolving from the compression is provided (Manceau *et al.*, 2012).

The axial compression imposed on the wall as an earth pressure, can only be within a feasible range of earth pressures for the soil mass are between the earth pressure at rest ( $K_0$ ) and the active earth pressure state ( $K_a$ ). A specific type of active earth pressure was adopted, known as Rankine's active earth pressure, which assumes that the failure plane of the failed wedge is at  $45 + \phi/2$  from the horizontal. This was later considered to be overly conservative in estimating the maximum active earth pressure envelope (Koerner, 2012; Wesley, 2010; Wu and Payeur, 2015) where the envelope also renders the zones of maximum tensile strain.

Taking the conservative approach in adopting Rankine's active earth pressure, the reinforced soil structure will now consist of two zones divided by the failure plane: an active and a passive zone (not to be confused with an active and passive state). This is



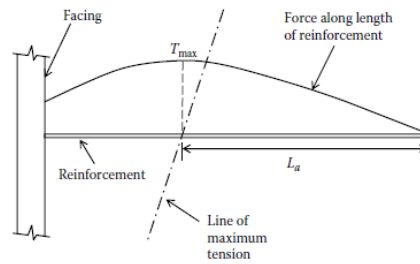


Figure 2.11. Collation of maximum tension line and  $T_{Max}$  from (Clayton, *et al.*, 2013).

The overall reinforcement length would provide the necessary capacity to withstand the horizontal pressure acting on the soil face, where the total horizontal pressure is divided amongst each reinforcement layer by its spacing in the vertical and horizontal axes (Wesley, 2010; Koerner and Soong, 2001).

It is important to appreciate that the tensile force developed at a distance from the face would vary as a manifestation of the shear stresses being transmitted from the soil to the reinforcement and vice-versa (Ingold, 1994).

Overall, the reinforcement must also be capable to carry the tensile forces transferred to them by the soil without rupture while also developing sufficient bond between the reinforcement and the soil in the resisting zone so that the reinforcement develops the required force without having to pullout. Such conditions are satisfied by ensuring adequate spacing and lengths for the reinforcements (Ingold, 1994).

### 2.2.1. Common Methods of Analysis for Internal Resistance Methods

As was shown earlier under Figure 2.7 and 2.8, the value of  $T_{max}$  would coincide with the failure line. An important precaution to consider is how much the available strength within the reinforcement varies over its full length. Although the reinforcement's central section would possess the maximum tensile strength, there is a diminishing tensile strength near the anchor sections of the reinforcement length. The varying tensile strength needs to be considered when determining the anchor length of the reinforcement (Wesley, 2010)

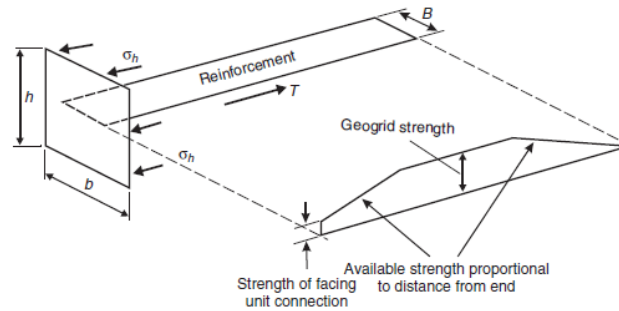


Figure 2.12. Available strength on geogrid reinforcement along length from (Wesley, 2010)

Another aspect to consider is how the earth pressures are distributed along with the wall as per the assumed kinematic constraints implied on the wall. This is vital in calculating how much tension force each of the reinforcements would be carrying. In turn, the designer would be able to discern the type, number of layers, embedment length to ensure that there is enough capacity to withstand the tension transferring to the reinforcements, which can manifest two modes of failure:

- Pullout failure by the reinforcement being pulled out from the passive zone behind the assumed failure surface
- Rupture failure acting upon the locus of maximum reinforcement tensile force.

Two common methods of analysis to determine the magnitude of  $T_{\max}$  are the Tie-Back Wedge method and the Coherent Gravity method.

The first method of analysis, known as the Coherent Gravity Method, is based on the assumption that the wall moves outwardly away about the top wall crest, inducing a maximum tension line which divides the soil mass into an active and a passive zone. Semi-empirical measurements taken on several full-scale walls lead to devising a variable earth pressure coefficient from at rest earth pressure at the top reducing to a depth of 6m, and remains constant at such value (Clayton *et al.*, 2013).

Jones (1996) further corroborates with Clayton *et al.* (2013) on this matter by in the provision of empirical equations for the coefficient  $K$  in how it varies with depth.

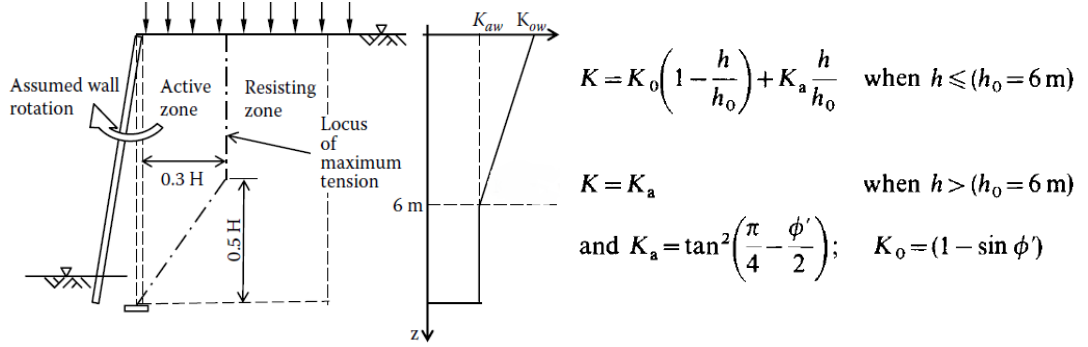


Figure 2.13. Coherent Gravity Method wall movement from (Clayton *et al.*, 2013).

This in turn would enable determining the maximum tension ( $T_{\max}$ ) on the reinforcement elements, with  $T_{\max}$  as:

$$T_{\max} = K \sigma_v \Delta H / N \quad (2.12)$$

with  $N$  as the number of reinforcing elements per unit structural width.

Furthermore, there is also an apparent coefficient of adherence between the reinforcement inclusions and the fills, which is determined from pullout tests. Such coefficient varies depending on the soil grading (in particular its uniformity coefficient  $C_u$ ) reinforcement type being used (geotextile or a geogrid) as well as where it is placed along the wall depth.

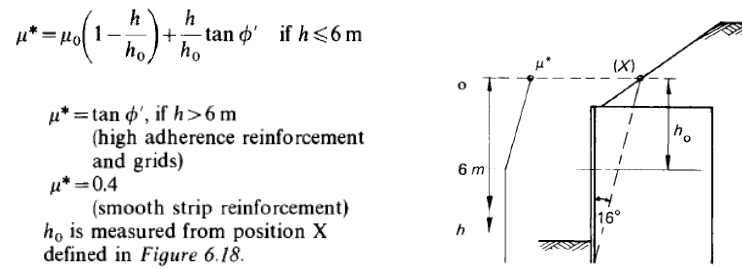


Figure 2.14. Adhesion coefficient profile from (Jones, 1996).

The maximum adherence of the reinforcement is:

$$T_{ad} = 2B \int_{L-L_i}^L (\mu^* \sigma_v dL) \quad (2.13)$$

Finally, a Factor of Safety for Bonding would be given as:

$FS_{\text{bond}} = T_{ad} / T_{\max}$  where a threshold factor of safety is to be met.

Based on this philosophy, the designer must ensure that there is enough bonding to prevent pullout by designating sufficient anchor length in the reinforcement beyond the passive zone as well as active length within the active zone defined by the varying earth pressure coefficient (Wesley, 2010)

The second method, Tie-back wedge analysis proposes a different wall movement, where the kinematic mechanism involves moves outwardly away about the bottom wall face. In this case, the earth pressure coefficient takes a constant active value,  $K_{aw}$  throughout the wall depth, with a critical failure mode of a Coulomb type wedge inclined at  $45^\circ + \phi'/2$  to the horizontal. This implies to attempt to mobilise the full shear strength of the reinforced fill in conducting the analysis. An assumption is made that the failure plane coincides with the locus of maximum tension reinforcement (Clayton *et al.*, 2013; Ingold, 1994).

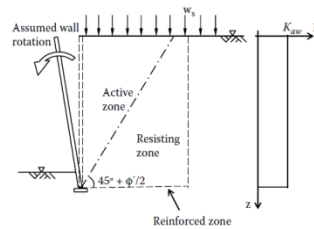


Figure 2.15. Wall movement associated with Tie-Back Method from (Clayton *et al.*, 2013).

Each individual reinforcing element must have enough capacity to withstand the maximum tensile force  $T_i$ , which is a summed combination from various contributing factors:

$$T_i = T_{hi} + T_{wi} + T_{fi} + T_{mi} + T_{si} \quad (2.14)$$

Where  $T_{hi}$  is the resultant tension from the reinforced fill height above element layer;

$T_{wi}$  Resultant tension from uniform surcharge on top of structure,

$T_{fi}$  Horizontal shear applied to contact area of width  $b$  at top of structure,

$T_{mi}$  Tension from Bending Moment caused by external loading acting on wall,

and  $T_{si}$  Vertical loading applied to a strip contact area of width  $b$  at top of structure.

From the maximum tension force, the necessary dimensions for the reinforcement inclusions are calculated as follows:

$$a_{si} = \frac{T_i}{P_{at}}$$

$$P_i = \frac{T_i}{\mu \frac{L_i}{2} (\gamma h_i + w_s) + c' r \frac{L_i}{2}} \quad (2.15)$$

$P_{at}$  is basic permissible axial tensile stress in reinforcement

$a_{si}$  structural cross-sectional area parallel to structure face of the  $i$ th reinforcement layer element

$h_i$  reinforced soil height above the  $i^{\text{th}}$  reinforcement element

$\mu$  coefficient of friction between fill and reinforcing element

$L_i$  length at right angles to the structure face of  $i^{\text{th}}$  layer of reinforcing element

$T_i$  total maximum tensile force resisted by the  $i^{\text{th}}$  layer of reinforcement

The Tie-back Wedge method is actually an extension of the Trial Wedge method (Ingold, 1994) in which a rigid coulomb wedge body is presumed to occur within a reinforced fill. The earlier is an assessment for local internal stability to prevent rupture and pullout, whereas the latter is a global internal stability check (Ingold, 1994) to ensure the summed resistances of the individual reinforcement layers withstand the total disturbing thrusts in the active zone.

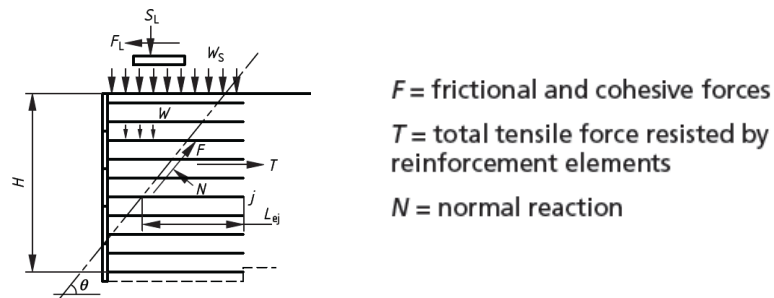


Figure 2.16. Trial Wedge Method forces from (British Standards Institution, 2012).

$W$ : Self-weight of the fill in the wedge;

$W_s$ : Uniformly distributed surcharge loads;

$S_L$ : Vertical strip loading;

$F_L$ : Horizontal shear;

$F$ : Frictional and cohesive forces acting along the potential failure plane; and

$N$ : Normal reaction of the failure plane.

By conducting several iterations for many wedge layouts, the optimised maximum  $T_{\max}$  value can be obtained.

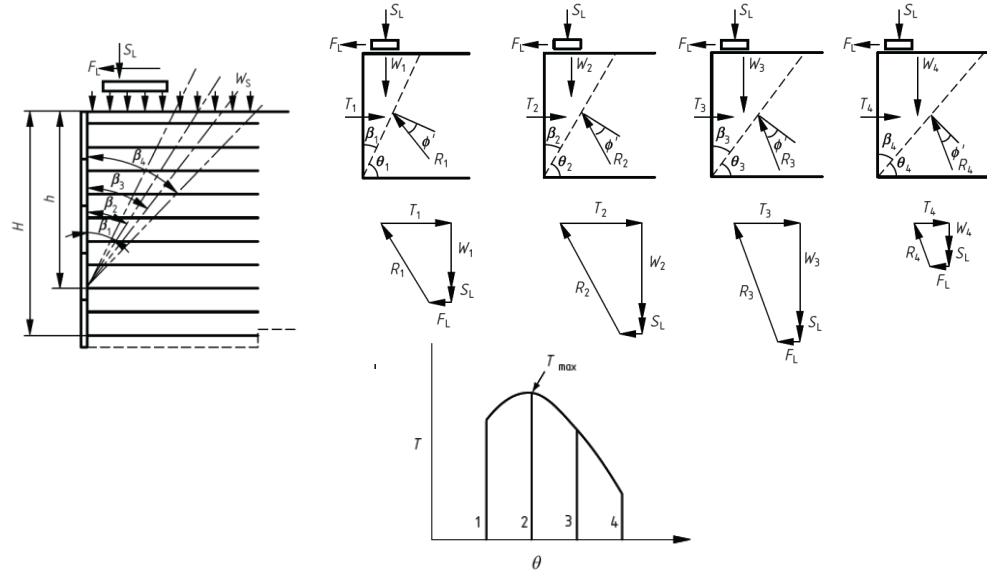


Figure 2.17. Trial wedge iterations to obtain  $T_{\max}$   
from (British Standards Institution, 2012).

The resistance provided by an individual layer of reinforcing elements is taken as the minimum of :

- Frictional resistance of the layer embedded in the fill beyond the potential failure plane; or
- Tensile resistance of the layer of elements.

The total resistance of the reinforcement layers within the wedge is satisfied by:

$$\sum_{j=1}^m \left[ \frac{T_{Dj}}{f_n} \right] \geq T \text{ or} \quad (2.16)$$

$$\sum_{j=1}^m \left[ \frac{P_j L_{ej}}{f_p f_n} \right] \left( \mu f_{fs} \gamma h_j + \mu f_f w_s + \frac{a'_{bc} c'}{f_{ms}} \right) \geq T \quad (2.17)$$

Where  $T_{Dj}$  is the design strength of the reinforcements at  $j$ th level in wall;

$f_n$  is the partial factor applied to economic ramifications of failure;

$P_j$  is the total horizontal width of the top and bottom faces of the reinforcing element;

$L_{ej}$  is the length of reinforcement in the resistant zone outside potential failure wedge;

$f_p$  is the partial factor for reinforcement pull-out resistance;

$w_s$  is the surcharge due to dead loads only;

$a'_{bc}$  is the adhesion coefficient relating soil cohesion to soil/reinforcement bond;

$c'$  is soil cohesion measured under effective stress conditions and

$f_{ms}$  is the partial material factor applied to  $c'$ ,

Overall, the general driving and resisting mechanisms of an internally stabilized wall system provides a starting point into understanding how Geosynthetic Reinforced Wall withstands failure mechanisms. To grasp a greater understanding of what factors contribute to or resist the driving mechanism, a look into the various international design standards to make more apparent the causes of the discrepancies in the Reinforced Earth Wall structure.

## 2.3 Conclusions

- A retained backfill, when allowed to move freely, can cause the stress state of the soil to reach plastic equilibrium. The soil stress state is either active (the horizontal stress state is lower than the vertical stress state) or passive (the horizontal stress state is greater than the vertical stress state). The limiting cases can be depicted using the coefficient of earth pressure (ratio of horizontal to vertical stress). The at-rest coefficient  $K_0$  (elastic equilibrium), Active and passive coefficients,  $K_a$  and  $K_p$  (plastic equilibrium) delineate the feasible stress state range preventing failure.
- The plastic equilibrium would have a failure wedge attributed with the backfill. The size of the failure wedge in passive state is appreciably larger than the active state due to the differing resistance mechanisms that prevent reaching the plastic equilibrium. The active state is caused by the soil initially confined, and once movement away from the retained backfill allowed, the soil mass exhibits a stress release allowing it to expand. The shear resistance is utilised to oppose such movement. The soil in the passive case however utilises the shear resistance to resist the induced shear stress caused by the wall moving towards the backfill.
- In both cases the induced shear stress counteracted by the shearing resistance is a component of the weight of the failure wedge. Therefore, the passive case requires a larger weight to counteract the resistance.

- As the soil approaches from  $K_o$  to  $K_a$  range, the soil will correspond with lateral tensile strain, which is as explained limited by its shear strength before reaching plastic equilibrium. The introduction of reinforcement inclusions would have synergised and enhanced soil shear strength, as it will be the sum of the soil peak shear strength and the tensile strength from the reinforcements, which is the key mechanism in GRS structures.

### **3. MODES OF INSTABILITY OF THE GRS AS PER DESIGN STANDARDS**

#### **3.1. Introduction to Mechanisms**

A postulation of all the failure mechanisms (modes of instability) common to GRS walls in current international design standards would enable to further decipher the factors the drive such mechanisms, along with the properties inherent to the soil and geosynthetics that constitute to resisting the mechanisms. Special attention will also be given to the synergy within the soil-geosynthetic interaction.

The standards that were assessed are as follows:

- United States: FHWA Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Slopes. Abbreviated as FHWA 1
- United States: FHWA Design and Construction Guidelines. Abbreviated as FHWA 2
- United States: Design Manual for Segmental Retaining Walls, National Concrete Masonry. Abbreviated as NCMA
- United Kingdom: British Standards BS 8006 Code of Practice for Strengthened/Reinforced Soils and Other Fills. Abbreviated as BS 8006
- Hong Kong: Geoguide 6 Guide to Reinforced Fill Structure and Slope Design. Abbreviated as Geoguide 6
- Germany: Recommendations for Design and Analysis of Earth Structures Using Geosynthetic Reinforcements-EBGEO. Abbreviated as EBGEO.
- New Zealand: Guidelines for Design and Construction of Geosynthetic-Reinforced Soil Structures in New Zealand-Transfund New Zealand Research Report No. 239. Abbreviated as NZ Report 239
- Australia: Australian Standard AS 4678-2002 Earth Retaining Structures. Abbreviated as AS 4678

It is worth mentioning that all these mechanisms are based on a 2 dimensional (2D) plane strain analysis, and do not directly tackle the essence of the problem being a 3 dimensional (3D) tight cornered block retaining wall with a longitudinally well-

defined dimension. This should not however discourage to look at what is readily available from the plain strain point of view and then seek to deduce the impacts on a 3D perspective.

Also, some mechanisms at first glance are less tangible to be a root cause for the situation present. Nevertheless, these were not discredited because the factors contributing to such mechanisms may still manifest to the case, but not on a primary basis.

External stability is more focused on treating the soil mass as a monolithic body similar to a rigid retaining wall with earth pressures acting behind the wall (British Standards Institution, 2012; Fang and Daniels, 2006). The design for external stability (against sliding, bearing capacity and overturning) determines the dimensions of the Reinforced Wall structure. (National Concrete Masonry Association, 2010).

The design for Internal stability (connection strength, rupture, pullout, internal sliding) is concerned with the integrity of the reinforced soil (Government of the Hong Kong Special Administrative Region, 2002) such that reinforcement failure through tensile rupture and bond loss is avoided. This would determine the length of the reinforcement embedded within the reinforced fill and its strength properties.

And finally, compound stability deals with slip surfaces passing within and beyond the reinforced soil mass onto the unreinforced backfill.

Table 3.1. Mode of Instability Mode of Instabilities, Driving and Resisting Factors.

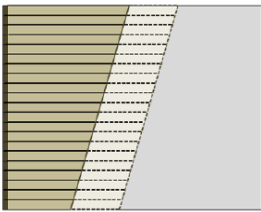
| Mode of Instability-Description and Schematic                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     | Factors Contributing to Driving Mechanism                                                                                                                                                                                                      | Factors Contributing to the Resisting Mechanism                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |
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| <p><u>Ultimate Limit State Mechanisms</u><br/> <u>Mechanism 1a. Sliding Failure (External)</u></p> <p>Destabilizing forces caused by the soil horizontal earth pressure and surcharge loads can be invoked, that once exceeding the retaining structure's resisting mechanisms, can cause translation movement.</p> <p>Translation movement involves GRS system as a coherent body to slide upon base, by either:<br/>         Sliding along the foundation soil with shear strength<br/>         Sliding along reinforced fill<br/>         Sliding along the weaker of the upper and lower soil—reinforcement layers</p>  <p>Figure 3.1. External sliding failure mechanism from (Federal Highway Administration, 2012).</p> | <p><u>Mechanism 1a. Sliding Failure (External)</u></p> <p>Soil Self weight and surcharge produces enough pressure to reach an active state earth pressure condition.</p> <p>Geometry of reinforced backfill-vertical and horizontal batter</p> | <p><u>Mechanism 1a. Sliding Failure (External)</u></p> <p>Frictional interaction of horizontal component of coherent body's vertical load</p> <p>The horizontal resistance from the vertical weight is related using the tangent component of the soil friction angle <math>\phi</math>, being the minimum of three types of cases:</p> <p>Sliding along the foundation soil where the shear strength <math>\phi_f</math><br/>         Sliding along the reinforced fill <math>\phi_r</math> (grid type of reinforcement)<br/>         Sliding along weaker of the upper and lower soil-reinforcement interfaces, which is represented by friction angle <math>\rho</math> where the reinforcement is a sheet type</p> <p>FHWA advises the friction angle <math>\rho</math> can be obtained from interface direct shear tests, or in the absence to adopt <math>\frac{2}{3}\tan \phi_r</math>.</p> <p>Consideration for Short Term (undrained cohesion) and Long term strength of soil (friction angle and effective cohesion).</p> |

Table 3.1. Mode of Instability Mode of Instabilities, Driving and Resisting Factors (Cont.).

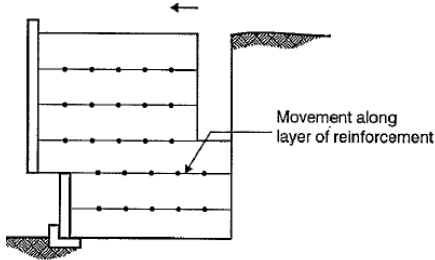
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| <p><u>Mechanism 1b. Sliding Failure (Internal)</u></p> <p>Same principle as the External, except that the movement caused between the reinforcement layers.</p>  <p>Figure 3.2. Internal sliding mechanism from (Transfund New Zealand, 2003).</p> | <p><u>Mechanism 1b. Sliding Failure (Internal)</u></p> <p>Horizontal earth pressure from the soil mass, but only the height above the reinforcement is taken under consideration.</p> <p>Vertical stresses from uniform surcharge or concentrated vertical loads.</p> <p>Connection instability is also a cause for wall instability.</p> | <p><u>Mechanism 1b. Sliding Failure (Internal)</u></p> <p>Mobilization of soil-reinforcement interaction from direct shear.</p> <p>Additional resistance from shear provided by the wall facing, as a manifestation of friction between segmental units and the geosynthetic reinforcement, and the friction between geosynthetic reinforcement and granular material filling the unit cavities.</p> <p>Such requires a local stability of the wall facing to mobilise the shear resistance.</p> |
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Table 3.1. Mode of Instability, Mode of Instabilities, Driving and Resisting Factors (Cont).

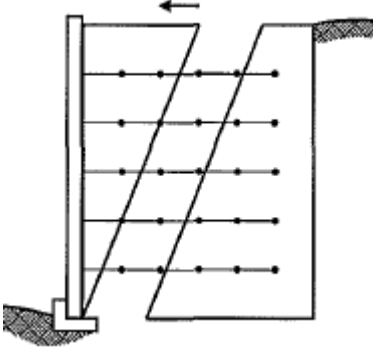
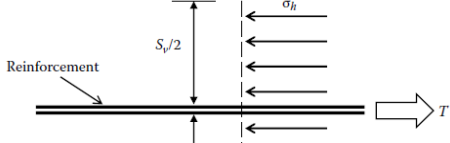
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| <p><u>Mechanism 2: Pullout Failure (Internal)</u></p> <p>Reinforcement pulled out from soil after the inherent maximum bond stress is mobilised to withstand the horizontal pressures.</p> <p>This can cause large movements and possible structural collapse when the tensile forces in reinforcement exceed pullout resistance.</p>  <p>Figure 3.3. Pullout mechanism from (Transfund New Zealand, 2003).</p> | <p><u>Mechanism 2: Pullout Failure (Internal)</u></p> <p>Horizontal earth pressure covering a tributary area between midpoints of above and below adjacent reinforcements, known as the contributory heights.</p>  <p>Figure 3.4. Tributary area of earth pressure contributing to Pullout Mechanism from (Clayton <i>et al.</i>, 2006).</p> $\sigma_h = K_r[(\gamma_r Z) \gamma_{EV-MAX}]$ $T_{max} = \sigma_h S_v$ <p>It is important to differentiate how the earth pressure coefficient is chosen, whether Coherent Gravity Method or Tie Back Wedge method is used for K. Each would have Yielding zones and Resisting zones unique to the method used.</p> | <p><u>Mechanism 2: Pullout Failure (Internal)</u></p> <p>The Reinforcement effective length must be as such to ensure enough embedment length covers the passive zone with a tensile capacity to overcome maximum tension.</p> <p>Tension is also not constant on the reinforcement length throughout; it varies along the length, and is critically important at the facing and along the maximum tension line (predefined depending on whether the Coherent Gravity Method or Tie Back Method is adopted).</p> <p>The soil strength from friction as well as cohesion contribute to the composite strength at the soil-reinforcement interface.</p> |
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Table 3.1. Mode of Instability Mode of Instabilities, Driving and Resisting Factors (Cont.).

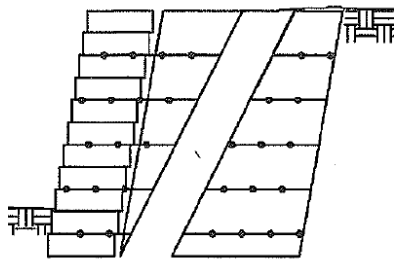
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| <p><u><i>Mechanism 3: Reinforcement Elongation and Breakage</i></u></p> <p>Generated tensile forces in the reinforcement reach a threshold amount causing them to elongate excessively or break, which results to large movements and/or structural collapse.</p>  <p>Figure 3.5. Reinforcement elongation and breakage from (National Concrete Masonry Association, 2010).</p> | <p><u><i>Mechanism 3: Reinforcement Elongation and Breakage</i></u></p> <p>Horizontal earth pressures generated between reinforcements</p> <p>Factors affecting the ultimate tensile capacity are installation damages, creep reductions and durability as a result of polymer chemical degradations contribute to the decrease in the tensile capacity. If unaccounted for, these can cause unanticipated failure as a result of excess tension acting.</p> | <p><u><i>Mechanism 3: Reinforcement Elongation and Breakage</i></u></p> <p>Reinforcement ultimate tensile strength must exceed the maximum tensile force acting on the reinforcement</p> |
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Table 3.1. Mode of Instability Mode of Instabilities, Driving and Resisting Factors (Cont.).

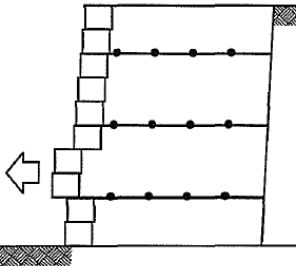
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| <p><u><i>Mechanism 4: Facing Instability</i></u></p> <p>Ensuring the stability of a block of unit columns need to remain intact and not prone to toppling/sliding.</p> <p><u><i>Mechanism 4a Shear Failure ( Bulging) of Facing Column</i></u></p> <p>Wall unit no maintaining relative position to the adjacent unit. Shear resistance maintains their relative positions with one another.</p> <p>Shear failure (bulging) between facing units can be induced when there is insufficient shear capacity between the block units and the block-to-reinforcement interaction.</p>  <p>Figure 3.6. Facing bulging from (Transfund New Zealand, 2003).</p> | <p><u><i>Mechanism 4a Shear Failure ( Bulging) of Facing Column</i></u></p> <p>Applied lateral pressure contributes to bulging instability.</p> | <p><u><i>Mechanism 4a Shear Failure ( Bulging) of Facing Column</i></u></p> <p>Shear capacity between the wall units</p> <p>Contribution of the reinforcement with adequate vertical spacing assigned to decrease the contributing lateral pressures.</p> <p>Normal component of wall unit self-weight gives a shear capacity magnitude.</p> |
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Table 3.1. Mode of Instabilities, Driving and Resisting Factors (Cont.).

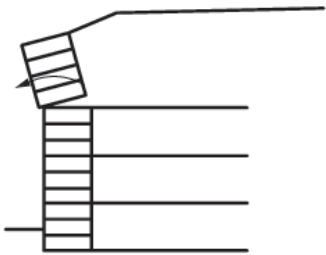
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| <p><u><i>Mechanism 4b Unreinforced Crest Failure</i></u></p> <p>Special attention is given to the block units above the highest reinforcement layer, to perform as a free standing retaining wall capable of withstanding sliding and toppling failure modes as if it was a conventional retaining wall.</p>  <p>Figure 3.7. Unreinforced crest toppling from (British Standards Institution, 2012).</p> | <p><u><i>Mechanism 4b Unreinforced Crest Failure</i></u></p> <p>Horizontal earth pressure acting along the segmental wall units through to the point where highest reinforcement is placed.</p> <p>Vertical batter of the wall, if any, should be considered in calculating the earth pressures.</p> | <p><u><i>Mechanism 4b Unreinforced Crest Failure</i></u></p> <p>Weight of the facing units is important for stability.</p> <p>Stacked units need to be proportioned in a manner that ensures the resultant force to act within the middle third of the base unit under worst loading combinations.</p> |
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Table 3.1. Mode of Instabilities, Driving and Resisting Factors (Cont.).

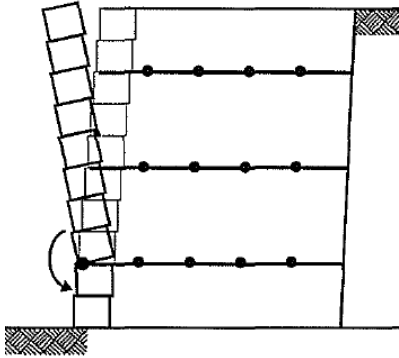
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| <p><u><i>Mechanism 4c Local Overturning</i></u></p> <p>Involves driving moments about the toe of a connection to cause wall segments to overturn. This effect is mostly exacerbated when the moment action is about the toe at the bottom of the wall.</p>  <p>Figure 3.8. Local overturning from (Transfund New Zealand, 2003).</p> | <p><u><i>Mechanism 4c Local Overturning</i></u></p> <p>Driving moments from active horizontal thrusts acting upon the wall segments from the backfill lateral earth pressures.</p> | <p><u><i>Mechanism 4c Local Overturning</i></u></p> <p>Shear resistance between the block units, as well as between block units and the reinforcement</p> <p>Resisting moments from the soil self-weight and the surcharge loads.</p> |
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Table 3.1. Mode of Instabilities, Driving and Resisting Factors (Cont.).

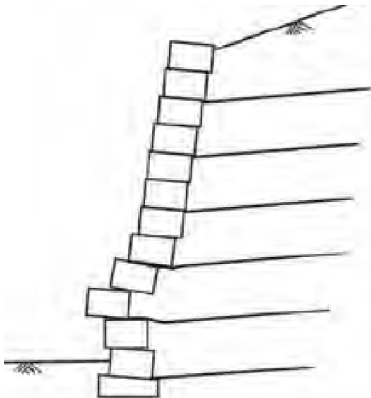
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| <p><i>Mechanism 4d Block Sliding Failure</i></p> <p>Caused by horizontal pressures applied between the reinforcement layers being greater than the shear capacity in all the facing units. This can cause the blocks to slide against each other.</p>  <p>Figure 3.9. Block sliding failure from (Government of the Hong Kong Special Administrative Region, 2002).</p> | <p><i>Mechanism 4d Block Sliding Failure</i></p> <p>Lateral earth pressure acting on the segments.</p> <p>Deformability of facing can insinuate different forms of vertical differential settlement between facing elements and reinforced earth structure.</p> | <p><i>Mechanism 4d Block Sliding Failure</i></p> <p>Facing units shear capacity, vertical spacing of reinforcement layers and facing column weights provide the resistance against block sliding.</p> <p>Block-to-Block shear strength comes from a greater normal load acting above a particular joint.</p> |
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Table 3.1. Mode of Instabilities, Driving and Resisting Factors (Cont.).

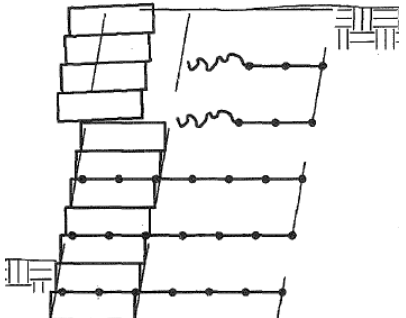
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| <p><u><i>Mechanism 5: Connection Failure (Internal)</i></u></p> <p>The connection interface between the reinforcement and the facing unit needs enough capacity to withstand rupture or slippage, although the full tensile load acting on the reinforcement will shed before reaching the facing.</p>  <p>Figure 3.10. Facing connection failure from (National Concrete Masonry Association, 2010).</p> | <p><u><i>Mechanism 5: Connection Failure (Internal)</i></u></p> <p>Applied tensile forces can cause rupture or slippage on the reinforcement near facing elements.</p> <p>Shear and bending stresses causing settlement of fill relative to the facing.</p> <p>Such tensile forces are a manifestation to resisting the fill self-weight, surcharge and overturning moment caused by earth pressures.</p> <p>Differential settlements can cause additional strain on the reinforcements and stresses on the connections (Government of the Hong Kong Special Administrative Region, 2002)</p> | <p><u><i>Mechanism 5: Connection Failure (Internal)</i></u></p> <p>Vertical and horizontal spacing between reinforcements</p> <p>Sufficient overlap between facing panels</p> <p>Friction developed between blocks and soil reinforcement.</p> |
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Table 3.1. Mode of Instabilities, Driving and Resisting Factors (Cont.).

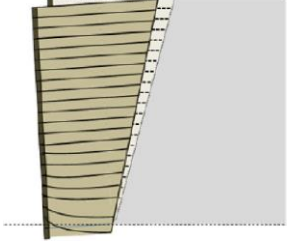
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| <p><u>Mechanism 6: Bearing Capacity (External)</u></p> <p>Foundation soils can undergo overstressing from the GRS structure leading to bearing capacity failure</p>  <p>Figure 3.11. Bearing capacity failure from (Federal Highway Administration, 2012).</p> | <p><u>Mechanism 6: Bearing Capacity (External)</u></p> <p>Bearing pressure from the GRS body acting on the base of the wall.</p> <p>A contribution of the GRS self-weight and any surcharge load acting on top of the wall.</p> | <p><u>Mechanism 6: Bearing Capacity</u></p> <p>Bearing Capacity of Underlying Foundation</p> |
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Table 3.1. Mode of Instabilities, Driving and Resisting Factors (Cont.).

|                                                                                                                                                                                                                                                                              |                                                                                                                                                             |                                                                                                                                                                                                             |
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| <p><u>Serviceability Mechanism 1</u><br/>Intrinsic Settlement of Fill Material</p> <p>The reinforced volume from within undergoes intrinsic settlement. Especially when the fill relative to the facing can settle and induce increased loading.</p>                         | <p>Vertical pressures acting within the fill.</p>                                                                                                           | <p>Proper compaction</p> <p>Construction methodology of depositing in layers must allow for tolerances to accommodate internal deformations of the fill material along with the reinforcement elements.</p> |
| <p><u>Serviceability Mechanism 2</u><br/>Differential Settlement</p> <p>Longitudinally, walls are prone to portions undergo elaborate settlement than other parts, causing an overall tilting of the wall, which can have a severe impact on the structural functioning.</p> | <p><u>Serviceability Mechanism 2</u></p> <p>Natural behaviour of a soil with varied ground properties induces settlement to occur in parts of the wall.</p> | <p><u>Serviceability Mechanism 2</u></p> <p>Understanding of tolerances acting on the wall</p>                                                                                                              |

Table 3.1. Mode of Instabilities, Driving and Resisting Factors (Cont.).

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| <p><u>Serviceability Mechanism 3</u></p> <p>Frontal and Horizontal Displacement of Wall Front</p> <p>Reinforcements develop an increase of strain, causing facings to undergo displacements as a manifestation in the length change of individual reinforcement layers.</p> | <p>Individual reinforcement layers change in length as loading is increased.</p> | <p>Axial stiffness of reinforcement layers</p> <p>Compaction efforts</p> <p>Reinforcement length</p> <p>Facing system deformability</p> |
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Table 3.1. Mode of Instabilities, Driving and Resisting Factors (Cont.).

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| <p>Surface Displacement from Shear Deformations</p> <p>Horizontal displacements occurring within the reinforcement from the above mechanism can cause vertical displacements to occur within the internal surface of the facing.</p> | <p>Tension in reinforcement layers undergoing strain causes horizontal displacements to occur within the reinforcements, and would manifest to vertical displacements occurring near the wall facing surface.</p> | <p>Connections between soil reinforcements and the blocks.</p> <p>Self-weight of blocks producing shear strength</p> <p>Connection strength between blocks and reinforcing elements</p> |
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### 3.2. Conclusions

- The GRS structure is designed against commonly postulated failure modes. These are categorised as External (Sliding Failure), Internal (Sliding Failure, Reinforcement Elongation and Rupture, Pullout, Facing Instabilities, Connection Failure) and Compound failures (Bearing Capacity failure) for Ultimate Limit State. Serviceability Limit State failures include Foundation Settlement (including Differential), Intrinsic Settlement of Fill Material, Wall Frontal and Horizontal Displacement as well as Surface Displacement from Shear Deformations.
- The key driving forces are self-weight of reinforced soil and external surcharge loads
- The key resisting forces are soil's inherent strength properties (friction angle and cohesion), settlement properties and the reinforcement tensile strength.
- Special design methods are carried out depending on the kinematic constraints postulated for the failure mechanism; Coherent Gravity Method (wall is able to move outwardly away about the top of the wall crest) and Tie-Back Method (wall is able to move outwardly away about the wall base)

## 4. SOIL BEHAVIOUR IN SOIL-REINFORCEMENT INTERACTION

### 4.1. Reinforced Soil Behaviour

A reinforced soil has a more enhanced shear resistance, which stems from both the soil shear resistance and the tension force in the reinforcement. The soil standalone is understood to have a high compressive strength but with a weak tensile strength. Once loaded under a compressive stress regime, tensile strains will evolve at an orthogonal direction within the soil mass. A reinforced soil undergoing the same compressive stress regime is capable to withstand both the compressive strain and the orthogonal tensile strain to a greater extent compared to an unreinforced soil. Figure 4.1 below illustrates this extent of vertical and horizontal strains for both soil element cases, where a compressive stress  $\sigma_1$  of magnitude greater than the confining pressure  $\sigma_3$  is being applied. Both the axial compression and lateral expansion in the reinforced case is lower than the unreinforced case. Such strain reduction in the reinforced case stems from the internal interaction between the soil and the reinforcement producing an additional confining stress  $\Delta\sigma_3$  (British Standards Institution, 2012; Manceau *et al.*, 2012).

As reinforcement has a higher tensile stiffness than soil, the lateral movement of the reinforced soil element is only manifested if the soil can move relative to the reinforcement. A reinforcement having a substantial roughness would prompt generation of shear stresses in the soil/reinforcement interface. This is reflected in the earlier internal confining stress  $\Delta\sigma_3$ , which as a result of the internal interaction brings about an overall external effect of a reduced axial and lateral deformations in the reinforced soil mass (British Standards Institution, 2012; Manceau *et al.*, 2012).

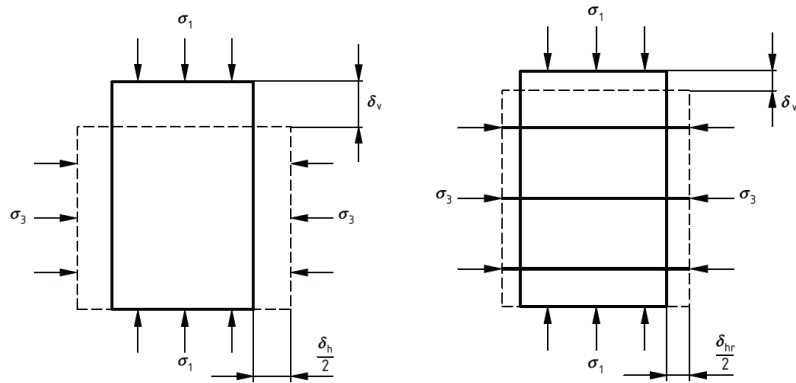


Figure 4.1. Comparison between unreinforced (Left) and reinforced (Right) in soil element from (British Standards Institution, 2012).

The added reinforcements had resulted to larger  $\sigma_1$  value needed to induce a sufficient amount of  $\sigma_3$  to substantiate a greater strain.

It may be conceived that the reinforced soil is analogous to reinforced concrete, but Jones (1996) (by referring to Vidal 1963, 1966, 1969) stresses otherwise. The mode of action with regards to reinforcement in the soil is not one of developed tensile stresses being carried, but rather to do with the anisotropic reduction or suppression of a normal strain rate. Considering soil particles being tied together to form a pseudo-cohesion, this mass of particles when expanded laterally reduce from lateral stress at-rest  $K_0\sigma_v$  to a limiting value  $K_a\sigma_v$ , as depicted in Figure 31. Figure 30 for the unreinforced soil element case encapsulates this where the final state of complete lateral expansion is the  $K_a\sigma_v$ . Colin (1996) corroborates with the British standard (2012) narrative with regards to the reinforcement in that the adhesion or the frictional interaction between the reinforcement and soil will cause the soil to be restrained by an equivalent at-rest lateral force, which restricts the anisotropic normal strain. Colin (1996) further states that failure is only manifested when the reinforcement ruptures, or the adhesion between the reinforcement and soil fails (i.e. slip out failure), and thus causing the stress circle to always lie below the rupture curve.

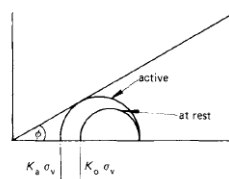


Figure 4.2. Mohr Circle reducing from at rest to Active state within Mohr Coulomb failure envelope from (Jones, 1996).

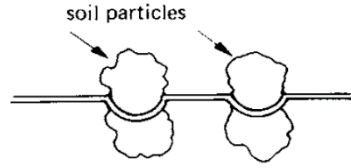


Figure 4.3. Soil particles restrained by the reinforcement interaction from (Jones, 1996).

As the restraining earth pressure is an at-rest pressure, the normal force transferred from the soil to the reinforcement is equivalent to the lateral stress  $K_0\sigma_v$ . To quantify tensile stress in any unit reinforcement would in turn be:

$$\text{Tensile stress in Unit Reinforcement} = \frac{K_0\sigma_v}{a_r} \quad (4.1)$$

By assuming that the soil's lateral strain is in the direction of the reinforcement, the reinforcement strain,  $\epsilon_r$  would be:

$$\epsilon_r = \frac{K_0\sigma_v}{a_r E_r} \quad (4.2)$$

where  $E_r$  is the elastic modulus of the reinforcement

A high effective stiffness would imply an infinitesimal strain to occur and on the other hand, a smaller effective stress (i.e. negligible) would cause the  $\epsilon_r$  to increase, and the pressure coefficients to reach active. (Jones, 1996)

The British standard (2012) however, though without any substantiation of proportional quantities, had taken a rather conservative approach in that lesser strain would occur in a reinforced soil mass rather than an infinitesimal strain as Jones (1996) states.

This now implores another inquiry into what extent the mobilization of each of the synergising strength properties of both the soil and the reinforcement are occurring.

Jewell (1996) highlights a phenomenon about the soil-reinforcement interaction behaviour, known as Strain Compatibility. It involves attaining an equilibrium state between the amount of strain developing in the soil and that counteracted by the reinforcement. Such understanding of the compatibility would enable to discern appropriate design values assigned for the reinforcement force so that there is a more

synergised mobilization of the soil shearing resistance along with the reinforcement strain. The equilibrium sought by this method would also constitute an acceptable deformation for the structure.

To further illustrate this, Jewel combines a compatibility curve for determining such equilibrium (Figure 4.6) from figures showing relation of the soil mobilisation along with strain, and the mobilised reinforcement force from the reinforcement which corresponds to a specific loading period  $t_d$  and temperature  $T_d^\circ$  in the reinforced soil structure ( $t_d, T_d^\circ$ ). The compatibility curve makes an assumption that there is equal tensile strain in the reinforcement and in the soil in the reinforcement direction.

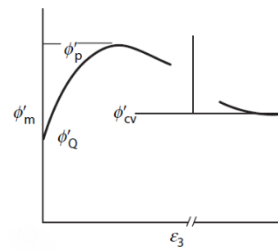


Figure 4.4. Mobilised soil shearing resistance from (Manceau *et al.*, 2012).

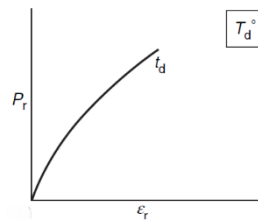


Figure 4.5. Mobilised reinforced force  $P_r$  at loading period  $t_d$ , and temperature  $T_d$  from (Manceau *et al.*, 2012).

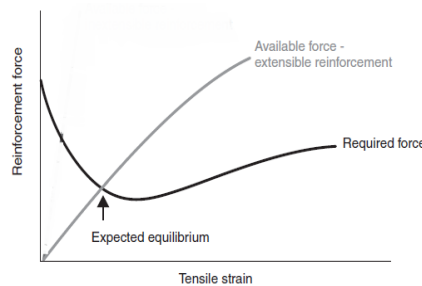


Figure 4.6. Compatibility curve for reinforced soil from (Manceau *et al.*, 2012).

The reinforced soil structure is first envisioned to be held in equilibrium by imaginary external forces (contrary to the function of an internally stabilised retaining wall). This equilibrium zero tensile strain will correspond to an earth pressure at rest in the soil and zero reinforcement force. Once the external support is removed, deformation and tensile strain starts to develop.

In concurrently inspecting at the soil behaviour curve and the compatibility curve, the soil shearing resistance will increase (depicted by an increased mobilised friction angle) until it reaches the peak shearing resistance  $\phi_p$ . Beyond this point (referred to Point A for the purpose of explanation), further strain deformation from the soil will not be catered for under the same capacity, as the angle of shearing resistance falls towards a plateaued value of constant-volume angle of friction ( $\phi_{CV}$ ) possessing a high large strain (Jewell, 1990b, 1996; Manceau *et al.*, 2012).

The force mobilised from the reinforcement is then sought after to compensate for the lower capacity available to withstand the strain. The available force mobilised as shown in the curve shows a rate of increase in available force (which would depend on the reinforcement stiffness properties, number of layers intersecting the potential rupture surface). A condition of equilibrium is only then achieved once the required and available force intercept on the compatibility curve. The need for such imaginary external support at this condition is completely removed at this stage (Jewell, 1990b, 1996; Manceau *et al.*, 2012).

Based on Jewell's Strain Compatibility interaction, an understanding the soil shear strength would provide insight in the extent into which it can be independently utilized to withstand the strain deformations before resorting to the mobilisation of reinforcement strength.

#### **4.2. Soil Properties and Its Interaction with the Reinforcement**

Earlier, the several modes of instability highlighted the importance of soil strength plays in resisting the failure mechanisms.

The load transfer mechanism between the soil and reinforcement also pronounces the importance of the soil characteristics. In particular is the pullout mode

of failure which depends upon the bond resistance type present which is dictated in whether or not the soil is cohesive; A cohesive soil would pertain to having an adhesive bond type and a cohesionless soil a frictional with cohesionless soil (British Standards Institution, 2012). The shear strength also impacts the load transference from the soil to the reinforcement, as was elucidated under the strain compatibility method in section 5.1. (British Standards Institution, 2012).

This steers the importance of investigating further the strength and deformation characteristics of soil, whether it is compact, granular soils or soft clays. However, the research is geared towards the more commonly used granular frictional type of soil i.e. silts, sands and gravels as the cohesive clayey soils is seldom used due to impracticalities in the construction process. For example, such soils are normally poorly drained and would require a comprehensive drainage regime during construction. The excess pore water pressure developments associated with the soils during construction cause a short-term instability to occur. There can also be post-construction movements as a result of the cohesive soils exhibiting elasto-plastic or plastic behaviour (Saran, 2010).

#### **4.2.1. Soil Shearing Resistance: Effective Stress Concept, Peak and Critical State Friction Angle**

The shearing resistance of the soil as a frictional material depends on the effective stresses (Atkinson and Bransby, 1982; Jewell, 1996). Atkinson and Bransby (1982) explain how Terzaghi's effective stress principle regards to a combination of total stress and pore pressure ( $\Delta u$ ), and that such relevant combination is what determines the soil behaviour. For explanation purposes, the granular material will be assumed to be highly permeable, with any potential pore water pressure build up to immediately dissipate. This in turn implies that the external stress applied is directly transferred between the solid soil particles, thus altering the soil strength, in line with Terzaghi (Atkinson and Bransby, 1982)'s statement on effective stress:

“All measureable effects of change of stress, such as compression, distortion, and a change in shearing resistance, are exclusively due to changes in effective stresses”

The applied external stress regime causes the shearing resistance of the soil to be mobilised to withstand the stress regime. The soil would remain in static equilibrium provided that its mobilised shearing resistance is less than the peak shearing resistance.

The mobilised angle of friction  $\phi'_m$  is used to highlight the effective stress state to withstand the applied external stress regime, with the peak angle of friction  $\phi'_p$  used to highlight the effective stress combinations at the soil's full strength capacity. These two angles are shown under the Mohr Circle in figure 4.7.:

$$\frac{t}{s'} = \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 + \sigma_3)} = \sin \phi'_m \quad (4.3)$$

$$t = \frac{\sigma_1 - \sigma_3}{2}$$

$$s' = \frac{\sigma_1 + \sigma_3}{2}$$

With regards to reinforced soil, the tensile strain in the soil being prevented by the stiff reinforcement implores the need to have a more deciphered approach with regards to specifying angles of friction that indicate when tensile strains begin to develop.

Jewell (1990) highlights two limiting angles of friction

- At-Rest Angle of Friction  $\phi'_o$
- Critical State Angle of Friction  $\phi'_{cv}$

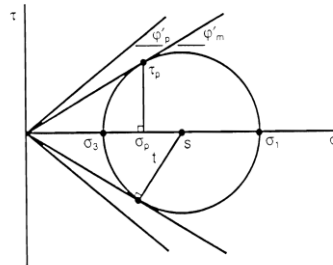


Figure 4.7. Mobilised and peak frictional forces for soil from (Jewell, 1990b).

Jewell (1990) highlights the first limiting friction angle, known as the at rest  $\phi'_o$ , which indicates that when the soil's effective stress state combinations are at or below  $\phi'_o$ , the soil is wholly compressible.

Based on the collating experiments made by Ladd *et al.* (1977) and Mayne and Kulhawy (1982), granular soil under at rest with zero net lateral strain is found to corroborate with the earth pressure at rest found by Jaky's equation from Wroth (1972):

Based on the collating experiments made by Ladd *et al.* (1977) and Mayne and Kulhawy (1982), granular soil under at rest with zero net lateral strain is found to corroborate with the earth pressure at rest found by Jaky's equation from Wroth (1972):

$$K_o = 1 - \sin \phi'_p \quad (4.4)$$

Using Jaky's equation relating the coefficient of earth pressure at rest and the peak friction angle, the corresponding at rest friction angle  $\phi'_o$  can be deduced geometrically from Mohr's circles to give

$$\phi'_o = \sin^{-1} (1-K_o)/(1+K_o) \quad (4.5)$$

However, in re-revisiting Ladd's work (shown in figure 4.8) , the corroboration does hold but up to a cut-off value of  $K_o = 0.35$ . Such cutoff is due to the mobilised friction angle exceeding the critical state friction angle  $\phi'_{cv}$  (i.e.  $\phi'_m = \phi'_o > \phi'_{cs}$ ) implies a dilatant behaviour, which is virtually incompatible with the boundary condition associated with the at-rest case of zero lateral strain. The zero lateral strain holds true only when the mobilised friction angle is less than the critical state friction angle  $\phi'_{cv}$ . The cut-off  $(K_o)_{min} = 0.35$  connotes a friction angle of  $29^\circ$ , which is slightly less than the minimum critical state angle of friction for granular soil  $\phi'_{cs} = 30^\circ$ .

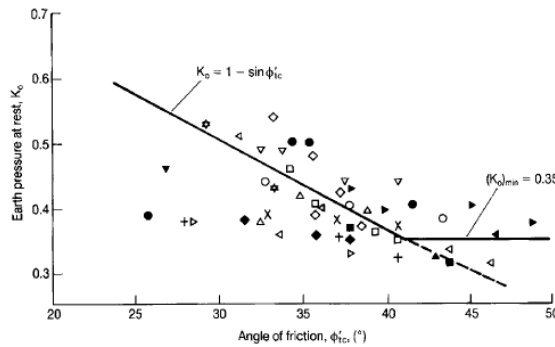


Figure 4.8. Ladd's work of earth pressure at rest versus angle of friction from (Jewell, 1990b).

The other limiting angle of friction, critical state angle of friction  $\phi'_{cv}$ , is best understood by discerning the components contributing to the mobilised angle of friction  $\phi'_{mob}$  (Rowe (1962) ;Terzaghi *et al.* (1996). Sadrekarimi and Olson (2011) describe how the shear resistance is mobilised through the sliding friction between the soil particles (denoted by the interparticle friction angle  $\phi'_\mu$ ) and geometrical interference i.e. particle interlocking (denoted by friction angle  $\phi'_g$ ).

The interparticle friction angle depends upon the surface roughness (Rowe, 1962), which stems from the strength, texture, hardness of the particles (Terzaghi *et al.*, 1996).

With regards to the particle interlocking, the shear resistance is mobilised by particles pushing against, climbing over and damaging the adjacent particles by abrading and fracture. Terzaghi *et al.* (1996) further distinguishes two inner mechanisms within particle interlocking and mobilised resistance. The first is by dilation,  $\psi$ , and the other is the mobilised resistance by particle rearrangement and damage ( $\phi'_{r \& d}$ ).

Dilation is understood through a simplified analogy of Houlsby (1991) saw-tooth model. The first model where one frictional block slides over a flat plane will not induce any volume change, whereas the second model is of the block sliding on a rough plane will consequently have the particles climb up over each other (Sadrekarimi and Olson, 2011) and invoke a volume change. The roughness is substantiated using the dilatancy angle,  $\psi$ .



Figure 4.9. Block Slide versus Saw-Tooth model from Houlsby (1991).

The constant-volume, or critical state friction angle  $\phi'_{cv}$ , is the friction angle of a soil with no inherent dilation, i.e. no tendency for volume changes to occur when

undergoing shear. Such a condition arises when the soil void ratio reaches the critical void ratio value (Casagrande, 1936; Ranjan and Rao, 2005)

Sadrekarami and Olson(2011) draws a conclusion from Sture *et al.* (1998) draws a conclusion from Sture *et al.* (1998) experiments in that as the initial void ratio increases, the contribution to dilation  $\psi$  decreases, to the point where the resistance mobilised from interparticle interlocking is predominantly from particle rearrangement and damage  $\phi'_{r \& d}$

The critical state friction angle  $\phi'_{cv}$  with no dilation contribution becomes:

$$\phi'_{cv} = \phi'_{\mu} + \phi'_{r \& d} \quad (4.6)$$

Overall, Jewell (1990) adopts the 3 friction angles,  $\phi'_o$ ,  $\phi'_{cv}$ , and  $\phi'_p$  to establish three regions of soil's shearing resistance being mobilised in relation to the tensile strain with shear, shown on figure 46 below. Any effective stress combinations for a friction angle below  $\phi'_o$  insinuates a complete compressive strain in the soil, whereas a strong development of tensile strain pertains to effective stress combinations between  $\phi'_p$  and  $\phi'_{cv}$ .

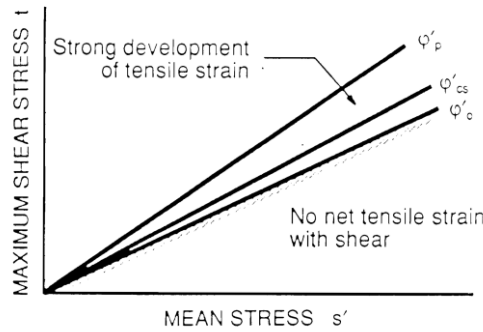


Figure 4.10. Friction angle envelopes for tensile strain ranges from Jewell (1990).

#### 4.2.2. Soil's Peak Shearing Resistance

Bolton (1986) developed an empirical relative density index  $I_R$  which relates the soil relative density and the mean effective stress of the soil.

$$I_R = I_D(10 - \ln s') - 1 \quad (4.7)$$

With relative density,  $I_D$  equal to  $(e_{max} - e) / (e_{max} - e_{min})$ ;

And mean effective stress  $s' = (\sigma'_1 + \sigma'_2 + \sigma'_3) / 3$  (4.8)

The relative density convention,  $I_D$ , is indicative of how dense the soil is relative to its densest state (depicted by void ratio  $e_{min}$  with an  $I_D$  value of 1) or from its loosest state (depicted by  $e_{max}$  with an  $I_D$  value of 0). Cornforth (1973) put forth that relative density has a stronger correlation to measuring the sand strength compared to void ratios as it is able to compensate for the particle grading and shape effects which influence  $e_{max}$  and  $e_{min}$ , albeit their use in directly calculating relative density.

The mean effective stress,  $s'$ , depicts the extent in which all the principal stresses are the same (Wood, 1990) while in the same time gives an appreciation on the size of the average principal stress acting upon the soil element.

Bolton (1986)'s empirical correlation for  $I_R$  however is only feasible in value ranges of 0 to 4, which insinuates a limiting cut-off at  $s'$  of 150 kPa. Cases where at 10 metres and less would have a simplified correlation of

$$I_R = 5 I_D - 1 \quad (4.9)$$

In which the mean effective stress component is now suppressed. Bolton then related the peak angle of friction with the relative density index depending on the loading regime acting upon the soil: Plane Strain and Triaxial Compression.

$$\phi'_{p-plane strain} = \phi'_{cv} + 5I_R \quad (4.10)$$

$$\phi'_{p-Triaxial Compression} = \phi'_{cv} + 3I_R \quad (4.11)$$

Plane Strain loading involves the spanning of the load in one direction to be significantly greater than the other two directions (Fredlund and Rahardjo, 1993), whereas the Triaxial compression is of an axisymmetric loading nature with a major principal load being greater than the minor principal stress loads. With such difference in nature for the two loading regimes, the coefficients of 5 and 3 multiplied with  $I_R$  above the critical state friction angle yield different peak angles of friction. The figures 4.11 and 4.12 below show the relationship between  $I_D$  and peak friction angle. An increase in  $I_D$  pronounces a greater increase in the peak angle of friction, which when

fixed whilst the mean effective stress increases causes a smooth decrease in the peak angle of friction.

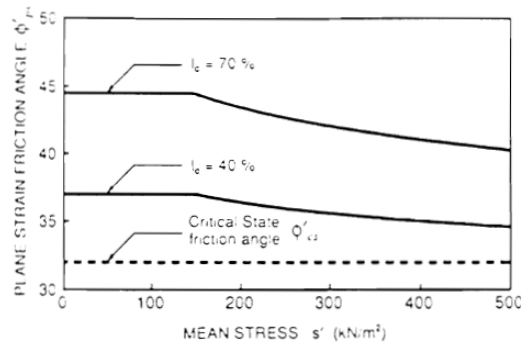


Figure 4.11. Peak plane strain angle of friction variation with relative density and stress level of granular soil from (Jewell, 1990b).

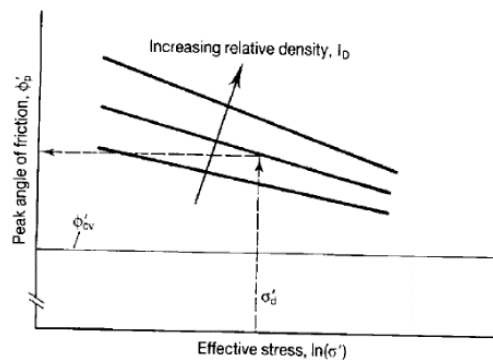


Figure 4.12. Peak angle of friction varying with mean effective stress in granular soil from (Jewell, 1996).

#### 4.2.3. Compaction and Peak Friction Angle

The peak friction angle with relation to relative density implores the need to how compaction, in its action to obtain the maximum dry density, is able to impact the soil strength. Compaction was accentuated in many design and construction standards as to how it impacts the overall lateral displacement of the wall or as an enhancer to the performance characteristics of soils with lower friction (Ingold, 1994; Wu *et al.*, 2006).

Compaction refers to the process of increasing the bulk density of a soil by the particles coming closer together by reducing the air volume (Knappett and Craig, 2012). Such effect soil particles become more closely packed together as to have their voids minimised and the particle interlock maximised (Manceau *et al.*, 2012). This is attributed by the particles reorienting, fracture of grains as well as reorientation and

bending/distortion of the particles, which all ultimately bring about increased contact between the soil grains (Aggour and Brown, 1974)

The soil whilst undergoing shearing would require the soil in the sheared region to dilate i.e. to work against the confining forces, as well as work against the frictional forces. It brings an increase to the strength of the granular material as a result of increasing the values of overall internal angle of friction (Cernica, 1995).

The degree of compaction of a soil is measured in terms of dry density and this dry density depends on the water content and the energy supplied by the compaction equipment which is called the Compactive Effort (Craig and Knappett, 2012). The dry density would equal to:

$$\rho_{dry} = \rho / (1 + w) \quad (4.12)$$

where  $\rho$  is the Bulk Density and  $w$  is the water content.

The goal of the compaction process is to achieve an optimized maximum dry bulk density  $\rho_{max}$ , which depends two-fold on the water content and the compaction effort. The best approach in understanding this is to look at the impact of water content stand-alone for a given compaction effort. An optimum level of water content would enable the densification with minimum compaction effort (Aggour and Brown, 1974).

Craig and Knappett (2012) explain that a test is carried out for a given soil sample in which bulk density and the water content are determined to calculate the dry density for a given compaction effort. The test is repeated 5 times at increased water contents to give a graph shown on the right. The maximum dry density value is achieved at a particular water content called the optimum water content,  $w_{opt}$ .

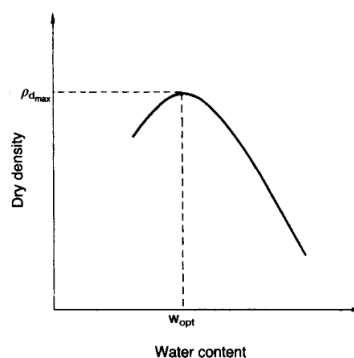


Figure 4.13. Relationship between dry density and water content from Craig and Knappett (2012).

When the water content is lower than  $W_{opt}$ , the soil material tends to be more stiff and difficult to compact, whereas when the water content is higher than  $W_{opt}$ , the soil would become more workable and facilitate compaction to result to higher dry densities (Craig and Knappett, 2012). This stems from a positive pressure build up would in turn resist any reduce the compressive capillary forces at particle contacts to make it easier for particles to slip amongst each other (Thom, 2008) to ease compaction. However, there is a inhibiting effect of the water content on the dry density in that a further increase in the water content would cause an increasing proportion of the soil to be occupied by the water, making the soil in a state of full saturation (Craig and Knappett, 2012; Ishibashi and Hazarika, 2011).

Associating water content back with compaction effort,  $w_{opt}$  can be reduced as the compaction effort is increased, along with manifesting a much higher maximum dry density (Craig and Knappett, 2012), as highlighted under figure below:

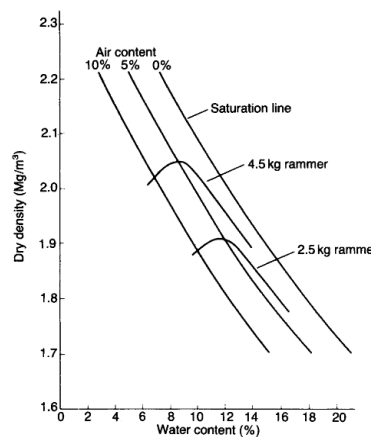


Figure 4.14. Relationship of dry density and water content under various compaction effort regimes from Craig and Knapett (2012).

Thom (2008) explains how compaction effort has a profound effect on the soil strength, when investigated under various uniformity coefficients for each individual compaction effort. The Uniformity Coefficient  $C_u$ , a ratio of the sieve size through which 60% of the material passes to that through which 10% passes, is used to indicate the particle size distribution. A granular material that is well graded tends to have a higher angle of friction than a gap-graded soil Thom (2008). The increased number of particle contacts in the well-graded soil causes a failure plane developing to be more

difficult (Thom, 2008), thus causing a tendency for the soil to gain strength. However, the extent at which the granular soil is well-graded poses a limitation in that a greater portion of small size particles can dominate the points of contact between the large particles causing them to be separate from one another (Thom, 2008). Figure 4.15 below shows that for each individual compaction level, though a higher compaction effort yields a greater stress ratio at failure, there is always a cut-off for the optimised uniformity coefficient, albeit that the compaction effort overall for an individual uniformity coefficient possesses a greater soil strength.

Stress Ratio at failure is the limiting ratio of two orthogonal stresses limited by the internal friction angle  $\phi$  by the Mohr Circle Geometry (Thom, 2008). The relationship is given as:

$$\sigma_1/\sigma_2 = (1+\sin\phi)/(1-\sin\phi)$$

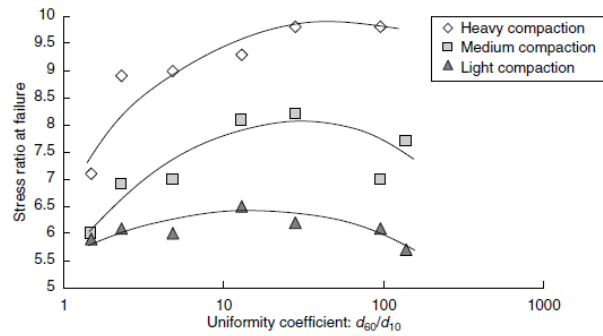


Figure 4.15. Relationship between Stress Ratio at Failure and Uniformity Coefficient from Thom (2008).

#### 4.2.4. Peak Angle of Friction of soil, Void Ratio and Specific Volume

*Using Maximum Dry Density to find Relative Density  $I_D$*

Jewell stated that the maximum dry density would correspond to the minimum void ratio when attempting to establish the relative density  $I_D$  for any given field condition.

$$e_{min} = \frac{G_s \gamma_w}{\gamma_{max}} - 1 \quad (4.13)$$

A field condition to be compacted to 95% would entail:

$$e_{field} = \frac{G_s \gamma_w}{0.95 \gamma_{max}} - 1 \quad (4.14)$$

This would directly fit in the relative density equation for  $I_D$  with the use of  $e_{\max}$  and  $e_{\min}$  of the soil.

A particular benefit to utilise the void ratio in understanding peak friction angle is by adopting its specific volume form,  $v' = 1 + e$ , which in essence is the volume of an element soil containing a unit volume of mineral grains, which is an indication of any volumetric strain  $\varepsilon_v$  on the soil based on the equations below:

$$\delta\varepsilon_v = \frac{\delta e}{(1+e)} = \frac{\delta v}{v} \quad (4.15)$$

The soil's specific volume and mean effective stress state when plotted with its positioning distance away from the critical state line gets inspected, the difference in its current and its inherently critical state specific volume highlights an increase in peak friction angle. This can also be seen with its initial over-compression value (current mean effective stress/critical state mean effective stress) (Jewell, 1990b; Wood, 1990)

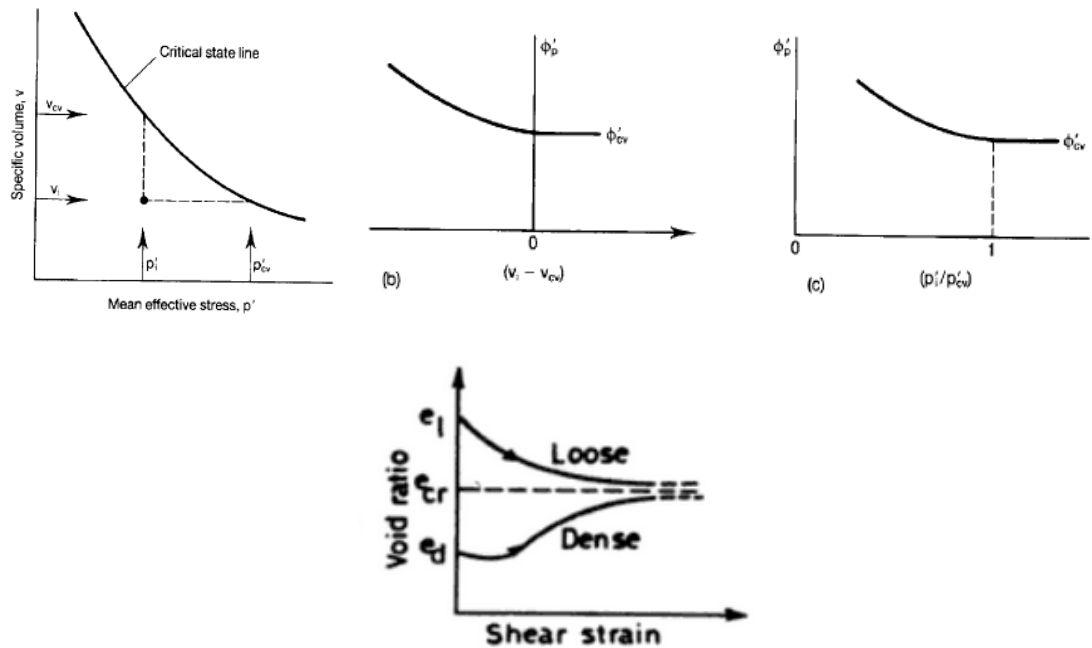


Figure 4.16. Various trends for peak frictional resistance with distance from Critical State Line from Jewell (1996). Figure 4.16.a. enables to indicate the distance. The information can be implemented in 4.16b to indicate magnitude of Peak Friction Angle, and Figure 4.16c provides the Initial Over-compression Magnitude

The behaviour of a soil possessing a specific volume different from its critical state is best understood by looking at the stress-strain and volume change behaviour of loose sand (possessing a high initial void ratio) and dense sand (possessing a low initial

void ratio (Ranjan and Rao, 2005). Figure 4.17 shows the difference in the results for deviator stress with axial strain for both dense and loose sands under the same confining pressure. The dense sand reaches a peak stress at a relatively low axial strain and then abruptly decreases reaching to a plateau as strain further increases, where no further change in deviator stress occurs (known as the ultimate stress) (Ranjan and Rao, 2005). The loose sand exhibits a different deviator stress-strain behaviour, in which the strain increase will induce a gradual increase in deviator stress with the maximum deviator stress magnitude approaches to a value close to the ultimate stress. The difference in the deviator stress-axial strain behaviour in the two soil types is also reflected in the volume change-axial strain behaviour. The dense sand receives a slight decrease in volume (contraction) and then undergoes dilation (expansion). Conversely, the loose sand would consistently decrease in volume and contract. (Aysen, 2002; Ranjan and Rao, 2005)

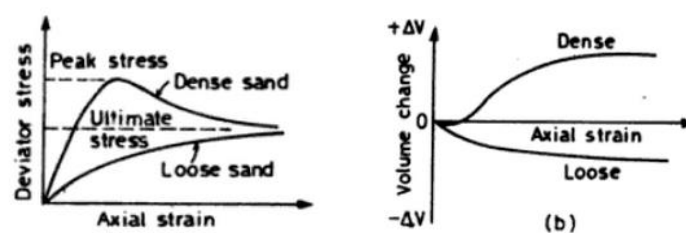


Figure 4.17. Stress Strain, volume change and void ratio curves from shear tests on sands from (Ranjan and Rao, 2005).

These behaviours, although invariably different, require the soil itself to be better discerned rather than using crude terms such as dense and loose sand.

A more pedantic approach is to assess volume change in terms of void ratio changes. Given the initial void ratio of the loose sand  $e_l$  and dense sand as  $e_d$ , the changes in void ratio with a given shear strain are examined. The sand coined as dense would have a slight decrease in void ratio followed by an increase, reaching a plateaued constant void ratio value, known as critical void ratio  $e_{cr}$  (Aysen, 2002; Ranjan and Rao, 2005). A soil would inherently not undergo further change in void ratio, hence volume change, with further shearing strain application. In a situation where the initial void ratio of the soil is equal to the critical void ratio  $e_{cr}$ , there will be no further volume change occurring.

The critical void ratio  $e_{cr}$  would essentially detect a benchmark for a volume change regime the sand soil can encounter. Sand with an initial void ratio greater than  $e_{cr}$  would undergo volume reduction and contrastingly when the initial void ratio is lower than  $e_{cr}$  would entail a volume expansion, or dilation. (Aysen, 2002; Ranjan and Rao, 2005)

The process of compaction increasing the bulk density of the soil explained earlier is analogous to having an increase in confining pressure. Such confining pressure increase is decisive in the actual behaviour of the soil dilating or contracting, as an increase in confining pressure results to a decrease in the critical void ratio. A dense soil undergoing shear can contract if the confining pressures are very high, similarly as a loose sand would dilate as if it was a dense sand if the confining pressure is very low (Aysen, 2002; Ranjan and Rao, 2005).

Figure 4.18. below shows the relationship between the Critical Void Ratio and Confining Pressure:

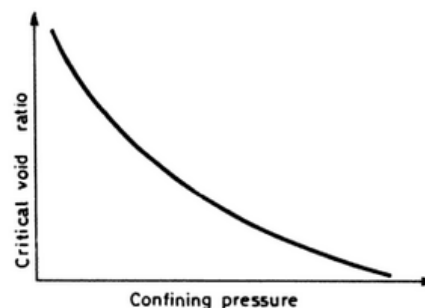


Figure 4.18. Relationship between Critical Void Ratio and confining pressure from Ranjan and Rao (2005).

#### 4.2.4. Dilation Effects

Aysen (2002) had represented the rate of volume change for the dense sand as a dilation angle  $\psi$  being the curve slope. The dilation angle is seen to decrease on the soil approaches the critical state where no further volume change occurs. Aysen (2002) had applied Rowe's definition of dilation in being a 'rate (or increment) of volume change with the rate (or increment) of shear strain (Jefferies and Been, 2006; Rowe, 1962).

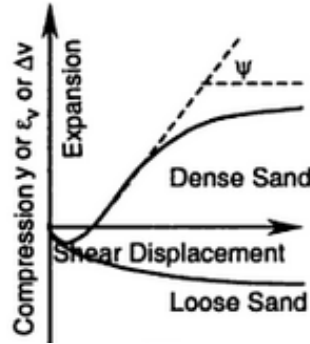


Figure 4.19. Dilation angle as a rate of volume change for dense sand from Aysen (2002).

Dilation, as previously explained earlier to be the phenomenon of volume change as a result of the particles overcoming the interlocking by climbing over each other (Sadrekarimi and Olson, 2011). To quantify dilatancy, it is more accurately defined as the ratio between volumetric strain rate to the shear strain rate (Houlsby, 1991; Jefferies and Been, 2006) as: (Houlsby, 1991; Jefferies and Been, 2006) as:

$$\sin\psi = \frac{-(\epsilon'_{1}-\epsilon'_{3})}{(\epsilon'_{1}+\epsilon'_{3})} \quad (4.15)$$

Jewell (1990) had underscored how the soil strain comprises of elastic and plastic components which while are respectively recoverable and irrecoverable. These would both occur when soil undergoes a form of shearing, which is in agreement with Jewell's statement on soil behaviour.

Referring back to the saw-tooth figure model in figure 46, the behaviour of particles climbing over each other is inherently a form of particle reorientation and plastic distortion (Jewell, 1990). Therefore, it is prudent that the dilation angle is defined in terms of plastic volumetric strain and plastic shear strain rate (Houlsby, 1991; Jefferies and Been, 2006; Jewell, 1990b) as follows:

$$\sin\psi = \frac{-(\epsilon'^p_{1}-\epsilon'^p_{3})}{(\epsilon'^p_{1}+\epsilon'^p_{3})} \quad (4.16)$$

This may come across as posing difficulties in undertaking experiments to determine the angle of dilation as it will depend on plastic properties of the soil. However, Houlsby overhauls this by explaining that the difference between using elastic and plastic strains is rather small, and given the high elastic stiffness nature of the soil, the elastic strains are much smaller than the plastic strains. Over the progression of shear tests, the stresses at peak value ranges are not changing causing the elastic strain rates to be set to zero (Houlsby, 1991).

The relevance in the importance of dilatancy with regards to unreinforced and reinforced soil design is attributed by Jewell (1990), in line with Bolton (1990)'s corresponding outlook of the elastic and plastic strain nature of the unreinforced soil. Jewell (1990) endorsed Bolton (1990)'s work into reinforced soil behaviour by focusing on the relationship between the mobilised angle of friction and tensile strain, in which two approximate upper and lower bound relationships are provided as to characterize the likely behaviour.

Distinguishing between the upper and lower bound relationships in a visual manner requires them to be benchmarked to a hyperbolic model of the soil strength relationship with the shear strain application (depicted as the stress ratio).

To deduce the hyperbolic model, the overall response of the dense granular soil is firstly illustrated. The shear modulus  $G$ , in its initial value reflects the true elastic behaviour of the soil. The change resulting from the shear strain increase is a manifestation of the plastic strain gaining prevalence.

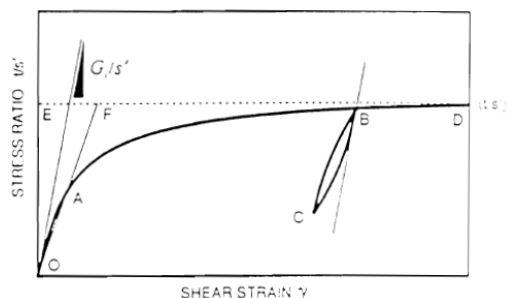


Figure 4.20. Parameter definitions for shear loading from Jewell (1990).

Jewell points out Kondner and Zelasko (1963)'s relationship between the maximum and initial shear stress difference to the shear Modulus. The graph on the left shows the hyperbolic curve of the stress ratio against shear strain based on the equation

$$(t - t_i) = \frac{G_i \gamma}{(1 + (\frac{G_i \gamma}{t_p - t_i}))} \quad (4.17)$$

Where  $t$  and  $\gamma$  are the current maximum shear stress and maximum shear strain.

$t_i$  is the initial shear stress

$t$  is the current shear stress

$t_p$  being the peak shear stress that the soil can sustain

$G_i$  being the initial (true) elastic Shear Modulus

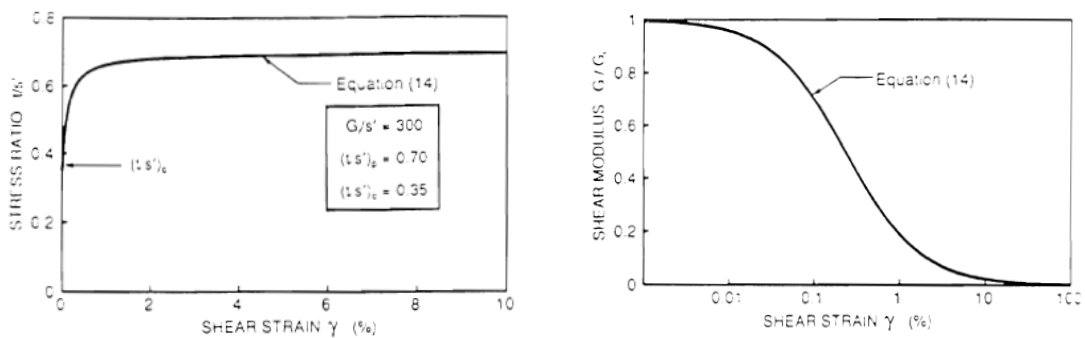


Figure 4.21.a. (Left) Hyperbolic model of a maximum shear stress (as Stress Ratio) versus shear Strain for compact granular soil sheared from at-rest conditions from Jewell (1990). Figure 4.21.b. (Right) is a relationship between shear modulus with shear strain from Jewell (1990).

The immediate left portion of the curve, with shear strain magnitude in the order of  $10^{-4}$  presents the true elastic shear modulus  $G$ , where the graph initiates in a more flat manner. Jewell points out to an expression derived by Hardin and Black (1966). Jewell points out to an expression derived by Hardin and Black (1966) and verified experimentally by Iwasaki *et al.* (1978) for the elastic shear modulus  $G$ , which renders the complexity in the shear modulus as it depends on the soil density and mean effective stress. Iwasaki *et al.* (1978) for the elastic shear modulus  $G$ , which renders the complexity in the shear modulus as it depends on the soil density and mean effective stress. Jewell (1990) states that the range of interest for a reinforced soil is found to be  $1000 > G/s' > 300$

$$\frac{G_i}{s_r} = 700 \frac{(2.17-e)^2}{(1+e)} \left(\frac{s'_i}{s_r}\right)^{\frac{1}{2}} \quad (4.18)$$

This renders the soil being sheared to its peak stress ratio has a true elastic component of shear strain in the order of magnitude  $10^{-3}$  to  $10^{-4}$ , which is relatively insignificant compared to the overall more pronounced plastic component.

Consequently, Jewel (1990) infers that the observed deformation under high stress ratio has been seen essentially as plastic. This leads to safely return to explaining the plastic strain's upper and lower bounds.

The upper bound treats the soil deformation to occur at maximum rate of plastic dilation  $\psi_{\max}$  from when shear is initiated, in which the maximum magnitude of tensile strain is induced developed is as per the following relationship of the tensile strain to shear strain (Jewell, 1990):

$$\frac{d\varepsilon_3}{d\gamma} = \frac{-(1+\sin\psi_{\max})}{2} \quad (4.19)$$

On the other hand, the lower bound refers to a tensile strain amount found by assuming the soil deforms with zero net tensile strain below the at-rest stress ratio, and beyond the at-rest ratio there will be no volume change in deformation. This deduces the minimum ratio of tensile strain to shear strain to be:

$$\frac{d\varepsilon_3}{d\gamma} = -0.5 \quad (4.20)$$

These bounds in fact coalesce with one another as the uncertainty in the soil behaviour between the two limits is finite (Jewell, 1990)

The same arrangement used for the elastic strain hyperbola to compare tensile strain with stress ratio to delineate the plastic strain bounds. The shear strain measured for generating the elastic strain hyperbolic models can be converted to the tensile strains using the following relationships:

For upper bound tensile strain:

$$\gamma = -\frac{2}{1+\sin\psi_{\max}} \varepsilon_3 \quad (4.21)$$

And the following equation for the lower bound (where the stress ratio  $t/s'$  is greater than  $(t/s')_0$  at rest:

$$\gamma = -2\varepsilon_3 \quad (4.22)$$

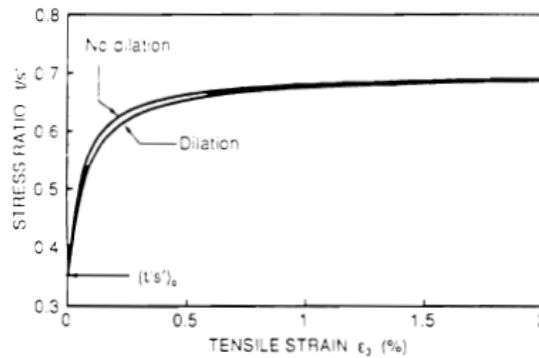


Figure 4.22. Hyperbolic models displaying small range of uncertainty between the two limiting models from Jewell (1990).

In conclusion, the graph deduces a remarkably small tensile strain can result to a rapid mobilisation of the shearing resistance of the soil. This is important considering that reinforced walls can be subjected to a tensile strain of up to 1% (Jewell, 1990).

#### 4.2.5. Compaction, Dilatancy and Critical State

The critical void ratio  $e_{cv}$ , specific volume  $v_{cv}$ , and friction angle  $\phi'_{cv}$  have all epitomised a clear-cut soil state where the volume remains constant as the soil is continuously sheared. These are based on the critical void ratio line, a work Roscoe *et al.* (1958) by coherently combining the works of Casagrande (1938) on the changes in volume in drained tests and Taylor (1948)'s work on undrained tests, in an attempt to define a unique line in which all loading paths converge in a  $p, e, q$  space irrespective of the loading path undertaken. Such line is known as the *critical void ratio line* (CVR).

Under drained test conditions, a sample reaches an ultimate state where any further increase in shear distortion will not cause any void ratio change, where the soil reaches a plateaued critical void ratio state. Conducting several tests at various confining pressures would result to a series of critical void ratio points to finely converge with the drained yield surface (Roscoe and Wroth, 1958). The yield surface

depicts a yield criterion, by adopting a 3-dimensional space, which defines a material to exhibit plastic deformations once the stresses satisfy the yield criterion (Verruijt, 2001)

Similarly, an undrained test condition, though no change of volume is permitted as a result of the undrained conditions, any effective stress alteration that would make the sample to reach ultimate limit state would do so at a particular void ratio, which remains constant during shear, which is again defined as the critical void ratio. Compared to the drained test conditions, the convergence of a series of critical void ratios the yield surface but to a less miniscule extent (Roscoe and Wroth, 1958).

Roscoe points out that should a unique line to which all the loading paths (drained or undrained) converge in the  $(p, e, q)$  space, such convergence would be to a CVR line, and that the yield surfaces whether drained or undrained would coincide with the CVR line. However, consideration must be taken into drawing such an analogy, as it takes a slight distortion to the sample to bring about the yield surface, whereas a larger sample distortion must be manifested for the load path to reach the CVR line. To avoid incongruities, Roscoe suggested that the loading paths converge to a 'bounded region' rather than characterizing a specific CVR line.

Atkinson and Bransby (1982), and Schofield and Worth (1968) adopted the specific volume  $v$  instead of  $e$  for the  $p, v, q$  space and refer the line as *critical state line* CSL, which portrays the same concept.

In summary, the critical state is where the soil would continue to undergo deformation at constant stress and at constant void ratio,  $e_{cv}$ . Another important feature to elaborate is that the void ratio has no propensity to change from such value and so with further shearing, it would remain constant (Jefferies and Been, 2006). Such nature of the critical state enables it to be a reference state to compare the actual void ratio of the soil, as the further distance from the critical state causes it to have a greater rate for dilation (or contraction) (Jefferies and Been, 2006).

An analogy will be drawn based on (Houlsby, 1991)'s explanation that the critical state line clear-cuts a situation where the dilation rate is zero.

This will be used again as a benchmark to quantify the dilatancy rate, which in turn would help appreciate the amount of plastic strain that the soil is susceptible to.

Houlsby uses parameters of  $v'=T$  on the Critical State Line and  $v'_\lambda$  for the specific volume of the soil under investigation. It is important to highlight that the establishing a critical state line for sand is not as readily obtained as it is for clay. (Atkinson and Bransby, 1982). Nevertheless, Stroud (1971) was able to construct a CSL for simple shear tests on sand, although  $\lambda$  was rather low (typically  $\lambda=0.1$ ) (Atkinson and Bransby, 1982)

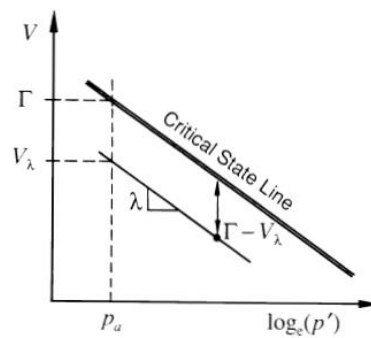


Figure 4.23. Defining  $T$ ,  $v'_\lambda$  and  $\lambda$  from Houlsby (1991).

$v'_\lambda$  is better represented by defining it as  $v + \lambda \ln (p'/p_a)$ , which is the value of the specific volume under atmospheric pressure on the line drawn through the soil state under investigation parallel to the critical state line. The distance  $T - v'_\lambda$  as a distance below the critical state line will be used in the expression derived by Wroth (1990) (in private conversation with Houlsby) between dilatancy rate and  $T - v'_\lambda$ .

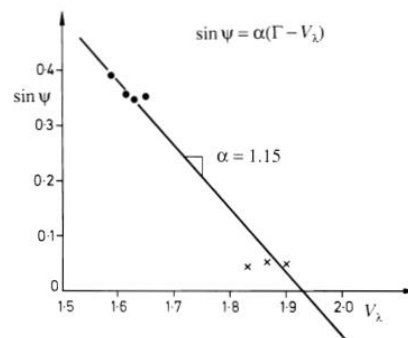


Figure 4.24. Stroud's simple shear test results on sand interpreted by Wroth in a private conversation with Houlsby (1991).

As sand has been termed as 'dense' and 'loose', it would be convenient to accentuate a relation between dilatancy rate with relative density, in which Houlsby (1991) rearranged Wroth's expression to obtain:

$$\sin \psi = \alpha (\Gamma - V_{max}) + \alpha(V_{max} - V_{min})I_D - \alpha \lambda \ln \left(\frac{p'}{p_a}\right) \quad (4.23)$$

By drawing a comparison with Bolton's (1986) equation for the angle of friction in plane strain:

$$\phi_p = \phi'_{cv} + 5(I_D[10 - \ln(\frac{p'}{1.5p_a})] - 1) \quad (4.24)$$

Houlsby (1991) was able to derive expression for the angle of dilation as follows:

$$\psi = -0.11 + 0.59 I_D - 0.11 I_D \ln \frac{p'}{p_a} \quad (4.25)$$

The advantage of this comes from the ease of estimating density of the granular material and the probable value of the working stresses can enable a first calculated estimate of the dilation rate as well as the peak friction angle (Houlsby, 1991).

#### 4.3. Conclusion

- Strain compatibility is used to depict the soil-reinforcement interaction behaviour, where the attainment of the equilibrium state between the amount of strain developing in the soil is counteracted by that of the reinforcement. Once the soil frictional strength is reduced from its peak value, to its constant-volume value (as is exhibited in densely packed soil), the force mobilised from the reinforcement is sought after to compensate for the lower capacity to withstand the tensile strain.
- The soil's strength standalone can be represented by its friction angle,  $\phi$ . The friction angle can be further characterized as the at-rest friction angle  $\phi_o$  (which indicates the soil's effective stress state combinations being wholly compressible), critical volume friction angle  $\phi'_{cv}$ , (friction angle of a soil with no inherent dilation, i.e. no tendency for volume changes to occur when undergoing shear) and peak friction angle  $\phi_p$  which corresponds to the maximum peak strength (comprising of  $\phi'_{cv}$  and the dilatancy angle  $\psi$ )
- Dilatancy angle is important especially with densely packed soils, is a quantitative measure of the tendency for volume change as a result of particles overcoming the interlocking by climbing over each other.

- The use of  $\phi'_{cv}$ ,  $\phi_o$  and  $\phi_p$  enables three regions of soil's shearing resistance to be mobilised in relation to the tensile strain with shear. Effective shear stress combinations for friction angles below  $\phi_o$  insinuates a complete compressive strain in the soil, whereas a strong development of tensile strain pertains to effective stress regions between  $\phi'_p$  and  $\phi'_{cv}$ .
- Soil strain comprises of elastic and plastic components which while are respectively recoverable and irrecoverable. These would both occur when soil undergoes a form of shearing
- In comparing the unreinforced and reinforced soil design, an upper and lower bound relationships were deduced for the reinforced soil behaviour, which had coalesced with one another. The upper bound treats the soil deformation to occur at maximum rate of plastic dilation  $\psi_{max}$  from when shear is initiated, in which the maximum magnitude of tensile strain is induced developed is as per the following relationship of the tensile strain to shear strain. The lower bound referred to a tensile strain amount found by assuming the soil deforms with zero net tensile strain below the at-rest stress ratio, and beyond the at-rest ratio there will be no volume change in deformation
- True elastic component of shear strain in the order of magnitude  $10^{-3}$  to  $10^{-4}$ , which is relatively insignificant compared to the overall more pronounced plastic component. This is important considering that reinforced walls can be subjected to a tensile strain of up to 1%.
- The increase in relative density  $I_D$  causes an increase in dilatancy, which entails a greater feasible tensile strain range.
- In a Critical State Soil Mechanics approach, the greater the difference between the soil's current state specific volume and its critical state specific volume  $v_{cv}$ , the greater the dilation that the soil would exhibit.

## 5. COMPACTION INDUCED ANALYSIS

### 5.1. Effects of Compaction on Stresses

Compaction also attributes in increasing the ratio of effective horizontal to vertical stresses, analogous to the effect of overconsolidation (Terzaghi *et al.*, 1996). Although the standards had prescribed the use of compaction for the soil placements to enhance its mechanical behaviours, there was no means of quantifying the degree of impact compaction would have on the lateral earth pressures. Duncan and Seed (1986) pointed out on the destitute nature of reliable analyses to investigate compaction induced earth pressures on retaining structures. The compaction affecting the stress levels on a soil mass can have implications on the strength and mechanical behaviour of the soil, which implores the need to analyse such effects in order to model the soil response.(Duncan and Seed, 1986).

Sherif *et al.* (1984) demonstrate this with Ottawa silica sand with  $\phi_{cv} = 32^\circ$  and  $K_{op} = 0.466$  ( $K_{op}$  is the at rest coefficient of earth pressure for Normally Consolidated soil). The frictional angle increase readily showcases how consolidation and deposition causes an increase in the friction angle with an overall increase in coefficient of earth pressure at rest.

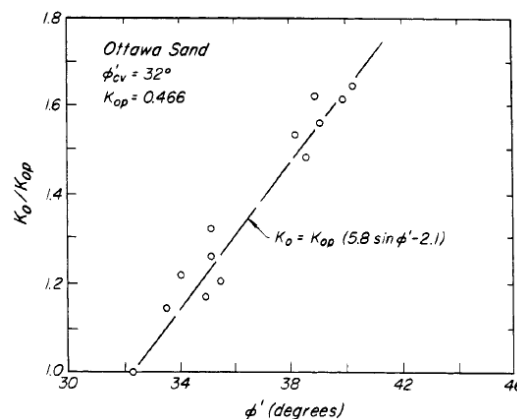


Figure 5.1. Sherif *et al.* (1984)'s relationship between  $K_0$  of sand with densification from (Terzaghi *et al.*, 1996).

Although Sherif *et al.* (1984)'s work summarises the overall effect of compaction, a more accentuated depiction of the very nature of the compaction as a construction process, in terms of its cycles of loading, unloading and reloading is need. This will be

looked upon by appreciating the compaction process to be of such hysteresis nature, as will be shown in Brom (1974)'s and Duncan et al (1986)'s hysteresis model.

Before embarking into the model, it is important to understand the components of compaction induced earth pressure that Duncan *et al.* (1986) refers to. Once the soil is subjected to a transient load effort, the maximum horizontal stress that occurs at any point as a result of the positioning of the surficial compaction plant load is known as the peak compaction-induced lateral pressure (Banerjee and Butterfield, 2005) Following the compaction effort, certain earth pressure sustains, known as the residual lateral pressure, which is prone to diminish with time (Wu *et al.*, 2006),

Duncan *et al.* (1986) had made the following observations on compaction induced stresses and deformations:

- The hysteretic effect of load application and removal can cause residual lateral earth pressures to increase. Such pressures may approach passive earth magnitudes, especially when these greatly exceed the at-rest values.
- At instances where the overburden pressures present at depths are sufficient to cause passive failure are of peak lateral pressure nature, these are preserved to be residual pressures at a 40—90% extent.
- Soil compaction on deflecting structures can manifest to deflections, which increases the near surface residual lateral pressures to the extent of becoming greater than the at-rest values, but would also decrease lateral pressures at depth as a consequence of the increase in structural deflections. Such pattern is influenced by the mode of structural deflections.
- A soil previously compacted would have only a small increase in peak earth pressures when subjected to a subsequent compaction loading compared to an un-compacted soil. A portion of such peak increase is retained as residual earth pressures following compaction completion.

The first hysteretic overview of loading and unloading was proposed by Broms (1974) (cited from (Duncan and Seed, 1986)) as an empirical analytical procedure displayed in Figure 5.2 in the subsequent page. The soil would possess an initial stress state (indicated as A). Once loaded vertically, the soil would have no lateral stress increase until the  $K_0$  line is approached (Point B). Any further loading would instigate a

horizontal stress increase as  $\sigma_h = K_0 \sigma_v$ . The same phenomenon occurs for unloading, where a subsequent decrease in vertical effective stress (unloading) again brings no lateral stress decrease unless a limiting condition is reached (point D) on line  $K_1$ , a limiting condition for the soil being at passive failure based on the Mohr Coulomb criteria:

$$K_1 = \left( \frac{\sigma'_h}{\sigma'_v} \right)_{lim} = K_{1,\phi'} + \frac{2c'}{\sigma'_v} \sqrt{K_{1,\phi'}} \quad (5.1)$$

$K_{1,\phi}$  is a parameter of the soil later explained under Section 5.1.

Duncan and Seed (1986) however found the Broms' (1974) method inconsistent because it models the peak lateral pressure increase from concentrated loading based on constant  $K_0$  (i.e. the lateral pressure is indirectly calculated from the vertical pressures). Duncan and Seed (1986) claims that the peak lateral stress increase in Brom's method may be larger or smaller than  $K_0$  times the vertical stress increase, rather than the peak lateral stress increase which can be larger or smaller than  $K_0$  times the peak vertical stress increase.

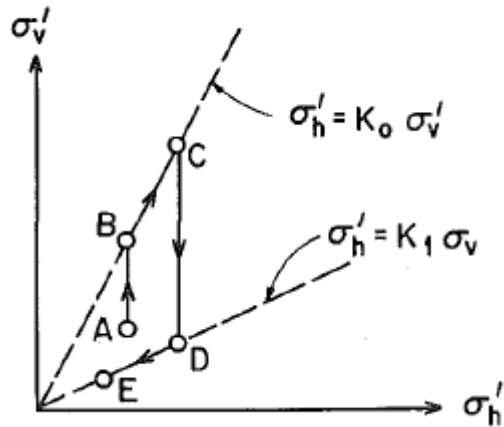


Figure 5.2. Broms' (1974) hysteretic model from (Duncan and Seed, 1986).

Terzaghi *et al.* (1996) attempt to explain the effect of placing and compacting a granular soil in 25 layers of equal thickness. Two benchmarks for comparison are made; the immediate placement (denoted as line  $C_2$ ) and compaction of entire backfill with a period of adjustment for pressures to relax to a long-term value (denoted as  $C_3$ ). It was overall observed that, the earth pressure at rest cannot be less than the at rest  $K_{op}$ , and near the surface it cannot exceed the passive earth pressure value  $K_p$ . Supposing an

intermediate level of 10 layers was placed, the same would be seen for the delineation of earth pressures (denoted as  $C_1$ ).

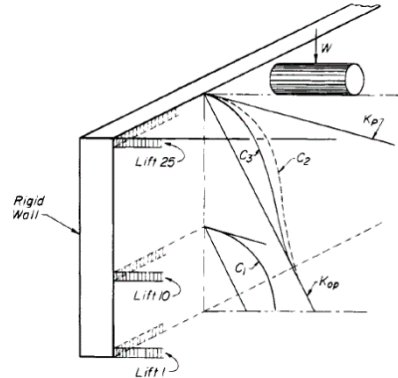


Figure 5.3. Horizontal pressure profiles during placement of Lift 10 (Curve  $C_1$ ) and Lift 25 (Curve  $C_2$ ) and post compaction of Lift 25 (Curve  $C_3$ ) from Terzaghi *et al.* (1996).

Terzaghi *et al.* (1996) provided another means of understanding the changes in stress behaviour with the induced compaction in a stress path form, as shown in figure 64 below. The stress representation is for the first lift of soil, with its sole placement and the subsequent layers inserted and compacted. Interest is vested particularly on this layer as it includes the stress history of all the lifts compacted to complete a backfill. In correlation with the earth pressure profiles for the full depth and the intermediate depth follow the same delineation pattern, it will be seen that the stress state changes of the first layer would also experience the stress history of the intermediate layers. The list below explains the different stress paths and the processes involved for the placement and compaction of the first and second layer.

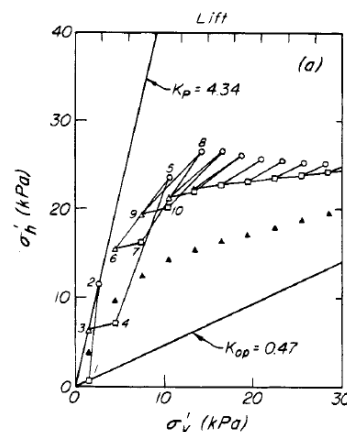


Figure 5.4. Effective stress paths during compaction within the vicinity of  $K_p$  and  $K_{Op}$  from Terzaghi *et al.* (1996).

- Path 0-1: Placement of first lift giving an effective vertical stress  $\sigma_v$  reflecting the layer's half thickness. It brings in a horizontal earth pressure equal to  $\sigma_h$  in correspondence with the  $K_{op}$  coefficient
- Path 1-2: Compactor's vertical force (as a weight and dynamic action) gives an altered stress state shown at 2. No passive failure would however occur (although the stress state is on the  $K_p$  failure line) because no overlying soil is present above the first lift yet.
- Path 2-3 Stress state reduces to point 3 along the  $K_p$  line as a relief from the compactor removal.
- Path 3-4: Addition of second lift causes the vertical pressure on the first lift to increase in accordance with the second lift's weight.  $\sigma_h$  also increases to a stress state in point 4.
- Path 4-5: The second lift is now compacted instigating a further increase in vertical stresses on the first lift.
- Path 5-6: Following the compactor removal, the stresses reduce to the point represented in point 6. In this instance, although there is overlying soil above the first layer for passive failure, such failure is prevented from materializing because the soil is now compacted to give a resisting nature.

It can be deciphered that the overall increase in  $\sigma_h$  at points 3 and 6 is relative to the slight overall increase in vertical pressure from the subsequent layers show how significant the compaction process elevates the horizontal pressure.

The stress points 6, 7, 8 and 9 again represent an additional layer imposed with the same repeated compaction action. The solid triangles indicate time passage accounted for the earth pressure prior to the consecutive level implemented and compacted.

Although the first layer had been separately put under examination, Terzaghi *et al.* (1996) adopts this method to depict the stress changes within all the lifts, analogous to the overall earth pressure diagrams of 10 and 25 lifts as shown previously in Figure 5.4. The various steps ab (layer insertion), bc (compaction), and cd (release) and de (post compaction release) depicts the stress condition in a Lift j immediately after the

entire backfill consisting of  $n$  lifts. Another scenario these steps represent is the compaction stress paths of other lifts when  $n-j$  lifts from the current backfill surface are subjected to a placement and compaction of a new lift i.e. Point d represents the stress condition of mid depth of level 22 immediately after completion of a 25 lift backfill, and concurrently represents the stress conditions of any lift after 4 consecutive placement and compactions added (e.g. stress state of lift 6 when a 10 lift backfill is complete).

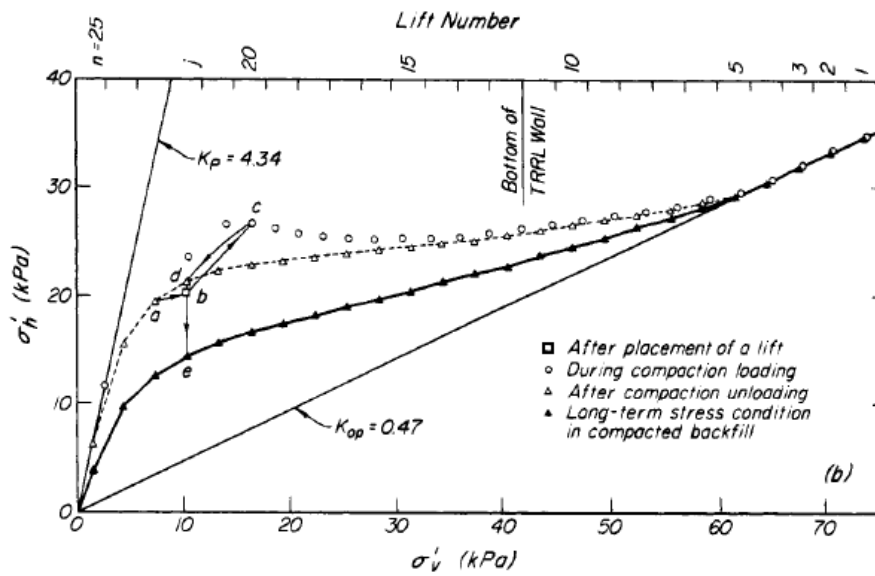


Figure 5.5. Profile of horizontal pressure of sand backfill versus vertical pressure of sand backfill in Non-Deflecting wall during Lift Placement (ab), Compaction (bc), Immediately After Compaction (cd) and Post Compaction (de) from Terzaghi *et al.* (1996).

Mathematical relationships based on elastic distribution are derived to find the end-result horizontal effective stress after unloading as follows:

$$\sigma'_h = K_m \left( \frac{\sigma'_{vm}}{\sigma'_v} \right)^{\sin \phi_{cv}} \sigma'_v \quad (5.2)$$

$\sigma'_{vm}$  is the maximum effective vertical stress reached during the compaction of the lift;

$K_m$  is the corresponding lateral pressure coefficient

$\sigma'_v = \sigma'_{vm} - \Delta \sigma'_v$  (where in which  $\Delta \sigma'_v$  is calculated from the linear-elastic stress distribution).

A second lift placement reloads the first lift along the recompression line with a slope  $K_r$  which is given as (Mayne, 1989 from Terzaghi *et al.* (1996)).

$$K'_r = 0.75 (1 - \sin\phi') \quad (5.3)$$

The change in horizontal stress becomes:

$$\Delta\sigma'_h = 0.75 (1 - \sin\phi') \Delta\sigma'_v \quad (5.4)$$

Overall, the graph shows a dominion of the earth pressures acting are between  $K_p$  and  $K_{op}$ , with  $K_p$  being the upper limit and  $K_{op}$  being the lower limit.

Compaction induces shear stresses that move a point representing a stress state away from the  $K_{op}$  stress condition. Compaction induced horizontal stresses decrease from d to e (from Figure 5.5) under the subsequently laterally constrained condition according to equation:

$$K_o = K_{OP} + \frac{K_{PS} - K_{OP}}{2.5} \frac{K_P}{3} \quad (5.5)$$

The  $K_{op}$  line indicates the lower limit for  $\sigma_h$ .  $K_{ps}$  in this equation indicates the coefficient of passive earth pressure during passive shearing. And  $K_p$  is strictly the maximum upper limit for the soil is undergoing shear stresses as passive pressure (Terzaghi *et al.*, 1996).

#### 5.1.1. Seed and Duncan's Hysteresis Model

Duncan and Seed (1986) propose a rather intricate, yet nevertheless expedient work in understanding the compaction behaviour on a vertical non-deflecting wall structure.

A cyclic one dimensional overconsolidation loading/unloading model is used to epitomise the hysteresis nature of the method of compaction by layers, on the premise that the principle effective stresses remain horizontal and vertical and no displacements occur (Duncan et al, 1986).

This model, known as  $K_o$ -loading/unloading used to evaluate the compaction induced stresses by assuming passive soil failure to occur in compatibility with Mohr-Coulomb's failure criteria. Being based on 5 material parameters, the limiting



The virgin loading would ultimately establish the New Maximum Past Loading Point (MPLP)

- ii. Virgin Unloading: unloading occurring at a vertical effective stress less than the achieved since the establishment of the MPLP, which follows the following stress path and establishes the Current Minimum Unloading Point (CMUP)

$$\sigma'_h = K'_o \sigma'_v \quad (5.7)$$

$$K'_o = K'_o(OCR)^\alpha \quad (5.8)$$

$$OCR = \frac{\sigma'_{v\,MPLP}}{\sigma'_{v\,ESS}} \quad (5.9)$$

$$\sigma'_h \leq K'_1 \sigma'_v \quad (5.10)$$

- iii. Virgin Reloading: involves the first reloading cycle after establishing the new CMUP, and follows a linear stress path from the CMUP to the reloading point R on the virgin loading line.

The slope for the linear path increases as the degree of unloading before reloading increases. The horizontal and vertical stress parameters for reloading point R become:

$$\sigma_{h,r}^* = \sigma'_{h,CMUP} + \beta \Delta \quad (5.11)$$

$$\sigma_{v,r}^* = \frac{\sigma_{h,r}^*}{K_0} \quad (5.12)$$

$$\Delta = \sigma'_{h,MPLP} - \sigma'_{h,CMUP} \quad (5.13)$$

Post Virgin Conditions involving stress states in non-exceedance of MPLP

- iv. Non-virgin reloading is categorized by a stress path from Recent Maximum Loading Point (RMUP)-corresponding to maximum  $\sigma_h$  achieved during most recent loading cycle to R and then continues to follow the  $K_o$  line to

MPLP. Any further loading from point R onwards is considered as virgin loading.

In the case that reloading causes a stress level greater than R (on the condition of being less than MPLP) occurs, the new point establishes as a new R for any future loading cycles.

- v. Non-virgin unloading which is a stress path of unloading from the newly established point R, defined as:

$$\frac{\sigma'_h}{\sigma'_v} = K_0 \left( \frac{\sigma'_{h,RMLP}}{\sigma'_{v,ESS}} \right)^{\alpha^*} \quad (5.14)$$

Where  $\alpha^*$  is chosen so that the unloading path passes through CMUP

$$\alpha^* = \frac{\ln\left(\frac{\sigma'_{h,RMLP}}{K_0 \sigma'_{v,CMUP}}\right)}{\ln\left(\frac{\sigma'_{v,RMLP}}{\sigma'_{v,CMUP}}\right)} \quad (5.15)$$

The  $\alpha^*$  constraint (always greater than  $\alpha$ ) is significant in its distinguishing of an apparent memory loss of the soil as a consequence of shearing during unloading along the  $K_1$  line. This is the case where the unloading continues below the CMUP would establish a new CMUP and a new R. If the  $\sigma_h$  was reduced but not to the extent of having a state below CMUP, would entail a pseudo CMUP which performs as CMUP but does not establish a new R.

The hysteretic model gave favourable agreement when compared with experimental results on both sand and clay. While procuring Duncan and Seed's model (either by using computer based or hand calculations), it provided calculations residual and peak earth pressures as shown in Figure 5.7, which resembles well with a typical profile of a line load.

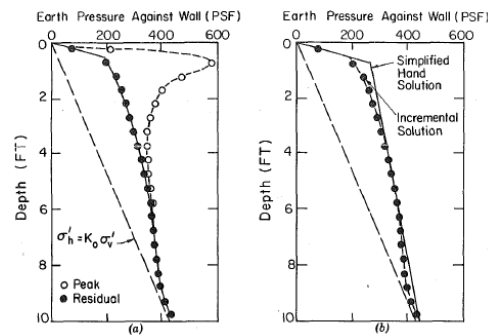


Figure 5.7. (Left) Calculated peak and residual earth pressures following compaction of final layer (Right) final pressure distribution from incremental analysis and hand solution from Duncan and Seed's (1986) model.

An attempt to verify Duncan and Seed (1986)'s model on a field observation was conducted on the TRRL wall by Carderet *et al.* (1977). The wall was essentially a non-deflecting wall but undergone certain amounts of displacements (as shown in Figure 5.9). It comprised of a 22 m reinforced concrete trough and backfilled with clean medium sand, placed at 0.15 m. backfill lifts (Duncan and Seed, 1986). The computed and measured lateral earth pressures were in good agreement near the top of the wall, but not towards the bottom of the wall as the computed lateral pressures of a non-deflecting wall were much larger than the measured of a deflecting wall.

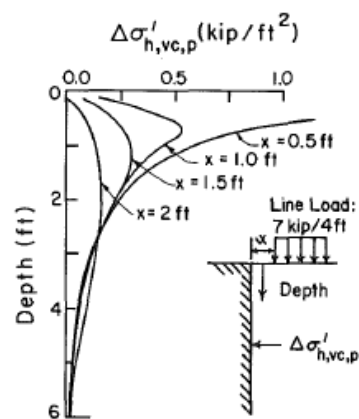


Figure 5.8. Change in horizontal stress profile from line load application from (Terzaghi *et al.*, 1996).

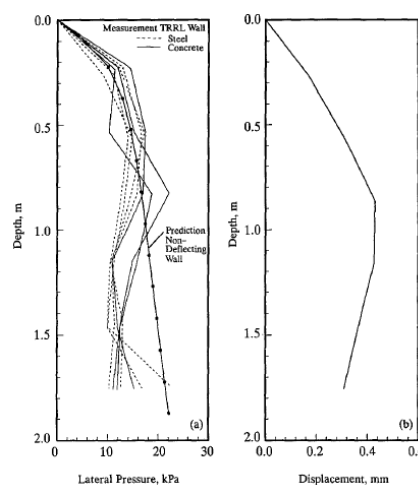


Figure 5.9. Compaction induced lateral pressure and horizontal displacement of TRRL Wall from Mesri *et al.* (1993).

Duncan and Seed (1986) presents an overall summary of the different methods:

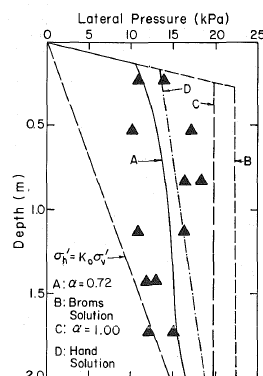


Figure 5.10. Results of various methods compared with experimental results:  
 A. Duncan and Seed (1986)'s Computerised Method B. Broms' (1971) Solution. C.  
 Computerised Method with  $\alpha=1$  D. Duncan and Seed's (1986) Hand Method.

The results show that Duncan and Seed (1986)'s computerised and hand solution results are in more favourable agreement with the empirical results compared to Broms' (1971) solution. Broms' (1971) solution was also found to be overly conservative, such that it was greater than the computerised method with the adoption of  $\alpha=1$ .

An interesting finding on the TRRL wall is that a maximum displacement of 0.42 mm had reduced the horizontal pressure against the horizontal strip to less than 50% of that expected on a non-deflecting wall. Carter *et al.* (1977) points out the importance of the wall rigidity, such that a larger lateral stress would be measured behind a completely rigid structure.

### 5.1.2. Ingold (1983) 's Work: Design of reinforced Soil Walls by Compaction Theory

Ingold (1983) presents a critique on the currently practiced design philosophies adopted for the reinforced wall design; particularly the Tie Back Wedge Method and the Coherent Gravity method. These methods fail to take into consideration compaction effects on the earth pressures, and may have discrepancies that can mislead the design engineer while incorporating these methods.

It was understood in Chapter 2 that the Tieback method assumes a kinematic constraint at the toe causing the top of the wall to move outwards. The earth pressure is

considered to be active throughout the depth and that the soil failure is a planar surface type at  $45 + \phi/2$  to the horizontal (as shown in Figure 5.11).

The Coherent method had different kinematic constraint for the purpose with the assumptions it is based on, in that the failure plane is logarithmic spiral (figure 72) and that the earth pressure states during failure varies from at-rest at the top to active further down the wall. Kinematically, the wall in turn would be fixed at the top and rotating at the toe (Ingold, 1983). Interestingly, field observations have shown this form of kinematic constraint.

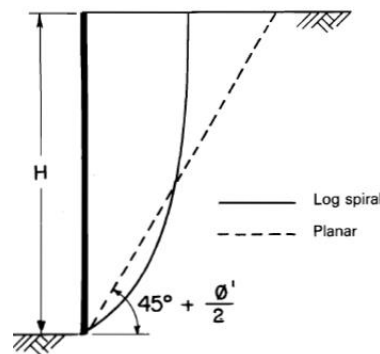


Figure 5.11. Failure surfaces: planar for Tie-Back Method and log spiral for Coherent Gravity Method from Ingold (1983).

Bearing in mind that the Coherent Gravity method was seen to be more tacit (Ingold, 1983) in predicting more realistically the earth pressures (as observations had found that the earth pressures were higher than the active pressure assumption at the top), it still required to be investigated on the premise of the exact profile of earth pressure variation and the wall rotation mode.

Based on Dubrova (1963)'s theory, the coefficient  $K$  drops below the active value for depths exceeding  $0.62 H$ , which made current design practice for Coherent Gravity to conclude the following for the earth pressures:

$$z < 6m: K = K_0 + \frac{1}{6}z (K_A - K_0) \quad (5.16)$$

$$z > 6m: K = K_A$$

Ingold (1983) however recalls a private conversation with Schlosser in which the latter conversely states otherwise and found deviations in the earth pressures for  $z/H$  ranges from 0.33 to 0.83, which varies significantly from the 0.62 value adopted.

With regards to wall rotation, Ingold (1983) presents results for 3 walls with the use of variant  $m$  to denote the wall movement, as  $m/H$ . The top walls were found to move from 1.7% of the wall height, and that the walls were rotating outwards about the toe (in corroboration with the Tie-Back method), which associates with an active condition.

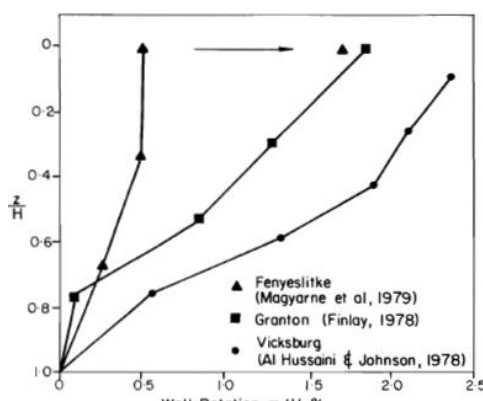


Figure 5.12. Various wall movements from Ingold (1983).

Studies by Nabonne *et al.* (1979) deduced that the variations in the results are being owed to the compaction effects, based on a study conducted of granular backfills with various load intensities of 6.6 kN/m, 7.1 kN/m, 9.8 kN/m and 14.7 kN/m. compactor rollers produced an outward rotation increase.

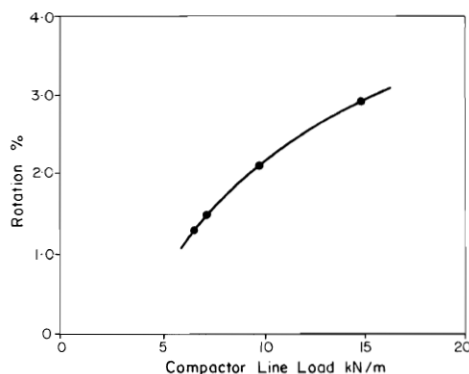


Figure 5.13. Wall deformation induced from line load action from Ingold (1983).

Such variations motivated Ingold (1983) to develop the Compaction Theory method. Later in this section a comparison with other methods will be shown along with case studies to draw up conclusions on the applicability of the Compaction Theory.

To begin with, Ingold provides an expression for maximum compaction induced earth pressure acting on the back of the wall manifested from a load intensity  $p$

$$\sigma'_{hm} = \alpha \sqrt{\frac{2py}{\pi}} \quad (5.17)$$

The coefficient alpha renders the degree to which the lateral earth pressures transmitted back to the facing units can be shed by the reinforcement; an attribute unique to the reinforced soil system. However, compaction can impact the bond stresses acting on the reinforcements, as Richardson and Lee had shown a drop to  $14^\circ$  when a vibratory roller was used from the  $44^\circ$  anticipated for sand with a static angle of friction.

In cases where the reinforcement is 0.1% of the overall reinforced mass, the value alpha can be reduced to unity. There is however an overriding constraint of a critical height constant, where heights greater than the critical value would cause compaction induced earth pressures from the unreinforced mass behind the reinforced soil zone. (Ingold, 1983) The critical height is defined, based on passive earth pressure failure, as:

$$H_c = K_p \sqrt{\frac{2p}{\pi y}} \quad (5.18)$$

This however is for a conventional retaining wall. The equation has been refined to include allowances for compaction within the reinforced mass to become:

$$H'_c = \frac{L^2}{6} \sqrt{\frac{\gamma\pi}{2p} \left[ \left( 1 + \frac{24p}{\gamma\pi K_a L^2} \right) - 1 \right]} \quad (5.19)$$

This is on the premise that the reinforcement of length  $L$  gives a rise in an additional vertical stress (reflected to a horizontal stress by incorporating the coefficient of lateral pressure  $K_a$ ).

When the height of the wall is less than  $H_c$ , the lateral earth pressures acting on the wall facings is that of a conventional wall. Conversely, a height greater than  $H_c$  would equate to a pressure distribution as follows:

$$\sigma'_{vf} = \gamma z + \frac{3z^2}{L^2} \sqrt{\frac{2py}{\pi}} + \frac{K_a \gamma (z - H_c)^3}{L^2} \quad (5.20)$$

A comparison is then made with three case studies: Little Bridge Abutment, Dunkerque Wall and the TRRL full-scale wall. Information about these 3 walls is provided under Table 5.1 below, and as a precaution that the first two walls in France had incorporated inextensible reinforcement (non-geosynthetic products).

Table 5.1. Comparison of Wall Case Studies.

| Lille Bridge Abutment, Lille, France                                                                                                                                                                                                                                                                                                    | Dunkerque Wall, France                                                                                                                                                                                                                                                                                                                                                                                                | TRRL Full-Scale Wall, UK<br>(Extensive experiment)                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                           |
|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <p>Wall dimensions: 5.6 m high and 15m long</p> <p>Facing Units: Conventional Reinforced Concrete cruciform Pattern</p> <p>Reinforcement: Stainless Steel</p> <p>Fill Type: Red Schist with average bulk density 1.85 Mg/m<sup>3</sup> with internal angle friction 45.5°</p> <p>Load Action: 8 kN/m</p> <p>Critical Height: 2.65 m</p> | <p>Wall dimensions: 12 m high and 630 m long with two walls between 15 and 18 m apart</p> <p>Facing Units: Non-stated</p> <p>Reinforcement: Galvanised Mild Steel</p> <p>Fill Type: Sand with average bulk density of 1.91 Mg/m<sup>3</sup> with angle of friction 44°</p> <p>Load Action: 15 kN/m</p> <p>Critical Height: unstated due to the reporting undertaken 9 years following the compaction application.</p> | <p>Wall dimensions: 6 m high embankment with 14 m wide and 25 m long body</p> <p>Facing Units: Conventional Reinforced Concrete cruciform Pattern</p> <p>Reinforcement: Various</p> <p>Fill Type: Lower fill-sandy silty clay compacted to an average dry density of 1.8 Mg/m<sup>3</sup></p> <p>Middle Fill: Gravelly sand separating layer (unstated dry density compaction)</p> <p>Upper Fill: Clayey silt compacted to an average dry density of 1.63 Mg/m<sup>3</sup></p> <p>Over height, average unit weight of fill is given as 20.7 kN/m<sup>3</sup> with a friction angle of 34°</p> <p>Load Action: 40.3 kN/m</p> <p>Critical Height: Unstated</p> |

The profiles for the earth pressure distributions along the depth for both wall cases show favourable agreement with the Theoretical Compaction Theory method. The Lille abutment wall profile demonstrates that once an exceedance of the critical height occurs, the lateral earth pressure reverts to an active earth pressure. The same pattern behaviour is seen for the Dunkerque Wall, although there was no explicit reference about the actual critical height to make a benchmark comparison due to the 9 year period of re-assessing the wall pressures.

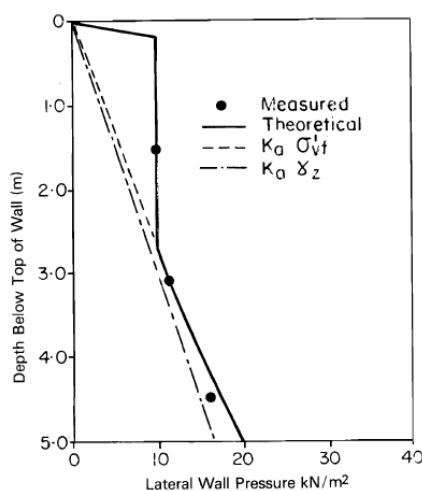


Figure 5.14. Lille Abutment measures of lateral wall pressure vs theoretical values from Compaction Theory Method from Ingold (1983).

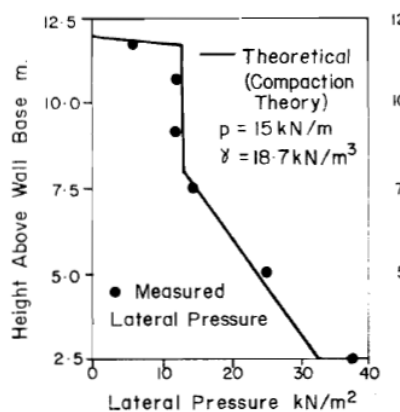


Figure 5.15. Lille Abutment measures of theoretical and measured reinforcement loads from Ingold (1983).

In both wall cases, when comparing the as-measured  $K/K_a$  ratio with that predicted using both the theoretical Dubrova's (1963) and Compaction theory (1983), the as-measured results corroborate stronger with the Compaction theory.

It was anticipated earlier in the Coherent Gravity method (which utilizes the Dubrova theory) that a value of  $z/H$  of 0.62 is anticipated for a structure to truly rotate about the top. The results however point out otherwise for the compaction theory case, where  $K/K_a$  of unity (implying  $K$  to equal  $K_a$ ) had a  $z/H$  value of 0.47 for Little Bridge and 0.33 for the Dunkerque wall (half the initial postulated value of 0.62). There is also no decrease in  $K/K_a$  once the ratio reduced to unity in both cases.

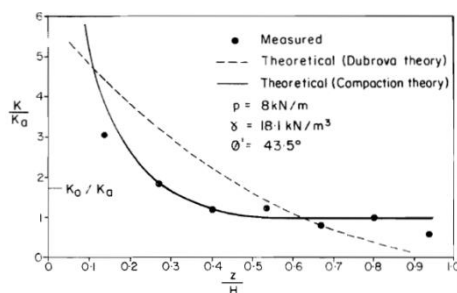


Figure 5.16. Theoretical and measured earth pressure coefficients for Lille Abutment from Ingold (1983).

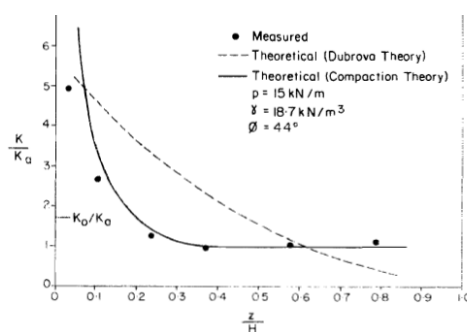


Figure 5.17. Theoretical and measured earth pressure coefficients for Dunkerque wall from Ingold (1983).

TRRL full scale wall results for the horizontal earth pressure profile also show the as-measured earth pressures to agree favourably with the theoretical compaction theory. An important point to decipher from the TRRL wall results is that if the critical depth was to be calculated using  $K_0$ , the compaction theory gives a closer agreement for very low values of  $z/H$ . Ingold emphasizes on the appropriateness of adopting  $K_0$  to the reinforced mass bulk as it would not have any lateral yield occurring. On this basis, the theoretical upper level value of  $K/K_a$  equal to 8 is more corroborating with the maximum measured value of 7.3. Coincidentally, the Coherent Gravity method gives a much lower  $K_0/K_a$  of 1.6. For comparison purposes, the  $z/H$  value for  $K$  to equal  $K_a$  was found to be 0.55, which is again lower than the 0.62 proposed by the Dubrova theory (1963).

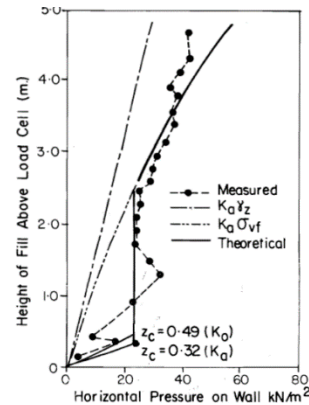


Figure 5.18. TRRL Full Scale Wall theoretical and measured earth pressures from Ingold (1983).

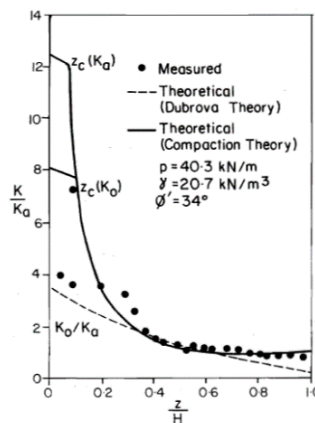


Figure 5.19. TRRL Full Scale Wall theoretical and as measured earth pressure coefficients from Ingold (1983)

### 5.1.3. Comparison between the Coherent Gravity, Tie Back and Compaction Theory Method

Ingold (1983) presented an overall comparison between the profiles of the three earth pressure methods. For a 6 metre high wall, the maximum pressures at the wall base range from 37.2 kPa (Coherent Gravity method) to 45 kPa (Tie Back Method), which is less than the Compaction Theory amount of 68.7 kPa by 60%. The extent over which such variation would occur would also depend upon the compaction plant employed.

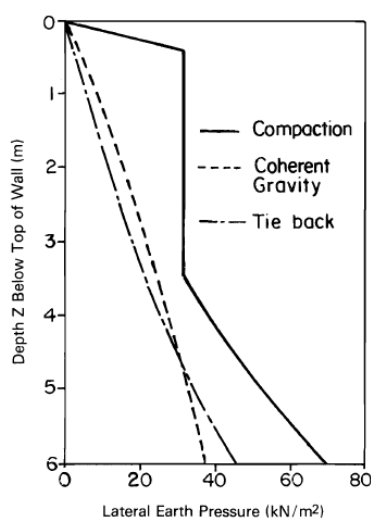


Figure 5.20. Comparison of the various design methods from Ingold (1983).

Ingold (1983) points out to a concern of the permanent transfer of the compaction pressures to the reinforcement can have ramifications in the adapted factors of safety for the tensile and bond failure to reduce. This is best explained by quantifying the drop in factor of safety from a compaction induced pressure taken into account by the compaction theory and not considered in the conventional methods.

Ingold (1983) conveys a re-evaluation of the factor of safety when induced compaction is considered. For instance, a wall based on the Coherent Gravity method was given a maximum pressure of 37.2 kPa, which was calculated based on a factor of safety of 2. With the compaction induced base pressure of 68.7 kPa, the factor of safety is revisited to be  $(37.2/68.7) \times 2 = 1.08$  at wall base, which is only sustained at the top 4 m. Similarly for the Tie Back method again based on a factor of safety of 2, when re-evaluating the refined factor of safety gives a drop of  $(45/68.7) \times 2 = 1.3$  (Ingold, 1983)

Such reduced factors of safety signify repercussions of distressed and failed structures, and is more pronounced if the rate at which the compaction induced stresses dissipate is slower than the detrimental decrease in the reinforcement thickness (Ingold, 1983).

Such effects also resonate in bond failure, but to a lesser extent as the walls yield, which in turn cause the compaction induced earth pressures to reduce to an active value, but can be reinstated with further yielding of the wall (Ingold, 1983). A recommendation to overcome this is by both employing longer and rougher

reinforcement at the upper portions of the wall, or by adopting lighter compaction equipment (Ingold, 1983).

## 5.2. Conclusion

- In relation to GRS wall construction, compaction causes an increase in the relative density.
- Compaction causes a transient load effort to occur, in which following the compaction effort can cause some certain earth pressure to sustain (known as residual lateral pressure). Although such residual pressure would diminish with time, it would be beneficial to probe the amounts
- Many methods modelled the hysteretic nature of compaction, such as Broms (1974), Duncan and Seed's (1986) as these methods can cause residual stresses to increase. In some instances, the residual stresses had been preserved at 40-90%
- The conventional methods for Tie Back Wedge Method and Coherent Gravity method are paramount in determining internal stability of the GRS structure, but do not address compaction efforts. A more suitable method is the Compaction theory devised by Ingold (1983), which corroborated well with many full scale experiments such as the infamous TRRL Wall. The compaction theory estimated the lateral earth pressure at the wall base to be greater than that of the Coherent Gravity and Tie Back methods by 40-60%.
- Compaction pressures being transferred to the reinforcement can have ramifications on the factor of safety for tensile and bond failure.

## 6. REINFORCEMENT

The shearing resistance of soil standalone is found to be rapidly mobilised when a small tensile strain is manifested, in which is noticeably of a more plastic, irrecoverable nature. A cumulative strain of up to 1% or less is enough to mobilise a substantial portion of the available shearing resistance in reinforced soil walls (Ingold, 1994; Jewell, 1990b). It was also seen how tensile strains are associated with soils of high peak friction angles.

This implores the need to understand the reinforcement inclusions as these primarily compensate the soil's incapacity to withstand such strains, and in doing so would decrease the earth pressures acting on the wall face driving the movement.

### 6.1. Reinforcement Interaction with Soil Backfill

The section will explore in detail the reinforcement behaviour standalone when undergoing tensile strain as well as its interaction with the soil under two major movements: Pullout and Sliding mechanisms, in parallel with Section 2.1. For proper scrutiny, the resistance contribution of the facing panels and connections, if any, (although such information was not provided in (Hamderi and Guler (2015) work) will be omitted.

Before delving into the interdependent interaction mechanisms, it is important to understand that the reinforcement behaviour standalone and how the tensile force induced from the tensile strains vary along the reinforcement length itself.

Ingold (1994) highlights how the tensile force,  $T_x$ , developed at a distance  $x$  from the face is not constant along the reinforcement length, but would vary due to the shear stress transfer between the soil and the reinforcement. At a distance  $x$  from the wall face, such shear stress, referred to as  $T_x$ , pertains on the upper and lower surface of the reinforcement. A short length in the reinforcement  $dx$  will cause a change in  $T_x$  by an amount  $dT$ , which can be expediently used to define  $t_x$  as  $dT/dx = 2T_x$  (Ingold, 1994)

The magnitude of  $dT/dx$  determines how the shear stress is being transferred. A positive  $dT/dx$ , as is the nature in the active zone, which would imply that the soil is

transferring shear stress into the reinforcement. The passive zone contrarily has a negative  $dT/x$  magnitude, which implies the shear stress being shed into the soil. The designer must ensure the length is extended beyond the active zone boundary (from the point of maximum tensile force build up) to extend to a length adequate enough to shed all the shear stress back into the restraint zone.(Ingold, 1994)

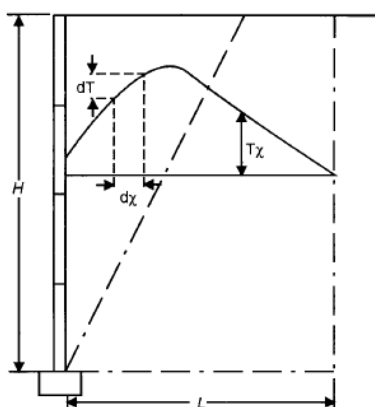


Figure 6.1. Tension distribution from Ingold (1994).

A pertinent failure mechanism was previously seen to relate to the GRS wall, is known as rupture failure; in which the axial load exceeds the axial tensile strength of the reinforcement. The tensile strength however may reduce as it is subjected to creep from the loadings imposed over a long period. Such reduction in strength, when not taken into account, can lead to drastic consequences as the failure of one reinforcement layer would cause the full load to be transferred into the other layers, thus resulting to a total progressive failure. Rupture failure will be explored in detail under section 7.5.

Considering again that structural contributions of the wall facing ignored, the soil in the active zone would require the strength of the soil-reinforcement interface to be mobilized.

The reinforcements embedded in the soil would cause its total frictional resistance to be mobilised by a combination of sliding, rolling, soil particle interlocking within the geosynthetic surfaces as well as shear strain within the geosynthetic specimen (Hayashi, Makiuchi, and Ochiai, 1994)

Lopes (2002) highlights a more distinct approach, in that the mobilization has to do with three interaction mechanisms:

- Skin friction along reinforcement
- Soil-soil friction
- Passive thrust acting upon the reinforcement bearing members

Lopez (2002) asserts that skin friction is the only attributable mechanism in geotextiles, whereas all the mechanisms are pertinent for geogrids (as relative movement for the soil along the grid apertures can occur). The figures below demonstrate the resistance behaviours for geogrids:



Figure 6.2. Skin friction mechanism to resist tension force from Lopes (2002).

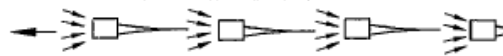


Figure 6.3. Passive thrust acting upon bearing members of geogrids from Lopes (2002).

The soil-reinforcement mobilized shear strength for geotextiles is interdependently based a twofold mechanism: such that the soil needs to inherently possess dilatancy (in that plastic tensile strain occurs) as well as the reinforcement having a certain extensibility extent. Geogrids on the other hand have an additional mechanism, which is an additional ability to mobilize the passive thrust on the grid's bearing members with increases in displacement. This third mechanism causes the geogrid to be less extensible than the geotextile, and as a corollary would have a greater shearing resistance that can be mobilized. For explanation purposes, the twofold mechanism will be referred to as a two dimensional phenomenon, and the threefold mechanism as a three dimensional phenomenon.

These interaction mechanisms concurrently involve relative movements to undergo between the soil-reinforcement interfaces, which are categorised as:

- Direct Sliding: soil slides across one reinforcement side that is linked to another soil block on the other side where failure occurs by direct shear (Shukla, 2002)
- Pullout: reinforcement moves in relation to the surrounding soil (Shukla, 2002) where failure occurs after the maximum available bond stress is exceeded (Jewell, 1990)

Palmeira and Milligan (1989) demonstrate where such mechanisms are more profound on a reinforcement structure, in which direct sliding dominates at A, Pullout occurs at B whereas a more complex interaction occurs at C.

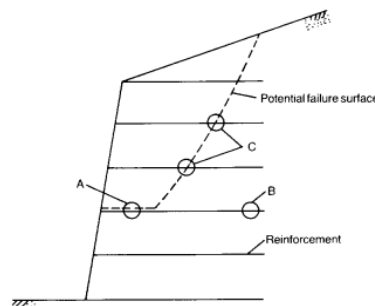


Figure 6.4. Dominating mechanisms depending on region within reinforcement structure from Palmeira and Milligan (1989).

Both movement mechanisms are restrained if the reinforcement specimen can cater for a greater effective normal (bond) stress. Lopes (2002) shows how an increased bond stress is capacitated for geogrids as opposed to geotextiles by in the stress profiles displayed for each:

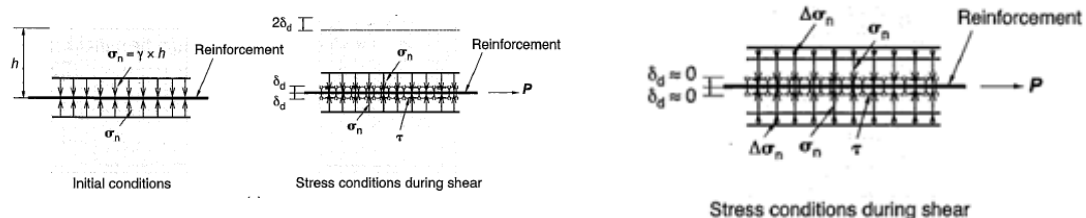


Figure 6.5. Catering for greater bond Stress mechanism and increased bond development during shear from Lopes (2002). Left: Initial conditions before shear. Middle: bond stress developing in geotextiles. Right: bond stress developing greater with greater application of normal stress.

Either way, an increased bond development would render the soil to withstand from sliding past the geosynthetic or for the geosynthetic to be pulled out of the soil as tensile load is being mobilised (Shukla and Yin, 2006)

Koerner (2012) evidently presents the direct shear test results of a soil-geotextile frictional behaviour, in which an increase in normal stress application results to an increase in peak shear and residual strength to withstand sliding. Shukla and Yin (2006) had also demonstrated that the normal stress results to an increased maximum pullout resistance, though this is more directly a function of the soil gradation, plasticity, as-placed dry unit weight, moisture content, embedment length, geosynthetic surface characteristics. Lopes and Ladiera (1996) the relationship between embedment length and pullout behaviour shows an increase in pullout force for a given front displacement, although the increase was more profound between increased lengths of 0.80 to 0.96 than it was in lengths increased from 0.96 to 1.12.

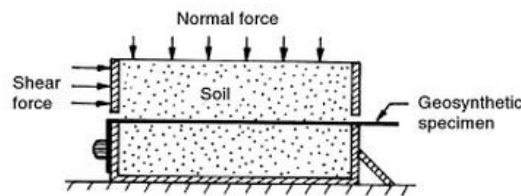


Figure 6.6. Setup to test geosynthetic during Direct Sliding from Shukla and Yin (2006).

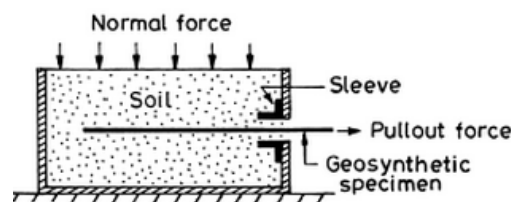


Figure 6.7. Setup to test Geosynthetic during Pullout from Shukla and Yin (2006).

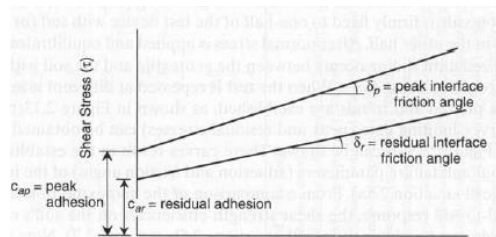


Figure 6.8. Peak and residual interface friction angle for shear testing from Koerner (2012).



Figure 6.9. Pullout force with frontal displacements from Shukla and Yin (2006).

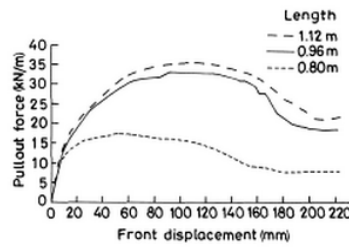


Figure 6.10. Maximum pullout resistance versus normal stress from Shukla and Yin (2006).

The factors influencing the soil-geosynthetic interaction relating to the mobilisation of direct shear and pullout will be explored. This would require establishing an understanding on parameters pertaining to coefficient of direct shear and pullout to make baseline comparisons. The nomenclature adopted by Jewell (CIRIA, 1996, 1990, 1984) will be applied to fulfil this.

The generalised interface shear strength of the soil-reinforcement is:

$$T = 2WL\sigma'_n f \tan \phi' \quad (6.1)$$

W and L are the width and length of the reinforcement, with  $\sigma_n$  as the effective normal stress and  $\phi'$  is the friction angle (peak or constant volume) in terms of effective stress.

The important notation in the equation is  $f$  which denotes the interaction mechanism mobilized (skin friction along reinforcement, soil-soil reinforcement, passive thrust on bearing members) and the relative movement occurring (direct sliding, pullout) on the same interface. Its value ranges between 0 to 1.

Other dimensions in the equation are demonstrated on Figure 6.11.:

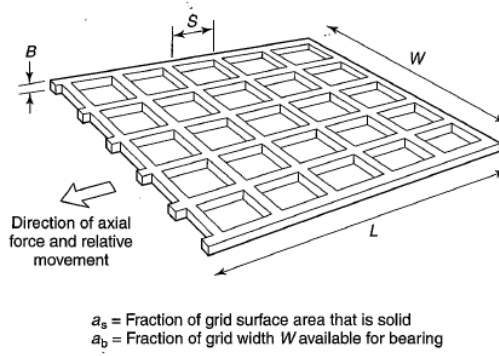


Figure 6.11. Geogrid dimensions from Lopes (2002) and Jewell (1996).

As geotextiles involve solely the skin friction mechanism being mobilised; it causes for the direct sliding and pullout movements to be very similar (Jewell) This results to the following relationship:

$$f = f_{ds} = f_b = \tan \delta / \tan \varphi' \quad (6.2)$$

with  $\delta$  is the friction angle for the soil-reinforcement interface. (Jewell, 1996)

The geogrids on the other hand derive its shear strength for the soil-reinforcement interface for direct sliding from two mechanisms:

$$\text{Skin Friction mechanism} \quad T_s = 2 a_s W L \sigma_n \tan \delta \quad (6.3)$$

$$\text{Soil-Soil Friction mechanism} \quad T_{s/s} = 2 (1 - a_s) W L \sigma_n \tan \varphi' \quad (6.4)$$

$a_s$  denotes the fraction of geogrid surface area that is solid

$$T = T_s + T_{s/s} \quad (6.5)$$

It is important to note that for the direct sliding movement mechanism, the passive thrust mobilised on bearing members is obsolete. Mathematically, the rearrangement of the equations would give interface coefficient for direct sliding to be:

$$f = f_{ds} = a_s (\tan \delta / \tan \varphi') + (1 - a_s); \quad (6.6)$$

The resistance for pullout movement in geogrids stems from the skin friction mechanism ( $T_s$ ) and the passive thrust mobilisation on the bearing members ( $T_p$ )

$$T = T_s + T_p \quad (6.7)$$

$$T_s = 2 a_s W L \sigma_n \tan \delta \quad (6.8)$$

$$T_p = \left(\frac{L}{S}\right) a_b W B \sigma'_{p} \quad (6.9)$$

With  $\sigma'_p$  being the effective passive stress mobilised.

Again, mathematically rearranging the equations would yield an  $f_b$  of:

$$f = f_b = a_s \left( \frac{\tan \delta}{\tan \phi'} \right) + \left( \frac{\sigma'_p}{\sigma'_n} \right) \left( \frac{a_b B}{S} \right) \left( \frac{1}{2 \tan \phi} \right) \quad (6.10)$$

$a_b$  is the fraction of the geogrid bearing width available.

A special condition is when  $a_s = 1$  and  $a_b = 0$  would entail  $f_b$  and  $f_{ds}$  to be equal.

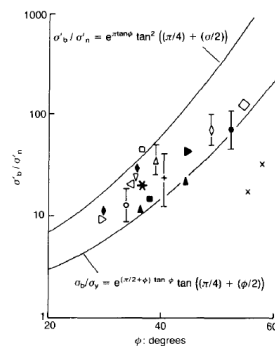
A paramount value that will be later seen involved in the soil interaction is the passive resistance component mobilised ( $\sigma'_p / \sigma'_n$ )

The theoretical value  $\sigma'_p$  mobilised is given as:

$$\sigma'_p = c' N_c + \sigma'_n N_q \quad (6.11)$$

where  $N_c$  and  $N_q$  are the bearing capacity factors analogous to a failure mechanism of a strip footing turned  $90^\circ$ .

Results gathered by Jewell *et al.* (1984) and Jewell (1990a, 1990b, 1996) had deduced a lower and upper bound limits for the passive resistance mobilised on the bearing members in which the test results had been encompassed within such dominion, as per the figure in the subsequent page.



$$\frac{\sigma'_p}{\sigma'_n} = \tan^2\left(\frac{\pi}{4} + \frac{\phi'}{2}\right) e^{\pi \tan \phi}$$

$$\frac{\sigma'_p}{\sigma'_n} = \tan^2\left(\frac{\pi}{4} + \frac{\phi'}{2}\right) e^{((\frac{\pi}{2}) + \phi') \tan \phi}$$

Figure 6.12. Collation of results for measured and predicted bearing stress from Jewell *et al.* (1984) (Obtained from (Jewell, 1990a))

## 6.2. Factors Affecting Soil-Geosynthetic Interaction

### 6.2.1. Soil Particle Size and Geogrids

Based on experiments conducted to investigate the coefficients of Direct Sliding, Jewell (1984) asserts a ratio between the minimum aperture dimension and the average particle size for the applicability of the full employment of geogrid resistance interactions as soil reinforcement.

The ratio as: Minimum Aperture Dimension/Average Particle Size  $\geq 3$

The full utilization occurs when the grain size is the same as that of the geogrid aperture. Initially a small grain size as that of a silt or a fine sand would cause the failure surface to adapt the grid's lateral surface. As the grain size increases, the failure surface is at a tangent to the bearing members of the grid. The maximum effect is obtained when the grain size is similar to that of the grid apertures, causing the soil particles placed against the bearing members causes overtopping to instigate a failure surface to be driven way into the soil mass.

A further increase in the particle size to the extent in becoming larger than the geogrid would cause the coefficient to be lower.

A relation had been deduced by Palmeira and Milligan (1989) between the mobilised bearing stress and the mean particle size in which a clear cut ratio  $B/D_{50}$  (where B is the bearing members thickness and  $D_{50}$  is the average particle size. A value of lower than 10 (i.e. an increase in the average particle size) would induce a change of the failure mechanism to be that of an upper bound passive resistance (as shown in Figure 6.12). Depending also on the shape of the bearing member, the mobilised bearing stress can also be improved to greater than two, as shown in figure 6.13 in the subsequent page.

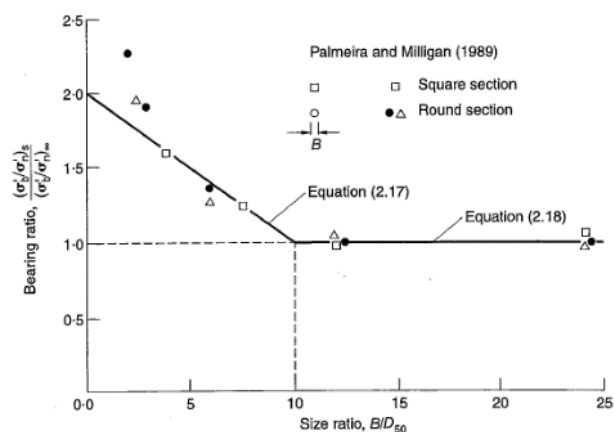


Figure 6.13. Relationship between bearing ratio and mean particle size from Palmeira and Milligan (1989).

Lopes (1998) undertook a similar study on the influence of soil grain size on the soil-geogrid interaction using pullout tests. The studies did corroborate with Palmeira and Milligan (1989) of an increase pertaining, but the extent of increase was to a lower extent by approximately 50% of that obtained by Palmeira and Morgan although such devised empirical equations were incorporated for the different soil and geogrid used by Lopez (1998).

Tables 6.1 and 6.2 below show the soil characteristics and the geogrids tested.

Table 6.1. Properties of Soil Types Used.

| Soil   | $D_{\min}$ | $D_{\max}$ | $D_{50}$ | $C_u$ | $C_c$ |
|--------|------------|------------|----------|-------|-------|
| Sand 1 | 0.074      | 2          | 0.43     | 2.94  | 0.94  |
| Sand 2 | 0.074      | 9.54       | 1.30     | 3.64  | 1.00  |

Table 6.2. Geogrid Properties.

| Geogrid | L (mm) | W   | B    | S  | Tensile Strength (kN/m) |
|---------|--------|-----|------|----|-------------------------|
| GG1     | 960    | 330 | 3.55 | 16 | 80                      |
| GG2     | 960    | 330 | 5.70 | 16 | 120                     |

With sand 2 exhibiting larger grain size properties than sand 1, the global strength at the soil-geogrid interface was seen to increase by 24 to 27%.

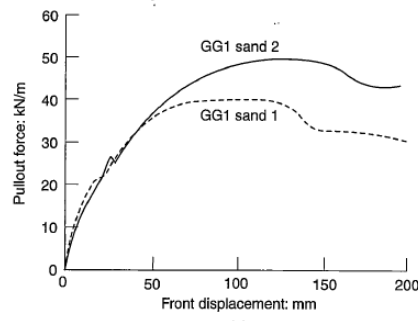


Figure 6.14. Influence of soil grain size on the impact of Pullout Force vs Frontal Displacement from Lopes (2002).

### 6.2.2. Confinement Stress

By re-revising the general equation for bond capacity, confinement stress is directly related as well as impacting the soil friction angle.

$$T = 2WL\sigma'_n f \tan \phi' \quad (6.14)$$

In revisiting the three dimensional phenomenon for bonding stress in geogrids, there is a more notable impact from confining stress in inhibiting that dilatancy would occur, which is attributable to dense soils. This consequently gives a higher soil-geosynthetic interface strength improvement.

Lopez's Geogrid GG1 with sand 1 was tested for confining stress effects (24.5 and 38 kPa) on the amount of pullout force generated from inducing a front displacement.

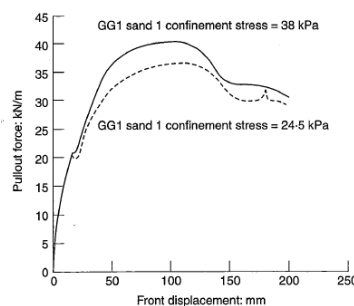


Figure 6.15. Influence of confinement stress on Pullout Force vs Front Displacement from Lopes (2002).

Although the soil was not considered to be of a ‘dense’ state given its ID at 50%, the increase of confining stress level by 55% led to an increase of the shear strength mobilised at the interface and an improvement in the soil-geogrid interface resistance by 11%. Lopez (Geosynthetics and Applications) predicts that a denser soil state would exhibit a much greater increase in interface resistance.

### 6.2.3. Soil Density

A denser granular soil was earlier understood to possess a greater dilatancy behaviour, thus inducing a higher confinement stresses needed to mobilise the maximum bonding stress.

Lopes and Ladiera (1996) conducted a pullout test where two soil samples of different soil relative densities were compared, ID at 50% and ID at 86%. Table x below summarises the properties of both the sands and the geogrids used, and the results of the pullout force registered from the induced frontal displacement shown on figure6.16.

The geogrid with sand at a looser state fails by a lack of adherence at a pullout force of 32.2 kN/m, but however would failure by a lack of tensile strength with a denser state. Although the lack of tensile strength caused the sand to fail at 45.3 kN/m, but it would have been higher if the geogrid had a higher tensile capacity.

The 40% increase in strength for the soil-geogrid interface owes to the greater relative density associated with the sand. The geogrid displacement decrease and the interface stiffness modulus increase, along with the pullout force (Lopes and Ladiera, 1996). It was also recognized that a higher soil density entails a third of the inclusion length to contribute to resistance.

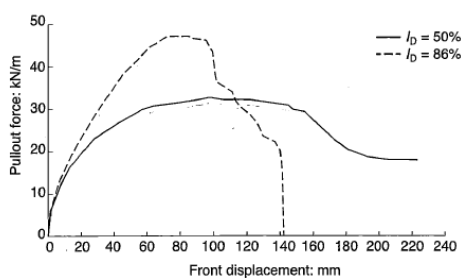


Figure 6.16. Influence of soil density on Pullout Force vs Front Displacement from Lopes (2002).

#### 6.2.4. Reinforcement Orientation and Stiffness

The reinforcement force enables an improvement to the shearing resistance based on the following equilibrium equation of forces provided by Jewell (1996)

$$\Delta P_s = P_r (\sin \theta + \cos \theta \tan \phi') \quad (6.15)$$

Where reinforcement angle is depicted in the figure below:

With a fixed tensile force,  $P_r$ , the optimum orientation  $\theta_{opt}$  is given at  $90^\circ - \phi'$ , although in practice the optimum orientation is intertwined with the magnitude of the tensile force induced.

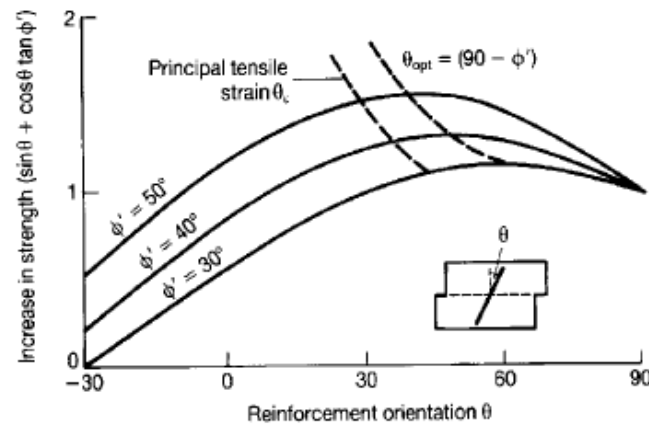


Figure 6.17. Shearing resistance variation as a function of Reinforcement Orientation from Jewell (1996).

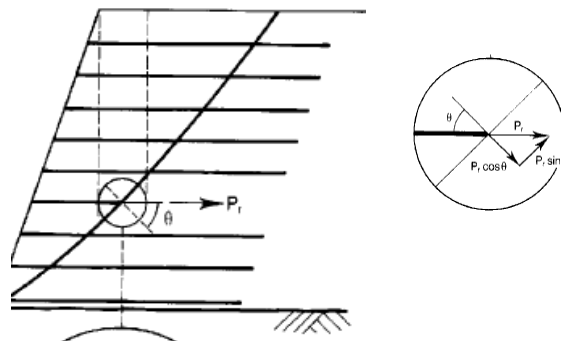


Figure 6.18. Reinforcement effect on equilibrium for reinforcement force from Jewell (1996).

Jewell (1996) asserts two mechanisms for the mobilised reinforcement force  $P_r$ .

The bond between the reinforcement and the soil limits the force acting on a reinforcement layer, which develops through frictional mechanisms (skin friction along reinforcement). Such bond is found to be maximised once the maximum compressive stress in the soil acts perpendicular to the reinforcement plane, in which the maximum bond angle  $\Theta_b$  as shown under Figure 6.19.

The tensile strain developed: which is a manifestation of the reinforcement stiffness properties influencing the soil shear deformation necessary to mobilise the reinforcement force. This is deduced by understanding that it equates to the tensile strain occurring in soil adjacent to the reinforcement. By the reinforcement orienting in the direction of maximum tensile strain, the greatest elongation will be experienced under a given shear deformation by the soil  $\Theta_\epsilon$

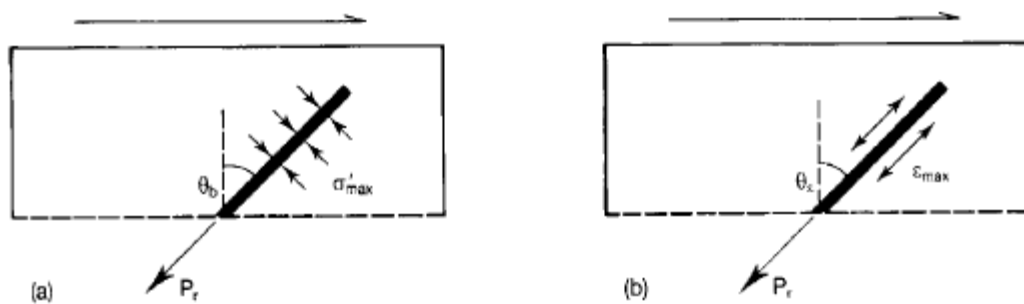


Figure 6.19. Jewell (1996)'s schematic of optimum reinforcement directions to the tensile force to maximise bond stress and tensile strain.

The principal axes for stress and incremental strain are closely aligned for soil undergoing shearing, which comes to the advantage in determining the optimised orientation.

Jewell recommends that the orientation is maximised in compact soils when the reinforcement is about  $45^\circ > \Theta > 25^\circ$ .

### 6.3. Reinforcement Spacing

The effect of reinforcement spacing is strongly intertwined with which failure mechanism that can occur on the GRS system, whether it is an Internal, Compound or an External failure mode.

Leshchinsky and Vulova (2001) provides critique on the Limit Equilibrium (LE) analysis adopted by the FHWA (1997) and NCMA (1997). Such widely used design standards postulate failure mechanisms based on the design strength of the reinforcement and its layouts. The argument put forth by Leshchinsky et al (2001) is that the LE method treats all the modes of failure to be equally critical, and with one ‘randomly serving as the predominant (actual) mode’. The method however is self-obscuring because it provides different factors of safety while holding the premise of equal criticality.

The classic flow of LE design in that the internal stability is first ensured by defining an active wedge in the soil body that must be restrained by the reinforcements intersecting the wedge surface. This follows on by ensuring compound stability in that the slip surfaces may also protrude away from the reinforcements; thus would entail a rotational failure mechanism to be insured. Finally, an external stability analysis analogous to the gravity retaining wall is conducted (direct sliding, overturning, bearing capacity and deep-seated failure are assessed) (Leshchinsky and Vulova, (2001)).

It may be assumed that designing against Internal stability failure would prevent the rest of the mechanisms from occurring. In reality however, the other mechanisms may also occur beforehand, and can drive the internal failure to become more feasible, especially in the case of the reinforcements although closely spaced, but would possess low strength (Leshchinsky and Vulova (2001)).

Leshchinsky and Vulova (2001) refers to Bell *et al.* (1983)’s full scale tests in that the internal stability is no longer an issue then the reinforcements are closely spaced and sufficiently strong, in which Bell *et al.* (1983)’s tests were at 200 to 400 mm spacing. This stems from a ‘confining effect’ manifested by the soil and geosynthetic layer interactions, which impede the movement of soils relative to the tensile inclusions.

The reduction in deformations would consequently prevent the occurrence of a slip surface developing in the reinforced soil.

Bellet *al.* (1983)'s results however did not entirely corroborate with Leshchinsky *et. al* (1994)'s experimental work, where geogrid layers at 200mm spaced and a surcharge load applied to the wall. Contrarily to expecting a failure mechanism as per Rankine or Coulomb's wedge of an outer facing movement occurring with an Active state developing, the soil actually behaved as a rigid mass i.e. no internal failure occurred even though large deformations and displacements developing in the geogrid front end (Leshchinsky *et al.* 1994).

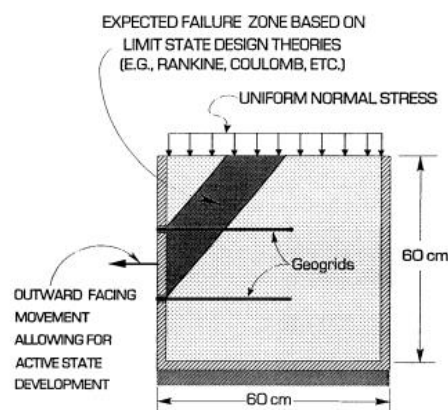


Figure 6.20. Postulated failure mechanism for block walls in typical standards) from (Leshchinsky and Vulova, 2001).

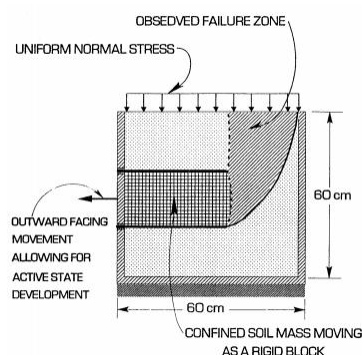


Figure 6.21. Observed mechanism from (Leshchinsky and Vulova, 2001).

Leshchinsky *et al.* (1994) offered an explanation in that soil confinement is only significant once the reinforcements are close, but what exactly constitutes to 'closely spaced reinforcements' is more complex than just spacing, as it is interrelated with soil

type (particle size, stiffness, strength and dilation), reinforcement strength and stiffness, foundation properties, types of facing connections (although it will be later seen that Wu et al (2001) had an opposing account on facing connections being in any way necessary to be dispensed).

Such variables geared Leshchinsky and Vulova (2001) into producing a numerical parametric study of a reinforced soil using the Finite Element package FLAC 2D, where a change in a parameter was investigated with a change in reinforcement spacing. The baseline comparisons required Leshchinsky and Vulova (2001) to define two modes of instability:

- Verge of Failure

Indicates a clear shear zone developing between the crest and the face (or toe), which is an initial state at which the shear zone is consistently developed. This implies a threshold stage being reached where the Mohr Coulomb failure criterion is satisfied along a well-defined slip surface (parallel with the Limit Equilibrium analysis) (Leshchinsky and Vulova, 2001)

- Post Failure

An evolution from the ‘Verge of Failure mode, Post failure is ultimately a catastrophic collapse subsequently occurring after the slip surface has developed. There is no longer a static equilibrium and the structure has reached a total collapse, where in which the various failure mechanisms can be recognised.

In practice however, the under-utilised reinforcements would undergo a load redistribution from the previously failing reinforcements, which must be appreciated in that FLAC had not taken this into account (Leshchinsky and Volova, 2001)

The benchmark variable in these failure mechanisms is the wall height that would cause failure (given that the reinforcement length in each model is constant).

The failure mechanisms against which such modes of instability would cause are:

- External (Direct Sliding and Toppling) which is characterised by a failure occurring at the block-block interface. There was no failure occurring within the

reinforced soil zone, which is a characteristic of strong interaction between the reinforcement layers and the soil confined.

The verge of failure state had a height of  $H=6.6$  m ( $L/H) = 0.23$

The failure mode for a numerical analysis is shown below:

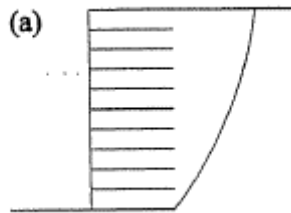


Figure 6.22. External slip surface postulation from Vulova and Leshchinsky (2002).

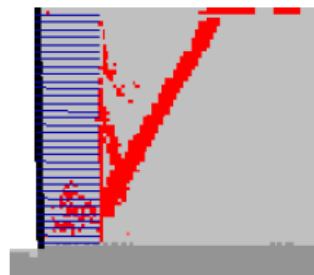


Figure 6.23. Typical numerical result of external slip surface on Verge of Failure from Leshchinsky and Vulova (2001).

- Deep Seated Failure is depicted as a mode of failure occurring when the reinforced soil has closely spaced reinforcements but is embedded by a relatively weak foundation.

The verge of failure possesses a height of  $H= 3.2$  m. ( $L/H) = 0.47$

The deep-seated failure can be analysed by employing Bishop's method to approximate a circular arc.

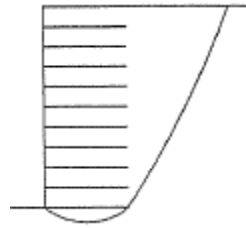


Figure 6.24. Typical numerical result of Deep-Seated slip surface on Verge of Failure from Leshchinsky and Vulova (2001).

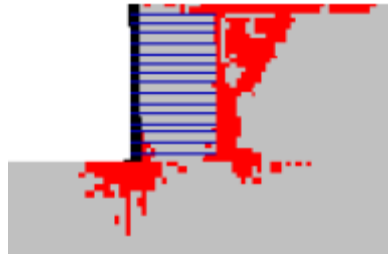


Figure 6.25. Deep Seated slip surface postulation from Vulova and Leshchinsky (2002).

- Compound is observed to have a shear zone developing within the reinforced and retained soil. The reinforced soil would not behave as a monolithic block but rather the collapse due to sliding occurs as a result of a compound surface. The verge of failure possesses a height of  $H = 3.2$  m.  $(L/H) = 0.47$

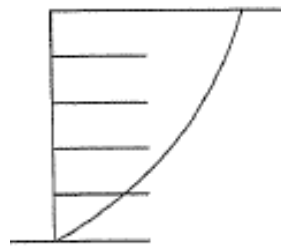


Figure 6.26. Compound slip surface postulation from Vulova and Leshchinsky (2002).

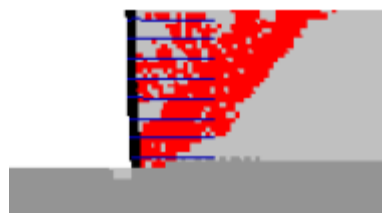


Figure 6.27. Typical numerical result of compound slip surface on Verge of Failure from Leshchinsky and Vulova (2001).

- Internal Connection Failure inherently consists of shear zones developing from within the reinforced soil. There is no coherent mass behaviour occurring in such failure mode.

The verge of failure condition possesses a height of  $H = 2.6\text{m}$ .  $(l/H) = 0.58$

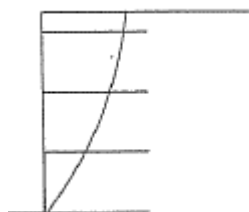


Figure 6.28. Internal connection slip surface postulation from Vulova and Leshchinsky (2002).

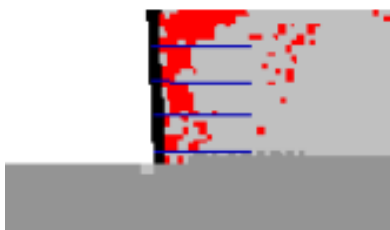


Figure 6.29. Typical numerical result of internal connection slip surface on Verge of Failure from Leshchinsky and Vulova (2001).

The summary of critical heights from the numerical models manifesting the ‘Verge of Failure’ and the ‘Post Failure’ mode is shown in Table 6.3 in the subsequent page:

Table 6.3. Typical Critical Heights for Verge of Failure and Post Failure for Various Mechanisms

| Mechanism             | Critical Height on Verge of Failure (m.) | Critical Height on Post failure (m.) |
|-----------------------|------------------------------------------|--------------------------------------|
| External Slip Surface | 6.6                                      | 8.6                                  |
| Deep-Seated           | 3.2                                      | 4.4                                  |
| Compound Slip Surface | 3.2                                      | 5.4                                  |
| Internal Connection   | 2.6                                      | 4.6                                  |

Below is a graph to delineate the patterns of changes in critical height from Leshchinsky et al's (2001) different models each with a varied spacing and a soil friction angle unique to the model. In all the soil strength parameters, the critical height for the most anticipated failure mode decreases as the spacing increases. For a fixed spacing however, the critical height decreases as the friction angle decreases, but the effectiveness of the frictional angle impact on the critical height is more profound at spacing amounts less than 0.6 m. One particular observation was the close merging of the critical heights with spacing for a friction angle of 45° backfill on the verge of failure with the critical heights of the backfill at 35° friction angle at post failure, which one can deduce that the friction angle greater than 35° pronounces a greater critical height with the spacing kept constant.

In all models, the smaller spacing of the reinforcement did attribute to an external mode of failure to proceed, though with the friction angles reducing below 35° had disputations as to whether external or compound failure manifests first, but it seems to be more pronounced at friction angles higher than 45° that the external mode of failure prevails.

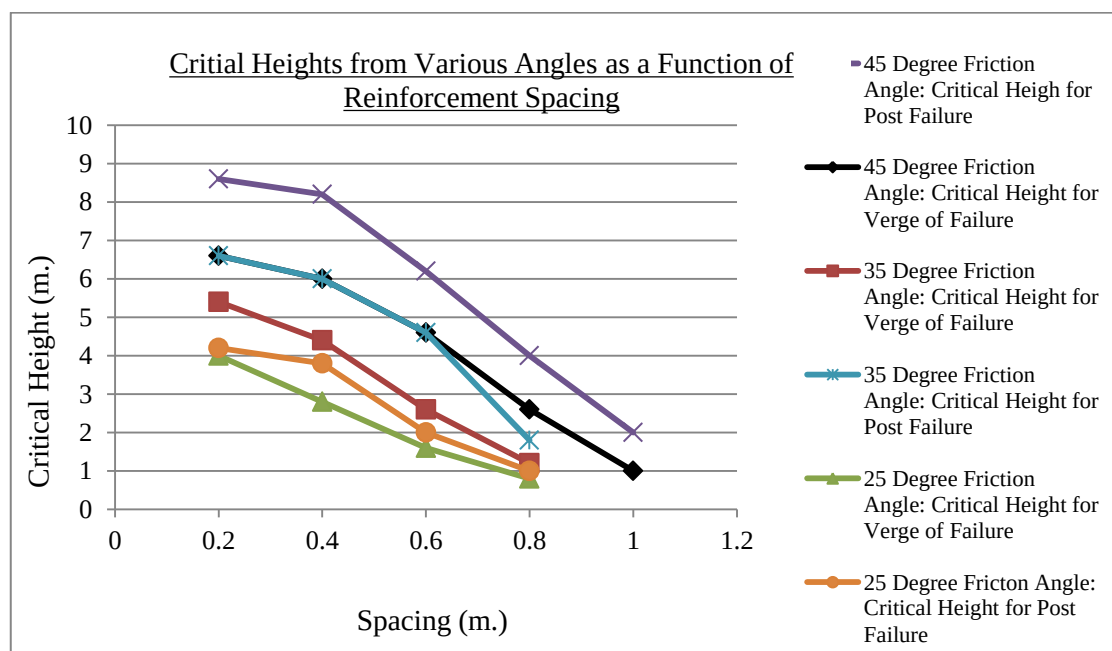


Figure 6.30. Summary of Leshchinsky and Vulova (2001) results for critical heights and various friction angles for Verge of Failure and Post Failure as a function of reinforcement spacing.

The other observation is the impact of the foundation stiffness assigned on the critical height manifested. By exclusively observing of a 35° friction angle backfill, an extremely stiff foundation exhibited greater critical heights for failure modes on the verge of failure compared to that of a foundation of equal stiffness but at post failure, which shows a profound impact of the foundation stiffness on the critical height manifested. Another interesting comparison was the impact of the base foundation stiffness when the same foundation is assigned as the retained wall backfill. The backfill friction angle reduction for such a scenario brings an inclination that the first instigating mode of failure diverts away from being an external mode of failure when smaller reinforcement spacing is assigned. i.e. Bell's observations hold more bearing when the underlying foundation.

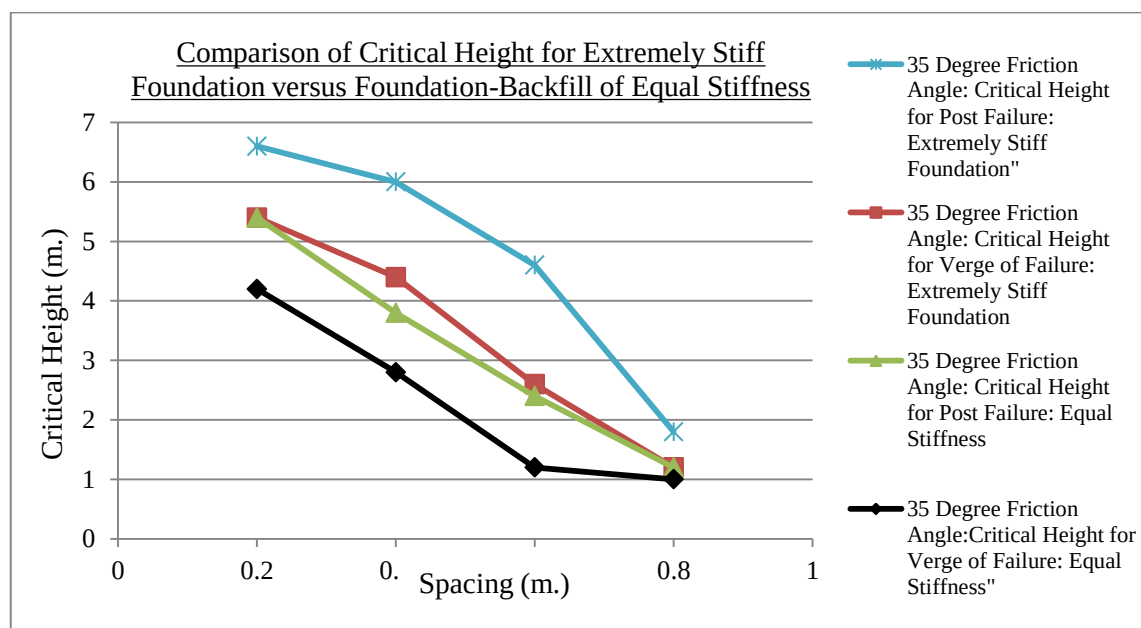


Figure 6.31. Summary of Leshchinsky and Vulova (2001) results for critical heights and various friction angles for Verge of Failure and Post Failure as a function of reinforcement spacing, with special focus on foundation stiffness.

Table 6.4. Leshchinsky and Vulova (2001) Summary Results for Extremely Stiff Foundation.

|                    | Spacing | H at Verge of Failure | H at Post Failure | Prevailing Failure Mode |
|--------------------|---------|-----------------------|-------------------|-------------------------|
| 45° Friction Angle | 0.2     | 6.6                   | 8.6               | External                |
|                    | 0.4     | 6                     | 8.2               | External/Compound       |
|                    | 0.6     | 4.6                   | 6.2               | Compound/Connection     |
|                    | 0.8     | 2.6                   | 4                 | Connection              |
|                    | 1       | 1                     | 2                 | Connection              |
| 35° Friction Angle | 0.2     | 5.4                   | 6.6               | External/Compound       |
|                    | 0.4     | 4.4                   | 6                 | Compound/External       |
|                    | 0.6     | 2.6                   | 4.6               | Connection/Compound     |
|                    | 0.8     | 1.2                   | 1.8               | Connection              |
| 25° Friction Angle | 0.2     | 4                     | 4.2               | Compound/External       |
|                    | 0.4     | 2.8                   | 3.8               | Compound                |
|                    | 0.6     | 1.6                   | 2                 | Connection              |
|                    | 0.8     | 0.8                   | 1                 | Connection              |

Table 6.5. Leshchinsky and Vulova (2001) Summary Results for completely normal foundation.

|                    | Spacing | H at Verge of Failure | H at Post Failure | Prevailing Failure Mode |
|--------------------|---------|-----------------------|-------------------|-------------------------|
| 45° Friction Angle | 0.2     | 6.6                   | 8.6               | External                |
|                    | 0.4     | 6                     | 8.2               | External/Compound       |
|                    | 0.6     | 4.6                   | 6.2               | Compound/Connection     |
|                    | 0.8     | 2.6                   | 4                 | Connection              |
|                    | 1       | 1                     | 2                 | Connection              |
| 35° Friction Angle | 0.2     | 5.4                   | 6.6               | External/Compound       |
|                    | 0.4     | 4.4                   | 6                 | Compound/External       |
|                    | 0.6     | 2.6                   | 4.6               | Connection/Compound     |
|                    | 0.8     | 1.2                   | 1.8               | Connection              |
| 25° Friction Angle | 0.2     | 4                     | 4.2               | Compound/External       |
|                    | 0.4     | 2.8                   | 3.8               | Compound                |
|                    | 0.6     | 1.6                   | 2                 | Connection              |
|                    | 0.8     | 0.8                   | 1                 | Connection              |

#### 6.4. Failure Mechanisms Exhibited as a Function of Reinforcement Stiffness and Geosynthetic Spacing

Leshchinsky *et al.* (2002) conducted a numerical study to assess the effect of reinforcement stiffness on the mode of failure mechanism. There is an additional interrelated component in the assessment: soil strength.

Two types of reinforcements were assessed:

(A) Baseline Reinforcement with  $E$  of  $106 \text{ kN/m}^2/\text{m}$

(B) Ductile Reinforcement with  $E$  of  $105 \text{ kN/m}^2/\text{m}$

Table 6.6. Reinforcement Type, spacing, Soil Strength and corresponding failure modes with critical heights.

|                                     | Low Strength Soil $25^\circ$ |          | Medium Strength Soil $35^\circ$ |          | High Strength Soil $45^\circ$ |          |
|-------------------------------------|------------------------------|----------|---------------------------------|----------|-------------------------------|----------|
| Reinforcement Type and Spacing (m.) | A/0.2                        | B/0.2    | A/0.4                           | B/0.4    | A/0.4                         | B/0.4    |
| Failure Mode                        | Compound                     | Compound | Compound                        | Compound | External                      | Compound |
| Critical Height                     | 4                            | 2.2      | 4.4                             | 3.2      | 6                             | 5        |

Irrespective of the backfill soil strength, the stiffer baseline reinforcement manifested a higher critical height than the ductile reinforcement, indicating that the ductile reinforcement enables larger deformations to occur, which associates with greater plastic zones. However, such exacerbation in the critical height is more prominent in soil with lower strength, although the higher strength soil had a fundamental change in the critical failure mechanism type.

##### 6.4.1. Failure Mechanisms Exhibited as a Function of Connection Strength and Geosynthetic Spacing

Leshchinsky and Vulova (2001) posed three types of connections in order to assess the impact of the connection strength on the failure mechanisms. These were:

- (i) Frictional connection with baseline strength (modelled interface of  $\phi=30^\circ$  and  $c=0$  kPa)
- (ii) Frictional connection with low strength (modelled interface of  $\phi=20^\circ$  and  $c=0$  kPa)
- (iii) Structural Connection (modelled interface of  $\phi=30^\circ$  and  $c=20$  kPa)

These were compared in parallel with two cases, the first with small reinforcement spacing of 0.2 m and the other was with a reinforcement spacing of 0.6m.

The results that were obtained by Leshchinsky *et al.* (2001) are as follows:

Table 6.7. Connection Type with Failure Modes Initiated.

|                                        | Spacing of 0.2m |          |          | Spacing of 0.6 m. |          |
|----------------------------------------|-----------------|----------|----------|-------------------|----------|
| Connection Type                        | A               | B        | C        | A                 | C        |
| 1 <sup>st</sup> Initiated Failure Mode | External        | External | External | Connection        | Compound |
| Critical Wall Height                   | 6.6             | 6.6      | 6.6      | 2.6               | 4.6      |

The small spacing case revealed that the connection strength had no influence on the wall behaviour, but was more profound for larger spacing arrangement of 0.6 m. This owes to the larger spacing not able to cater for preventing connection failure in the same way as smaller spacing. Therefore, when a stronger connection arrangement is assigned for the larger spacing, the first failure mechanism instigating was a compound failure mode, as oppose to the baseline connection causing a connection failure to materialise first.

Intermediate reinforcements contribute significantly to the stability of the reinforced wall.

Leshchinsky and Vulova (2001) investigated the impact of insertion of intermediate reinforcements between adjacent primary reinforcements. At a short length of 300 mm. for a large spacing configuration of 600mm. between the primary reinforcement demonstrated a significant increase in the failure height from 2.6 m. to

5m. The internal failure mode associated between the large spacing configurations also transforming into become a compound failure.

This was due to the intermediate layers facilitating to carry out what was otherwise to be carried by the primary layers.

#### 6.4.2. Soil-Geosynthetic Interface Strength Effects

Leshchinsky and Vulova (2001) attempted to assess the impact of the frictional shear strength  $\delta$  (a combination of the soil friction angle  $\phi$  and dilatancy angle  $\psi$ ) on the critical height obtained. A baseline model of interface strength  $\delta=35^\circ$  symbolises a perfect bonding condition between the reinforcement and the confining soil, which is exemplary in geogrids having both friction along longitudinal ribs and passive resistance along traverse ribs.

A set of models were created, where all had a varying soil strength but equal reinforcement spacing of 400 mm, shown in Table 6 below:

Table 6.8. Typical Soil Properties and Interface Strengths.

| Soil and Interface Strength                      | H at Verge of Failure (metres) | $T_{\max}$ in Reinforcement | Prevailing Failure Mode |
|--------------------------------------------------|--------------------------------|-----------------------------|-------------------------|
| $\Phi=45^\circ, \psi=15^\circ (\delta=35^\circ)$ | 6                              | 8.8                         | External/Compound       |
| $\Phi=35^\circ, \psi=7^\circ (\delta=26^\circ)$  | 4.4                            | 9.6                         | External/Compound       |
| $\Phi=25^\circ, \psi=0^\circ (\delta=18^\circ)$  | 2.8                            | 8.3                         | Compound                |

The results displayed an impact of the dilatancy on the height manifested at the verge of failure, but does not significantly govern the prevailing failure mode that occurs. The amount of tensile force on the reinforcement, T-max seems to exhibit an optimised behaviour, where it increases with a reduction of friction angle from  $45^\circ$  to  $35^\circ$  but then reduces again.

## 6.5. Reinforcement Length

### 6.5.1. Impact on the Reinforcement Stresses

Leshchinsky *et al.* (2001) attempted to examine reinforcement length on the wall stability by revisiting 3 models that exemplified the three failure modes: A- External Failure, B-Compound Failure and C- Deep Seated Failure. The baseline models were given heights equal to the critical heights they had possessed with their initial overall layouts, and were modified with a series of 'stable states' in such that their reinforcement lengths were increased. An assessment was conducted on the stress distributions at two points of interest, the first at 0.15metres behind the wall and the second at 0.15metres behind the reinforced soil.

#### (i) Model A

A high strength  $45^\circ$  friction angle soil with a very stiff foundation, reinforcement spacing of 0.2m.and an associated critical height of 6.6 m. ( $l/h=0.23$ ) experienced an external failure.

The three stable states investigated having a consecutively increasing reinforcement length were  $l=2.0$ ,  $l=2.6$  and  $l=3.3$  m. and gave failure modes emulating wall toppling, where there was stress concentration at 0.15 metres behind the wall and a smaller stress at behind the soil wall. The increase in reinforcement length manifested to a decrease in stresses behind the facing wall and increase at the back of the soil wall; which entailed an increase in wall stability and decrease in a rotation about the toe.

#### (ii) Model B

A high strength  $45^\circ$  friction angle soil with a very stiff foundation, reinforcement spacing of 0.4 m. and an associated critical height of 5 m. ( $l/h=0.3$ ) experiencing a compound failure. The reinforcements this time were of ductile nature.

The same behaviour improvement was exhibited as in Model A with the increase in reinforcement length.

(iii) Model C

A low strength  $25^\circ$  friction angle soil with a low foundation, reinforcement spacing of 0.4 metres and an associated critical height of 3.2 metres ( $l/h=0.47$ ) experiencing a compound failure.

The same behaviour improvement as that was exhibited in A and B with the increase in reinforcement length.

### 6.5.2. Impact of Reinforcement Length with Various Factors

Bilgin (2008) had a different account on the reinforcement spacing, but the investigation was to investigate the impact on the vertical spacing on the minimum required length.

Bilgin (2008)'s research theme was geared towards investigating whether the external or internal stability modes dictated the reinforcement length and under what parameters are certain modes more influential.

Bilgin (2008) identifies deviations present amongst few agencies from the general  $L_{\min}$  adoption of  $0.7H$  (FHWA, AASHTO), such as the Hong Kong and Brazilian guidelines adopt  $0.5H$  and  $0.8H$  respectively, and the common standard NCMA adopting minimum of  $0.6H$ .

There was no overall consensus in the literature and standards regarding the governing criteria determining the reinforcement length, for example the NCMA chooses the sliding and bearing capacity external failures as the leading mechanisms determining the reinforcement length, whereas FHWA chooses the internal stability to control the reinforcement length. Whitlow explicitly concludes, in agreement with the British standards, that the reinforcement length is externally governed and the spacing internally governed.

Bilgin (2008) provides minimum reinforcement lengths for external and internal mechanisms as shown below, and on these bases, Bilgin produced parametric studies to investigate each parameter's impact on  $L_{\min}/H$  ratio. The parametric studies included

Wall height, Backfill soil unit weight, friction angle, vertical spacing and the reinforced soil unit weight and friction angle.

The minimum reinforcement lengths for external and internal modes s that Bilgin (2008) investigated are as follows:

$L_{min}$  from External Mode

$$L_{min,E} = \sqrt{\frac{0.5H^2 K_{a,b} (\frac{1}{3}\gamma_b H + q_s)}{\gamma_r H + q_s}} .6 \quad (6.16)$$

$L_{min}$  from Internal Mode (Pullout)

$$L_{min,P} = L_a + L_e \quad (6.17)$$

$$L_e = \frac{T_{max}}{F \cdot \alpha \sigma_v C R_c} \times F S_P \quad (6.18)$$

### 6.5.3. Effect of Wall Height

The studies had shown that the increase in wall height decrease the required minimum length to height ratio. The governing mode failure is indicated to be the Pullout failure mode, with the highest length ratio required to satisfy minimum required reinforcement length is 0.65H

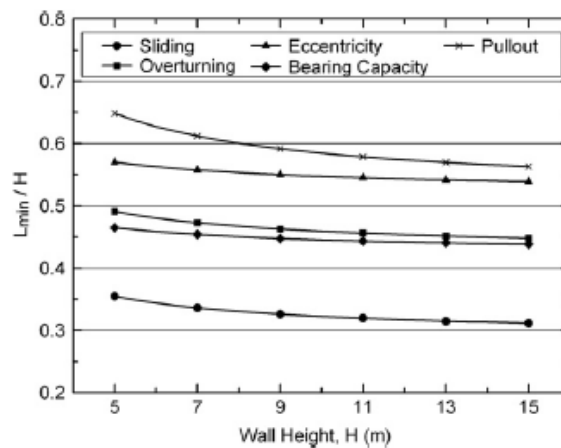


Figure 6.32. Minimum reinforcement requirements as a function of wall height.

#### 6.5.4. Retained Soil Unit Weight and Friction Angle

External failure modes were the most affected by the retained soil unit weight (assessment ranges 16-20 kN/m<sup>3</sup>) and friction angle (investigated between angles 25-45°), but the friction angle increase had a greater impact on the  $L_{min}/H$  ratio. A higher unit weight would attribute to a greater horizontal force action, which entails a longer reinforcement required to create a larger monolithic body capable of producing higher resisting forces and moments to provide stability. The friction angle increase directly produces a reduced lateral earth pressure. In both cases, the internal failure mechanism pullout failure is not affected by the retained soil friction angle, as no reinforcement is within the retained soil.

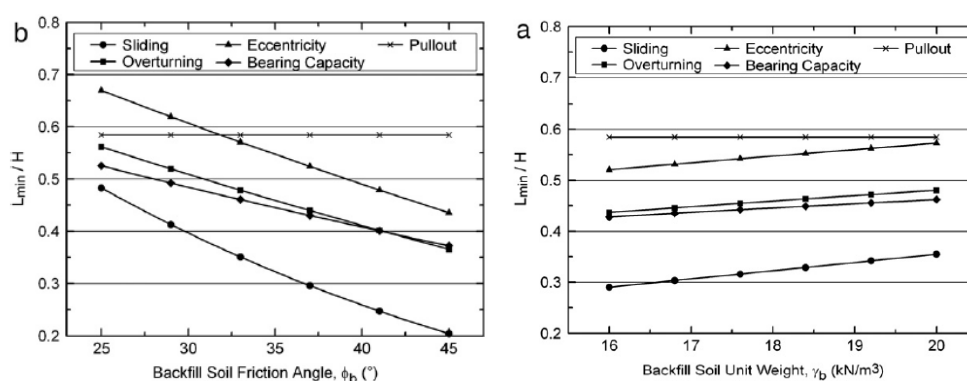


Figure 6.33. Minimum reinforcement length as a function of backfill soil friction angle from Bilgin (2009).

#### 6.5.5. Reinforced Soil Unit Weight and Friction Angle

The internal Pullout failure mechanism was the more critical mechanism in determining the required minimum length required.

The necessity for longer reinforcements in external failure modes is disposed of when lower soil reinforced unit weight is lower. The sliding failure mode in particular demonstrated the largest decrease. It was also the only external failure mode impacted by the reinforced soil friction angle, and is particularly evident once the reinforced soil friction angle becomes less than the foundation soil friction angle.

The pullout mechanism governs the minimum length required when the soil has low friction angles, but the external mechanisms were found to be governing in the higher friction angle cases.

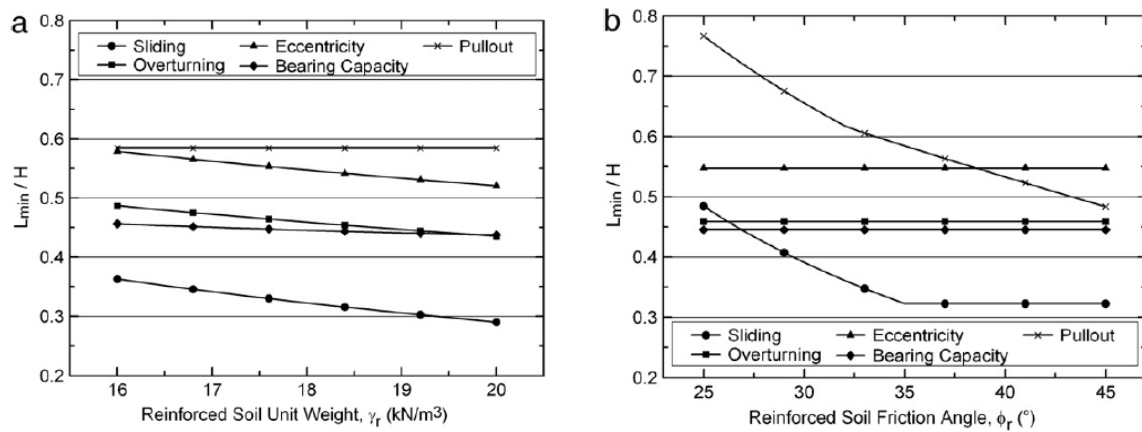


Figure 6.34 Minimum reinforcement length required as a function of reinforced soil unit weight from Bilgin (2009).

#### 6.5.6. Foundation Soil Unit Weight and Friction Angle as Function of Reinforcement Length

Pullout failure mode is again a governing factor for the foundation soil unit weight, although as with all the failure modes is unaffected by the unit weight. The only exception is the bearing capacity mode which has a slightly diminished  $L_{min}/H$  ratio.

The soil friction angle however strongly influences the bearing capacity failure modes, where a 50% reduction in the reinforcement length was observed with friction angle increasing from  $25^\circ$  to  $45^\circ$ .

External mode of failure (Bearing Capacity) governs when the friction angle is at low ranges of  $25^\circ$  -  $30^\circ$ . Once exceeding will see the Pullout internal failure mode to govern.

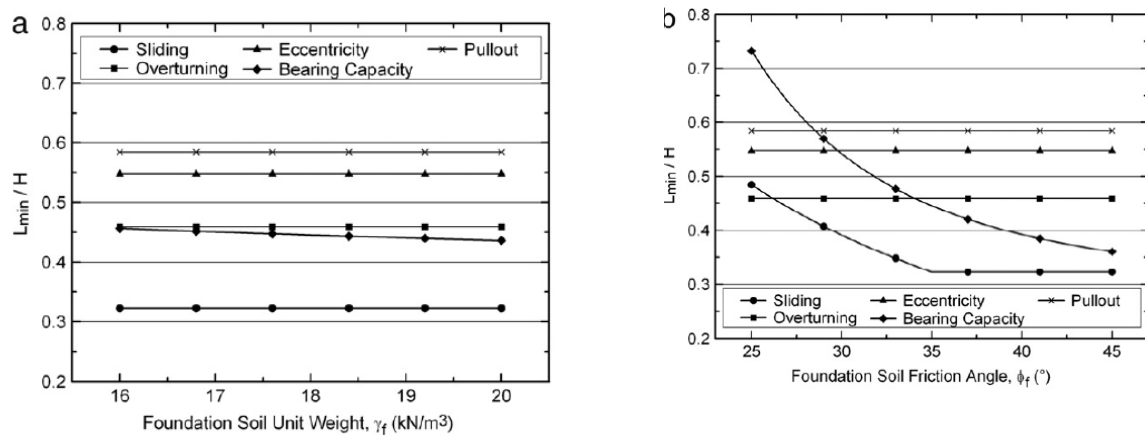


Figure 6.37. Minimum reinforcement length as a function of foundation soil unit weight from Bilgin (2009).

Overall, the most profound parameter influencing the reinforcement length was the friction angle in all its forms, whether as the foundation, retained backfill and the reinforced soil friction angle.

Especially for the cases of backfill and foundation soil where at low friction angles, external failure (bearing capacity) had governed the reinforcement length and with larger friction angles, internal failure (pullout) was the foremost parameter in assigning the reinforcement length

The works from Smith (1977) and Jewell (1980) put forward a different account, in that a stronger reinforced soil is not always directly proportional, as there is an optimised spacing such that a further spacing reduction would have a reduction in the amount of shear strength increase (Jones, 1996).

Proper quantification of this can be stemmed by the ratio of the spacing ( $S$ ) to the effective length ( $L_e$ ), where  $L_e$  is the reinforcement length extending away from the soil critical plane. In the event of a critical ratio  $S/L_e$  of 1 would exhibit a diminishing influence for each reinforcement on the reinforced soil shear strength (Jones, 1996).

## 6.6. Conclusions

- Two major movement mechanisms are predominant in the interaction of the reinforcement with the soil: Pullout and Sliding failure mechanisms.

- The mobilization mechanisms in skin friction along reinforcement (the only mechanism attributable to geotextiles), soil-soil friction and passive thrust acting upon reinforcement bearing members.
- The magnitude in the change of tensile force with distance ( $dT/dx$ ) determines how the shear stress is being transferred. A positive  $dT/dx$ , as is the nature in the active zone, which would imply that the soil is transferring shear stress into the reinforcement. The passive zone contrarily has a negative  $dT/dx$  magnitude, which implies the shear stress being shed into the soil. The reinforcement length is extended beyond the active zone boundary (from the point of maximum tensile force build up) to extend to a length adequate enough to shed all the shear stress back into the restraint zone.
- Rupture is a failure mode associated exclusively with reinforcements which occurs when the axial load exceeds the axial tensile strength of the reinforcement. This has drastic consequences as the failure of one reinforcement layer would cause a full load to be transferred to the other layers, thus manifesting a total progressive failure.
- The soil-reinforcement mobilized shear strength for geotextiles is interdependently based a twofold mechanism: such that the soil needs to inherently possess dilatancy (in that plastic tensile strain occurs) as well as the reinforcement having a certain extensibility extent (referred to as a 2-dimensional phenomenon). Geogrids on the other hand have an additional mechanism, which is an additional ability to mobilize the passive thrust on the grid's bearing members with increases in displacement. This third mechanism causes the geogrid to be less extensible than the geotextile, and as a corollary would have a greater shearing resistance that can be mobilized (referred to as a 3-dimensional phenomenon).
- Particle grain size of the reinforced soil has a profound effect on the geogrid performance, in which maximum performance is achieved when the grain size is the same as the geogrid aperture size.
- Confinement stress has an important role on bond capacity as well as directly impacting the soil friction angle. A soil in a dense state would have an increased

shear strength mobilised at the interface. A denser granular soil was earlier understood to possess greater dilatancy behaviour, thus inducing a higher confinement stresses needed to mobilise the maximum bonding stress.

- Reinforcement spacing can impact the overall failure mechanism of the GRS system, whether it will fail internally, externally or exhibit compound failure.
- The most dominant property impacting the reinforcement length suitable for providing adequate tensile strength and impacting which failure mode is most critical is the friction angle in all its forms (whether it is that of the reinforced fill, foundation or backfill).

## 7. FACING STABILITY

Many standards such as the NCMA and the British Standard dedicate provisions to the wall facings and connections of the Reinforced Soil Walls, which also raises the question on whether or not the internally stabilised wall system fulfils the prophesied perception of fully dispensing the requirement for any external body necessary to provide a structural stability contribution.

The following chapter explores three studies tackling this matter; the first by Bathurst *et al.* (2006) which assesses the wall performance to withstand surcharge loads with regards to facing stiffness, the second by Ehrlich and Mirmoradi (2013) which assesses how such contribution is dependent on toe resistance and thirdly by Wu and Payeur (2015) which distinguishes in dispensing the necessity of adopting heavy segmental blocks commonly used in the construction industry.

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### 7.1. Study 1: Bathurst *et al.* (2006) Wall Facing Stiffness Effects on GRS Wall Stability

Bathurst *et al.* (2006) conducted a study to investigate whether the facing stiffness contributes as a structural component in the stability of GRS walls. The research was geared as a result of AASHTO's simplified method not tackling the stiffness of the facing type influence on the reinforcement loads. By carrying out tests on two full scale walls, with one comprising of a flexible wrapped-face reinforced soil wall and the other a stiff-face segmental reinforce soil wall, a quantitative comparison was made on the extent of reinforcement load reduction for both wall cases.

The wall arrangements, (displayed in Figure 7.1 and 7.2 below), possessed the same soil and reinforcement property arrangements. Particular interest was the soil properties peak  $\phi_p$ , constant volume  $\phi_{cv}$  and direct sliding friction angle  $\phi_{ds}$ , which will be elaborated further when investigating the maximum Reinforcement Load imposed versus the surcharge amount. Other soil properties worthy of mention are unit weight Unit Weight =  $16.7 \text{ kN/m}^3$ ,  $C_c = 2.25$   $C_u = 1.09$  and  $D_{50} = 0.34 \text{ mm}$ . The reinforcements were poly-propylene geogrids of 2.52 metre lengths and arranged at a spacing of 0.6m. The tensile loads acting are calculated by multiplying the secant stiffness  $J$  (an isochronous function) with the amount of strain generated (shown in figure 7.3). Figure 7.4 describes the surcharge history for the test walls i.e. at what period and magnitude was the surcharge of a construction nature and of an external surcharge nature.

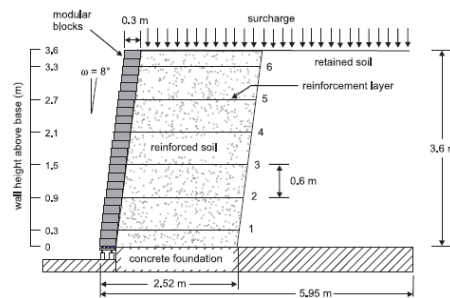


Figure 7.1. Stiff-face segmental wall from Bathurst et al (2006).

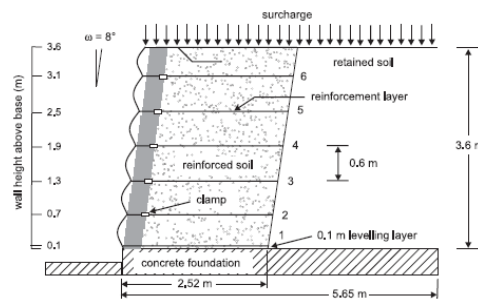


Figure 7.2. Flexible wrapped-face reinforced wall from Bathurst *et al.* (2006).

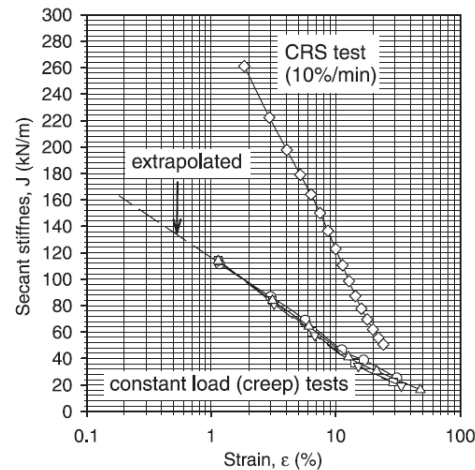


Figure 7.3. Secant stiffness  $J$  Index property for reinforcement based on Creep Load tests from Bathurst *et al.* (2006).

At the end of construction, the stiff-face structure demonstrated very small strains of 1% compared to the wrapped-face structure (maximum of 4%). The fourfold magnitude difference indicates the rigid facing carrying a portion of the lateral earth loads.

The largest strains for both walls were exhibited close to the wall facings (shown in Figure 124), especially with the stiff-faced walls being a manifestation of the downward movement of the soil behind the facing. This is caused by settlement of backfill during compaction and outward rotational movement of the facing column during construction (Bathurst *et al.*, 2006). The wrapped-face experienced its strains as a result of the downward sagging of the wrapped portions at the wall front.

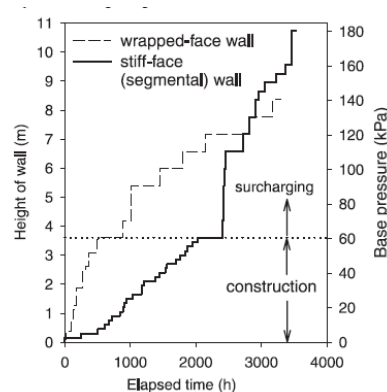


Figure 7.4. Test walls surcharge history from Bathurst *et al.* (2006).

Another ballpoint comparison is with an 80 kPa surcharge load application (the maximum surcharge application the wrapped face configuration has experienced). Each wall exhibited a strain magnitude 5 times larger than their respective end of construction situation (as depicted in Figure 7.5). However, there was no clear defined peak strain location along the reinforcement layers for the stiff wall facing, but when adopting a 90 kPa for the stiff-face wall, the peak reinforcement strain started to materialise, which corresponded to an internal failure plane. The 80 kPa surcharge however was enough to define such pattern for the wrapped-face wall.

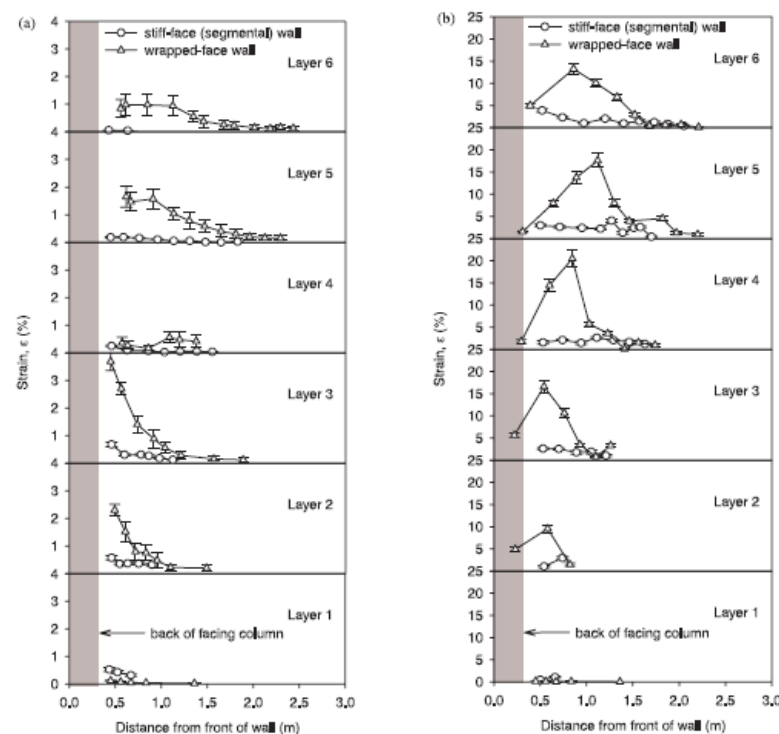


Figure 7.5. Comparison of reinforcement strains at (a) end of construction and (b) 80 kPa surcharge from (Bathurst *et al.*, 2006).

### 7.1.1. Results for Tensile Loads

Based on the time of interest (the time duration is taken as the time elapsed since the layer was placed on the wall), the reinforcement load  $T$  was measured based on an isochronous stiffness value  $J$  and measured strain value. For comparison purposes, the maximum tensile load was also calculated based on the simplified method as

$$T_{Max} = S_v K_{AH} (zy + q) \quad (7.1)$$

The coefficient  $K_{ah}$ , the horizontal component of the active earth pressure calculated based on Coulomb earth pressure theory, is a function of  $\phi$  (soil friction angle),  $\delta$  (peak wall-soil interface friction angle) and  $\omega$  (wall facing batter from the vertical)

By calculating the predicted loads using three different peak friction values for the soil ( $\phi = \phi_{ps} > \phi_{cs} > \phi_{cv}$ ) using the Simplified Method, the measured reinforcement loads at all surcharge loads demonstrated more congruence at the peak plane strain angle than the peak friction angle from direct shear and constant volume friction angles. The extent of congruence was more pronounced with the flexible wrapped-face wall (Figure 7.7) than the stiff-face wall (Figure 7.6) (i.e. the stiff face wall had lower tensile forces than the flexible wrapped-face wall).

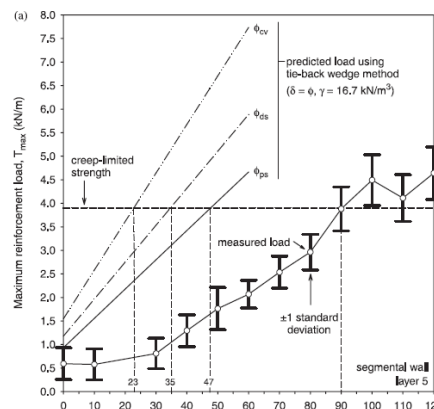


Figure 7.6. Predicted and as measured reinforcement tensile loads at end of construction and during surcharging for Stiff-Face segmental wall from Bathurst *et al.* (2006).

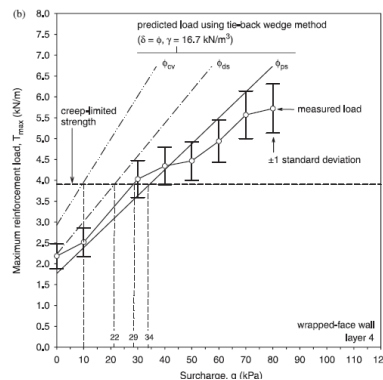


Figure 7.7. Predicted and as measured reinforcement tensile loads at end of construction and during surcharging for Flexible Wrap-Face Segmental Wall from Bathurst *et al.* (2006).

In order to make quantitative judgements on the difference between the wrapped-face and stiff-face walls, the creep-limited strengths of the reinforcements were superimposed on both graphs, and the surcharge corresponding to the creep-strength tensile force amount of 3.9 kN/m were extracted from the graphs above (depicted as a dashed line) as follows:

Table 7.1. Surcharge Amounts Calculated for Given Friction Angles.

| Friction Angle Type | Surcharge Amount (kPa) Corresponding to Creep-limited Strength of 3.9 kN/m |            |
|---------------------|----------------------------------------------------------------------------|------------|
|                     | Wrapped-Face Wall kPa (Factor)                                             | Rigid Wall |
| $\phi_{ps}$         | 47 (1.9)                                                                   | 34 (2.65)  |
| $\phi_{cs}$         | 35 (2.6)                                                                   | 29 (3.1)   |
| $\phi_{cv}$         | 23 (3.9)                                                                   | 22 (4.09)  |

It was observed that the measured data displayed that a surcharge load of 90 kPa was required to be reached to obtain the creep-limited strength of the critical reinforcement layer, but when using the friction angles for  $\phi$  as  $\phi_{ps}$ ,  $\phi_{ds}$ ,  $\phi_{cv}$ , the surcharge levels predicted to reach such critical state re reduced 47 kPa, 35 kPa and 23 kPa respectively. Consequently, the surcharge load levels required were underpredicted by 1.9, 2.6 and 3.9. Such underpredictions were also present for the rigid wall configuration but at a higher extent (2.65, 3.1, 4.09) which ascribes to the relatively higher facing column stiffness for rigid wall structure.

## 7.2. Study 2: Ehrlich and Mirmoradi (2013)'s Evaluation Facing Stiffness Effects in Conjunction with and Toe Resistance

A similar study was conducted by Ehrlich and Mirmoradi (2013), only this time the effect of toe resistance was taken into consideration in its impact on engaging or disengaging the facing stiffness contribution. A similar experiment of two prototypes: stiff concrete block facing (Wall 1) and flexible wrapped facing (Wall 2) were compared to monitor the tension along the reinforcements. The nature of the study was twofold, where firstly an experimental comparison was made between wall 1 and wall 2 with regards to the mobilised tension on the reinforcements and the lateral and vertical

displacements occurring. The other study was a parametric study involving a numerical model with various stiffness moduli to investigate the impact of a fixed and free toe condition.

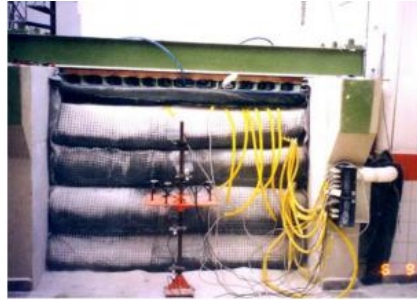


Figure 7.8. Wrapped facing experiment from Ehlich and Mirmoradi (2013).



Figure 7.9. Modular blocks experiment from Ehlich and Mirmoradi (2013).

### 7.2.1. Empirical Comparison of Stiff Concrete Block Facing (Wall 1) versus Flexible Wrapped Facing (Wall 2)

The purpose of the model scale experiments was to conduct a comparison of the amount of tensile forces and displacements from the surcharges imposed on both wall types. At the bottom of both walls, a 1 m wide zone was lubricated by sandwiching rubber sheets and silicon grease in order to allow movement for the potential failure surface. The surcharge additions imposed started from 10 kPa to 100 kPa.

Both walls had the same backfill and reinforcement installation assigned. The backfill was a well-graded sand ( $D_{50} = 0.25$   $C_c = 1$ ,  $C_u = 8.9$ ) with a compacted unit weight of 21 kN/m<sup>3</sup>. The geosynthetics implemented were spaced at 0.4 m. with the final reinforcement at 0.2 m, below the upper surface (as shown on the schematic

arrangement in Figure 7.10.). The reinforcement was of a longitudinal and transverse tensile strength of 80 kN/m and 30 kN/m.

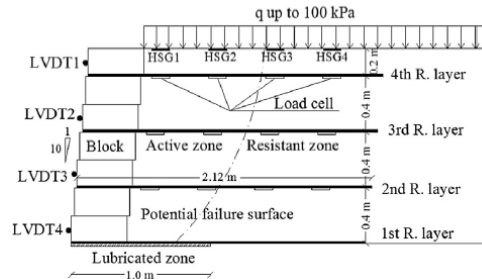


Figure 7.10. Testing configuration schematic for both wall types from Ehrlich and Mirmoradi (2013).

### 7.2.2. Tensile Force Comparison

The summation of the measured mobilised tensions along the second, third and fourth reinforcements show that for both walls, the value of  $\Sigma T_{\max}$  was at 0.6-0.7 m. from the back of the facing were of similar values were pertinent for both wall types. The reinforcement loads at the connections  $\Sigma T_o$ , were much lower than  $\Sigma T_{\max}$  for both walls at  $<30\% \Sigma T_{\max}$ .

Another analogy of ‘equivalent depth’ was conducted in which a soil equivalent depth layer  $Z_{eq}$  would corresponds to an external load applied at the wall top divided by the soil unit weight ( $Z_{eq} = q/\gamma$ ) added to the real depth of a specific reinforcement layer of  $T_{\max}$  showed an agreeable correlation between the two wall types.

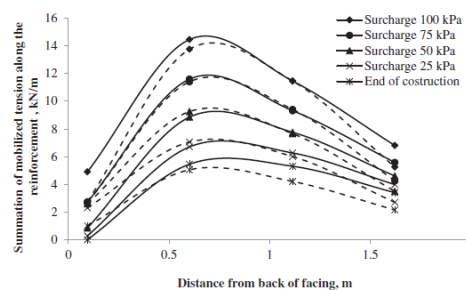


Figure 7.11. Mobilised friction versus distance from back facing with respect to surcharge from Ehrlich and Mirmoradi (2013).

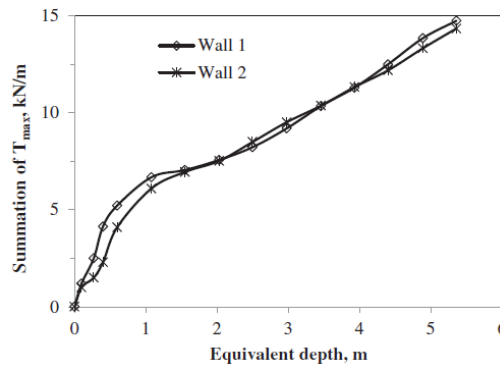


Figure 7.12. Summation of  $T_{max}$  with equivalent depth from Ehrlich and Mirmoradi (2013).

Ehrlich and Mirmoradi (2013) concluded that the facing stiffness does not play a significant contribution to the amount of tensile reinforcement mobilised when toe resistance is not present. Ehrlich and Mirmoradi (2013) corroborated this by presenting a force body diagram of the soil wedge weight ( $W_s$ ), facing weight ( $W_f$ ), a mobilised reaction of the stable soil mass at potential failure surface ( $F_1$ ) and mobilised force at facing base ( $F_2$ ).  $F_2$  is dependent on the increase of the shear stress mobilisation  $\phi_{bf}$  at the facing base, which in turn would reduce the maximum tension summation at the reinforcements  $\Sigma T_{max}$ . As the experiments had a lubricated base, the stiffness would not make a difference on the value of  $F_2$ .

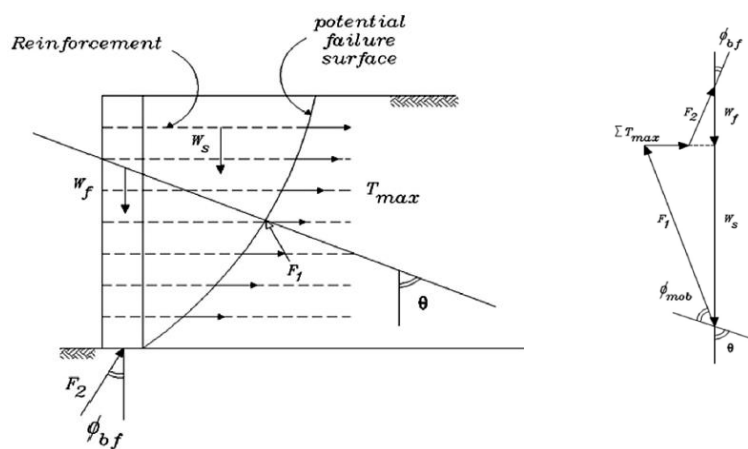


Figure 7.13. Force body diagram of GRS from Ehrlich and Mirmoradi (2013).

### 7.2.3. Displacement Comparison

The horizontal displacement generated was compared at the end of construction and at a surcharge application of 100 kPa. Wall 2 demonstrated a greater horizontal displacement amount compared to Wall 1. At an exemplar height of 0.75m, Wall 2 at the 100 kPa external surcharge application had a horizontal displacement of 60 mm whereas Wall 1 exhibited 18 mm displacement. The difference at the end of construction and with a 100 kPa external surcharge was more profound for Wall 2 compared to Wall 1.

With regards to vertical displacement, the vertical displacement decreased as the distance from the wall facing increased. Wall 2 as a flexibly faced wall had very large movements near the face. At a maximum surcharge of 100 kPa, Wall 1 had a vertical displacement of 2 mm and Wall 2 had a 17 mm displacement associated with it.

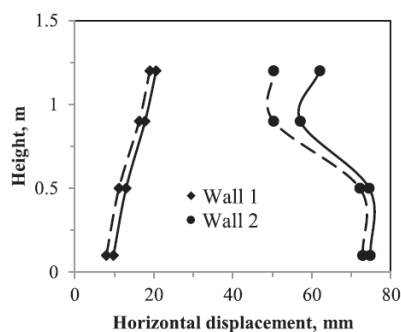


Figure 7.14. Horizontal displacement at end of Construction (Dashed) and with external surcharge Load (Solid) from Ehrlich and Mirmoradi (2013).

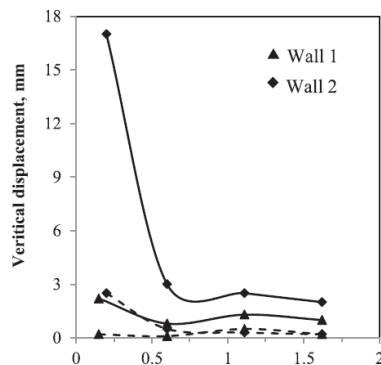


Figure 7.15. Vertical displacement impact with external surcharge load of 10 kPa (Dashed) and 100 kPa (Solid) from Ehrlich and Mirmoradi (2013).

#### 7.2.4. Numerical Study on the Wall Stiffness with Amount of Displacement Occurring

Ehrlich and Mirmoradi (2013) further conducted a numerical study, in which a numerical model was made (with parameters validated based on the experimental results of Wall 1 and 2) to investigate the effect of the combination of facing stiffness and toe resistance on the GRS wall behaviour. By adopting various stiffness moduli for the stiff wall along with the performed analysis of the wrapped facing wall, the summation of  $\Sigma T_{\max}$  produced and the mobilised friction load at the base collected for each stiffness modulus implemented.

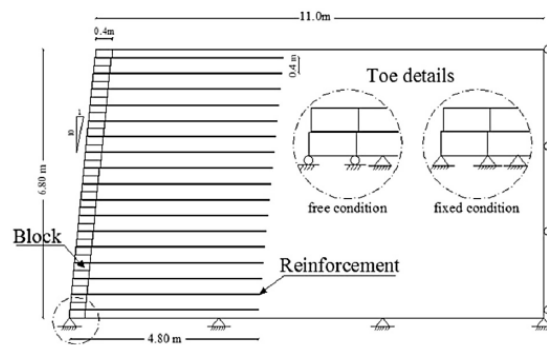


Figure 7.16. Numerical model for Ehrlich and Mirmoradi (2013)'s wall arrangement.

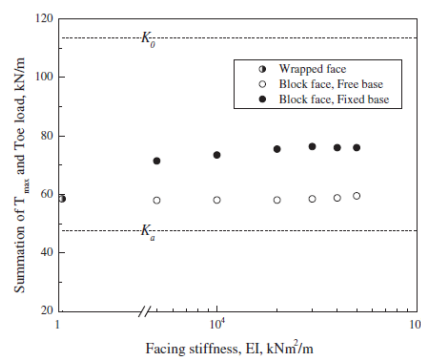


Figure 7.17. Summation of  $T_{\max}$  versus facing stiffness from Ehrlich and Mirmoradi (2013).

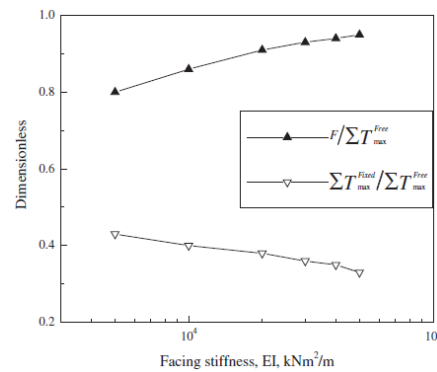


Figure 7.18. Impact of  $T_{\max}$  versus facing stiffness from Ehrlich and Mirmoradi (2013).

### 7.3. Study 3: Wu-Wall Facing and Earth Pressure

Wu *et al.* (2006) held a different account that the wall facing contributes to a structural component altogether, on the premise of the earth pressures induced on the wall facing. The revision on the AASHTO design guidelines asserts that the objective in designing a reinforced soil mass is so that it is sufficiently stable independently without any necessity of an external support. The reinforcement inclusions are strictly the sole mechanism to internally stabilise the retaining wall. Therefore, the facings are strictly to prevent sloughing of the vertical surface, as well as a construction aid and promoting a façade to the GRS wall, but not a load carrying element.

Based on experimental work conducted by the FHWA, Wu (2001) also points out that such strong wall performance when internally stabilised stems from the reinforcement spacing, where 4 inches of spacing would produce ‘a practically indestructible reinforced soil mass’.

The lateral earth pressure, when assigned as Rankine active earth pressure, is not in similitude with the reality for the segmental GRS walls. This is due to the Rankine earth pressure making an assumption that the soil mass is uniform and monolithic, whereas the geosynthetic inclusions in reality make it otherwise.

Wu (2001) and Wu and Payeur (2015) state that the nature of the internally stabilised GRS system would exhibit a nil lateral earth pressure at any depth encountering a reinforcement sheet, and that the earth pressure is strictly the sum of

earth pressures between the reinforcement sheets. Wu refers this to be the 'bin pressure', and is independent from the wall weight, but strongly impacted by the reinforcement spacing.

The pressure at zero at any reinforcement layer depth however can be refined on the premise that the reinforcement may be subjected to slight deformation and that the reinforcement and soil may not have perfect bonding, and so a fine-tuned bin pressure as follows is proposed:

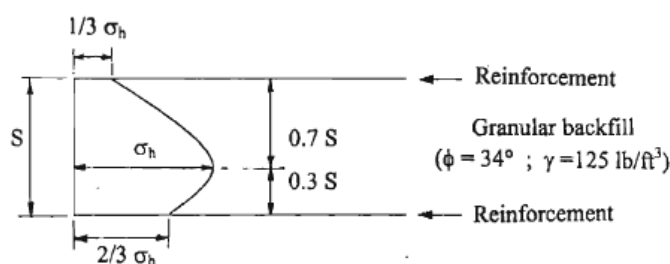


Figure 7.19. Bin Pressure concept proposed by Wu (2007).

An analogy was drawn in that the facing would not experience any lateral pressure providing that the reinforcement can effectively restrain the lateral deformation. The lateral earth pressure magnitude will in turn be dependent on the facing rigidity, where a greater rigidity will induce a larger lateral pressure. If the facing rigidity was to a point that would allow for an active condition to materialise, the resultant earth pressure would nevertheless be small for the closely spaced reinforcement. This precludes the requirement of using heavy concrete blocks altogether, as well as any connection configurations, as these were found redundant when small reinforcement spacing is allocated.

In another study, Wu and Chou (1993) from Wu (2007) attempt to assess the whether the assigned Rankine and Coulomb earth pressure theories employed on the segmental concrete block facing are realistic with the measured earth pressures. The assessment chose three planes to be put under scrutiny: a. wall face, b. back of reinforced zone, c. the plane of maximum tensile force in the reinforcement. The wall face lateral earth pressure was the smallest, and exhibited constant value at wall height ranges 1 to 3 m. The results shown on Figure 7.20 display that there was not much of deviation from the Rankine active earth pressure and that at the back of the reinforced

zone, which warrants the use of Rankine for the external stability of the GRS, but not for the facing internally for the connection strength purposes.

This had also corroborated with another study at the University of Strathclyde by Yogarajah *et al.*, (1996), where one wall was fully propped during construction and the other was propped only until the fill reached the top of panel.

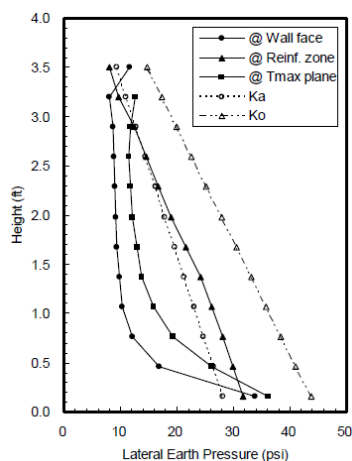


Figure 7.20. Results for lateral earth pressure results for the wall face, back of reinforced zone and plane of maximum tensile Force from Wu (2007).

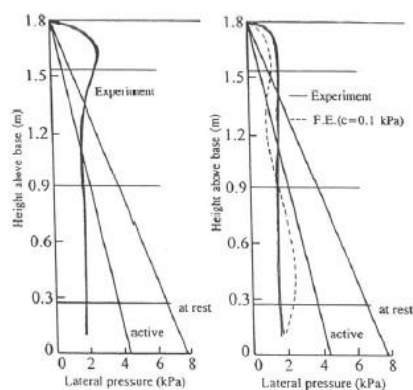


Figure 7.21. Results from Yogarajah and Saad (1996) displaying similar results in that the Earth Pressure is lower than Active state everywhere apart from the wall top from (Wu, 2007).

### 7.3.1. Connection Stability Analysis for Segmental Geosynthetic Reinforced Soil (GRS) Walls

In another study, Wu and Payeur (2015) went further to investigate if connection enhancement elements such as pins, lips or keys are needed for the facing stability of segmental reinforced walls with geosynthetic reinforcement were even necessary in par with the argument that an internally stabilising reinforcement wall does not require a structurally contributing wall facing.

In doing so, the investigation involved a parametric study of segmental retaining wall with purely frictional connections and employment of Rankine's Active earth pressure as per the FHWA recommendation and also providing a conservative approach to examine if the resisting forces are able to overcome the driving forces.

The generalised equations for the Connection forces are shown herein, in which enable the driving and resisting forces to be evaluated for any given geometric conditions and material properties.

The driving and resisting connection forces for the geosynthetic-block interface are:

$$R_i = 2B\gamma_b z_i (\tan \delta_{gb}) + 2nh\gamma_b \left[ \frac{(\tan \delta_{gb})(\tan^2 \delta_{gb})}{1 - (\tan \delta_{gb} \tan \delta_{gb})} \right] \quad (7.2)$$

$$D_i = 2nhK_a(\gamma_s z_i + q) - 2nh \left[ \frac{(B\gamma_b)(\tan \delta_{gb})}{1 - (\tan \delta_{gb} \tan \delta_{sb})} \right] \quad (7.3)$$

Whereas for the block-block interface are:

$$D_i = 2(n-1)hK_a(\gamma_s z_i + q) - 2(n-1)h \left[ \frac{(B\gamma_b)(\tan \delta_{bb})}{1 - (\tan \delta_{bb} \tan \delta_{sb})} \right] \quad (7.4)$$

$$R_i = 2B\gamma_b z_i (\tan \delta_{gb}) + 2(n-1)hB\gamma_b \left[ \frac{(\tan \delta_{sb})(\tan^2 \delta_{bb})}{1 - (\tan \delta_{bb} \tan \delta_{sb})} \right] \quad (7.5)$$

### 7.3.2. Parametric Study and Discussions

A baseline model was first undertaken with the aim to assess the driving and resisting forces at the geosynthetic-block and block-block interfaces along the depth. The results and the model parameters are highlighted below, with values in parentheses used for the baseline model:

Table 7.2. Parameter Values Adopted for Wu and Payeur (2015) Baseline Model

| Parameter                                                         | Value           |
|-------------------------------------------------------------------|-----------------|
| Wall Height (m)                                                   | (6)             |
| Facing Block Height, $h$ (m)                                      | (0.2)           |
| Number of Facing Blocks Between Adjacent Geosynthetic Sheets, $n$ | (1), 2, 3       |
| Soil Unit Weight, $\gamma_s$ (kN/m <sup>3</sup> )                 | (19)            |
| Facing Block Unit Weight, $\gamma_b$ (kN/m <sup>3</sup> )         | (19), 29, 39    |
| Equivalent Depth of Facing Block, $B$ (m)                         | (0.2), 0.3, 0.4 |
| Soil Friction Angle, $\phi$ (°)                                   | 34, 37, (40)    |
| Soil-Block Friction Angle $\delta_{sb} = 2/3 \phi$ (°)            | 23, 25, (27)    |
| Block-Block Friction Angle $\delta_{bb}$ (°)                      | 30, (35), 40    |
| Geosynthetic-Block Friction Angle $\delta_{gb}$ (°)               | 25, (30), 35    |

A positive net connection force implies that the resisting connection forces are greater than the driving, and the opposite would entail a negative net connection force (i.e. connection failure).

In both connection forces, the difference between the resisting and driving forces would increase with depth, though such difference was found to be smaller for the geosynthetic-block interface.

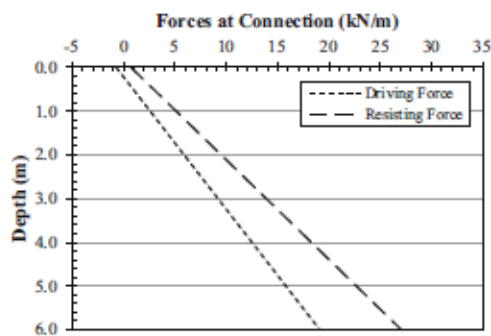


Figure 7.22. Driving and resisting connection forces for Baseline case at Geosynthetic-Block (left) and Block-Block (right) from Wu and Payeur (2015).

Expanding from the baseline case, Wu and Payeur (2015) conducted an assessment on the net connection force under impact of a typical traffic surcharge of 10 kPa with  $n=1, 2$  and 3 blocks between the successive reinforcement layers. In the case for  $n=2$  and 3, with  $\delta_{bb}$  and  $\delta_{gb}$  respectively at  $35^\circ$  and  $30^\circ$  the net connection forces for the geosynthetic-block interfaces were found to be significantly smaller than that of a block-block interface, from which one can deduce that the margin of safety at connection is higher than at the block-block interface. An elusive inference that can be made is in comparing the net connection forces for  $n=1$  and that for  $n=3$ , were both respectively have a spacing of 0.2 metres and 0.6 metres. The net connection forces for the larger spacing falls in the dominion of an overall negative connection force especially at greater depths, thereby indicating the significance of the spacing on the net forces. Such results for facing connection stability go in favour with the FHWA recommendations for a spacing to remain as close as  $<0.3$  metres which will give an increased stiffness and strength of the reinforced soil mass.

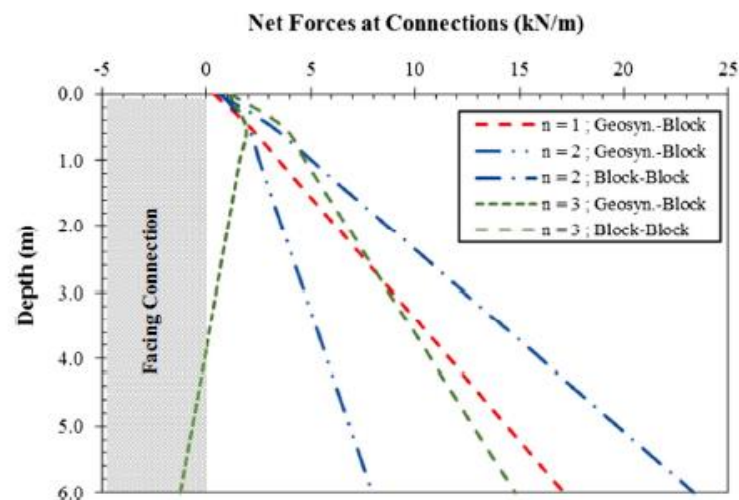


Figure 7.24. Net connection forces at Geosynthetic-Block and Block-Block Interface with 1, 2 and 3 blocks between successive reinforcement layers Under 10 kPa surcharge from Wu and Payeur (2015).

The effect of block-block interface friction angle on the net connection forces for  $n=2$  and  $n=3$  are also investigated for different  $\delta_{bb}$  value of  $30^\circ$ ,  $35^\circ$  and  $40^\circ$  while  $\delta_{gb}$  is kept constant at  $30^\circ$  (shown on figure 7.25). Overall, the net connection force in  $n=2$  is found to be larger than that of  $n=3$ , and there is always an increase in net connection force with an increase the magnitude of  $\delta_{bb}$ . The slope variation in  $n=3$  was

more noticeable at depths 0 to 1 m. from the top of the wall, which is a consequence of the driving forces within these depths pertaining to be a negative value.

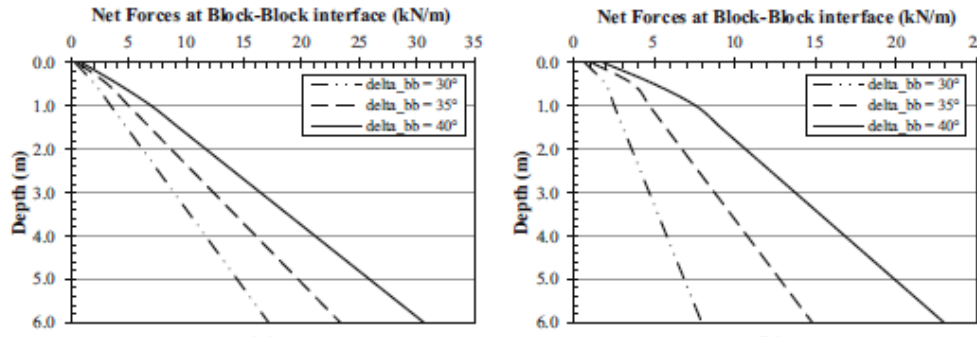


Figure 7.25. Net Block-Block Connection Forces versus depth for  $n=2$  blocks (left) and  $n=3$  blocks (right) for various Block-Block friction angles from Wu and Payeur (2015).

A similar study only this time the geosynthetic-block interface friction angle  $\delta_{gb}$  was assessed for  $n=2$  and  $n=3$  while  $\delta_{bb}$  was kept constant at  $35^\circ$  with  $\delta_{gb}$  at various values of  $25^\circ$ ,  $30^\circ$  and  $35^\circ$ . Although the net connection forces were also larger for  $n=2$  similarly to the increase in  $\delta_{bb}$ , but there was a distinct behaviour pertinent to  $n=3$ , especially for  $\delta_{gb}$  at  $25^\circ$  and  $30^\circ$ . Negative net connection forces (indicating failure) was observed for nearly the entire wall for  $\delta_{gb}$  equal to  $25^\circ$  and for  $\delta_{gb}$  equal to  $30^\circ$ , it was negative for the bottom third of the wall (so such failure phenomenon would not exist if the wall height was lower than 4m.). The reason for the negative net connection force was due to the rate of driving force increase with depth being greater than the resisting force rate of increase. Overall, a conclusion can be drawn that the net connection force is found to be sensitive to  $\delta_{gb}$  especially when it is less than  $35^\circ$ .

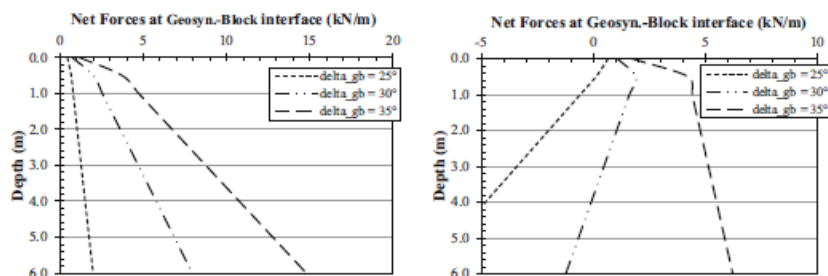


Figure 7.26. Net Geosynthetic-Block connection forces versus depth for  $n=2$  blocks (left) and  $n=3$  blocks (right) for various Geosynthetic-Block friction angles from Wu and Payeur (2015).

Another parametric assessment was the facing block inherent properties, particularly the bulk unit weight  $\gamma_b$  (magnitudes 19, 29, 39 kN/m<sup>3</sup> and the facing block equivalent depth, B (0.2, 0.3 and 0.4 m.). In both cases, the net connection forces corresponded to an increase in depth with the constituent magnitudes assigned.

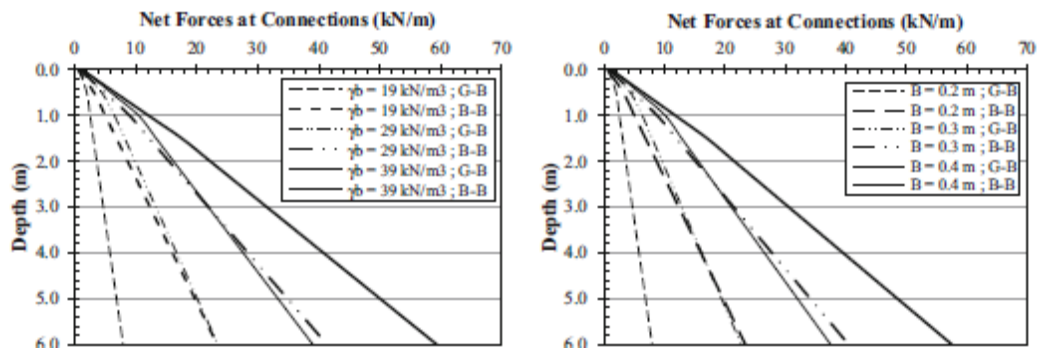


Figure 7.27. Net connection forces versus depth affected by Bulk Unit Weight of Facing Block (Left) and Equivalent Facing Block Depth, B (Right from Wu and Payeur (2015)).

A final parametric study was to investigate the behaviour of the change in friction angle ( $34^\circ$ ,  $37^\circ$ ,  $40^\circ$ ) with the corresponding soil-block interface friction angle  $\delta_{sb} = 2/3 \phi$ . The lower bound friction angle ( $34^\circ$  for reinforced soil walls gave a rather low net connection forces of an order of 1 kN/m and the higher bound  $40^\circ$ ) gave much higher connection forces. This accentuates the benefits of good compaction for connection by the higher friction angles present.

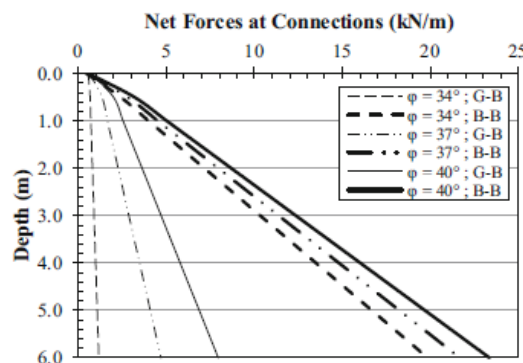


Figure 7.28. Net connection forces for Geosynthetic-Block and Block-Block interfaces affected by various Soil Friction Angles from Wu and Payeur (2015).

In conclusion, Wu and Payeur (2015)'s study had shown was that employing a conservative approach for the driving mechanism in adopting Rankine's Active earth

failure wedge had displayed an overall positive net connection forces, which were derived entirely from friction between the contact blocks.

Wu and Payeur (2015) also promote the use of close reinforcement spacing in the GRS wall systems bring about a significant improvement on the stiffness and strength of the reinforced soil mass.

At typical traffic loads, the net connection forces were positive for segmental reinforced walls with lightweight segments without any connection enhancements (Jonathan T Wu and Payeur, 2015). Contrary to what builders employ of using pins, blocks with lips or keys or heavyweight blocks, Wu and Payeur (2015) point out that such measures exacerbate the loading to the connections and consequently cause the earth thrust to exert on the wall face to the extent in being greater than the Rankine thrust.

#### **7.4. Conclusions**

- A comparison between a wrapped-face wall and a rigid wall at the end of construction demonstrated that the earlier exhibited more strains than the rigid wall facing, where in which the most strains were being experienced near the wall facing as a result of the downward movement of the soil behind the facing. A greater surcharge load of 90 kPa was needed for the stiff wall facing in order for the peak reinforcement strain in one reinforcement to materialise whereas the later required a lower surcharge load of 80 kPa.
- In a separate study, the benefits of a stiff wall facing compared to a rigid wall facing was more profound when the GRS wall possessed toe resistance, as the toe resistance is able to engage and disengage the facing stiffness contribution.
- Another study was challenging the conventional methods in international standards in that the use of the Coefficient of Active earth pressure  $K_a$  was more conservative and in similitude with the reality of the segmental GRS walls, and it was more suitable to use a lower earth pressure coefficient based on the Bin Pressure concept. The case put forward in the study was that the reinforcements are strictly the sole

mechanism to internally stabilise the retaining wall. The facings are only to prevent sloughing of the vertical surface as well as construction aid and a façade for the GRS wall, but not a load carrying element.

- A suggestion for the reinforcement spacing of 4 inches produces a ‘practically indestructible reinforced soil mass’, where it exhibits a significant improvement on the stiffness and strength of the reinforced soil mass.
- An analogy was drawn in that the facing would not experience any lateral pressure providing that the reinforcement can effectively restrain the lateral deformation. The lateral earth pressure magnitude will in turn be dependent on the facing rigidity, where a greater rigidity will induce a larger lateral pressure. If the facing rigidity was to a point that would allow for an active condition to materialise, the resultant earth pressure would nevertheless be small for the closely spaced reinforcement. This precludes the requirement of using heavy concrete blocks altogether, as well as any connection configurations, as these were found redundant when small reinforcement spacing is allocated.
- A separate study was carried out for connection strength. The net connection forces for block-block connections were greater than that for geosynthetic-block connections.
- Wall heights greater than 4 metres experience negative connection forces when the spacing connection is greater than 0.2 metres.
- The critical friction angles for the geosynthetic-block connections were in the range of 25-30 degrees.
- Contrary to what builders employ of using pins, blocks with lips or keys or heavyweight blocks, such measures exacerbate the loading to the connections and consequently cause the earth thrust to exert on the wall face to the extent in being greater than the Rankine thrust

## **8. THREE-DIMENSIONAL APPROACH TO ADDRESS GRS SYSTEM USING LIMIT EQUILIBRIUM METHOD**

As mentioned in Chapter 1, the stability of a GRS wall with a corner turn is essentially a 3D problem as oppose to the international standards treating a GRS design as 2D plane-strain problem. Two studies by Zhang *et al.* (2014) and Leshchinsky and Baker (1986) particularly targeted reinforced slopes in that the 3D stabilising characteristics (primarily the 3D end-effects) were overlooked when approaching the design in a 2D manner. Although it may give conservative values for reinforcement strengths required, the contribution of reinforcement stability can be overestimated which can lead to under-conservative consequences.

### **8.1. Postulation of 3D Mechanism for GRS Structure**

Zhang *et al.* (2014) highlighted the deviations from reality by idealising a 2D approach, for example, a long, straight GRE structure may have alternating toe elevations as a consequence of the naturally varying ground surface topography, thus causing the structure to have varying height. Another instance is in critical structures such as bridge abutments, in which although may be straight with uniform height, but may not warrant being of a plane strain nature by virtue of the limited length to its depth.

A 3D failure mechanism has been derived to account for the 3D effects on the required reinforcement strength in a reinforced earth structure. The mechanism however does not adopt the conventional methods of applying Rankine's Earth pressure to derive the lateral earth pressures at each elevation, in which a maximum tensile force is sought after based on the tributary area covered between the reinforcements. Instead, the mechanism adopts a Limit Equilibrium approach (where a state of limited equilibrium is defined as the mobilised shear stress being a fraction of the shear strength along a sliding surface) (Aryal, 2006). In doing so the resistive forces of the reinforcements are incorporated in the limit equilibrium equations (Zhang *et al.*, 2014)

The 3D failure mechanism proposed by Zhang *et al.* (2014) was nevertheless developed from a 2D Limit Equilibrium (LE) approach by Leshchinsky *et al.* (2010) in

which global reinforcement strength,  $R_t$  is calculated to ensure sufficient internal stability of the reinforced soil mass. A log spiral failure mechanism is postulated for this purpose, and the  $R_t$  is divided amongst the individual layers using three different assumed distribution functions (D- $T_{\max}$ ) - uniform, linear and multilinear. The factor would convert the global reinforcement strength into local strengths ( $T_{\max}$ ). In doing so, the locus of the maximum tension experienced on the reinforcement,  $T_{\max}$ , is assumed to intercept the critical log spiral trace. The impact of the distribution functions on a vertical slope is found to be more profound in reflecting the maximum unfactored load in individual layers, which is what is used to select geosynthetics.

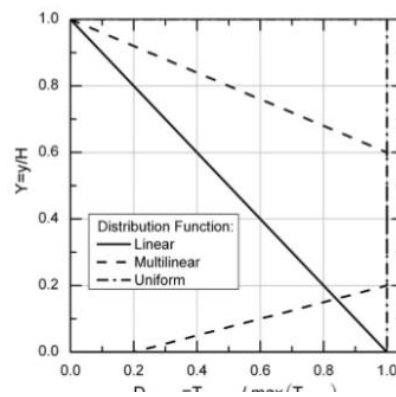


Figure 8.1. Distribution functions to Translate  $D_{T-\text{Max}}$  to  $T_{\text{Max-I}}$  including max ( $T_{\text{max}}$ ) from (Leshchinsky *et al.*, 2010).

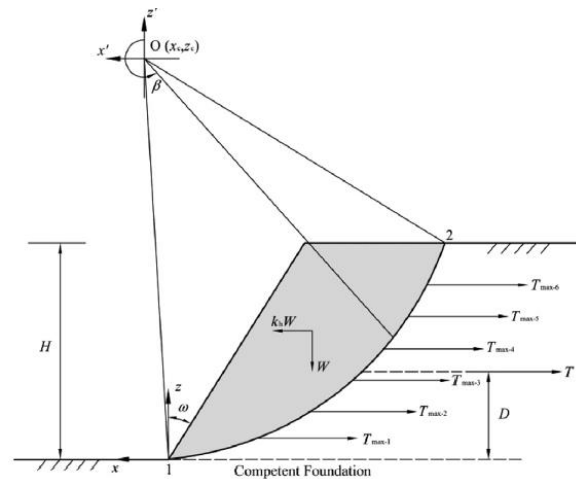


Figure 8.2. 2D Log spiral failure mechanism with notation and convention from (Zhang *et al.*, 2014).

The log-spiral slip surface used is defined by the radius vector:

$$R = A \cdot \exp(-\Psi_d \beta) \quad (8.1)$$

$A = \log \text{ spiral coefficient}$

$$\Psi_d = \tan \phi_d \quad (8.2)$$

$$\phi_d = \text{Design internal friction angle} = \tan^{-1}(\tan \phi / \text{FOS})$$

$\beta$  is the angle in polar coordinates defined relative to the Cartesian coordinates  $(x_c, y_c)$  from the toe at  $(0,0)$ .

The philosophy behind the slip surface is to obtain a test body that requires the reactive forces from the reinforcements to be at a maximum amount in order to satisfy the LE state. Consequently, a critical slip surface is generated which defines the location of  $T_{\max-l}$  for each  $n$  intersecting reinforcement.

To expand into the 3D dominion, Leshchinsky developed a differential equation for 3D slip surface as follows:

$$\frac{d\rho}{d\beta} = -\psi_d \sin \sqrt{\rho^2 + \left(\frac{d\rho}{d\alpha}\right)^2} \quad (8.3)$$

Where  $\rho = \rho(\alpha, \beta)$  depicts a slip surface  $z(x,y)$  in polar coordinates  $(\rho, \alpha, \beta)$ . The centre of this spherical system is located at the unknown point  $(x_c, y_c, z_c)$ . Figure 8.3a below illustrates the spherical coordinates relative to the unknown Cartesian coordinate system and Figure 8.3b displays the 3D mechanism in relation to the coordinate system.

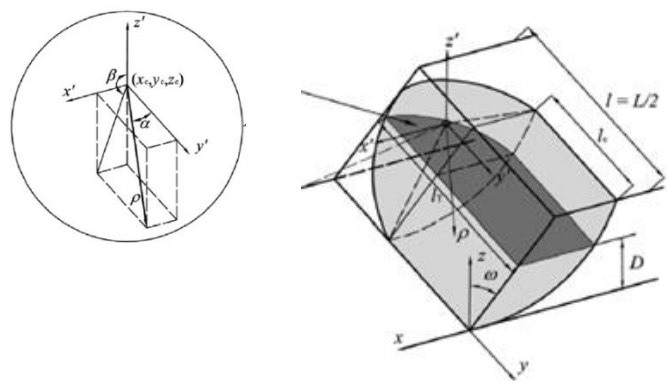


Figure 8.3. Spherical coordinates and conventions of 3 D Slip Surface from (Zhang *et al.*, 2014).

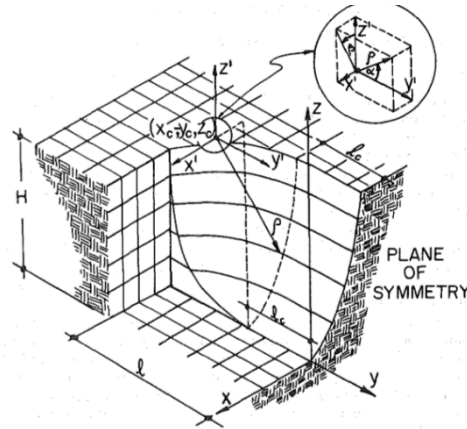


Figure 8.4. Isometric view of 3D mechanism conventions from (Leshchinsky and Baker, 1986).

Solving the differential equation produces two fundamental solutions for the potential slip surface geometry:

$$\rho = A \exp(-\psi_a \beta) \sin \alpha \text{ for } |y| \geq l_c \quad (8.4)$$

$$\rho = A \exp(-\psi_a \beta) / \sin \alpha \text{ for } |y| \leq l_c \quad (8.5)$$

Where  $A$  is an unknown constant and  $l_c$  is half the length of a central cylinder shown in Figure 8.3a and b.

The solutions of these equations are illustrated in figure 8.4a (for equation 1) and Figure 8.4b (for equation 2) below. Equation 1 delineates the end caps attached smoothly to the central cylinder, and equation 2 is the central cylinder ( $2l_c$ ) with the associated log-spiral geometry:

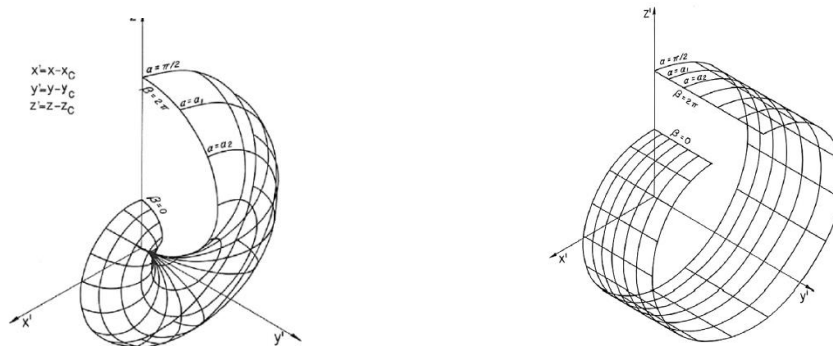


Figure 8.5. (a) Left: Graphical solution for  $y > |l_c|$  for the End Caps and (b) Right:  $y < |l_c|$  for the Central Cylinder from (Zhang *et al.*, 2014).

In Cartesian system terms, the coordinates are represented as:

$$x = x_c + A \exp(-\psi_d \beta) \sin^2 \alpha \sin \beta \quad (8.6)$$

$$y = y_c + A \exp(-\psi_d \beta) \sin \alpha \cos \alpha$$

$$z = z_c + A \exp(-\psi_d \beta) \sin^2 \alpha \cos \beta$$

$$x = x_c + A \exp(-\psi_d \beta) \sin \beta \quad (8.7)$$

$$y = y_c + A \exp(-\psi_d \beta) \cos \alpha / \cos \alpha$$

$$z = z_c + A \exp(-\psi_d \beta) \cos \beta$$

For carrying out a moment equilibrium calculation, the potential sliding mass shown on figure 150a and b showing one-half of the potential sliding mass is only considered by virtue of the symmetrical nature. By solving the equilibrium equations, the amount T can be calculated.

The master equation for Moment equilibrium is given as:

$$M = \left( \iint_{D_1} \gamma M_1 dx dy + \iint_{D_2} \gamma M_2 dx dy \right) - T l_T (z_c - D) = 0 \quad (8.8)$$

Where  $M_1$  and  $M_{1s}$  correspond to the end caps and  $M_2$  and  $M_{2s}$  are the central cylindrical portions.

$$M_1 = [A \exp(-\psi_d \beta) \sin^2 \alpha \cos \beta - (Z - z_c)] A \exp(-\psi_d \beta) \sin^2 \alpha \sin \beta \quad (8.9)$$

$$M_{1s} = [A \exp(-\psi_d \beta) \sin^2 \alpha \cos \beta - (Z - z_c)] x$$

$$[A \exp(-\psi_d \beta) \sin^2 \alpha \sin \beta + (Z - z_c)] \quad (8.10)$$

$$M_2 = [A \exp(-\psi_d \beta) \cos \beta - (Z - z_c)] A \exp(-\psi_d \beta) \sin \beta \quad (8.11)$$

$$M_{2s} = \frac{1}{2} [A \exp(-\psi_d \beta) \cos \beta - (Z - z_c)] x [A \exp(-\psi_d \beta) \cos \beta + (Z - z_c)] \quad (8.12)$$

Where  $Z = Z(x, y)$  being the slope surface geometry.

The acting length of resultant tensile force is expressed as:

$$l_T = -A \exp(-\psi_d \beta_T) \sin \alpha_T \cos \alpha_T - y_c \quad (8.13)$$

$$\tan \beta_T = \frac{x_c + D \tan \omega}{z_c - D} \quad (8.14)$$

$$\sin \alpha_T = \sqrt{\frac{D - z_c}{A \exp(-\psi_d \beta_T) \cos \beta_T}} \quad (8.15)$$

The unfactored strength then becomes:

$$K = \frac{T}{\frac{1}{2} \gamma H^2} = \frac{\frac{2}{H^2} [\iint_{D1} M_{12} dx dy + \iint_{D2} M_{22} dx dy]}{l_T (z_c - D)} \quad (8.16)$$

The reinforced structure in its geometric properties of its batter  $\omega$ , length-to-height ratio  $L/H$ , depth-to-height ratio  $D/H$  and design friction angle  $\phi_d$  and potential failure surfaces iteratively depicted by the assumed parameters  $A$ ,  $x_c$ ,  $y_c$  and  $z_c$  can be used to devise the required unfactored strength,  $\max(T)$  from the moment equilibrium equation. The corresponding critical value of  $K$  (i.e.  $\max(K)$ ) with its be obtained via an optimisation procedure to identify the magnitudes of the unknown parameters defining the slip surface.

$$K = \max f(A, x_c, y_c, z_c | w, \frac{L}{H}, \frac{D}{H}, \phi_d) \quad (8.17)$$

Optimising for  $K$  for limited condition for a given reinforced structure (batter  $w$ ,  $L:H$ ,  $D:H$ , design friction angle  $\phi_d$ , potential failure surfaces defined by iteratively assumed parameters  $A$ ,  $x_c$ ,  $y_c$ , and  $z_c$ .

It is important to emphasise on the length  $L$ , in that it is the feasible length of failure dictated by the physical constraints. When carrying out the analysis maximum  $K$  or  $T$ , the 3-D failure surface is then defined by  $L$  during the optimisation process (i.e. the internal stability of the GRS body).

## 8.2. Inferences from Zhang *et al.* (2014)'s Results

Zhang *et al.* (2014) collated instructive charts to highlight the impact of 3D analysis. Two distribution factors were chosen  $D/H=1/2$  and  $1/3$  for uniform and linear force distribution function. Furthermore two characteristic values for design friction

angles  $\phi_d$  of  $34^\circ$  (AASHTO's default value for MSE walls) and  $40^\circ$  (AASHTO's maximum value allowed in design).

The ordinate  $K_R = K_{3D}/K_{2D}$  is plotted in figure 8.5 for both cases versus the normalised feasible failure length,  $L/H$ . The  $K_{3D}$  coefficient represents the 3D problem and  $K_{2D}$  is the 2D plane-strain problem. The  $K_R$  ratio denotes the 3D end effects on the required geosynthetic strength. Concurrently with  $K_R$ , the normalised volume potential sliding mass  $V=v/H^3$  is also graphed as a function of  $L/H$ .

In both cases, the  $L/H$  increase (which highlights the feasible length of failure increasing) causes the  $K_R$  ratio to increase an asymptotic unity, which implies that the 3D end effects reduce for larger values of  $L/H$ . At lower  $L/H$  values however, the 3D effects increase stability which entails less tensile strength needed to resist failure.

The difference in required strength between 3D and 2D is less than 10% when an  $L/H$  ratio value greater than 5, which allows the 2D plane strain solution to be applicable. Conversely, at narrow lengths  $L/H=1.0$ , the difference would exceed 50%, which ends up causing the 2D solutions giving overly-conservative estimates for the required reinforcement strength. The two charts of different  $\phi_d$  values demonstrated that  $\phi_d$  is insignificant with regards to  $K_R$ , but does impact the normalised volume potential sliding mass.

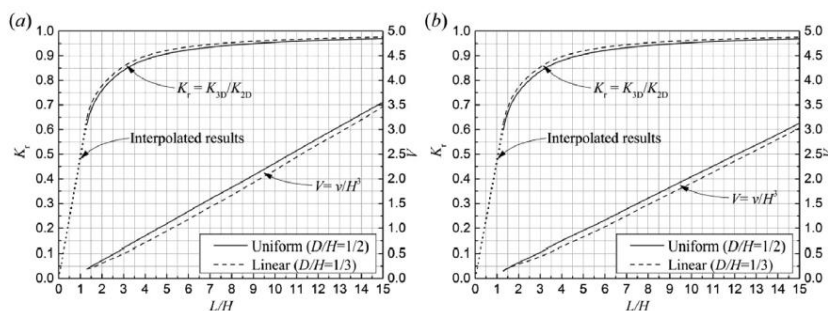


Figure 8.5. Design charts for  $K_R$  for Uniform and Linear Distribution from Leshchinsky *et al.* (2014).

In another collated chart, the distribution function for the reinforcements between 2D plane strain and 3D analysis was also compared. A vertical slope constrained to a narrow length of  $L/H=2$  has a curved slip surface as oppose to greater lengths of  $L/H=5$  and  $10$  where the slip surface degenerates to be planar. Such

difference in the slip surface curvature is more attributable in the uniform force distribution than that for linear force distribution

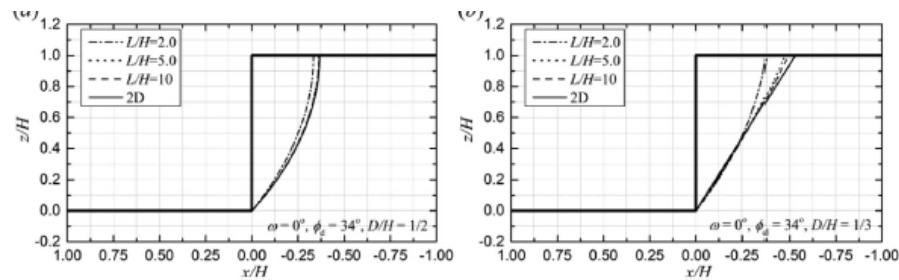


Figure 8.6. Critical slip surfaces for Uniform (Left) and Linear Distribution (Right) from Leshchinsky *et al.* (2014).

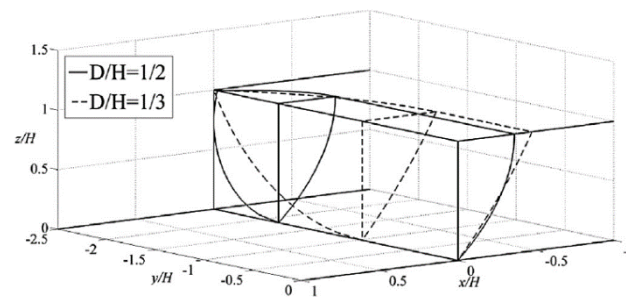


Figure 8.7. 3D view of the slip surfaces for a vertical Slope for Uniform (D/H=1/2) and Linear (D/H=1/3) from Leshchinsky *et al.* (2014).

### 8.3. Hamderi and Guler's (2015) Numerical Study on GRS Segmental Walls with Corner Turns

The discrepancies of cracking and separation of the segmental blocks of GRS walls with corner turns (shown in Figures 8 in Chapter 1) had geared Hamderi and Guler (2015) to conduct a 3D finite element model of the GRS walls to explore the driving mechanisms that augmented such discrepancies and what remedies can be used to mitigate them. The 3D modular block retaining wall of dimensions 3 m. x 3 m. x 3 m., consisted of staggered modular blocks with geosynthetic reinforcement spaced at 0.02 metres.

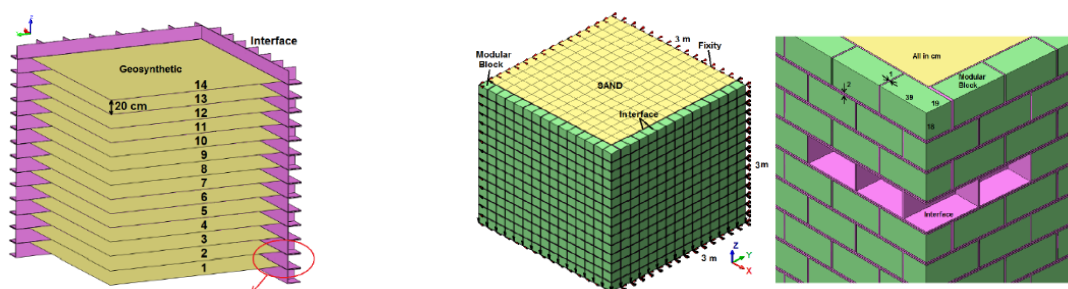


Figure 8.8. Modular block of GRS segmental wall from Hamderi and Guler (2015).

The material properties of all the elements are as follows:

Table 8.1. Hamderi and Guler (2015)'s Parameter Values Tested.

| Material                   | Constitutive Model | Modulus of Elasticity MPa | Poisson's Ratio | Internal Friction Angle | Cohesion (kPa) | Unit Weight kN/m <sup>3</sup> |
|----------------------------|--------------------|---------------------------|-----------------|-------------------------|----------------|-------------------------------|
| Granular Material          | Mohr-Coulomb       | 20<br>40<br>60<br>80      | 0.3             | 36                      | 5              | 18                            |
| Block                      | Linear Elastic     | 200                       | 0.15            | -                       | -              | 20                            |
| Interface                  | Mohr               | 1                         | 0.25            | 30                      | 5              | 20                            |
| Geosynthetic Reinforcement | Linear Elastic     | 100<br>250<br>500<br>1000 | 0.15            | -                       | -              | 1.2                           |

Two parametric studies were conducted in the study: the first was the change in modulus of Elasticity of the soil (which can vary in practice by virtue of compaction effects) and second on the geosynthetic strength used for the reinforcements.

### 8.3.1. Impact of Varying Soil Elastic Modulus

Hamderi and Guler (2015) ran the numerical analysis using solely the self-weight of the structure in a series of steps, without any external loading. In each analysis, with a fixed geosynthetic reinforcement strength of 100 kN/m., only the Elastic modulus of the soil was modified for series values 20, 40, 60 and 80 MPa. The greater the soil elastic modulus, the less prone the GRS wall was to excessive deformations. In ascending order of soil elastic modulus, the displacements manifested were 7.2, 4.05, 2.25 and 1.35 mm. These values are plotted in figure 8.9 below.

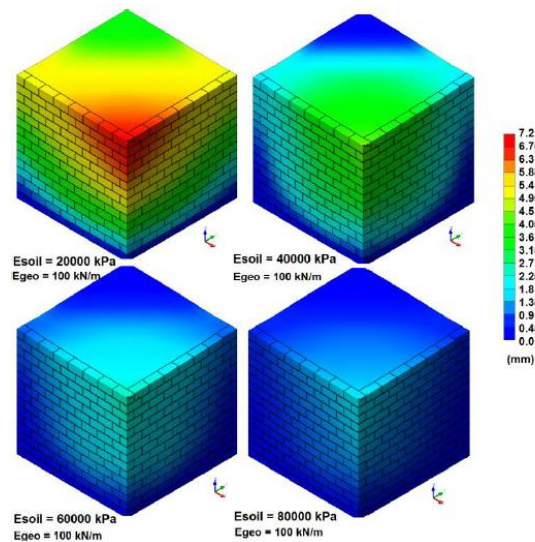


Figure 8.9. Varying displacements with increase in Soil Modulus of Elasticity from Hamderi and Guler (2015).

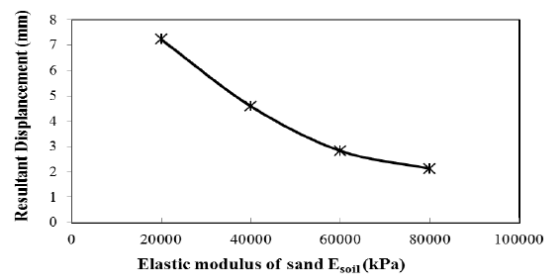


Figure 8.10. Plot of decrease of resultant displacement with increase in sand Elastic Modulus from Hamderi and Guler (2015).

The wall stress distributions for various soil moduli were also presented, where both sides of the walls experienced tension, and compression was more profound in the

lower corner. Such compressive stresses diminished with increasing Soil Moduli. An interesting inference deduced from the model is the location where compression builds up strongly correlates that of the cracking discrepancy in the GRS wall.

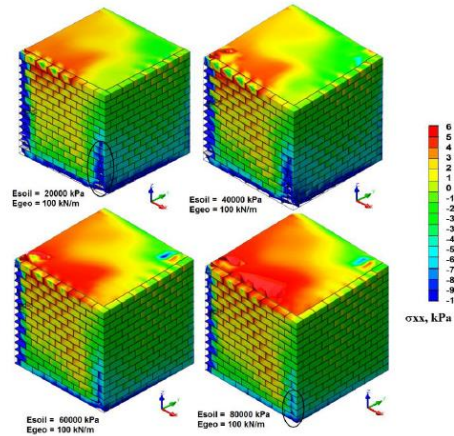


Figure 8.11. Stress distribution of corner wall with increase in Elastic Modulus from (Hamderi and Guler, 2015).

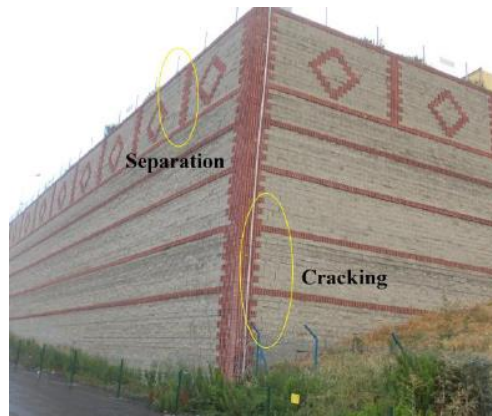


Figure 8.12. Discrepancy location on GRS Wall from (Hamderi and Guler, (2015).

### 8.3.2. Impact of Reinforcement Strength

Another analysis was conducted having the granular fill compacted moderately at an Elastic Modulus of 40 MPa, with various geosynthetic reinforcement stiffness magnitudes of 100, 250, 500, 1000 and 2000 kPa. The greatest wall displacement of 4.6 mm was experienced for the reinforcement with stiffness magnitude of 100 kN/m, and

the minimum value of 0.8 mm for the highest reinforcement stiffness of 2000 kPa. By observing where the two walls intersect, it can be deduced that the reinforcement stiffness plays a vital role on the integrity of the wall, which further signifies Zhang *et al.* (2014)'s work on when it is important to consider the 3D mechanism based on the wall dimensions (as the L/D in this case is 1. Figure 8.13 below plots the variation in resultant displacement with the geogrid Elastic Modulus.

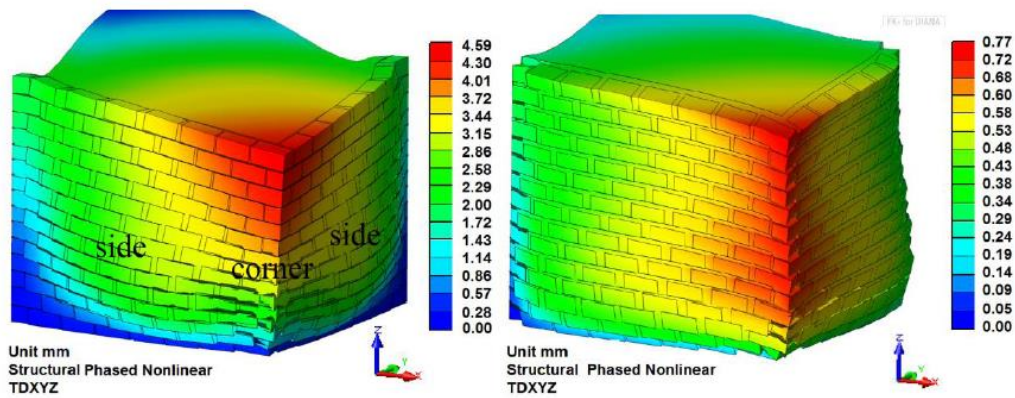


Figure 8.13. Significant deformation behaviour between reinforcements from Hamderi and Guler (2015).

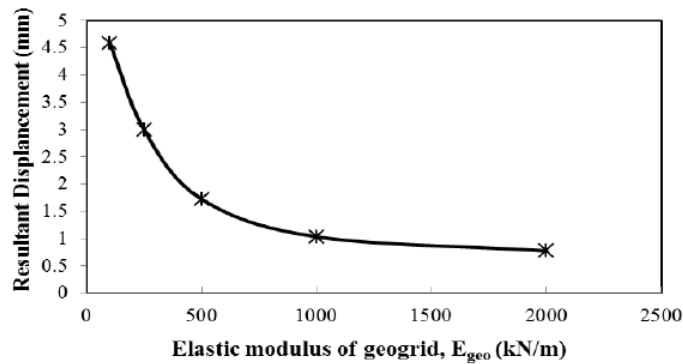


Figure 8.14. Plot of variation of resultant displacement with change in Geogrid Strength from Hamderi and Guler (2015).

### 8.3.3. Determining Location for Cracks and Block Separation

By observing the stress distribution variation  $\sigma_{xx}$  of the blocks in the x-direction (shown in Figure 8.15), two blocks were dissected to further evaluate the stress distribution on a typical tension and compression block. The tension block had a unique positive magnitude of stress whereas the compression block is under compression on

the outer surface and tension in the inner surface as shown in Figure 8.16, which consequently leads to cracking.

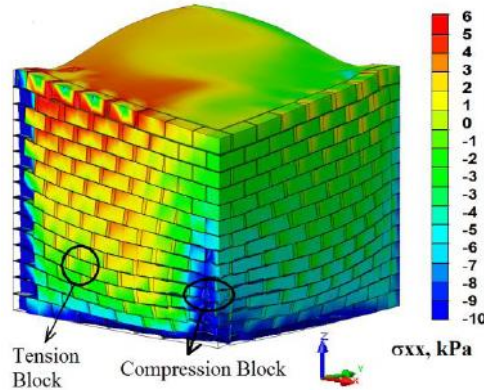


Figure 8.15. Stress distribution along X Axis for segmental wall from Hamderi and Guler (2015).

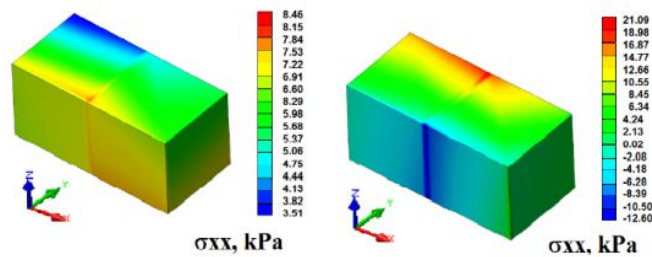


Figure 8.16. Isolated blocks experiencing Tension (Left) and both Compression and Tension (Right) from Hamderi and Guler (2015).

By observing the wall displacements along the x-direction for a case with a stronger reinforcement of 2000 kN/m and 100 kN/m, the earlier had experienced a more ostentatious degree of uniformity in the displacement distribution, whereas the latter had a more distorted displacement distribution which can manifest to local block separation.

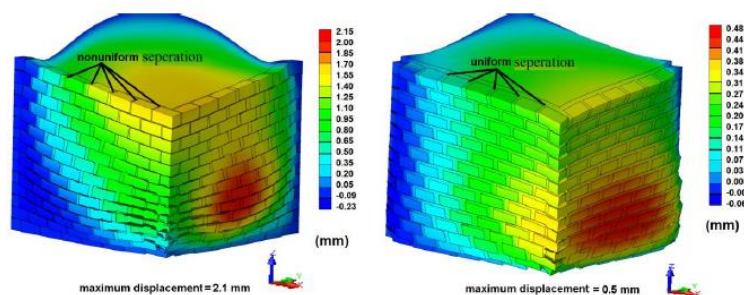


Figure 8.17. Nonuniform separation (Left) with 100 kN/m Reinforcement and Uniform Separation (Right) for 2000 kN/m Reinforcement from Hamderi and Guler (2015).

### 8.3.4. Corner Walls in Design Standards

In line with Appendix A on the critique on how the international standards approach the design of the GRS walls, special attention will be given to any aspect related to Corner Walls.

Both Geoguide 6 (Government of the Hong Kong Special Administrative Region, 2002) and Federal Highway Administration (2009) provided provisions for wall corners. The wall corner has a tendency to open up as a result of the wall segments orthogonal to one another moving laterally from the earth pressure (shown in Figure 8.18). To mitigate such phenomenon, corner elements must be added to resist any form of differential movements. In doing so, the reinforced soil will be prevented from moving through the crack (Federal Highway Administration, 2009).

The NCMA (National Concrete Masonry Association, 2010) provides special details for the placement of geosynthetic reinforcements in convex corners, in that 76 mm of soil is required to be between any overlapping arranged for the corners. Alternatively, reinforcement placement should be put perpendicularly to the principle direction in the cross over area (shown in Figure 8.19).

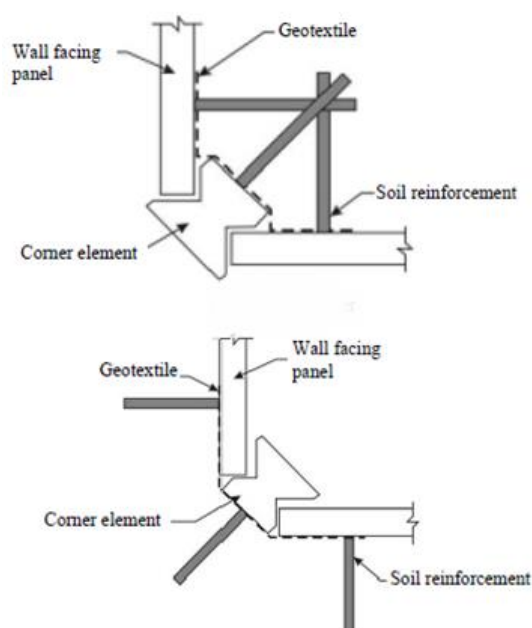


Figure 8.18. Treatment of external corners (90°) and internal corners (270°) using corner elements from Federal Highway Administration (2009).

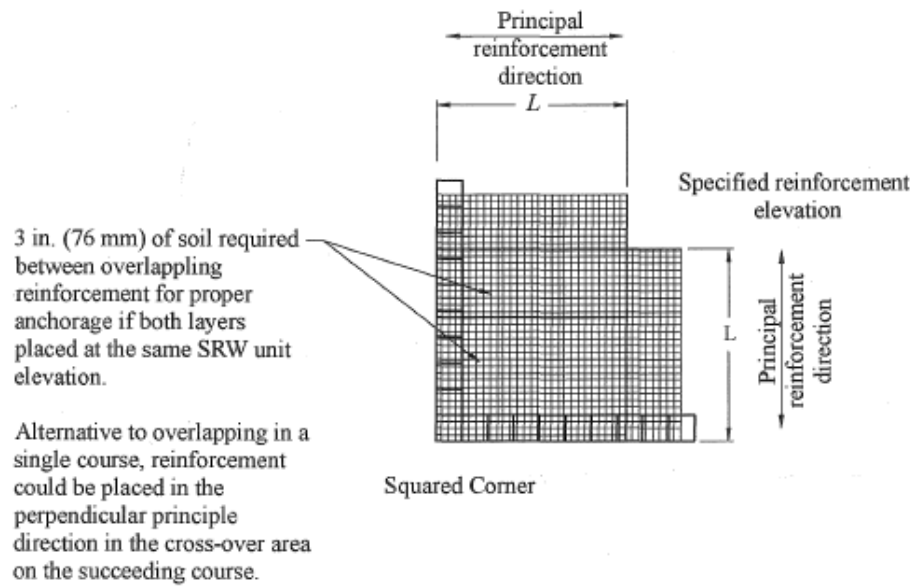


Figure 8.19. Reinforcement placement for convex curves from (National Concrete Masonry Association, 2010).

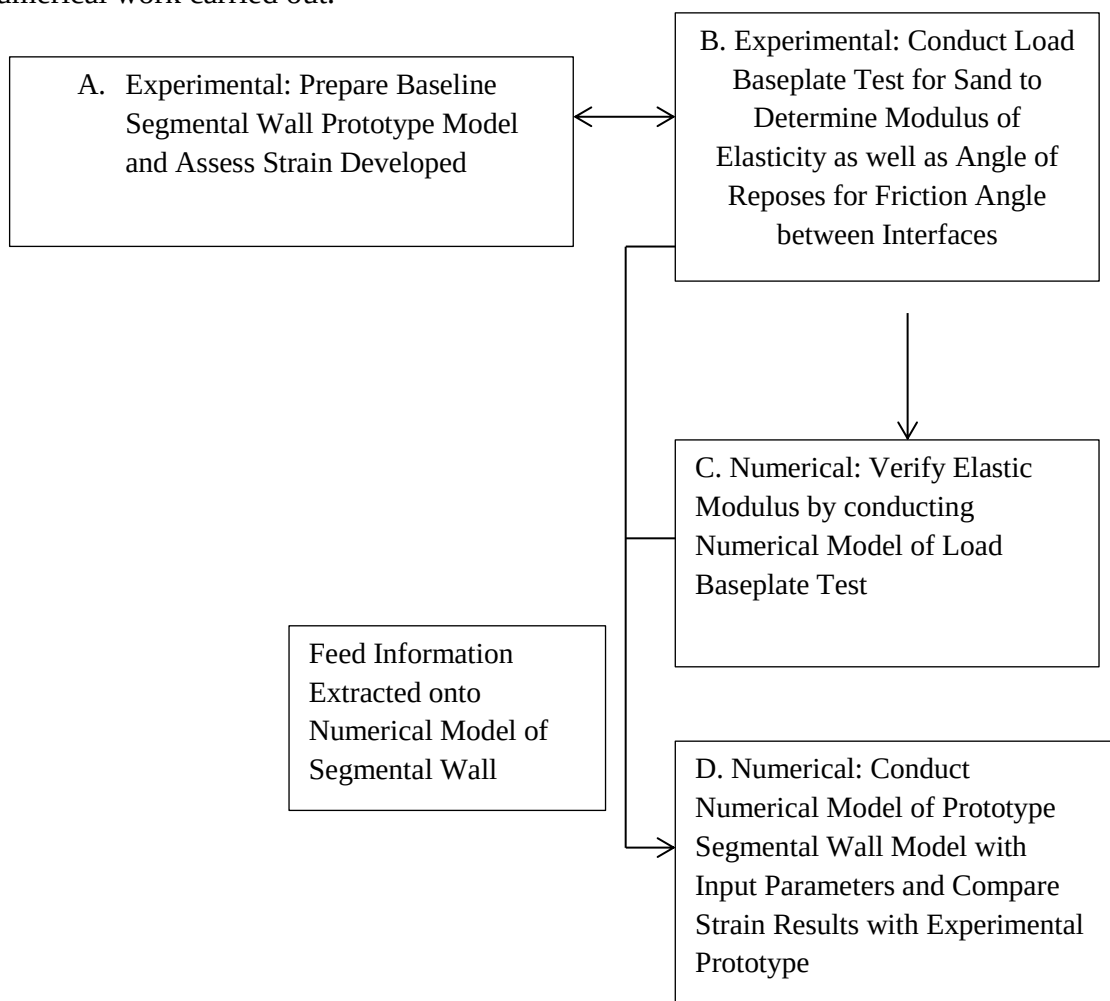
## 9. Experimental Work and Numerical Studies

### 9.1. Introduction

A Baseline Prototype model was prepared to examine the compressive strains occurring from the surcharge loads applied. The purpose of the model is to examine the strains induced along the wall with the addition of surcharge, which would provide evidence towards the causes of the discrepancies present on Hamderi and Guler (2015) wall. Once conducted, a numerical model using TNO Diana will be conducted of the baseline model and compared with the experimental results and that of the full scale segmental wall from Hamderi and Guler (2015) work.

### 9.2. Methodology

The following is a flowchart of the processes involved in the experimental and numerical work carried out.



## A. Baseline Segmental Wall Prototype Model Preparation

The preparation of the baseline model involved usage of modular wooden blocks as the wall facing segments, paper as reinforcement and granular sand as the reinforced soil. The blocks had standardised dimensions, although near the rigid wall end interfaces, the blocks had been sawn to

### *Prototype Setup*



Figure 9.1. Right side of the prototype model.



Figure 9.2. Left side of prototype model.

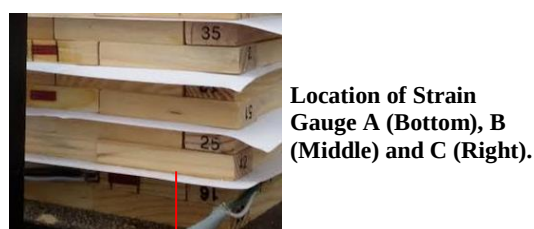


Figure 9.3. Isometric view of Pre-Loading (Left) and Post-Loading (Right) of prototype model.



Figure 9.4. Isometric view of Post-Loading of prototype model.

Strain gauges were fitted in the outer surface of the modular blocks, specifically to the blocks near the corner at the bottom, as it was hypothesised that most strain would be experienced at the lower wall part. This hypothesis was also tested by adopting strain gauges at various heights (refer to table 9.1 on the coordinates of the strain gauges).

For each layer, the sand was placed with minimal compaction for the sole purpose of having the granular layer equally levelled throughout the area before placing subsequent paper reinforcement.

Surcharge weights were then placed consecutively at the surface to measure the strain produced for each implementation, with the aim to investigate the correlation between the strains manifested with an increase in load increment.

Table 9.1. Strain Gauge Coordinates Origin at Bottom Corner; x is horizontal spacing from origin, y is vertical spacing from origin.

| Strain Gauge | Coordinate x,y (mm) |
|--------------|---------------------|
| A            | 60, 13.5            |
| B            | 108, 39.5           |
| C            | 108, 58.5           |
| D            | 45, 13.5            |

## B. Load Baseplate Test for Sand Elastic Modulus

A special experiment similar to a load baseplate experiment was conducted to find the amount of displacement occurring on the sand with load application on the sand equivalent to the load conditions experienced in the baseline model.

The setup involved placing sand into a steel mould of 149.5 mm diameter, with the load hanger placed to applying the load onto. The weight would be focused onto the load hanger baseplate, whereas on the top of the hanger there is a horizontal soffit with a dial gauge attached to measure displacement readings. The amount of displacement with each load is measured, in which a preliminary calculation of the modulus of elasticity,  $E$ , was calculated.

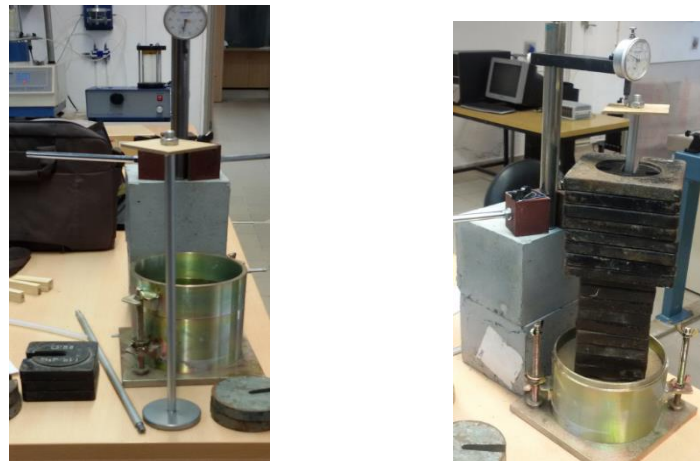


Figure 9.5. Load baseplate test for Displacement- $E$  calculation.

Table 9.2. Properties of Mould.

| Property                                  | Magnitude |
|-------------------------------------------|-----------|
| Disc Inner Diameter (mm)                  | 149.5     |
| Sand Poisson's Ratio                      | 0.35      |
| Hanger Baseplate Area (mm. <sup>2</sup> ) | 4417.86   |
| Exposure Stress Range (kPa)               | 0-15      |
| Initial Sand Height (mm)                  | 860       |

### C. Numerical Model on TNO Diana

The results of the Load Baseplate test were verified by the use of a numerical model on TNO Diana. A surcharge load of 71 kPa covering the area of the load baseplate (4417.86 mm.<sup>2</sup>) is applied (which resembles the working stress load application on the prototype experiment of 15 kPa. A trial and error for the modulus of Elasticity E was carried out to obtain a displacement amount matching that of the experiment. T

The boundary conditions used involved a fixed rigidity on all directions on the base and fixed only on the horizontal directions (x and y direction) along the wall circumference.

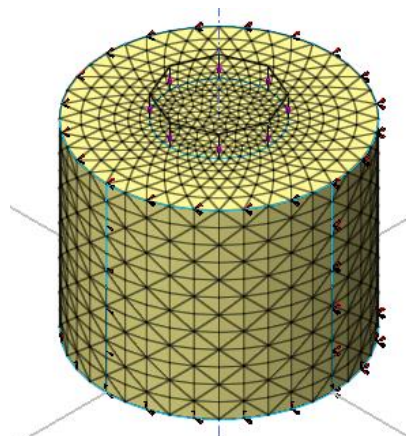


Figure 9.6. Numerical model of load baseplate on TNO Diana

D. The numerical analysis using TNO Diana also produced similar strain pattern results in terms of compression and tension but at different magnitudes.

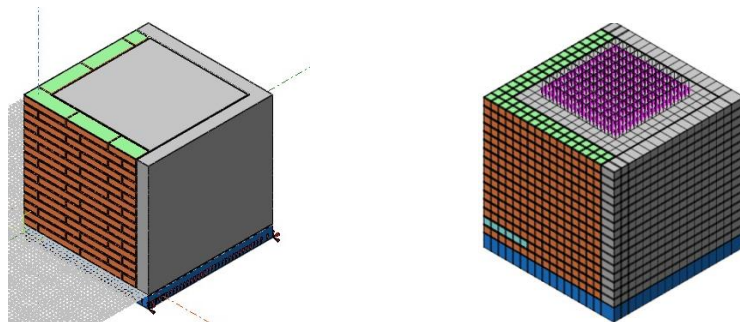


Figure 9.7. Diana Models prepared for analysis

The properties of the materials used are collated in Table 9.3 below:

### 9.3. Materials Used

Table 9.3. Various Properties Used in Prototype Model Experiment and Numerical Model

| Property                                   | Magnitude       |
|--------------------------------------------|-----------------|
| Standardised Block Dimension (L:W:H) (mm)  | 72:20:9         |
| Block Category A Dimension (L:W:H)         | 20:20:9         |
| Block Category B Dimension (L:W:H)         | 45:20:9         |
| Block Category C Dimension (L:W:H)         | 20:20:9         |
| Block Category D Dimension (L:W:H)         | 50:20:9         |
| Reinforced Soil Spanning Dimension (L:W:H) | 175 x 175 x 144 |
| Modular Block Compressive Strength (MPa)   | 40              |
| Paper Tensile strength (N/mm)              | 10              |
| Sand Friction Angle °                      | 35              |
| Block-Block Interface Angle °              | 30              |
| Block-Paper Interface Angle °              | 32              |
| Block-Sand Interface Angle °               | 35              |
| Block-Woodbase °                           | 25              |
| Surcharge Increment (kPa)                  | 1.09            |
| Total Surcharge (kPa)                      | 14.2            |

### 9.4. Results

#### A. Experimental Prototype Basemodel of Segmental Wall

Figure 9.7 below shows the strain gauge results over a period totalling 7200 seconds where 2 kg surcharge weights were placed for 10 minute duration intervals before the consecutive surcharge load was put in place, giving a total of 24 kilograms (13.4 kPa based on a surcharge area of 0.018 m.<sup>2</sup>). The results show symmetry by virtue of the close merging of the strain results between Gauge A (Bottom left Edge) and

Gauge D (Bottom Right Edge), which indicates that the strains manifested on both sides of the wall are equal. The variation in the left edge amongst Gauge A, B and C provides an outlook on the strain variation along the depth in that towards the mid-depth of the wall, the strain increases in negative magnitude (greater tension) being experienced.

Another inference that can be made from the results is that the compression experienced at the bottom is a sign for cracking potential to occur, whereas the tension, especially in greater magnitude towards the mid-height of the wall indicates proneness to segmental separation. Starting from the bottom of the prototype and going up, Gauge A, Band C had strain magnitudes of 26.09 (Tension), 12.50 (Tension) and -3.72 (Compression). Gauge A and D both being in the lower part of the wall experienced strain values consistent in the positive dominion.

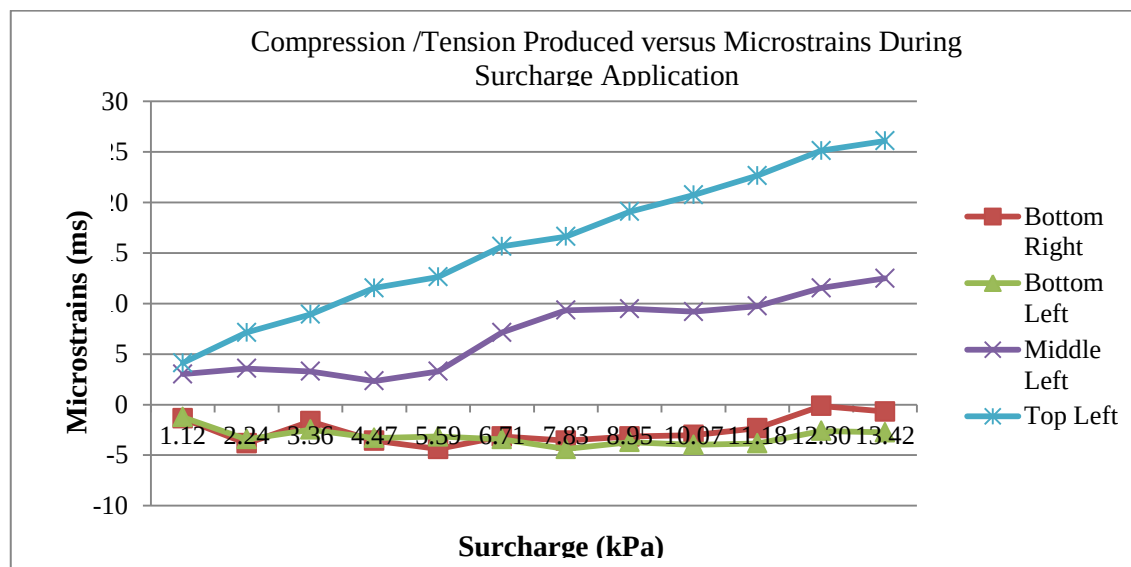


Figure 9.8. Compression and Tension in microstrains versus surcharge application for the various strain gauges.

The results do corroborate well with the numerical model for a full scale segmental wall from Hamderi and Guler (2015), in that the wall segments and the wooden blocks from the base model experienced compression near the wall corner whereas further away from the corner tension was more dominant.

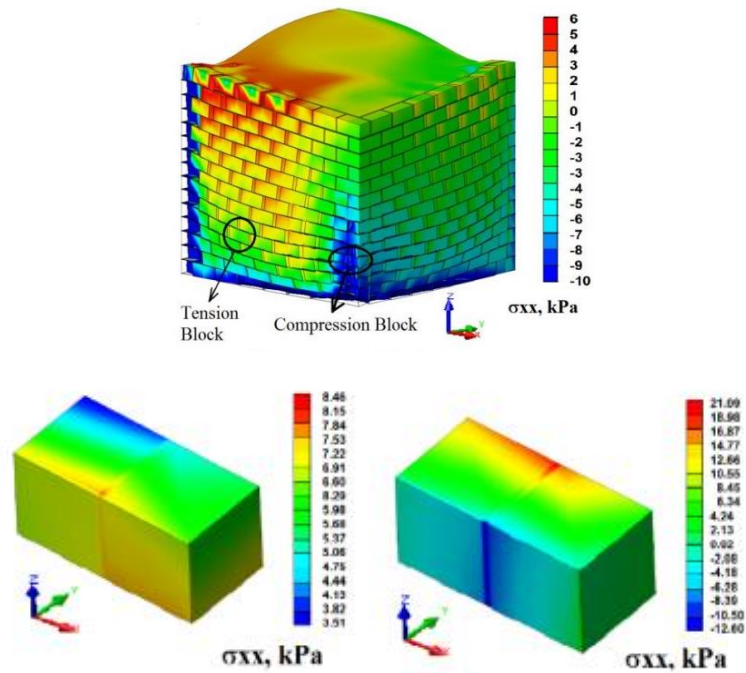


Figure 9.9. Compression and Tension in segmental wall model from Hamderi and Guler (2015).

## B. Load Baseplate Results

The preliminary based on the experimental results for strain are as follows:

Coefficient of Volume Compressibility:

$$m_v = -\frac{(\varepsilon_z)_2 - (\varepsilon_z)_1}{(\sigma'_z)_2 - (\sigma'_z)_1} \quad (9.1)$$

$$E'_c = \frac{1}{m_{vr}} \quad (9.2)$$

$$E'_c = \frac{\Delta\sigma_z}{\Delta\varepsilon_z} = \frac{E'(1-v')}{(1+v')(1-2v')} \quad (9.3)$$

$$E' = \frac{\Delta\sigma_z}{\Delta\varepsilon_z} = \frac{Ec(1+v')(1-2v')}{(1-v')} \quad (9.4)$$

Average  $E'$  over Stress Range 0-15.5 kPa = 19.84 MPa

The following results are the Surcharge-Strain amounts for the first test with a mass density of  $1.5756 \text{ kg/m}^3$ ,  $1.545 \text{ kg/m}^3$  and  $1.5563 \text{ kg/m}^3$ . The average elastic modulus for the sand calculated for the three tests were 19.83 MPa, 26.79 MPa, and

17.83 MPa, with each giving total displacement amounts of 0.445 mm, 0.37 mm and 0.51 mm.

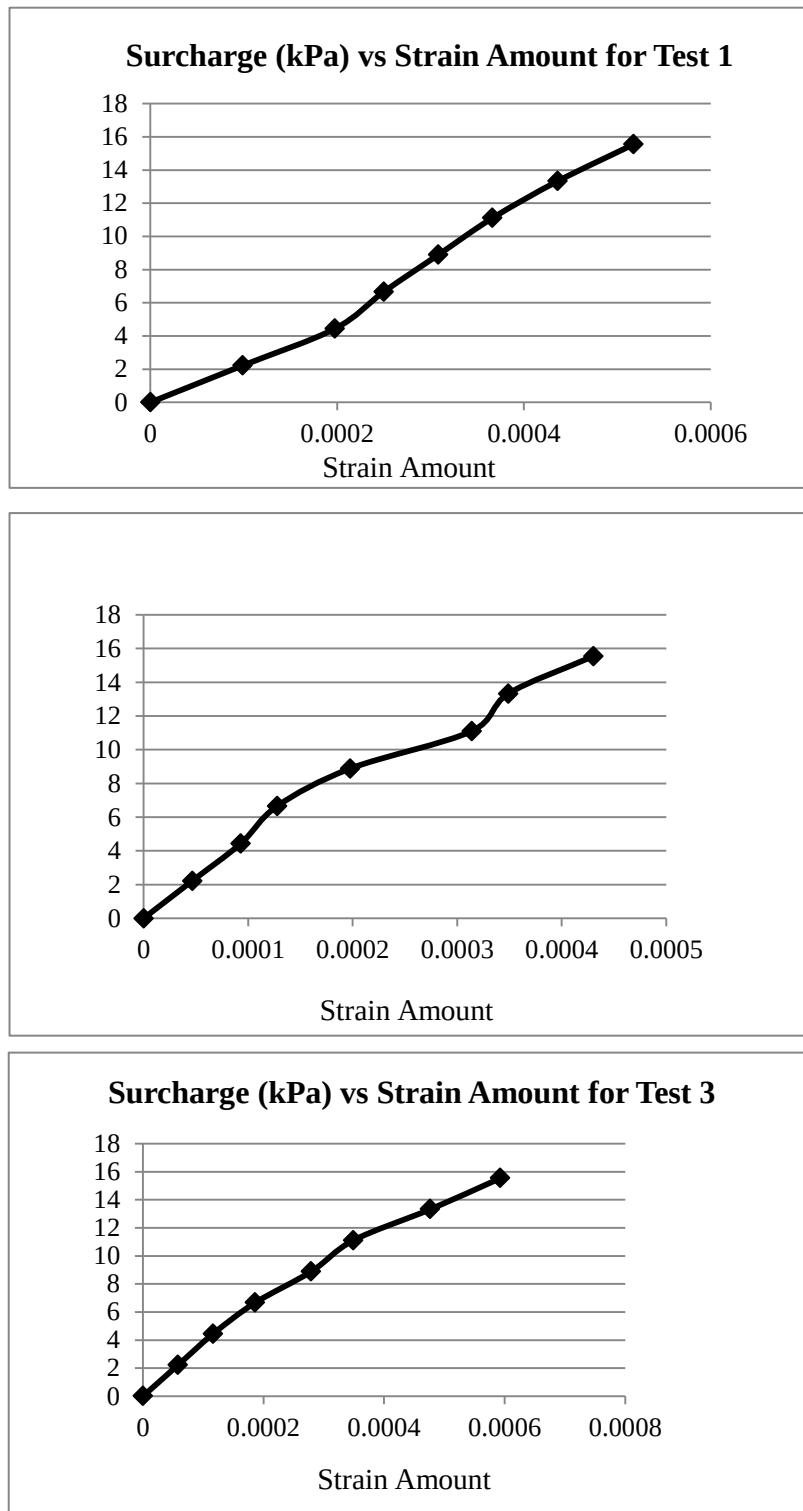


Figure 9.10. Surcharge versus strain for various tests.

### C. Numerical Analysis Model to Back-Track Elastic Modulus, E

Attempts were made using TNO Diana to verify the results for the Elastic Modulus. By assigning an elastic modulus value of 18 MPa, the amount of displacement achieved from a surcharge load application of 71 kPa was 0.458 mm.

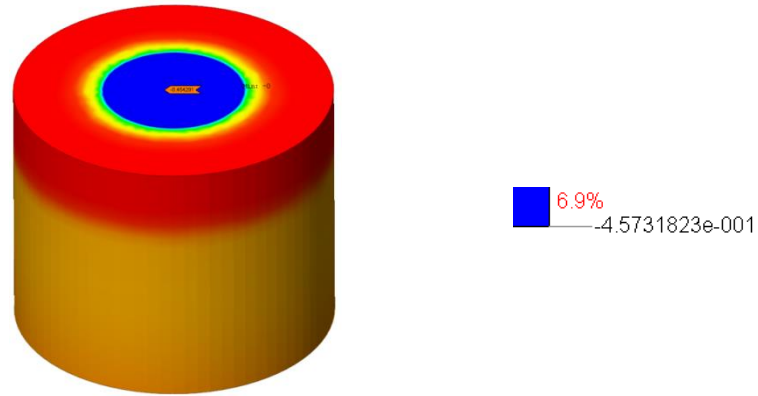


Figure 9.11. Displacement Result for Diana Model for Load Base Plate Test  
Maximum Displacement of 0.458.

### D. Numerical Analysis of the Prototype Model

The numerical analysis using TNO Diana also produced similar strain pattern results in terms of compression and tension but at different magnitudes. A magnitude of 18 MPa was adopted for the reinforced soil as per the numerical analysis carried out to back-track the Elasticity Modulus. Thin interfaces were used between the different wall materials to cause for a separation between the meshes. The two perpendicular planes on the back sides were allowed to only move in the in-plane direction causing restriction in the out-of-plane direction. Consequently, inaccurate stresses are prevented as a result of the backsides allowed to move freely in the horizontal and vertical directions. The base of the model however is fixed in the x, y and z directions, to avoid foundation failure.

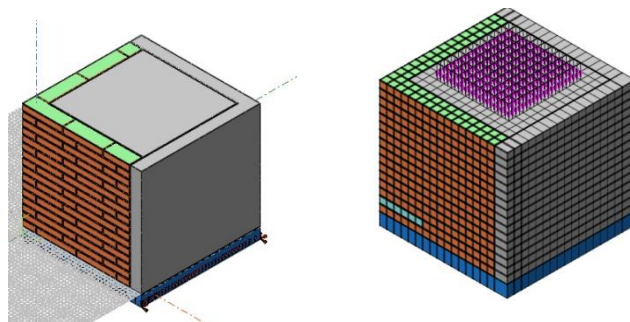


Figure 9.12. Diana Models prepared for analysis.

The nature of the strains in the numerical analysis (in terms of compression and tension) did match the experimental results, but in terms of magnitude. This however does not discredit the use of the numerical analysis method for analysing the strains and stresses manifested in a 3D mode.

Figure 9.12. below displays the results of a Diana model carried out for the Basemodel experiment. The model was not runned fully, though the preliminary results had favourably resembled the full scale segmental wall model from Hamderi and Guler (2015), as well as the pattern of the compressive/tensile nature of the wooden blocks of the experiment. The blocks near the corners were undergoing compression and away from the corners exhibited tension. It is worth mentioning that the reinforced sand used had a Modulus of Elasticity of 30 MPa, which is different from that foreseen by the experimental and the load baseplate Diana model carried out. An interesting observation was in block C, in that along its thickness there was a bending moment effect as a result of the sudden switch in the strain nature, which matched that of Figure 8.12 from Hamderi and Guler (2015)'s study.

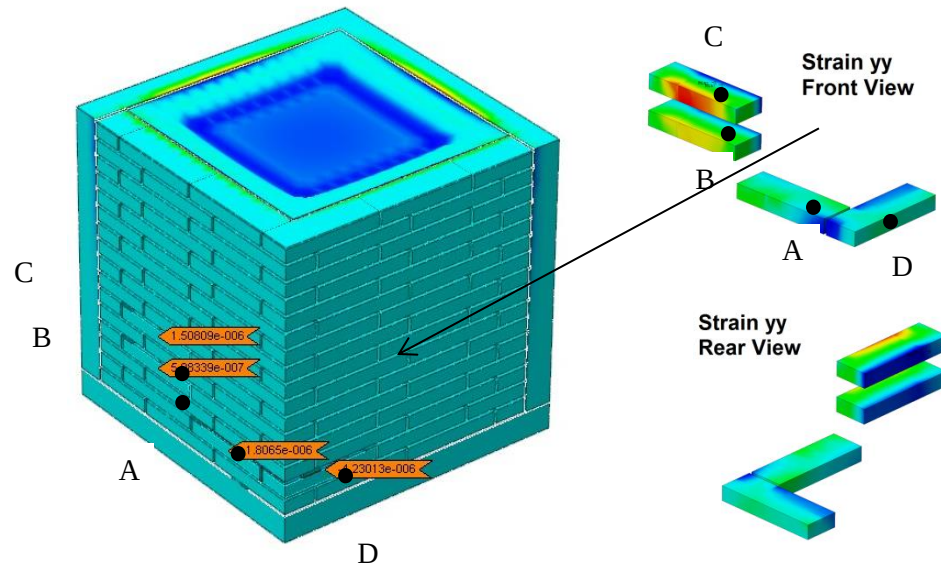


Figure 9.13. Strain results for Diana Model of Baseline experiment with the strain amounts on Gauge A, B, C and D.

## **10. CONCLUSIONS DRAWN FROM EXPERIMENTAL AND NUMERICAL WORK**

### **10.1. Experimental and Numerical Conclusions**

The numerical model of the prototype modular wall experiment demonstrated favourable results with that of the experimental model in terms of the strain distribution pattern, but not in magnitude values.

The prototype model had also exhibited the same strain behaviour as that of the full scale segmental wall model from Hamderi and Guler (2015). The expectations of which blocks would experience compression and tension was met in both the numerical and experimental models of the prototype experiments.

One inference deduced is that although the wall was very small in height, the stresses were not enough to cause failure. In a highwall case however, the stresses can reach to values for failure causing cracking from compression or block separation from bending moment generation.

The majority of the thesis material covered a 2D analysis approach, due to the lack of literature conveying 3D design. A reason for this can be due to the lack of full understanding of the 2D soil-reinforcement interaction, which makes the conversion into 3D having greater complexity. Leshchinsky *et al.* (2014) did attempt to tackle this problem, but at the expense of ignoring the facing contribution, which is paramount to the behaviour of GRS walls.

## **APPENDIX A: CRITIQUE ON VARIOUS INTERNATIONAL STANDARDS**

The standards assessed are the following:

- (i) United States: FHWA Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Slopes. (Federal Highway Administration, 2009)

Abbreviated as FHWA 1

- (ii) United States: FHWA Design and Construction Guidelines. (FHWA National Highway Institute, 1998)- Abbreviated as FHWA 2

- (iii) United States: Design Manual for Segmental Retaining Walls, National Concrete Masonry. (National Concrete Masonry Association, 2010) - Abbreviated as NCMA

- (iv) United States: Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide(Federal Highway Administration, 2012)-

Abbreviated as FHWA 3

- (v) United Kingdom: British Standards BS 8006 Code of Practice for Strengthened/Reinforced Soils and Other Fills. (British Standards Institution, 2012)

Abbreviated as BS 8006

- (vi) Hong Kong: Geoguide 6 Guide to Reinforced Fill Structure and Slope Design. (Government of the Hong Kong Special Administrative Region, 2002)

Abbreviated as Geoguide 6

- (vii) Germany: Recommendations for Design and Analysis of Earth Structures Using Geosynthetic Reinforcements-EBGEO (German Geotechnical Society, 2011)

(Note: This is an English language version which is based on the actual standard used by Germany known as DIN1054)

- (viii) New Zealand: Guidelines for Design and Construction of Geosynthetic-Reinforced Soil Structures in New Zealand-Transfund New Zealand Research

Report No. 239. (Transfund New Zealand, 2003). Abbreviated as NZ Report 239

- (ix) Australia: Australian Standard AS 4678-2002 Earth Retaining Structures. Abbreviated as AS 4678

### **A1.1. Sliding Failure (External and Internal Stability)**

The horizontal components of the earth pressure thrust acting upon the wall act as destabilizing forces capable of invoking a translation movement once these exceed the retaining structure's resistance mechanisms (Clayton *et al.*, 2013).

Such translation movements may involve the GRS system as a coherent body to slide upon its base (external stability), or from within, where the soil slides along or between the reinforcements (internal stability).

#### **Mechanism 1a External Sliding Failure**

For external stability, FHWA postulates the driving mechanism by the self-weight of the retained soil and surcharge acting upon produces enough active state horizontal pressure (as a means of plastic equilibrium being reached). The overall geometry of the reinforced backfill plays a role in defining Rankine's Active earth pressure, whether it is battered under 10 degrees from the vertical or traversal whether the back slope is horizontal or graded.

On the other hand, FHWA treats resistance as per the methods adopted by AASHTO, in which the failure is initiated once the factored shear resistance of the soil or the factored sheared resistance at the interface between soil and foundation (whichever bares minimum) per unit wall length is exceeded.

The shear resistance stems from the frictional interaction of the horizontal component of the vertical load of the coherent soil body with the underlying surface. FHWA conservatively omits any external live load although it increases the sliding stability, unless the live load is permanent.

The horizontal resistance from the vertical weight is related using the tangent component of the soil friction angle  $\phi$ , being the minimum of three types of cases:

Sliding along the foundation soil where the shear strength  $\phi_f$

Sliding along the reinforced fill  $\phi_r$  (grid type of reinforcement)

Sliding along weaker of the upper and lower soil-reinforcement interfaces, which is represented by friction angle  $\rho$  where the reinforcement is a sheet type

FHWA advises the friction angle  $\rho$  can be obtained from interface direct shear tests, or in the absence of such tests to adopt  $\frac{2}{3} \tan \phi_r$ .

The design procedure is to ensure that the capacity demand ratio for sliding ( $CDR = R_r/P_d$ ) exceeds unity. If not, the reinforcement length is increased for more contact surface for the resistance.

EBGEO adopts a ULS-STR type of analysis (in parallel with the Eurocode 7 standard), which is a limit state used in cases related to rupture or excessive deformations (Bond and Harris, 2008). In doing so, the load combinations for the driving mechanism adopt the ULS-STR type of loading,

EBGEO distinctly differs from the FHWA where it considers that the reinforcement will strictly always be underneath the retaining structure. The tensile strength sliding resistance is taken as a horizontal tangential of the self-weight of the retaining structure along with the surcharge load, with the angle of friction in this case to reflect a contact between the lowest reinforcement layer put at the base of the earth structure. The critical value for the angle of friction is chosen to be the lowest friction angle above ( $\phi_1$ ) or below ( $\phi_2$ ) the reinforcement.

In turn, the characteristic frictional coefficient between the reinforcement and the ground becomes:

$$f_{sg} = \tan \delta_{s,k} = \lambda \tan \phi_{\text{governing}} \quad (\text{A.1})$$

The British standard BS 8006 refers to the external sliding stability as stability against forward sliding at the interface between reinforced fill and the subsoil. It enables flexibility as does FHWA 1 on the contact at the base of the structure, whether

it is soil-to-soil contact or reinforcement-to-soil contact. The difference is that there is consideration for the time duration, where both the long-term (effective stress analysis) and the short-term (total stress analysis) analyses for both case types.

$$\text{Short Term Analysis: } f_s R_h \leq \frac{a'_{bc} c_u}{f_{ms}} L \quad (\text{A.2})$$

$$\text{Long Term Analysis: } f_s R_h \leq \frac{\tan \phi'_p}{f_{ms}} + \frac{cl}{f_{ms}} L \quad (\text{A.3})$$

### *Mechanism 1b Internal Sliding Failure*

The New Zealand guide, NZ Report 239 includes a mechanism involving internal sliding where movement occurs along a reinforcement layer, where an examination is carried out for each reinforcement elevation.

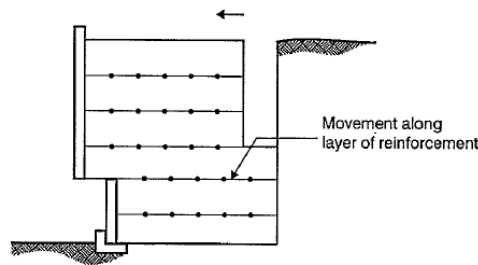


Figure A.1. Internal Sliding from Transfund New Zealand (2003)

The horizontal active earth pressures driving the mechanism are calculated in a manner similar to that for external stability, but with a difference of only taking the height above the reinforcement under consideration. Therefore, the overall geometry shape is also considered, with strictly being the mass over the reinforcement structure.

The resisting mechanism is similar to that of the external stability mechanism, in that the interaction strength of the soil-reinforcement for direct shear is mobilized. There is an additional resistance mechanism of utilizing the shear resistance provided by the wall facing.

The facing units and connections between facing and reinforcement possess a shear resistance due to friction between segmental units and geosynthetic

reinforcement, as well as shear resistance from friction between geosynthetic reinforcement and granular material filling unit cavities. These segmental units need to be locally stable to ensure the shearing resistance by the wall facing can be mobilised.

This connection instability can be seen as a corollary to cause internal sliding of the retaining structure.

Something noteworthy to mention is the NZ guide is more explicit than the FHWA in pointing out all the driving mechanism

- vertical stress due to sloping backfill;
- vertical stress due to uniformly distributed surcharge;
- vertical stress due to concentrated vertical loads

The Hong Kong Geoguide 6 recognises an additional mode of instability involving the sliding of planes between the reinforcement, where the shear resistance of the fill material along the potential failure plane is mobilised. The failure plane (as shown on figure A2) coincides between the two reinforcement layers spaced as per the requirement for maximum tensile strength.

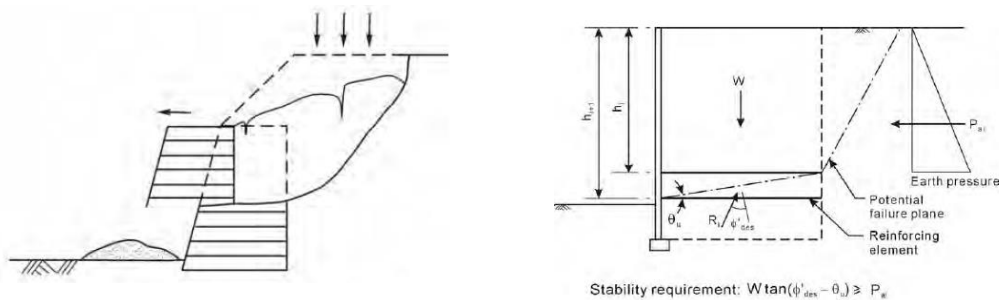


Figure A.2. Internal Sliding between the reinforcement Layers (Left) along with Stability requirement (Right) from (Government of the Hong Kong Special Administrative Region, 2002).

### A1.2. Pullout Failure (Internal)

Pull-out failure is an internal stability mechanism which involves a reinforcement layer pulling out of the soil once the total available bond stress to withstand the horizontal pressures has been mobilised (Manceau *et al.*, 2012)

Each reinforcement layer is individually checked for pull-out capacity, as well as the equilibrium of planar wedge mechanisms through reinforced zone being restrained by several layers.

FHWA 1 and FHWA 2 point out that large movements and possible collapse of the structure can happen in the event that the tensile forces in the reinforcement exceed the pullout resistance. In doing so, an effective length is prescribed to ensure that there is enough embedment length covered in the passive zone with tensile capacity to overcome the maximum tension,  $T_{max}$ , as follows:

$$L_e \geq \frac{T_{Max}}{\phi F^* \alpha \sigma_v C R_c} \geq 3 \text{ ft (1m)} \quad (\text{A.4})$$

A significant resistance component of the equation is  $F^*$ , known as the pullout resistance factor, which reflects the resistance acting from the reinforcement. It encompasses the passive and frictional resistance components as follows:

$$F^* = F_q \alpha \beta \text{ (Passive Resistance)} + \tan \rho \text{ (Frictional Resistance)}$$

$\rho$  being the soil-reinforcement interaction friction angle. FHWA 1 considers  $F^*$  for geosynthetic to be  $2/3 \tan \phi_p$ , the peak friction angle of the soil)

The passive resistance are only provided for geogrid type structures, which depend on the open grid spaces.

A further important resistance factor is the correction factor  $\alpha$  which accounts for non-linear stress reduction over embedded length of a highly extensible reinforcement. This stems from the strain softening of the compacted granular backfill material and the extensibility and length of the reinforcement.

$T_{max}$ , the maximum tension that can be applied on the reinforcement is the product of the vertical spacing between the reinforcements and the horizontal earth pressure. The horizontal earth pressure covers a tributary area between midpoints of the above and below adjacent reinforcements, known as the contributory heights. At each level, the lateral earth pressure becomes calculated as follows:

$$T_{Max} = \sigma_H S_V \quad (\text{A.5})$$

$$\sigma_H = K_r[(\gamma_r Z)\gamma_{EV-MAX}] \quad (A.6)$$

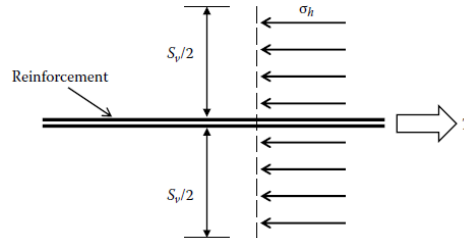


Figure A.3. Pullout resistance mechanism from Clayton *et al.* (2013).

The  $K_R$  denotes the use of the simplified coherent gravity method (explained further under section) , where in which Rankine's Active earth pressure is used for the horizontal earth pressure coefficient, on the basis that the reinforcement being extensible would elongate more than the soil, but will not change the soil failure surface. It must be emphasized that a metallic reinforcement would have a different coefficient of earth pressure greater than the active for the top part of the wall, with this being based on empirical results.

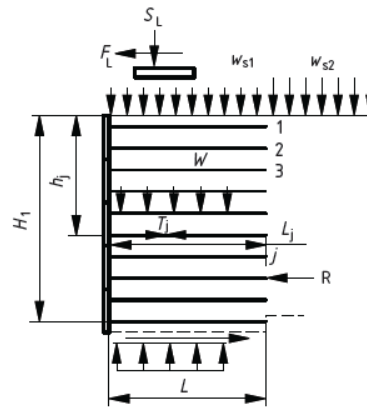


Figure A.4. Components for internal stability from Institution (2012).

The rotation mechanism for the Coherent gravity method is involves outward rotation about the top, which is elaborated in Chapter 2.

BS 8006 is more extensive than FHWA by providing three various methods for determining internal stability for pullout resistance (referred to as adherence) provided. The first method is as the earlier called Coherent Gravity method, with a difference of

having a varying coefficient of earth pressure within the structure which is applicable to all types of reinforcements (extensible and inextensible). The top of the wall has the coefficient of earth pressure taken as  $K_o$  and decreases linearly with depth to a value of  $K_a$  at 6 m below top of structure.

$$K = K_o(1 - z/z_o) + \left(\frac{K_a z}{z_o}\right) \text{ for } z \leq z_o = 6 \text{ m} \quad (\text{A.7})$$

$$K = K_a \text{ for } z > z_o \quad (\text{A.8})$$

A local stability check is first sought to ensure that the maximum ultimate limit state tensile force  $T_j$  is being resisted by the reinforcement layer at depth  $h_j$  below the top of the structure. BS takes into consideration the cohesion of the material to undercut the maximum tension on the reinforcement.

$T_{pj}$  denotes tensile force to withstand vertical loading from fill self-weight, surcharge and bending moment acting, with the use of coefficient of earth pressure  $K$  for the horizontal stresses.  $T_{sj}$  and  $T_{fj}$  respectively correspond to vertical strip loading and horizontal shear applied to a strip contact area. Cohesion effects are found to diminish the tension effects applied on the reinforcement.

$$T_j = T_{pj} + T_{sj} + T_{fj} - T_{cj} \quad (\text{A.9})$$

Once obtained, the reinforcement embedment length must be enough to satisfy stability considerations for pullout, where each is checked individually to ensure there is enough resistance to the maximum reinforcement tension. This resistance is rendered as embedment length in the passive zone, which provides resistance greater than the maximum tension as shown below:

$$T_j \leq \frac{2B\mu}{f_p f_n} \int_{L-L_{aj}}^L f_s \sigma_v(x) dx \quad (\text{A.10})$$

The combination of the loading effects deems the necessity to recognise how tension changes along the reinforcement length at 3 positions: the facing, along maximum tension line 1 and along maximum tension line 2.

The second method promoted by BS is the Tie Back Wedge method, which ensures both local and global stability is met by investigating pullout failure on an individual reinforcement basis as well as a wedge body shape.

The fundamental difference is the mode of instability movement mechanism, where the wall moves outward about the wall toe (Clayton) as oppose to the top of the wall for Coherent body method.

Also, the active earth pressure coefficient is used for finding all the horizontal earth pressures throughout the depth. The local stability of each reinforcement layer is sought similar to the Coherent Gravity Method.

The pullout check locally is to ensure that the perimeter of embedment (top and bottom) is enough to withstand the maximum tension force.

$$P_j \geq \frac{T_j}{\mu L_{ej}(f_{fs}\gamma_1 h_j + f_{fw}s) + \frac{a'_{bc}C' L_{ej}}{f_{ms}f_p f_n}} \quad (\text{A.11})$$

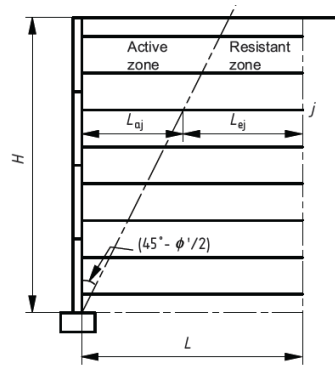


Figure A.5. Rankine's failure plane dividing Active and Resistant zone from Clayton *et al.* (2013).

A striking difference from the Coherent Body method is that the failure tension plane is strictly a Rankine failure plane, and there is description of varying earth pressure across the reinforcement length.

Reference to Hong Kong's Geoguide 6 elucidates the Yielding Zone and Resisting Zone for both the Tieback method and the Coherent Gravity method and how important it is that the length of the reinforcement within the resisting zone is sufficient enough to mobilise the shear resistance. Otherwise, the reinforcing elements become pulled out and produce gross distortions in the structure, with the potential of causing a collapse.

BS 8006 also conducts a global wedge stability check as explained in Chapter 2.

New Zealand Standard applies the same method as FHWA, with an addition of further meticulous criteria:

- (i) Allowable Displacement: soil-to-reinforcement displacement required to mobilise design tensile force to be smaller than the allowable displacement.

The movement required to mobilise the design tensile force depends mainly on the load transfer mechanism, reinforcement extensibility and the soil type.

- (ii) Long-term Displacement: pullout load to be smaller than the critical creep load.

The creep characteristics of the soil and reinforcement play a role in this criteria, bearing in mind that cohesive soils susceptible to creep will not use reinforcement systems.

NZ standards further consolidate the FHWA regarding the mechanism for passive resistance as a load transfer mechanism being only present for geo grids.

The German Recommendations following DIN1054 applies Eurocode's jargon of GEO in the design for pullout for a retaining wall. GEO is a Eurocode ultimate limit state that gives significance to soil strength in providing resistance to avoid failure or excessive deformation to the ground (Bond and Harris, 2008; Standard, 2005 ) In doing so, a partial safety is applied to the characteristic pullout resistance of the reinforcement.

The characteristic pullout resistance is as follows:

$$R_{A,k} = \sigma_{v,k} \cdot L_A \cdot f_{sg,k} \cdot n \quad (\text{A.12})$$

where  $L_a$  denotes anchorage length behind the reinforcement plane in the passive zone. A significant distinction with EBGeo is how it treats the friction coefficient between the fill soil, the plane formed by the geosynthetics and the intermediate ground. This friction coefficient is denoted as  $f_{sg,k}$  is more comprehensible in quantifying the composite action between the reinforcement and the soil in the transfer of forces between the soil and the geosynthetics.

$$f_{sg,k} = \lambda \tan \phi_k \quad (\text{A.13})$$

$\lambda$  being the composite action of friction, a ratio of the composite coefficient of geosynthetic/soil to the composite coefficient of soil.

$$F_{seg,k} = \lambda_c ck \quad (A.14)$$

$\lambda_c$  being the composite coefficient of cohesion, equal to the ratio of adhesion of geosynthetics/soil to the cohesion of the soil backfill itself.

### A1.3. Reinforcement Elongation and Breakage (Internal)

FHWA 1 states that rupture is manifested as a result of the tensile forces in the inclusions reach a threshold amount causing them to elongate excessively or break, which results to large movements and/or structural collapse.

A check is carried out on every individual reinforcement, where the factored tensile resistance,  $T_r$  must exceed the maximum tensile force acting on the reinforcement (as per the methods outlined under Mechanism 2: Pullout Capacity).

$T_r$  is found as by applying a resistance factor and reduction factors to  $T_{ultimate}$ , (the ultimate tensile strength of the inclusion). The reduction factors specifically take into consideration any detrimental effects over the life the reinforcement layer (see figure A6 below), including installation damages (from placement and soil compaction), creep reductions and durability as a result of the polymer chemical degradation.

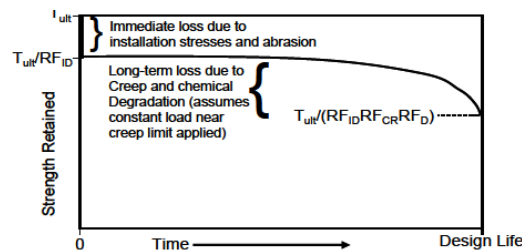


Figure A.6. Long term tensile strength from Federal Highway Institute (2009).

$$T_r = \phi T_{al} \quad (A.15)$$

$$T_{al} = \frac{T_{ult}}{RF} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_D} \quad (A.16)$$

A mechanism more prevalent with elongation and breakage as oppose to pullout is the interaction of the reinforcement with the failure surface, in this case a linear surface for an extensible reinforcement. There is a tendency for the tensile force to cause the reinforcement to rotate, where the component of tension in the failure surface direction would increase and the normal component may increase or decrease. This phenomenon is found to be common amongst geosynthetics, although the FHWA instructs to ignore such an effect in the case of a simplified coherent method being adopted in the analysis for finding out the maximum tension forces (as per outlined under the Pullout method).

BS 8006 treats rupture (the jargon used in the BS for breakage) in a similar manner as FHWA as a long term effect. There is an additional reduction factors to account for weathering effects (not to be confused with chemical weathering).

With regards to Geoguide 6, rupture is treated as either an internal stability problem, where the failure planes are entirely within the reinforced block or as a compound instability, where the failure planes are within and outside the reinforced block.

The design tensile strength is checked to ensure there is enough capacity to withstand the tensile force due to the fill's self-weight, with any contributions from the surcharge and the overturning moments, and tensile forces from isolated vertical and horizontal vertical pad loads.

There are references to stress rupture as a result of creep occurring in polymeric reinforcements, where constant load applications cause the polymeric material to extend. Considerations of the lifetime over which polymeric reinforcements need to carry a load are derived from stress-rupture curves.

The German guide also emphasizes on the ramifications pertaining in a long term sense where viscose, time-dependent creep processes can occur, and result to reduction in strength to lead to creep rupture and increase in creep strain, which is reflected as unacceptable deformations in the structure.

#### **A1.4. Facing Elements Stability and Connection Strength**

*The standards have treated both mechanisms as intertwined with one another. The standards however follow guidance provided by the NCMA, which provides exclusive sections for the Facing Stability and the Connection Strength.*

Not all the standards corroborate the same methods regarding facing and connections instability.

FHWA 3 for instance precludes any structural contribution from the facing element to the GRS system. This is especially the case if the designer was to follow the recommendations in using a concrete masonry unit (CMU). The manual recommends any facing unit to be used, although special design considerations would precedence for connections should there be any. Wu's studies in Chapter 7 further elaborate on this matter.

FHWA 1 however states otherwise, especially where it elaborates on the connection strength between the facing unit and the inclusions as being critical, and recommends a conservative approach to design for  $T_{\max}$  (refer to Mechanism 2: Pullout Failure) It further expands on this in that the facing elements are to resist the horizontal forces developed.

With the use of Modular Block Wall Units, the inclusions are connected by a means of the following:

An actual structural connection provided that it does not damage the facing

Friction between units and the reinforcement, including friction developed from the aggregate contained within the core of the units

Connection devices providing a friction and shear combination

The strength obtained is unique to the combination of geometry, unit batter, normal pressure, unit depth and unit infill gravel. This may require testing unique to the combination pertained.

Nominal Long term connection strength,  $T_{alc}$  is based on the geosynthetic tensile strength ( $T_{ult}$ ), long-term connection strength reduction factor ( $CR_{cr}$ ) and also reduction for chemical and biological degradation ( $RFD$ ), as per the relationship below:

$$T_{alc} = \frac{T_{ult}}{RFD} \quad (A.17)$$

$R_{Fd}$  for nominal long-term and long term are different due to the fact that the environment between and directly behind modular blocks at the connection may not be the same.

Connection strength developed is a function of normal pressure developed by the weight of the units. Such connection strength varies along the wall depth from a minimum at the top of the wall to a maximum at the lower portion.

FHWA provides  $CR_{cr}$  calculations for evaluating both the short and long term connection strength conditions by using varied  $CR_{cr}$  values.

For long-term testing:

CRCR is the long-term connection strength reduction factor for reduced ultimate strength resulting from the connection. This is obtained by carrying out creep tests over extended period of times to evaluate creep rupture at the connections

CRCR (Creep Reduced Connection Strength) is considered as:

With regards to facing elements, FHWA explicitly states that the facing elements are to be designed for the purpose of resisting the horizontal loads acting upon it. In doing so, the panel connector must be able to resist the  $T_{max}$  loads acting upon it.

Should a flexible facing panel be used, it must be designed to be able to withstand excessive bulging as backfill behind the facing elements compresses as a result of the compaction stresses, self-weight of the backfill or the lack of section modulus.

Bulging at the face between the reinforcement elements in the horizontal and vertical directions must be limited to 25 to 50 mm in the theoretical wall line. Such tolerance can be achieved by:

- Decreasing the vertical and horizontal spacing between reinforcements
- Increasing facing material section modulus
- Placement of nominal wide fill or cobbles directly behind facing
- Sufficient overlap between facing panels

Top of the flexible facing panel at the top of the wall should be attached to a soil reinforcement layer to provide stability to the top of the facing panel

Lateral movements can occur during construction can prompt mechanisms 4a, 4b, 4c and 4d. FHWA 1 describes how the magnitudes would depend on the fill placement techniques, compaction effects, reinforcement extensibility, reinforcement length, the connection details between the reinforcement and the facing (hence the intertwining of the Mechanism 4 and 5) and the wall facing detailing. Once the Length: Height surpasses the theoretical lower limit from  $0.5H$  to AASHTO's specified limit of  $0.7H$ , the deformation would decrease by 50 percent.

FHWA 1 however adopts the design philosophies primarily from NCMA, especially when it comes to defining the sufficient inter-unit shear capacity and the maximum spacing between the reinforcement layers to be twice the front-to-back width  $W_u$

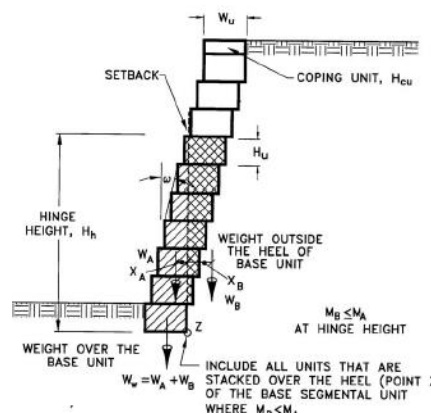


Figure A.7. Front-to-Back width of coping units from Federal Highway Institute (2009).

With regards to Geoguide 6, an individual tensile force evaluation is undertaken on each reinforcement is undertaken between the reinforcement elements and the connections. This is to avoid failure of the connections between the individual

reinforcement elements and the connections on the facing. Consideration is also to be given to shear and bending stresses generated from the fill settlement relative to the facing needed.

The required tension of the  $i^{\text{th}}$  reinforcement layer,  $T_i$  is:

$$T_i = T_{ei} + T_{pi} + T_{fi} \quad (\text{A.18})$$

$T_{ei}$  as the fill self-weight and any surcharge and overturning caused by earth pressure

$T_{pi}$  as the tensile force due to vertical loading applied to top of structure

$T_{fi}$  as tensile force due to horizontal shear applied to the top of the structure needed to be resisted by the  $i^{\text{th}}$  reinforcement layer

The connection strength of segmental block walls is found to be less than the long term design strength of the reinforcement, which deems to be more critical. This would therefore lead to a stability assessment of the segmental block facing to be carried out for the connection strength.

Global stability checks regarding polymeric connections, which take into account design actions of in tension, shear, combined tension and shear as well as the performance characteristics of different jointing.

BS 8006 provides lucid explanations design actions for connections only when metallic reinforcements are used (including provisions for combined actions of tension and shear, tension with bending etc.) but with polymeric reinforcements, it refers to the mechanisms as highlighted in Chapter 3 and 6 for Tie-Back Wedge method and Coherent Gravity method.

BS 8006 also provides provisions adopted for the structural loads on the facings. It stipulates that the facings must be designed to withstand the loads and effects from:

- (i) Horizontal soil pressures and the corresponding reinforcement tension reactions developed in the connections between the facing and the reinforcement;
- (ii) Forces generated from the superimposed facing units;

- (iii) Vertical shear forces developing as a consequence of the relative movement between the facing and the fill along with any additional tensile reactions generated;
- (iv) External load applications (temporary or permanent);
- (v) Possible differential longitudinal settlement

For the latter differential longitudinal settlement, whether the Tie Back Wedge method or Coherent gravity method is adopted, the capacity of the connection,  $T_{\text{conn}}$  must be 100%  $T_j$  throughout the whole segmental wall depth.

Similar to FHWA, BS also refers to NCMA with regards to the segmental block walls for the vertical spacing of reinforcements and the interface shear connections.

#### **A1.5. Shear Failure of Facing Column (Bulging)**

The block units of the column must remain intact and not prone to toppling or sliding in the units near the wall crests, especially at the unreinforced height of the block units at the structure top. Within the wall depth, shear failure must be prevented between facing units by ensuring there is sufficient interface shear capacity between the block units and the block-to-reinforcement interaction.



Figure A.8. Crest toppling (Left) and bulging from middle wall (Right) from Society (2011).

The block layers exhibit shear transfer by being developed primarily through shear keys and interface friction. In cases where interface layers are under low normal stress, i.e. upper part of the structure, the shear transfer is predominantly carried out by the mechanical connectors or the reinforcement layer provisions to resist toppling and sliding failures of the wall crest.

Another point of consideration is how the earth pressure acting on the facing is manifested. The overall geometry of the backfill will be decisive on the active earth

pressure coefficient to be adopted i.e. Coulomb equation for  $K_a$  for non-vertically battered backfill with a back-slope angle.

The shear capacity of the facing on a unit-to-unit and unit-to-reinforcement interface is dependent on the normal stress transmitted between the units. Such normal stress stems the weight of the units, which is at a minimum from the upper wall to a maximum at the lower wall.

FHWA 1, Geoguide 6 and NZ Report 239 refer to the NCMA Hinge Height Concept (although the latest 4th edition has refrained from adopting this concept), which is adopted in cases where the facing is non-vertical. The hinge height is the maximum height of an isolated column of units that can be stacked at a facing inclination  $\alpha$  without toppling.

$$H_h = \frac{2(W_u - G_u)}{\tan \alpha} \quad (\text{A.19})$$

Or for a true vertical facing,  $H_h = H_i$

A point of concern in designing the segmental walls is ensure that the facing units above the highest reinforcement layer must perform as a free standing retaining wall, such that these are capable to withstand sliding and toppling failure modes as If it was a conventional gravity wall. The resistance mechanism preventing such instability would be stemmed from the weight of the facing units.

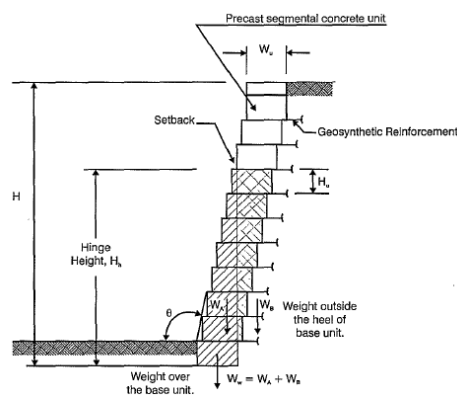


Figure A.9. Free standing Hinge Height concept from National Concrete Masonry Association (2010).

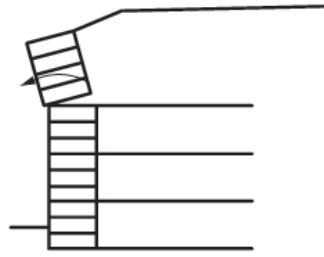


Figure A.10. Unsupported crest from British Standards Institution (2012).

BS8006 advises tilting can be limited by having the stacked units proportioned to ensure that the resultant force acts within the middle third of the unit base under the worst loading combination. The middle third rule being applied would enable the overturning and resisting movements to be calculated from the permanent and live loads as well as the fill material's shear strength parameters.

The Bulging stability for each segmental unit face is assessed as follows:

$$F^*_h - \sum_{j+1}^n F^*_i \leq \phi V_u \quad (\text{A.20})$$

The ultimate shear capacity  $V_u$  is found as  $V_u = a_u + N (\tan \lambda_u)$

$F^*_h$  being the resultant of all factored horizontal disturbing forces including the factored active thrust  $F^*A(z)$ , from the sloping backfill as well as any horizontal forces as a result of concentrated loads at the wall top.

$F^*_i$  is the sum of the design tensile forces in the geosynthetic reinforcement layers above the interface considered.

BS 8006 points out that the structural integrity of the block facing can be attained by the virtue of shear connections between successive modular blocks (either by friction of the blocks and by connection pins, shear keys etc.)

This is also found in the NZ Report 239, in Various modal forms bring about shear connections between segmental units, such as:

- Built-in mechanical interlock (shear keys, leading/trailing lips, etc.);

- Mechanical connectors (pins or clips linking successive vertical courses of units);
- Flat interfaces: shear resistance due to friction between segmental units and
- Geosynthetic reinforcement and to friction between geosynthetic reinforcement and granular material filling unit cavities (for units with cavities).

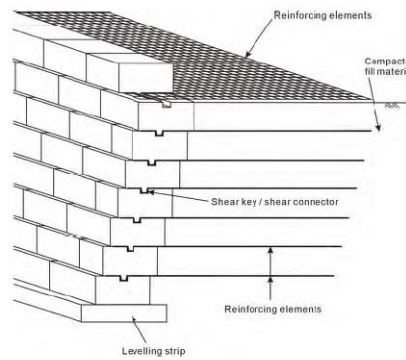


Figure A.11. Shear key connector on segmental wall from Transfund New Zealand (2003).

### A1.6. Unreinforced Crest Failure

NCMA stipulates a requirement that the Segmental Reinforced Wall (SRW) units above the highest reinforcement placement elevation needs to perform as a free standing retaining wall. In doing so, it needs to withstand overturning or sliding of the units near the wall crest.

By predesignating the toe to be at the highest reinforcement, the resisting moment is calculated as:

$$M_R = W_w X_w + P_{sV}(W_u + Y_s \tan \omega) + P_{qdv}(W_u + Y_q \tan \omega) \quad (\text{A.21})$$

The dimensions, subsequently collated, are based on the following schematic:

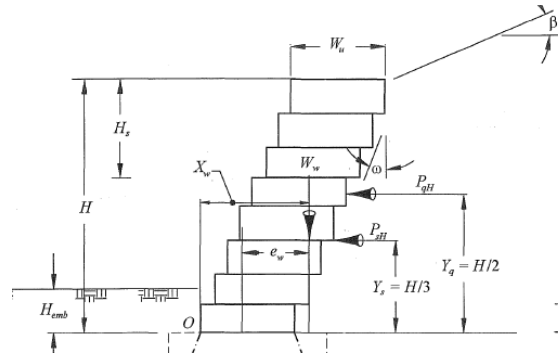


Figure A.12. Dimension spans from (National Concrete Masonry Association (2010)).

$$X_W = G_u + 0.5H \tan(\omega) - 0.5\Delta_u \quad (\text{A.22})$$

Where the driving moments as a result of the horizontal component of the earth pressure forces are:

$$M_o = P_{sH} Y_s + P_{qH} Y_q \quad (\text{A.23})$$

$$Y_s = H/3$$

$$Y_q = H/3$$

which in turn gives a factor of safety for overturning about the toe as:

$$FS_{ot} = Mr/M_o \quad (\text{A.24})$$

A factor of safety of 1.5 would require the maximum unreinforced height to be lowered.

With regards to sliding, the sliding of the underlain segment resistance is:

$$R_{sc} = \mu_b (W_w + P_{sY} + P_{qdV}) \tan \phi \quad (\text{A.25})$$

Where  $W_w$  is the weight of the column of SRW units being:

$$W_w = H \gamma_u W_u \quad (\text{A.26})$$

And  $\mu_b$  as the masonry friction reduction factor

A target minimum factor of safety of greater of 1.5 is sought which is accomplished by adjusting the unreinforced height of the wall H accordingly:

$$FS_{sl} = R_{sc} / P_{ah} \quad (\text{A.27})$$

with  $P_{ah}$  being the total horizontal active earth force acting on the back of the wall units, calculated as:

$$P_{ah} = P_{sH} + P_{qdH} + P_{qlH} \quad (A.28)$$

$P_{sH}$  is the horizontal active earth force due to soil self-weight,  $P_{qdH}$  and  $P_{qlH}$  are the surcharge loads (dead and live) loads acting on the surface.

### A1.7. Local Overturning

With regards to NCMA, Local Overturning per Mechanism 4b

The NZ Guide No. 239 provides a schematic to conduct a local overturning analysis of the segmental facing column against local overturning. The driving forces come from the earth pressures imposed on the segmental facing column, and the restoring forces are that from the segments.

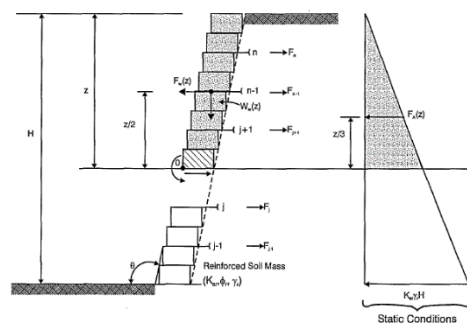


Figure A.13. Ultimate shear capacity calculation as per wall force actions from Transfund New Zealand (2003)

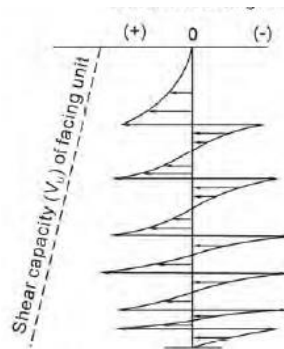


Figure A.14. Pressure distribution for overturning stability calculations from Transfund New Zealand (2003)

The equation is given as:

$$M_d^* \leq \phi M_r^* \quad (\text{A.29})$$

With  $M_d^*$  is the resultant of all driving moments; and

$M_r^*$  is the resultant of all restoring moments

Mechanism 4d: Block Sliding Failure

With regards to NCMA, Block Sliding done as per Mechanism 4b

Geoguide 6 explains the shear capacity between the facing units, vertical spacing of the reinforcement layers and the facing column weights are able to provide resistance against block sliding. The horizontal pressures being applied between the reinforcement layers must be resisted by sufficient shear capacity in all the facing units.

In carrying out the analysis, the lateral earth pressure is envisioned to act as a distributed load on a facing unit modelled as a continuously supported beam. A shear force diagram can be obtained in such a manner to find out the maximum shear forces occurring at the reinforcement elevations. These shear stresses are resisted by the facing column weight, reinforcement layer vertical spacing and the shear capacity between the facing units.

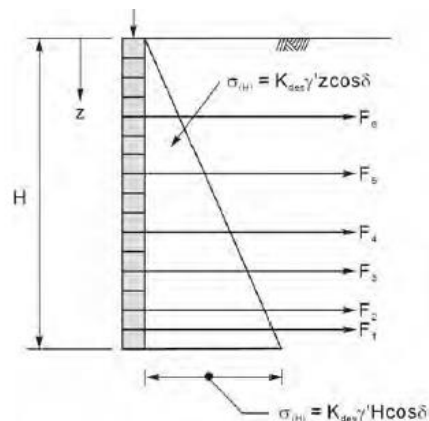


Figure A.15. Earth pressure distribution along wall depth

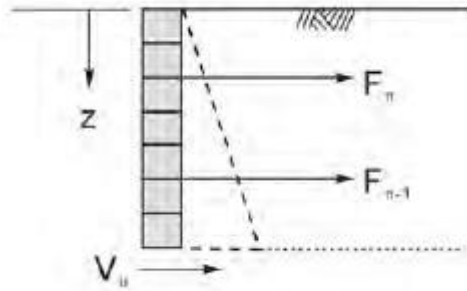


Figure A.16. Shear force diagram distribution to calculate F forces upon facing units

The available design shear capacity at a unit-to-unit and unit-to-reinforcement interface level is calculated.

$N_i$ : normal load per unit length across the  $i$ th level interface, which relates to the hinge height as follows:

$$N_i = H_h \gamma_u W_u \quad (\text{A.30})$$

$\gamma_u$  and  $W_u$  are the respective unit weight and width of the facing units.

$\lambda_{\text{des}}$ : design friction angle at the unit-to-unit or unit-to-reinforcement interface

Block sliding failure is prevented when the design shear capacity at an interface ( $V_{iD}$ ) must equal or exceed the maximum required shear force ( $V_i$ ) at that interface:

$$V_{iD} = V_i \quad (\text{A.31})$$

EBGEO contrasts from the previous standards in how it treats the Earth Pressure Coefficient adopted. It firstly considers the facing elements classified as rigid in that the blocks or panels have low compressibility and high flexural stiffness. The magnitudes of the horizontal pressure in turn would be critically influenced by the facing deformability as well as the composite properties of the reinforced earth structure, which does not make it necessary to adopt active earth pressure for the full height of the GRS structure.

For both the GRS structure and the surrounding ground, there must be an examination on the deformations manifested, particularly prevent any form of vertical differential settlement between the facing elements and the reinforced earth structure (which can occur from the differences in the facing and the reinforced earth structure supports, or inadequate compaction behind the facing). The facing's horizontal movement for which the design is based on shall be guaranteed for the entire height.

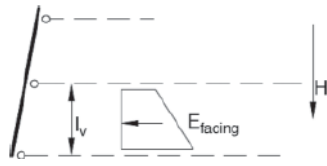


Figure A.17. Earth pressure distributions between reinforcement spacing from (German Geotechnical Society (2010))

$$E_{\text{facing}} = e_{\text{facing}} \cdot l_v \quad (\text{A.32})$$

$$e_{\text{facing}} = \text{Permanent and Live } K_a$$

$$R_{\text{bid}} \text{ or } R_{\text{aid}} > E_{\text{facing}}$$

This facing analysis is undertaken for each reinforcement layer.

### A1.7. Connection Failure

NCMA advises that the interface between the geosynthetic reinforcement and the modular unit at each reinforcement placement elevation needs connection strength adequate enough to prevent rupture or slippage of the reinforcement due to the applied tensile force. A full tensile load experiences on a reinforcement layer occurs at the intersection of the failure plane with the geosynthetic. This load in turn must be shed before reaching the facing. NCMA however adopts a conservative approach by treating the tensile to be fully exerted at the connection facing rather than at the failure plane intersection, and ignores any influence of the vertical component of the earth pressures. The limit state connection model in figure A18 below is used to determine the ultimate strength of the connection.

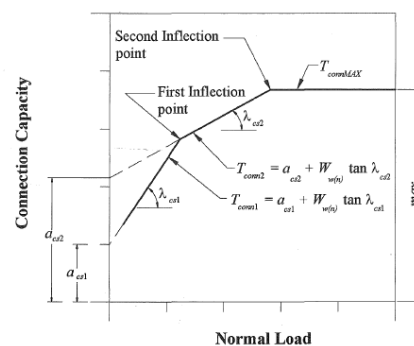


Figure A.18. Connection capacity versus normal load application

$W(n)$  connotes to the normal load action coming from the weight of the segmental units.

$\lambda$  as the apparent peak interface friction angle between the segmental units and  $a_{cs}$  as the apparent shear capacity adhesion.

The connection capacities  $T_{con1}$  and  $T_{con2}$  highlight the connection capacities above and below the failure zone.

It is worth noting on how NCMA treats the contribution of the connection to the failure surface. Generally, the location where the failure surface intersects with the facing causes a 100% connection contribution.

Below are the three cases adopted from NCMA where the failure surface may coincide with on the wall facing:

Case 1: Failure surface coinciding with the bottom block

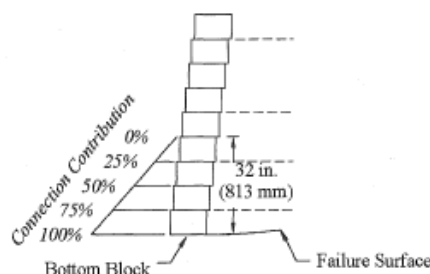


Figure A.19. Failure Surface intercepting with bottom block from National Concrete Masonry Association (2010).

Case 2: Failure surface coinciding with the reinforcement layer

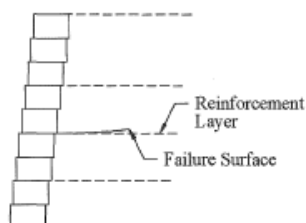


Figure A.20. Failure surface intercepting with reinforcement layer from National Concrete Masonry Association (2010).

Case 3: Failure surface coinciding between the reinforcement layers.

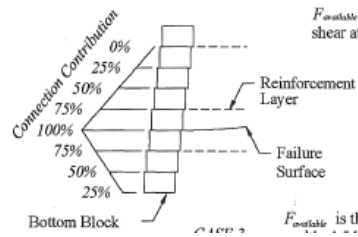


Figure A.21. Failure surface intercepting with Spacing between reinforcement layers from National Concrete Masonry Association (2010).

### A1.8. Bearing Capacity

Based on the block width, the GRS wall as a coherent body is designed externally to withstand bearing capacity. In doing so, the underlying foundation soil must meet the bearing capacity criteria (NCMA) such that the calculated bearing pressure  $Q_u$  applied must be greater than the allowable bearing pressure ( $Q_{allowable} = Q_{ult}/FOS$ ).

$Q_{ult}$  as per the classical soil mechanics principles, is given as:

$$Q_{ult} = c_f N_c + 0.5 \gamma_f B'_f N_\gamma + \gamma_f G_{emb} N_q \quad (A.33)$$

All the standards adopt the same procedure as NCMA

### A.1.9. Serviceability Limit States

#### (i) Ground and Differential Settlement

BS 8006, EBGeo and Geoguide 6 treats the reinforced earth structure and the surcharges imposed on it on the underlying ground as a flexible loaded area. Ground settlement effects warrants attention especially in cases where the underlying soil is soft. Consideration must be given to long term settlements arising from consolidation. FHWA 2 had explicitly stated that the vertical settlement is a prime cause for vertical settlement of the wall.

An interesting finding from BS 8006, Geoguide and EBGeo is that the reinforced retaining structure is more tolerable to settlement as oppose to the conventional retaining wall systems, and that the structure itself is able to compensate

the differential settlements generated. Geoguide however does stress upon the importance of investigating any effects of tilting and wall settlement as it can cause severe distresses on the wall. Overall, measures need to be undertaken to ensure the construction and post construction movements are within acceptable limits.

The British standard compared to EBGEO treats the total settlement being a confined effect of settlement from the foundation impacting as pressure on the soil along with the internal compression of the reinforced backfill, whereas the latter treats both manifested settlements separately.

However, the British standard does corroborate the advantage of the reinforced earth wall to withstand large settlement effects. Only in severe situations should further investigation on the internal settlement of the reinforced soil fill and differential settlement to be considered.

NCMA however instructs an assessment on the type, thickness and compressibility of the proposed foundation soils in all situations, especially faces an in excess during pre-construction activities. This is especially in saturated cohesive soils where these experience large, time-dependent settlements.

#### (ii) Intrinsic Settlement of Fill Material

The British standards and New Zealand guides instruct that the fill properties, the compaction, vertical pressures within the fill (which are a function of the structural geometry and the surcharge loading) impact the amount of intrinsic settlement occurring within the reinforced volume.

There are cases where the fill may settle relative to the facing which in turn will induce increased loading on the connections. This is especially prevalent with segmental blockwalls or in the use of non-sliding connections. The British standards BS EN 14475 2006 treats this as differential settlement between the reinforced fill and the facing panel. The kinematic restrictions of the connections between the reinforcement and facing panel would determine whether or not differential movement would occur. It is then that the deformations and differential movement that would affect what type of backfill to select and how it is compacted.

The German guide emphasizes that the intrinsic settlements of the fill soil occurs predominantly during the construction period. To investigate any settlements occurring within the soil from imposed surcharge, use of elastic analysis methods is recommended to predict them.

Geoguide 6 explains about the how the fill being deposited in layers whilst getting compressed causes the reinforcement elements move together along with the fill material as compaction occurs. Construction tolerances must be able to accommodate such internal deformations. Failure to do so will result in differential movement causing loss of serviceability or instability.

### (iii) Differential Settlement

Both earlier mechanisms can cause differential settlement longitudinally along the GRS length. The British standard highlights that the most severe impact on any form of structural functioning is from differential settlement, and it is the facing that dominates the permissible tolerances to differential settlement. Geoguide advises to incorporate special slip joints in the event where large differential settlements are anticipated. The anticipations are prevalent when the foundation material is likely to be variable, or when compressible spots are present.

Alternatively, the British standard recommends using the maximum tensile force acting on the reinforcement length when designing the connection to counteract differential settlements between the facing and the inclusions, whether the Tie Back Wedge method or Coherent body method is used.

NCMA provides explicit dimensional tolerances for the GRS wall. For example,

Vertical Control: 32 mm over a 3 m distance

Horizontal control: 32 mm over a 3 m distance

Rotational control: established wall batter  $\pm 2^\circ$

Frontal and Horizontal Displacement of the Slope Front

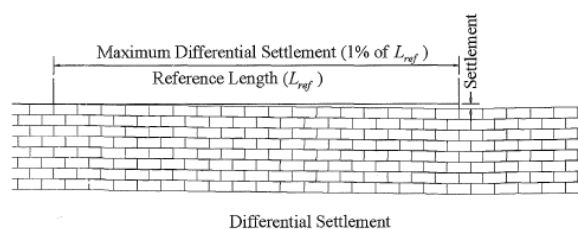


Figure A.34. Differential settlement in accordance with reference length from Association (2010)

EBGEO asserts the intricacy behind accurately determine the deformation behaviour of a composite structure for frontal and horizontal displacement (refer to figure A22), and that it can only be approximately described.

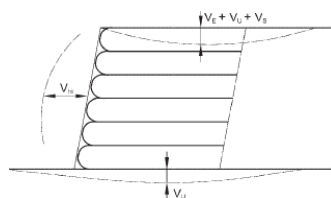


Figure A.35. Frontal displacement schematic from Geotechnical Association Society (2011)

There was no pronounced plastic deformation that may occur in the soil within the service load range, let alone any form of sliding occurring between the ground and the reinforcement. There is a linear increase in the strain developing the reinforcement as the load is increased. On this basis, the amount of facing displacements occurring is derived from the changes of length in individual reinforcement layers within the service load range. The linearity behaviour will also require knowledge on the load-extension behaviour of the reinforcement materials when immersed with the ground.

To estimate the horizontal facing displacements, one must analyse the tensile forces and their distributions in all reinforcement layers for Serviceability Limit State, followed by the axial stiffness' of the reinforcement layers.

This will enable the strain distributions for all reinforcement layers to be determined, and once integrated; the change of length of each layer is obtained.

Important determinants required for estimating the horizontal facing displacements include:

- Tensile force analysis and their distribution in all reinforcement layers for serviceability limit state.

- Determination of the associated axial stiffnesses of the reinforcement layers
- Determination of the strain distributions of the reinforcement layers
- Integration of the strains in all reinforcement layers in order to determine the length changes

The NZ Report No. 239 highlights that assessments have been developed empirically for determining the horizontal displacements, but not theoretical derivations. The key factors that were found to influence lateral displacements include compaction intensity, reinforcement to soil stiffness ratio, length of reinforcement, connections to the wall face and the facing system deformability.

FHWA 2 dictates that the major factors affecting lateral displacement are compaction intensity, reinforcement:soil stiffness ratio, length of reinforcement, lack in reinforcement connections at wall face and deformability of facing system. However, FHWA 2 asserts that the horizontal displacements are significantly much smaller than the vertical displacements.

FHWA 3 specifies that the fill material and the fill elements need to be chosen such that the post construction lateral deformation is ensured to be limited. Therefore, soil types of fine-grained and open-graded course aggregates with a maximum grain size of 2 inches (5.5 cm).

#### Resulting Surface displacement from Shear Deformations

NCMA asserts how strains in the reinforcement layers being produced as a result of achieving equilibrium conditions within the reinforced retaining structure, would consequently cause shear deformations to materialise. Any horizontal displacements occurring within the reinforcements would manifest to vertical displacements occurring in the surface. The shear deformations contribute to 30-50% of the frontal displacement.

BS 8006 points out that shear connections are adopted between the blocks by using shear key enhancements, pins etc. Only this time the block facing is seen to have a structural component (contrary to some of the research carried out by Wu in Chapter 8). However, it does not omit that the friction between the blocks is also a contributable factor.

### A1.10. Post-Construction Activity

The Post-Construction activities will be addressed based on three categories:

- (i) Reinforcement Inclusions
- (ii) Reinforced Soil
- (iii) Foundation Fill

#### (i) Reinforcement Inclusions

Polymeric reinforcements can be subject to internal creep strain from prolonged loading, which in which the strain build-up require attention that was not taken into consideration during the end of the construction period (which the period in which its durability is considered is reflected in the 'End of Design Life' period). The prolonged creep would reduce the stiffness of the reinforcement inclusion and would thereafter not work at its full predetermined capacity.

BS 8006 assigns for permanent structural loading and transient live loading on retaining walls to have a strain limit of 0.5% and 1% respectively.

#### (ii) Reinforced Soil

Isochronous curves are used to quantify the strain occurring at the end of the design life and the end of construction, if a capping value of a prescribed post-construction strain limit is provided.

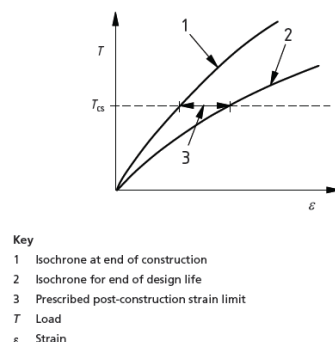


Figure A.36. Isochrones for end of Construction and end of design life from British Standards Institution (2012)

Such creep strain associated with such reinforcement inclusions however is limited due to the strain hardened backfill used for the reinforcing soil (as long as the reinforced fill is as prescribed in Geoguide 6). However, this is ignored in design which gives an overly conservative design value allocation.

Geoguide 6 emphasises on considering the ambient operating temperatures in the case whether these temperatures coincide with the visco-elastic phase, which causes creep to be an important design factor in the long-term load-carrying capacity.

Saturated fine grained soils, as per BS 8006, can be subjected to internal creep strain when used as the reinforced soil.

BS 8006 advises the use of well-graded granular material which will provide long-term durability for the reinforcing elements and stability during construction. It is not perceived for any internal yielding can materialise as these granular fill materials under normal working stress and post construction stress ranges would behave elastically (although this contradicts the works of Jewell under Chapter 3). A well-graded granular material also has the advantage of sought-after properties for enabling proper compaction, which as according to BS 8006, is the most advantageous condition to optimise the fill-reinforcement interaction.

There are however implications of using fine-grained fill materials as these are difficult to compact with the high moisture contents associated with the heavy rainfalls. These also exhibit elasto-plastic, or plastic behaviour, which consequently leads to post-construction movements.

On a further note, plastic deformations at post-construction can occur when the soil is poorly or gap-graded.

AS 4678 assesses special attention is needed for backfill soil which has shrink/swell characteristics which can have shrink/swell movements associated with them.

## (ii) Foundation Fill

The NCMA instructs to make an assessment of the maximum total and differential settlement after wall construction, especially if collapsible soils are present

within the vicinity of the GRS structure. Large, time-dependent deformations will be present in saturated cohesive soils, and by adopting the conventional one-dimensional consolidation theory, an estimate for the potential settlements can be obtained.

Geoguide 6 also points out the importance of conducting a consolidation settlement analysis when the foundation soil contains a compressive layer.

Emphasis of settlement for post-construction is more profound for cohesive soils than for cohesionless soils, as the latter experiences settlement during construction.

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