REINFORCED EARTH RETAINING WALL DESIGN USING REINFORCEMENT STEEL WITH GABION FACING

A THESIS PRESENTED TO SCHOOL OF GRADUATE STUDIES ADDIS ABABA UNIVERSITY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN CIVIL ENGINEERING

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OCTOBER, 2006

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Acknowledgement

This study was conducted under the supervision of my Advisor Professor Alemayehu Teferra, Faculty of Technology, Addis Ababa University.

I would like to express my gratitude to my Advisor Professor Alemayehu Teferra for his guidance and help throughout the study period. Acknowledgment is also due to Dr. Dereje Hailu Head of the Civil Engineering Department, for his advice and help throughout the study. My appreciation and thanks go to my colleague Fekede Wakuma for assisting me in technical matters, providing me with relevant materials and also for his valuable suggestions, during the study period.

Finally I would like to thank my family and friends for their encouragement throughout the study.

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List of Symbols (Notations)

γ_b = Bulk unit weight of back fill soil [KN/m^3]

ρ_b = Bulk Density [gm/cm^3]

ϕ = Angle of internal friction of the backfill soil [deg.]

ϕ_f = Angle of internal friction mobilized between reinforcing element and the backfill soil [deg.]

μ = co- efficient of friction between the backfill soil and reinforcing element

σ_n = Normal stress to the reinforcement [KPa]

σ_h = Lateral horizontal pressure from the backfill soil[KPa]

σ_a=Active tensile stress in the reinforcement. [KPa]

K_a = coefficient of active earth pressure

L_t = Total length of the reinforcement[m]

L_e = Effective length of the reinforcement[m]

L_r = Non-effective length of the reinforcement[m]

L_d = Anchorage length of the reinforcement[m]

T_i = Tensile force in the i^{th} reinforcement [KN]

F_t = Factor of safety against tension failure.

F_p = Factor of safety against pullout failure.

F_s = Factor of safety against sliding failure.

F_o = Factor of safety against overturning failure.

M_o = Over turning moment [KN-m}

M_r = Resisting moment [KN-m}

S_v = Vertical spacing of the reinforcement[m]

S_h = Horizontal spacing of the reinforcement[m]

FHWA = Federal Highway Administration

ASCE = American Society of Civil Engineers

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ABSTRACT

This paper presents reinforced earth model retaining wall made of reinforcement steel and gabion facing. Four model reinforced earth retaining wall were designed and constructed at selected site in Technology Faculty (North) campus. Rigid walls were provided at the sides and rear face to simulate the plain strain conditions prevailing in retaining walls. A gabion was used as facing to keep the backfill soil from flowing. Reinforcement steel was used as reinforcement with vertical spacing of 40cm and horizontal spacing of 36cm. Sand with different relative densities was used as backfill and a surcharge for each model wall. The surcharge was applied over the model wall step by step and deformation of the reinforcement was measured under different surcharge load until certain group of reinforcement observed to fully mobilize their frictional resistance. To identify the level of loading at which frictional resistance was fully mobilized, a graph of applied surcharge load versus wall face displacement was plotted and carefully observed the level of loading at which frictional resistance was fully mobilized. This level of loading was taken as ultimate load that the particular reinforcement steel can carry. Based on the observed ultimate load, the coefficient of friction between the backfill soil and reinforcement was determined. It is this coefficient of friction which will be used for the design of actual retaining wall.

INTRODUCTION

Soil is an abundant construction material which has compressive strength and no tensile strength. To overcome this weakness, the soil can be reinforced with materials with high tensile strength. The basic principle of earth reinforcement is the generation of frictional resisting force between the back fill soil and the reinforcing element. The frictional force is directly dependent on normal effective stress and mobilized angle of internal friction between the soil and reinforcement. For walls reinforced with longitudinal reinforcement, the anchorage strength, or pullout resistance, is a result of the shear strength along the interfaces between the reinforcement and backfill soil along the longitudinal reinforcement. Reinforced earth retaining wall is composed of back fill soil, facing (skin) and reinforcing element. The reinforcing element can be geotextile, metal strips rods, bars, etc. This paper comprises three parts. In part one (chapter 1), covers objectives of the research work Part two (chapter 2) literature review and back ground development of reinforced earth will be discussed. Part three (chapter 3, 4, 5, 6, 7 and 8) deals with model wall studies. Finally based on the result of analysis, conclusion and recommendation will be provided.

1. OBJECTIVE OF THE RESEARCH WORK

Reinforced earth wall have gained substantial acceptance as an alternative to conventional masonry and reinforced concrete cantilever retaining wall structures due to its simplicity, rapidity of construction, less site preparation and space requirement for construction operation. In addition to technical and performance advantages, another primary reason for the acceptance of reinforced earth retaining wall has been its inherent economy. It is reported that, reinforced earth retaining structures, beside its outstanding performance, a cost saving of up to 30% to 50% below alternate solutions have been achieved. Seismic loading, differential heave and settlement requirements make rigid masonry and concrete cantilever walls very difficult to achieve the desired safety factor. Whereas, reinforced earth system when subjected to seismic loads and differential earth movement has shown exceptional performance due to its flexibility and inherent energy absorption capacity. Even though reinforced earth is widely used in different parts of the world, it has not been well introduced in our country. Hence, it requires detail study to adapt this technology for locally available material. This thesis aims at evaluating and verifies the performance of reinforced earth walls through experiments on model reinforced earth retaining wall constructed from reinforcement steel with gabion facing.

2 FUNDAMENTAL PRINCIPLES AND DEVELOPMENT OF REINFORCED EARTH STRUCTURES

2.1 Development of Reinforced Earth Structures

The Concept of reinforcing soil with tensile members is not new. Dikes constructed from earth and tree branches, have been used in China for at least 1000 years. In England wooden pegs, bamboo and wire mesh have been used for erosion and land slide control. [3] In the modern context, reinforced soil began to be used during the early 1970's where, firstly steel strips reinforcement and later, geotextiles reinforcement were used in the construction of reinforced soil walls for slope stabilization. The present concept of systematic analysis and design of reinforced earth was first developed by a French Engineer, Henri Vidal in1966 and later on, numerous works have been done by Darbin in 1970, Schlosser and Long in 1974, and Schlosser and Vidal in1969 on the use of metallic strips as a reinforcing material. Reinforced earth retaining walls have been constructed around the world since Vidal started his work. The first reinforced earth retaining wall with metal strips as reinforcement was constructed in 1972 in USA in the San Gabriel Southern California. [1]

The use of geotextiles in soil reinforcement started in 1971 in France after their beneficial effect was noticed in the construction of embankments over weak sub grades. The use of geogrids for soil reinforcement was developed around 1980 [3]. By placing tensile reinforcing elements in the backfill soil of reinforced earth wall, the strength of the soil is improved. With the addition of a facing system, very steep slopes and vertical walls can be constructed safely.

The US Department of transportation in its technical report FHWA-SA-96-038, published in 1997 [3], describes a segmental precast facing mechanically stabilized earth as follows. "The system employs metallic (strip or bar) or geosynthetic (geogrid or geotextile) reinforcement that is connected to a precast concrete or prefabricated metal facing panel to create a reinforced soil mass. The reinforcement is placed in horizontal layers between successive layers of granular soil backfill. Each layer consists of one or more compacted fills. For walls reinforced with geogrids, the anchorage strength, or pullout resistance, is a result of three different mechanisms. The first is shear strength along the top and bottom interfaces between the geogrid and soil along the longitudinal ribs. The second is the shear strength along the top and bottom interface between the geogrid and soil along the transverse ribs. The third is the passive resistance against the front face of the transverse ribs. In this last mechanism, the soil goes into a passive state and resists pullout by means of bearing capacity."

2.2. Fundamental principles of reinforced earth structures

The fundamental principle of reinforced earth system lies in the mobilization of the shearing strength of the soil. The stability of a soil reinforced system is derived from the composite action of the horizontal reinforcing element which extends in the reinforced soil mass elements providing tensile strength to the soil mass. A retained earth wall system can be thought as a gravity dam supporting itself and the earth behind it. It is very important to understand the basic concept of soil reinforcement, as well as identify the types of applications where reinforced soil is of particular advantage. The soil must deform in shear before failure along slip surface occurs. For a slip surface to develop, the driving forces causing failure in the soil mass must overcome the frictional shearing resistance. The Shear deformation in the soil causes compressive and tensile strain. The reinforcement is placed in the direction of tensile strain so that deformation in the soil generates tensile force in the reinforcement. Consequently, the generated tension force in the reinforcement, improve the stability of the soil mass by increasing the shear resisting force [4]. Studies have indicated that an active failure wedge, similar to that used in coulomb or rankine's lateral earth pressure theory will satisfactorily apply, and maximum tensile forces of reinforcement are well predicted by assuming a rankine active earth pressure condition that they are proportional to the overburden pressure and located approximately at the point along the potential failure surface..

Consider a sliding wedge of Fig.1, the sliding wedge for unreinforced slope condition,

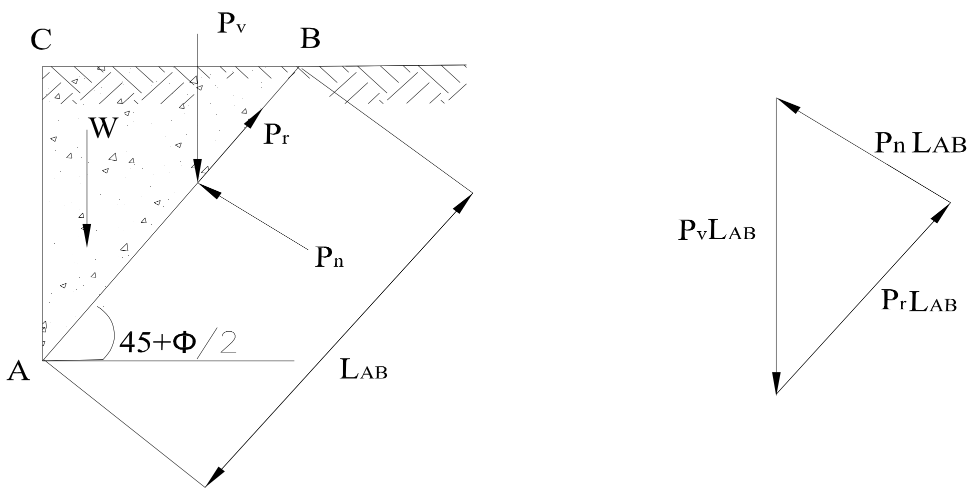


Figure 1: Sliding wedge for unreinforced slope

Where, P_r = Average resisting stress along the slip surface

P_n = Average normal stress perpendicular to the slip surface

P_v = Average vertical stress over the slip surface

W = Weight of the sliding wedge.

L_{AB} = Length of the slip surface of sliding wedge

The component of the applied disturbing force Wsin $(45 + \phi/2)$, is purely resisted by the frictional resistance generated along the slip surface in the soil which is $P_n \tan\phi \, L_{AB}$.

The total shearing resistance of the sliding wedge is:

$$P_n L_{AB} \tan\phi = P_r L_{AB}$$

$$P_r L_{AB} = W\cos(45+\phi/2)\tan\phi \qquad (1.1)$$

Again consider a sliding wedge of Fig. 2, the sliding wedge for reinforced slope condition.

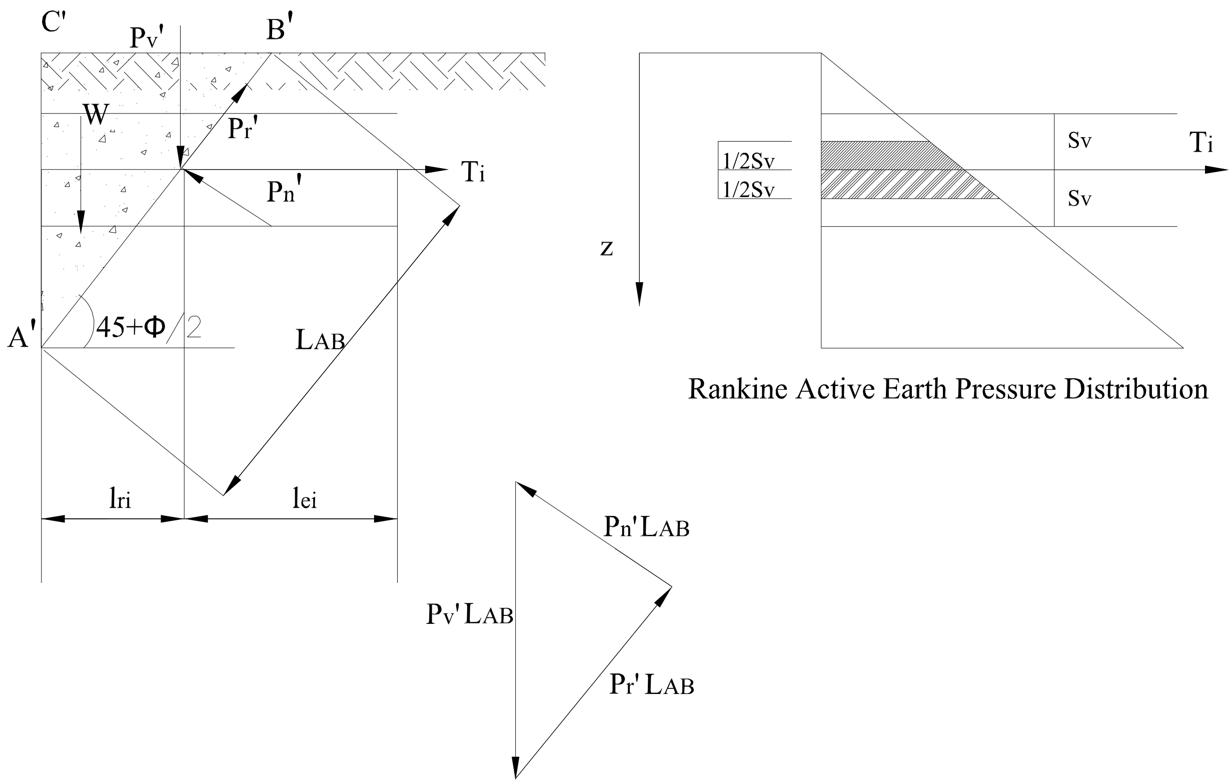


Figure 2: Sliding wedge for reinforced slope

Where, P_r' = Total average resisting stress along the slip surface

P_n' = Total average normal stress perpendicular to the slip surface

P_v' = Total average vertical stress over the slip surface

W = Weight of the sliding wedge.

T_i = Reinforcement tension force at layer (i)

L_{AB} = Length of the slip surface of sliding wedge.

Here, in addition to the acting forces on sliding wedge of Fig. 1, two additional forces can be observed.

a) The component of the reinforcement tension force T_i along the shear surface would be $T_i \cos(45+\phi/2)$, which is contributing to the resisting force.

b) The component of the reinforcement force T_i perpendicular to the shear surface would be $T_i \sin(45+\phi/2)$, which increases the compressive force over the slip surface, and there by increasing the frictional shearing resistance. Hence, the disturbing force $W \sin(45+\phi/2)$ is resisted by the sum of frictional resistance in the soil which is $P_n' L_{AB} \tan\phi$ and the component of reinforcement tension force $\Sigma (T_i \cos(45+\phi/2))$ along the shear surface.

$$P_n' L_{AB} = P_n L_{AB} + \Sigma T_i \sin(45+\phi/2)$$

$$\text{Resisting force } (P_r' L_{AB}) = (P_n' L_{AB}) \tan\phi + \Sigma (T_i \cos(45+\phi/2))$$

$$P_r' L_{AB} = (P_n L_{AB} + \Sigma T_i \sin(45+\phi/2)) \tan\phi + \Sigma (T_i \cos(45+\phi/2))$$

$$P_r' L_{AB} = P_n L_{AB} \tan\phi + \Sigma (T_i (\cos(45+\phi/2) + \sin(45+\phi/2) \tan\phi))$$

$$P_r' L_{AB} = W \cos(45+\phi/2) \tan\phi + \Sigma (T_i (\cos(45+\phi/2) + \sin(45+\phi/2) \tan\phi)) \tag{1.2}$$

These concepts lead to the following details regarding reinforced soil.

a) The main difference between unreinforced and reinforced soil is the addition of the term Σ (T_i (cos (45+ϕ/2). +sin (45+ϕ/2) tanϕ)) which increases the shear resisting force in reinforced earth structures

b) The reinforcement has to remain in equilibrium with the surrounding soil, and must bond sufficiently to transmit the required reinforcement force to the soil.

c) Major tensile strain develops in a horizontal direction in soil when the major loading is due to gravitational forces. Hence, reinforcement is usually placed in horizontal layers because the reinforcement works best in tension. The tensile strain in the soil mobilizes tensile force in the reinforcement, and the reinforcement force acts to reduce the forces which cause failure in the soil and increase the forces which resist failure.[5]

2.2.1 **Designing Principle of reinforced earth structures**

Reinforced earth retaining walls are retaining structures constructed of a facing and selected fill material that is stabilized with embedded reinforcement elements. The interaction of the backfill material and the reinforcement form a flexible, coherent block that can sustain significant loads and movements. The main objective of designing reinforced earth is to determine the layout and required tensile strength of the reinforcements. The existing design methodology of reinforced earth wall is based on internal and external stability analysis using a modified version of classical limit equilibrium slope stability methods in which reinforcement tension forces are introduced in the calculations. The required forces for equilibrium depend on soil geometry, shearing resistance, and applied load. The central concept in designing reinforced soil is to calculate the required forces to maintain stability with a margin of safety consistent with existing geotechnical engineering practice. In order to estimate the factor of safety against internal stability using these methods, the distribution of the reinforcement tensile force with height must be assumed. A linear distribution of reinforcement tension force with height, with zero tension at crest and maximum peak tension at toe of the structure has often been assumed. The stability analysis of reinforced slopes needs additional assumptions on top of those already made for the analysis of unreinforced slopes. These additional assumptions include the inclinations (orientation), distributions (spacing) of the reinforcement and peak tensile force along the potential failure surface.

Current design methods for reinforced soil are based on vertical wall and uniform horizontal reinforcements with the assumption that reinforcement peak forces are proportional to the overburden pressure, as measured from the top of the wall. Studies have indicated that maximum tensile forces are well predicted by assuming a rankine active earth pressure condition [6]. The principal components of reinforced earth retaining wall are indicated in Fig. 3.

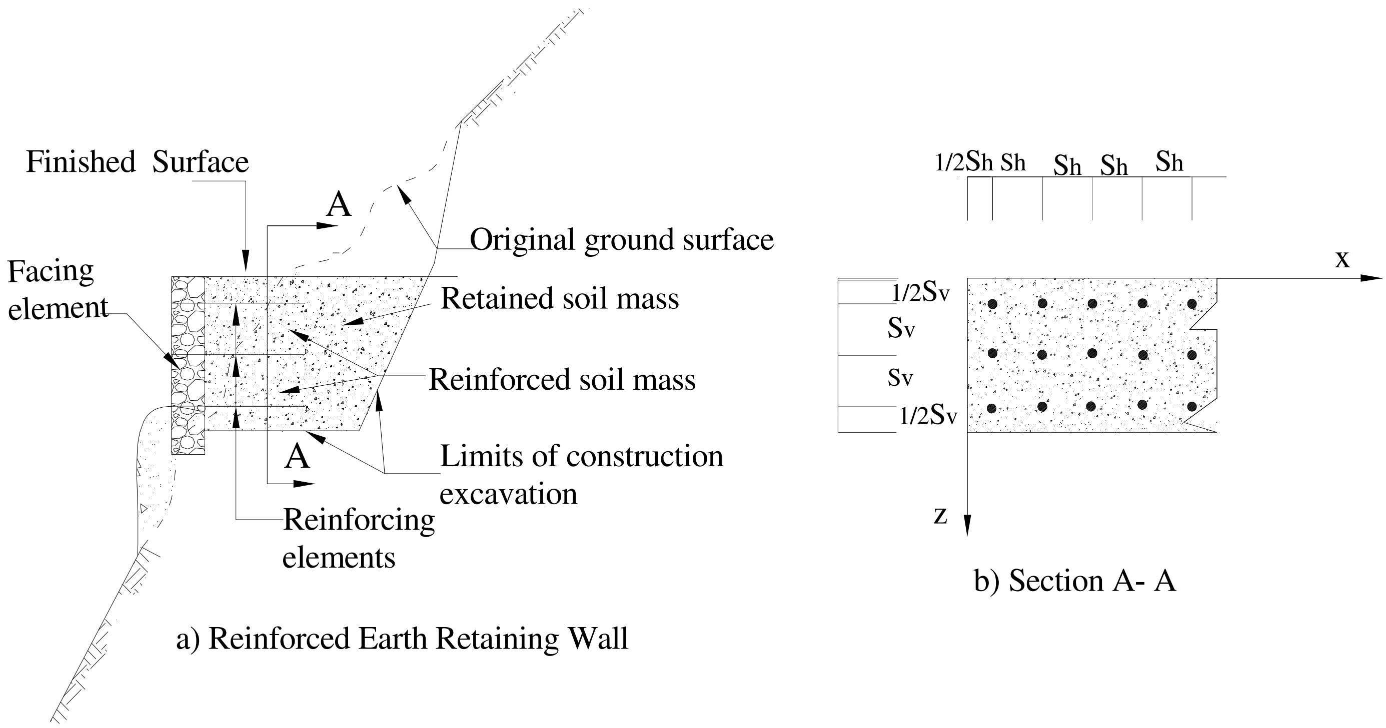


Figure 3: Principal components of reinforced earth retainaining wall

The main component of reinforced earth retaining wall includes:

3. Reinforcing element

3. Granular backfill soil

3. The facing (skin) element

At any depth the reinforcements are placed with a horizontal spacing of S_h and vertical spacing of the reinforcement S_v center - to- center.

According to Rankine active earth pressure theory, for granular soils with no surcharge load

$$\sigma_a = \sigma_v k_a \qquad (1.3)$$

$$\sigma_v = \gamma_b z \qquad (1.4)$$

$$k_a = \tan^2(45 - \phi/2) \qquad (1.5)$$

Thus;

$$\sigma_a = \gamma_b z k_a \qquad (1.6)$$

Where; σ_a = Rankine lateral active earth pressure at any depth z_i.

ϕ = Angle of internal friction of the backfill soil.

σ_v = Total vertical pressure at any depth z_i.

k_a = Rankine earth pressure coefficient.

γ_b = Bulk unit weight of the backfill soil.

If a surcharge is added at the top of the retaining wall, the vertical pressure due to surcharge will be added to the pressure due to backfill

$$\sigma_v = \sigma_{v_1} + \sigma_{v_2} \qquad (1.7)$$

Where

$\sigma_{v_1} = \gamma_b z$, Vertical pressures due to backfill soil

$\sigma_{v_2} = q$, Vertical pressure due to the applied surcharge load

$$\sigma_a = (\sigma_{v_1} + \sigma_{v_2})\, k_a \qquad (1.8)$$

Reinforced earth retaining walls reinforced with bars or strips have reinforcement ties at depth (z) from top and horizontal distance (x) from one edge of the wall respectively (Fig.3b).

Where:

$$z = \frac{1}{2}s_{v,}\ 1\frac{1}{2}s_{v,}\ 2\frac{1}{2}s_{v,} \ldots N\frac{1}{2}s_{v} \qquad (1.9)$$

$$x = \frac{1}{2}s_{h}\ 1\frac{1}{2}s_{h}\ 2\frac{1}{2}s_{h} \ldots N'\frac{1}{2}s_{h} \qquad (1.10)$$

s_v = Vertical spacing of the reinforcement

$N =$ Total number of reinforcement layers

$s_h =$ Horizontal spacing of the reinforcement

$N' =$ Total number of reinforcement used per each layer

The selection of the trial values of vertical spacing (s_v) and horizontal spacing (s_h) are based on assumption that every reinforcement at all levels should safely support the lateral active pressure force (P_{Ai}), acting over the area $(s_v) x (s_h)$, of the retaining wall. i.e. at any level, the active lateral pressure acting over the area $(s_v) x (s_h)$ is assumed to be resisted by tension force (T_i) developed in each reinforcement.

$$P_{Ai} = T_i = (q + \gamma z_i) k_a s_v s_h \qquad (1.11)$$

$$F_r = (q + \gamma z_i) p_r l_e \tan\phi_f \qquad (1.12)$$

Where;

F_r = The frictional resisting force

p_r = The perimeter of individual reinforcement

l_e = Effective length of the reinforcement

ϕ_f = Angle of mobilized friction between the backfill soil and reinforcement

The friction angle ϕ of the back fill soil will be determined in laboratory using direct shear test. At ultimate loading condition, the tension force (T_i) developed in each reinforcement bar should be equal to the frictional resisting force (F_{ri}) developed between each reinforcement and backfill soil.

$$F_{r_i} = T_i \quad (1.13)$$

$$(q+\gamma z)\, p_r l_e \tan\phi_f = (q+\gamma z_i)\, k_a s_v s_h \quad (1.14)$$

The only unknown variable in the above equation is the coefficient of friction between the reinforcement and the back fill soil which can be determined from the above equation

$$\tan\phi_f = \mu = \frac{s_v s_h k_a}{p_r l_e} \quad (1.15)$$

The value of equation (1.15) is called coefficient of friction (μ) between the reinforcement and the backfill soil.

2.2.1.1 Stability Analysis of Wall

The stability analysis of reinforced earth walls is based on internal and external stability analysis using limit equilibrium methods which is essentially based on the analysis of the following potential failure mechanisms and the corresponding factor of safety are determined and compared to acceptable values.

2.2.1.1.1 Internal Stability Analysis

Design of reinforced earth retaining structures for internal stability involves the determination of the required tensile and pullout resistance of reinforcing elements of each layer needed to ensure the reinforced mass is safe against internal collapse due to its own weight and applied surcharge load. Internal stability analysis is based on the assumption that the most critical slip surface will develop through the reinforced soil. The possibility of collapse with in the reinforced soil mass is due to insufficient strength or embedment length of the reinforcement [7].

The frictional resistance at the soil-reinforcement interface should be known and the load induced to the soil at a point must be transferred to the reinforcing element. In order to determine the shear strength at soil-reinforcement interface several tests have been devised, some of which includes: Pullout test, large scale shear test and direct shear test.

Getu Lemma, in his research work for M. Sc. Thesis on Reinforced earth retaining wall using geotextiles, had conducted an experiment to determine the interface friction angle (ϕ_f) (Pullout resistance) in 2000. He followed a modified direct shear test and obtained average mobilized angle (ϕ_f) = 39.35^0. He also determined the average friction angle (ϕ) of the backfill material using convectional direct shear test as; (ϕ) = 45.45^0. Hence, the ratio of mobilized friction angle (ϕ_f) to the actual friction angle (ϕ) of the backfill materials (ϕ_f/ϕ) = 0.87 [8].

Later on, Nuriye Mohammed, in his research work for M. Sc. Thesis on Feasibility of Reinforced earth retaining wall using locally produced metal - strip, had conducted an experiment to determine the interface friction angle (ϕ_f) (Pullout resistance) in 2002. Here, he followed two approaches to determine the mobilized friction angle (ϕ_f). These were:

a) Direct Pullout test method, b) Modified direct shear test method and obtained average mobilized angle (ϕ_f) = 25.76^0.and (ϕ_f) = 39.06^0 for the direct pullout test and modified direct shear test respectively. He also determined the average friction angle (ϕ) of the backfill material using convectional direct shear test (ϕ) = 44.175^0. Hence, the ratio of mobilized friction angle (ϕ_f) to the actual friction angle (ϕ) of the backfill materials was (ϕ_f/ϕ) = 0.583 and (ϕ_f/ϕ) = 0.884 for the direct pullout test and modified direct shear test respectively [9]. Even though Getu's and Nuriye's experiment were done at different time on different material sample, the research were done on similar material as we can observe from the frictional angle (ϕ) of the backfill material determined by direct shear test. Similarly, in both research works with modified direct shear test, the value of the mobilized friction angle (ϕ_f) obtained had nearly similar value. But the mobilized friction angle obtained from Nuriye's work with direct Pullout test was different. Hence, it needs additional investigation on both methods to identify a better method for mobilized friction identification. However, only direct shear test was used to determine the friction angle (ϕ) in this research work, due to absence of the other facilities.

The three potential internal stability failure mechanisms specific to reinforced earth structures illustrated in Fig. 4 are: a) Breaking of the reinforcement. b) Reinforcement pullout and c) Facing Connection Failure

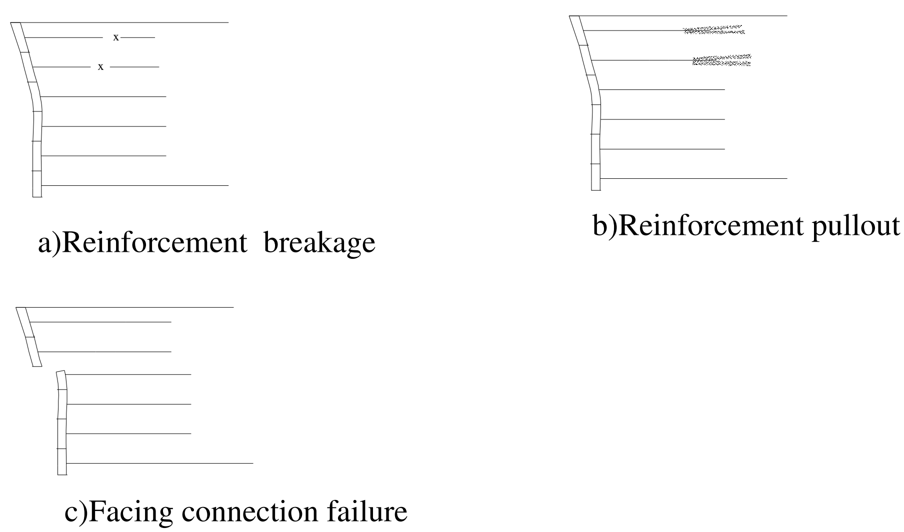


Figure 4: Potential failure mechanisms for intarnal stabilty of reinforced earth retaining wall.

Internal stability analysis yields the spacing and strength parameters of reinforcement necessary to ensure the integrity and internal stability of the reinforced soil. The internal stability checks are made at each level of reinforcement, and the most critical state defines the final length, spacing and strength parameters of the reinforcement. In most cases, the internal stability calculations control the total dimensions of the wall and the reinforcement characteristic [4].

a) Tension failure of the reinforcement

The check against reinforcement breakage is done at each level of reinforcement. It must ensure that the required tensile resistance of the reinforcement is less than its allowable strength. The tension force distribution, with in each reinforcement increases proportionally with lateral earth pressure (Fig. 2) from lightly loaded top layer to a maximum value at bottom layer. The tension force $(T_{z_{ii}})$ per individual reinforcement developed at any layer having a depth z_i below the top of the retaining wall is equal to the lateral active pressure at z_i multiplied by the area of the wall that must be supported by individual reinforcement[2].

$$T_{z_i} = \sigma_{v_i} s_v s_h k_a \tag{1.16}$$

The maximum Tension capacity (T_{max}) of individual reinforcement would be:

$$T_{max} = A_s f_s \tag{1.17}$$

Where: T_{max} = the maximum tensile force required to resist the active lateral active Pressure at the face of the wall;

A_s = Cross-sectional Area of individual reinforcement

f_y = Yield strength of the reinforcement

The factor safety (F_t) of the reinforcement against tension failure is given by:

$$F_t = \frac{\text{Resisting force in tension}}{\text{Tension force}}$$

$$F_t = \frac{A_s f_y}{\sigma_a s_v s_h k_a} \qquad (1.18)$$

b) **Pullout Failure of the Reinforcement**

The capacity of the reinforcement to develop the required tensile resistance depends on its pullout resistance. The pullout resistance of reinforcement is the result of friction or shear strength along the surface area of the embedded effective length of reinforcement. The factors affecting the pullout resistance of the reinforcement are; the type of the backfill soil, the reinforcing material properties, and the normal stress acting on it. A pullout failure of the reinforcement will occur if the frictional resistance developed along surface area of the reinforcement is less than the lateral active pressure to which the reinforcement is being subjected. The bond must resist the maximum tensile load carried by the reinforcing element. [2] The maximum frictional resisting force developed by each reinforcing element at depth z is given by:

$$F_p = \sigma_{vi} \tan \phi_f \, p_r l_e \qquad (1.19)$$

Where σ_{vi} = total vertical pressure at depth z_i

ϕ_f = coefficient of friction between the reinforcement and the back fill

p_{r} = perimeter of the reinforcement

l_e = effective length of the reinforcement placed at depth z_i

The factor of safety against pull-out failure (F_p) is given by:

$$F_p = \frac{Mobilized\ frictional\ resisting\ force}{Tension\ force}$$

$$F_p = \frac{\sigma_{v_i} \tan\phi_f P_r l_{e_i}}{\sigma_{a_i} s_v s_h} = \frac{P_r l_{ei} \tan\phi_f}{k_a s_v s_h} \qquad (1.20)$$

When the frictional resisting force is fully mobilized

i. e, $F_p = 1$

$$p_r l_{ei} \tan\phi_f = k_a S_v S_h \qquad (1.21)$$

$$l_{ei} = \frac{k_a s_v s_h}{p_r \tan\phi_f} \qquad (1.22)$$

From the assumed failure plane in Fig. 2, the embedment length (l_{ri}) of the reinforcement can be given by:

$$l_{ri} = \frac{h - z}{\tan\left(45 + \frac{\phi}{2}\right)} \qquad (1.23)$$

Combining equation (1.22) and (1.23) the total required length of reinforcement (l_t) at any depth is given by:

$$l_t = l_{ei} + l_{ri}$$

$$l_t = \frac{k_a s_v s_h}{p_r \tan \phi_f} + \frac{h - z}{\tan\left(45 + \frac{\phi}{2}\right)} \qquad (1.24)$$

c) Facing and Facing connection Design

The facing and facing connection design ensures that the forces at the connections can be sustained by both the reinforcement and the connection. The connection force in reinforcement is expressed by the active lateral earth pressure acting on the surface area of the wall. The most likely mode of failure is punching shear failure of facing and shearing of facing connection. The design of facing and facing connection involves calculation of the required allowable shear strength of facing and connection to maintain shear strength requirement with a margin of safety consistent with existing engineering practice. [4]

2.2.1.1.2 External Stability Analysis

External stability analysis of reinforced earth retaining wall is similar to the stability analysis of convectional (gravity and cantilever) retaining walls. Since the procedure for investigation of the external stability of convectional retaining walls has been well established for decades, the same procedure is used for reinforced earth retaining walls.

The first four failure mechanisms illustrated in Fig. 5 represents potential failure mechanism in external stability of the wall. It verifies whether the dimensions of the retaining wall ensure its global stability under the loads induced by the retained soil. The reinforced mass is considered as a solid block, and only failure surfaces through the adjacent retained soil are considered. The first four failure mechanisms pertain to the external stability of the wall: a) overturning, b) sliding, c) bearing capacity, and d) global stability for a given wall [3].

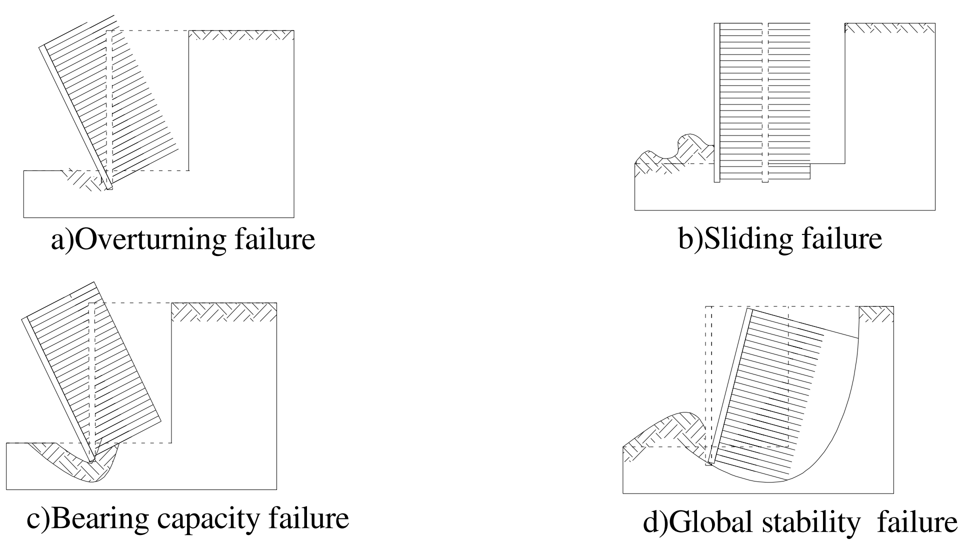


Figure 5: Potential failure mechanisms for external stability of reinforced earth retaining wall.

a) Overturning Failure

This is to ensure the stability of the reinforced earth retaining wall, against rotation about its toe. The factor of safety against over turning (F_o) is given by:

$$F_o = \frac{Resisting\ \ Moment(M_r)}{Overturning\ \ Moment(M_o)}$$

$$F_o = \frac{\frac{1}{2}\gamma_b h l^2}{\frac{1}{2} q h^2 k_a + \frac{1}{6}\gamma h^3 k_a}$$

$$F_o = \frac{\gamma_b l^2}{q h k_a + \frac{1}{3}\gamma h^2 k_a} \qquad (1.26)$$

Where

F_o = The factor of safety against overturning

h = height of the retaining wall

l = Length of the reinforced earth retaining wall

k_a = Rankine active pressure coefficient

γ_b = Bulk unit weight of the backfill soil

q = applied surcharge load

b) Sliding Failure

Since, the reinforced earth retaining wall acts as single gravity mass, the whole mass will tend to move horizontally. The frictional resisting force will be generated between the base of the wall and foundation soil which resists the active lateral earth pressure force. The factor of safety against sliding (F_s) is given by:

$$F_s = \frac{\text{Re}\,sistingForce}{DisturbingForce}$$

$$F_s = \frac{(q + \gamma_b h)\, l \tan\phi_f}{(qh + \frac{\gamma_b h^2}{2})k_a} \qquad (1.27)$$

Where h = height of the retaining wall

l = Length of the reinforced earth retaining wall

k_a = Rankine active pressure coefficient

γ_b = Bulk unit weight of the backfill soil

q = applied surcharge load

c) Foundation failure

Foundation failure includes Bearing foundation failure and deep (global) foundation failure. However, in most case the actual pressure under reinforced earth retaining wall is much lower than the bearing capacity of the foundation soil and hence foundation failure is not a common problem in reinforced earth retaining wall structures.

2.2.1.2 Preliminary Design of the model wall

In designing the model wall, first pullout test would have been done to determine the mobilized friction angle (ϕ_f) between the reinforcement and the backfill. However, due to the absence of the pullout test facility, it was not possible to carry out the pull out test. Hence, to determine reinforcement spacing and other necessary dimensions, the shear strength parameter (ϕ), mobilized friction angle (ϕ_f) between the reinforcement and the backfill soil were assumed.

Similarly, the maximum wall height (H), width (B) and effective length (l_e) were assumed as follows.

Friction angle of the backfill soil (ϕ) = 38^0

Mobilized friction angle (ϕ_f) = 50 %(ϕ)

Model wall height (H) = 1.8m

Width of the model wall (B) = 1.8m

Effective length (l_e) of reinforcement = 3m

The diameter of reinforcement = 10mm

From equation (1.11) the lateral pressure force (P_A) $= (q + \gamma_b z) k_a s_v s_h$

From equation (1.12) the frictional force (F_r) = $(q+\gamma_b z) p_r l_e \tan\phi_f$

Where, $F_r =$ the frictional resisting force

$p_r =$ The perimeter of individual reinforcement

$l_e =$ Effective length of the reinforcement

ϕ_f = mobilized friction angle between the backfill soil and reinforcement

ϕ = Internal friction angle of the backfill soil

k_a = Rankine earth pressure coefficient.

$$k_a = \tan^2(45 - \phi/2)$$

γ_b = Bulk unit weight of the backfill soil.

$\gamma_b z$, =Vertical pressures due to backfill soil

q = Vertical pressure due to the applied surcharge load

s_v = Vertical spacing of the reinforcement

s_h = Horizontal spacing of the reinforcement

At ultimate loading condition, the frictional resisting force (F_{ri}) developed between each reinforcements and backfill soil should be equal to the lateral pressure force (P_A)

i.e $F_r = P_A$

$$(q+\gamma_b z)p_r l_e \tan\phi_f = (q+\gamma_b z_i)k_a s_v s_h$$

$$p_r l_e \tan\phi_f = k_a s_v s_h \tag{1.28}$$

Substituting the assumed values in equation (1.28),

$P_r = 0.0314$m, $l_e = 3$m, ka = 0.238, $\tan\phi_f = 0.344$

$$s_v s_h = 0.136\text{m}^2$$

Let the vertical spacing (S_v) = 0.40m and hence the horizontal spacing (S_h) = 0.34m. Adjusting the horizontal spacing with assumed width of the model wall, the corrected horizontal spacing (S_h) = 0.36m.

Now use, the vertical spacing (S_v) = 0.40m

The horizontal spacing (S_h) = 0.36m

And to make the reinforcement mobilize their frictional resistance at lesser surcharge load, it is advisable to use shorter reinforcement. Hence, use total reinforcement length (l_t) = 3m for all layers

2.2.1.3 Generalized Flow Chart for Design Procedures of Reinforced Earth Retaining Wall Using Reinforcement Steel

From general known facts and result of analysis, the design procedures of reinforced earth retaining wall can be summarized as follows:

Step One

Proportion the Wall

- Determine wall height (m)
  - $\Rightarrow$ As required
- Determine applied load.
  - $\Rightarrow$ Expected applied load
- Determine reinforcement length and spacing
  - $\Rightarrow$ Trial value

↓

Step Two

- Determine geotechnical resistance parameters of backfill and foundation soil
  - $\Rightarrow$ Use Shear Strength Parameter (ϕ) from Laboratory Test Result.
- Determine Coefficient of friction between reinforcement and backfill
  - $\Rightarrow$ Take Mobilized Shear Strength Parameter (ϕ_f) as determined in experiment

↓

Step Three

Select the type of wall facing and facing/reinforcement connections

$\Rightarrow$ as required

Step Four

Check for internal stability

- **Tensile Failure of reinforcement**

$$\Rightarrow F_t = \frac{Tension\ capacity\ of\ \ reinforcement\left(A_s f_y\right)}{Generated\ \ tension\ \ force\left(\sigma_v s_v s_h k_a\right)} \geq 1.5$$

- **Reinforcement pullout Failure**

$$F_p = \frac{Mobilized\ \ frictional\ \ resisting\ \ \ f\square\square\left(\sigma_v \tan\phi_f\, p_r l_e\right)}{Generated\ \ tension\ \ force\left(\sigma_v s_v s_h k_a\right)} \geq \ \ 1.5$$

- **Facing connections failure**

  $\Rightarrow$ Check for shear strength requirement

If Step four is satisfied

- Yes, go to step five
- No, go to step one

Step Five

Check for external stability

- **Overturning of wall**

$$\Rightarrow F_o = \frac{\text{Re}\, sisting\ \ Moment(m_r)\left(\frac{1}{2}\gamma h l^2\right)}{Over\, Turning\ \ Moment(m_o)\left(\frac{1}{2}qh^2k_a + \frac{1}{6}\gamma h^3 k_a\right)} \geq 1.5$$

- **Sliding**

$$\Rightarrow F_s = \frac{\text{Re}\, sistingForce((q+\gamma h)\tan\phi_f l_e)}{DisturbingForce\left(qhk_a + \frac{1}{2}h^2 k_a\right)} \geq 1.5$$

If step five is satisfied

- Yes, go to step six
- No, go to step one

Step Six

The design ends up with

- Required allowable strength, length and spacing of reinforcement
- Required allowable strength (shear capacity) of facing/connection

3. THE RESEARCH WORK CONDUCTED

Four model reinforced earth retaining walls with plan area of 2.8m×1.8m and height of 0.8m, 1.2m and two walls with 1.6m were designed and constructed. The experiment was initially intended to work with concrete coated reinforcement steel. But due to the difficulty to sufficiently load the reinforcement to fully mobilize its frictional resistance, the research was concentrated to work with unreinforced reinforcement steel. In the first three model wall with 0.8m, 1.2m, and 1.6m wall height, uncoated reinforcement was used. In the fourth model wall 1.6m height, four reinforcements placed at the top layer of the wall were coated with 2.5cm thick concrete and the remaining reinforcement used uncoated to demonstrate the effect of reinforcement coating. Sand with different relative densities was used as backfill and a surcharge for each model wall. The friction angles of the backfill material (ϕ) were determined using the direct shear test at different relative densities and moisture content for the entire three model wall. A gabion was used as a facing to keep the backfill soil from flowing. Reinforcement steel was used as reinforcement with vertical spacing of forty centimeters and horizontal spacing of thirty six centimeters. This spacing is determined in preliminary design of the retaining wall from assumed values of the shear strength parameters of the backfill material. The surcharge was applied over the model wall step by step and displacement of the reinforcement was measured under the different surcharge load until frictional resistance of certain group of reinforcement was observed to be fully mobilized. To identify the level of loading at which frictional resistance of the reinforcement was fully mobilized, a graph of total vertical stress versus reinforcement displacement was plotted (Fig. 16) and carefully observed the level of loading at which its frictional resistance was fully mobilized. The level of loading identified as a point at which the frictional resistance of the reinforcement was fully mobilized was taken as ultimate load that the particular reinforcement can carry. Based on observed ultimate load, the coefficient of friction (μ) between the backfill soil and reinforcement was determined (Table 3). Coefficient of friction is usually obtained from experiment. But it is not feasible to make experiment in all practical cases. Hence, it is required to develop empirical relations based on experimental values. In this research work, the empirical relation considered is to determine the ratio of mobilized friction angle to actual friction angle of the backfill soil (ϕ_f/ϕ) using the coefficient of friction of the backfill soil (ϕ) obtained in direct shear test and the mobilized friction angle (ϕ_f) determined from direct shear and model test (Equation 1.15) for all the three model wall. This ratio is important to apply in designing reinforced earth retaining wall. Therefore, the main objective of this research work is to determine average values of the ratio (ϕ_f/ϕ) of mobilized friction to actual friction angle of backfill soil. From this analysis of the test result, appropriate ratio (ϕ_f/ϕ) and design procedures for designing actual reinforced earth retaining wall will be proposed.

4. MATERIALS USED IN MODEL WALL STUDY

There are three major components which constitute a reinforced soil system. These are the soil, the reinforcement, and the facing (skin). In order to design safe and economical reinforced soil structures it is necessary to have detailed characteristics of all three components. It is, therefore, essential to characterize all the relevant properties or design parameters of each component through laboratory or field test

4.1 The Backfill Material

High quality backfill material is required for durability, good drainage, and good reinforcement interaction, which can be obtained from well graded, granular materials. The backfill soil properties have a great influence on behavior of reinforced soil. Hence, it is very important to study these properties of the backfill soil.

4.1.1 Grain Size Distribution

The ideal nature of backfill soils used in reinforced earth structures is a well - graded and well- drained granular material. For a given slope, the disturbing forces remain about constant, but the shearing resistance provided by the soil decreases if the angle of friction in the soil decreases. Thus, for a given slope geometry a less frictional soil requires more reinforcement force for its stability and much less reinforcement is required to stabilize good quality granular soils than mixed or clay soils. It is also known that, rapid loading of saturated soft clay do not allow time for dissipation of pore water pressures. Hence, the initial effective stress, which exists before loading still apply and makes the reinforcing element ineffective. A high quality granular backfill has the advantages of being free draining, which avoids the danger of instability due to pore water pressure. This is the main reason why compact granular soil is the standard construction material for most reinforced earth systems, since they solely rely on the friction between the soil and reinforcing for anchoring strength or pullout. Therefore, the quality of backfill material is critical and this performance requirement generally eliminates soils with high clay contents as a backfill material in reinforced earth construction [4]. .

For the backfill material used for the model wall test in this research work, a sieve analysis was done**.** The gradation curve of the backfill soil is shown in Fig. 6. Table 1 shows the details of the sand. Even though it would have been preferable to use well graded sand, circumstance has forced the researcher to use this sand.

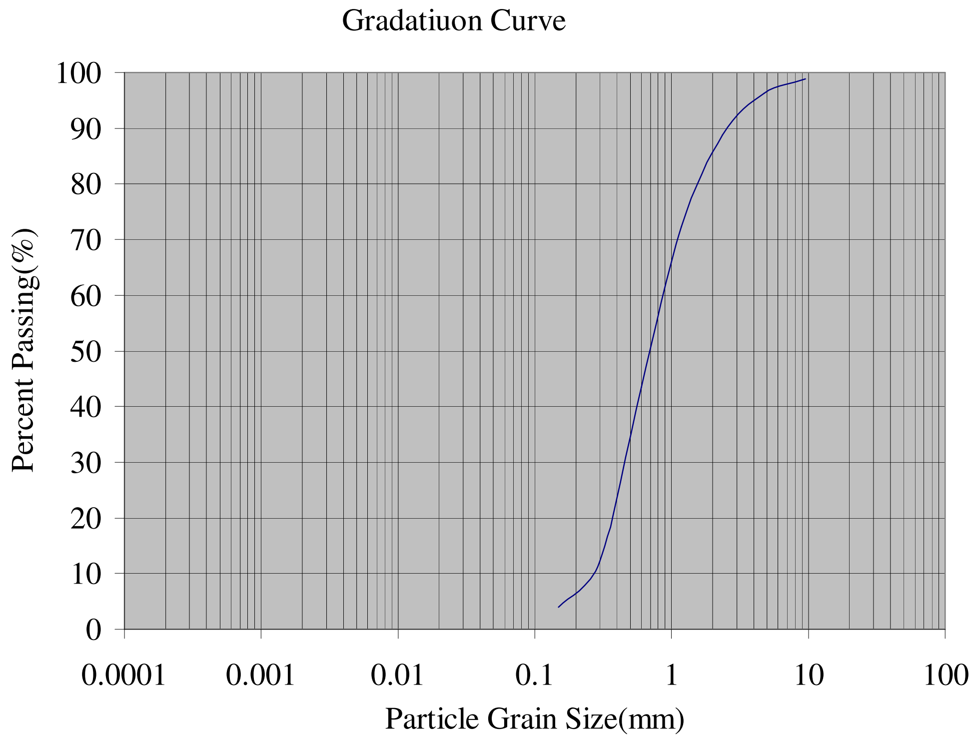


Figure 6: Grain size distribution curve of the backfill Soil used.

Table 1: Summarized data for Grain Size distribution of the backfill Soil

No	Description	Values
1	specific Gravity	2.33
2	Gravel percent	11.2
3	Coarse sand percent	45.5
4	medium sand percent	39.5
5	Fine sand percent	3.8
6	Silt percent	0
7	Clay percent	0
8	Particles size 10% Passing (D_{10})mm	0.25
9	Particles size 30% Passing (D_{30})mm	0.42
10	Average diameter(D_{50})mm	0.69
11	Particles size 60% Passing (D_{60}) mm	0.83
12	Uniformity coefficient(C_u)	3.32
13	Coefficient of gradation(C_c)	0.85
14	Soil classification(USC)	Poorly graded sands(SP)

4.1.2 Shear Strength

All soils develop tensile strains during shear deformation; consequently the reinforcement can develop tensile force when placed in any soil, provided that the reinforcement is in an appropriate orientation. Therefore, all soils may be reinforced, but there are some important distinction needs to be made between different soils. The stability of depends significantly on angle of friction (ϕ) the soil. Compact granular soil has a higher angle of friction than soil containing large percentage of clay. There are two components of frictional shearing resistance for compact granular soils. The first component is constant and depends mostly on the soil mineralogy. This frictional (2) shearing resistance component can always be mobilized in the soil, and represents a safe lower limit to the available soil strength. The second component of frictional shearing resistance is caused by the interlocking which occurs when particles are densely packed together. In order for shear deformation to occur, the particles must be able to move past one another, so the particles move apart as they shear, causing an increase in volume. This is called dilation. The rate at which the soil must dilate on shearing depends both on the relative density and on the magnitude of the mean stress in the soil. When loose granular soils are sheared, dilation does not occur, with the result that the maximum mobilized friction angle approaches the large strain friction angle for dense granular soils [5] as indicated in Fig. 7 below.

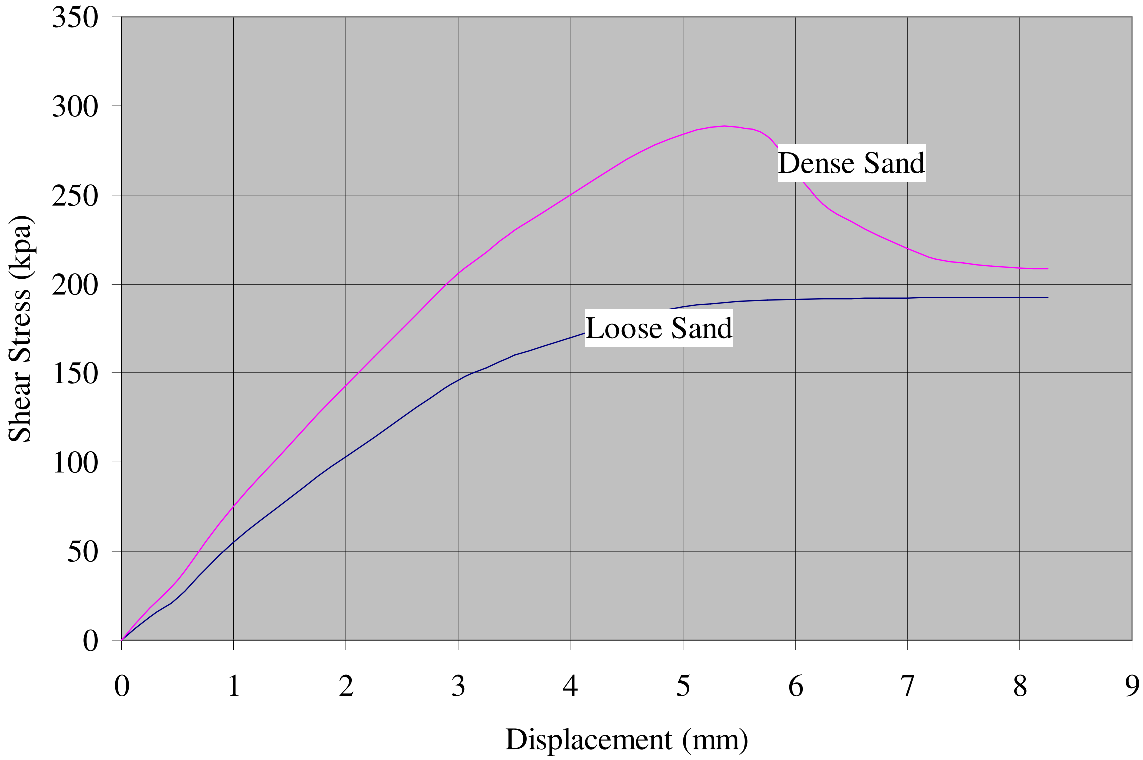


Figure 7: Frictional Shear resistance for dense and loose sand

In this model wall study, the Shear strength properties of the backfill material were determined using the direct shear test. The shear test was done with the applied normal stress of 100kPa, 200kPa and 300kPa, at shearing rate of 1.8mm/min. The initial densities before test were 1.49, 1.59, 1.61 and 1.60 gm/cm^3 and at moisture content of 8.8%, 15.85%, 13.84% and 14.05 respectively for the first, second, third and fourth model wall test. From the result of laboratory test, the angle of internal friction was found to be 39.69, 41.24 and 42.52 for the first, second and third model test respectively (Table 2). In the fourth test, the applied surcharge load is not sufficient to fully mobilize the frictional resistance; hence direct shear test was not conducted.

Table 2: The Summarized Data of the direct shear test.

No	Description	The First Model Wall Test	The Second Model Wall Test	The Third Model Wall Test	The Fourth Model Wall Test
1	Average Moisture Content w (%)	8.8	15.85	13.84	14.05
2	Average Bulk Density ρ_b (gm/cm$^{3)}$	1.49	1.59	1.61	1.60
3	Angle of internal friction of the backfill(ϕ)	39.69	41.24	42.52	–

.

4.2 The Reinforcing Element

The soil reinforcing element is designed and positioned within the compacted back fill to give the composite structure tensile strength. The mechanism of soil to reinforcement stress transfer is through the pressure developed due to overburden of the backfill soil on to the reinforcing elements. The pressure developed due to overburden along the surface area of the reinforcing element, results tensile resisting force which has to support the lateral earth pressure acting on the wall over the area that is shared by individual reinforcement. To prepare the reinforcing element, deformed concrete reinforcement bars of ten millimeter diameter with twelve meter in length were purchased from the market and curtailed to the selected length of three meters. Since the expected surcharge load is not very large, it is unlikely for the reinforcement to fail in tension in this model wall. Even though, determining the yield strength (f_y) is not necessary for designing this model wall, it is very important for actual reinforced earth retaining wall design.

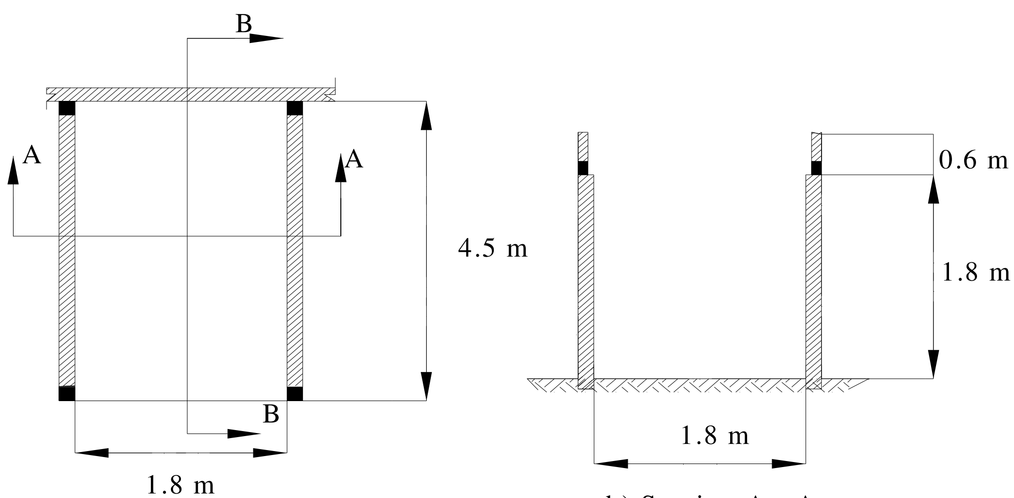
4.3 The facing (skin) element

The facing (skin) element protects the soil and reinforcing elements from weathering effect and used to keep the backfill soil from flowing. Since the facing is the visible part of the structure, it also controls the aesthetics of the reinforced earth wall. The facing can be constructed with section of relatively flexible material such as, galvanized sheet metal, wire mesh (gabion), and reinforced precast concrete plate e.t.c. The gabion facing is relatively cheaper material and locally available in sufficient quantities. To prepare the gabion facing, wire mesh having five centimeters opening, length of five meters and width of two meters was purchased from the market and sewed to the selected depth like a sack opened only from the top and filled with rock boulders grater than the opening of the wire mesh.

5. SET UP AND CONSTRUCTION PROCEDURE OF THE MODEL WALL

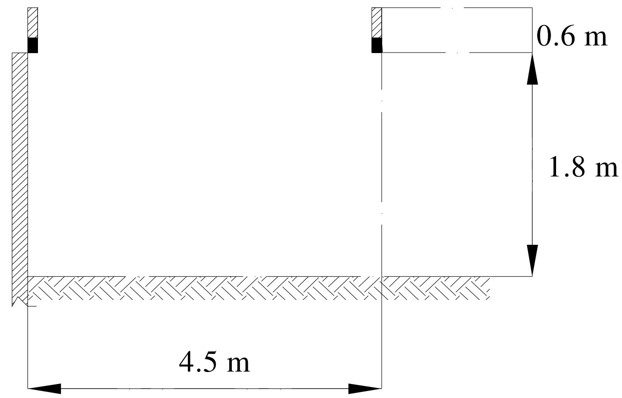
5.1 Construction of the Lateral Wall

To simulate the plain strain condition inherent in the design, rigid masonry walls were constructed on both sides of reinforced sand and at the back. The plan area of the constructed model wall is 1.8mx4.5m with 1.8m wall height. To insure the rigidity of the wall, reinforced concrete column at four corners and top tie beam was provided. In addition, block wall with depth of 0.60m was constructed at the top of the wall over the tie beam to provide support for surcharge load (Fig.. 8).



a) Plan view of the side walls

b) Section A - A



c) Section B - B

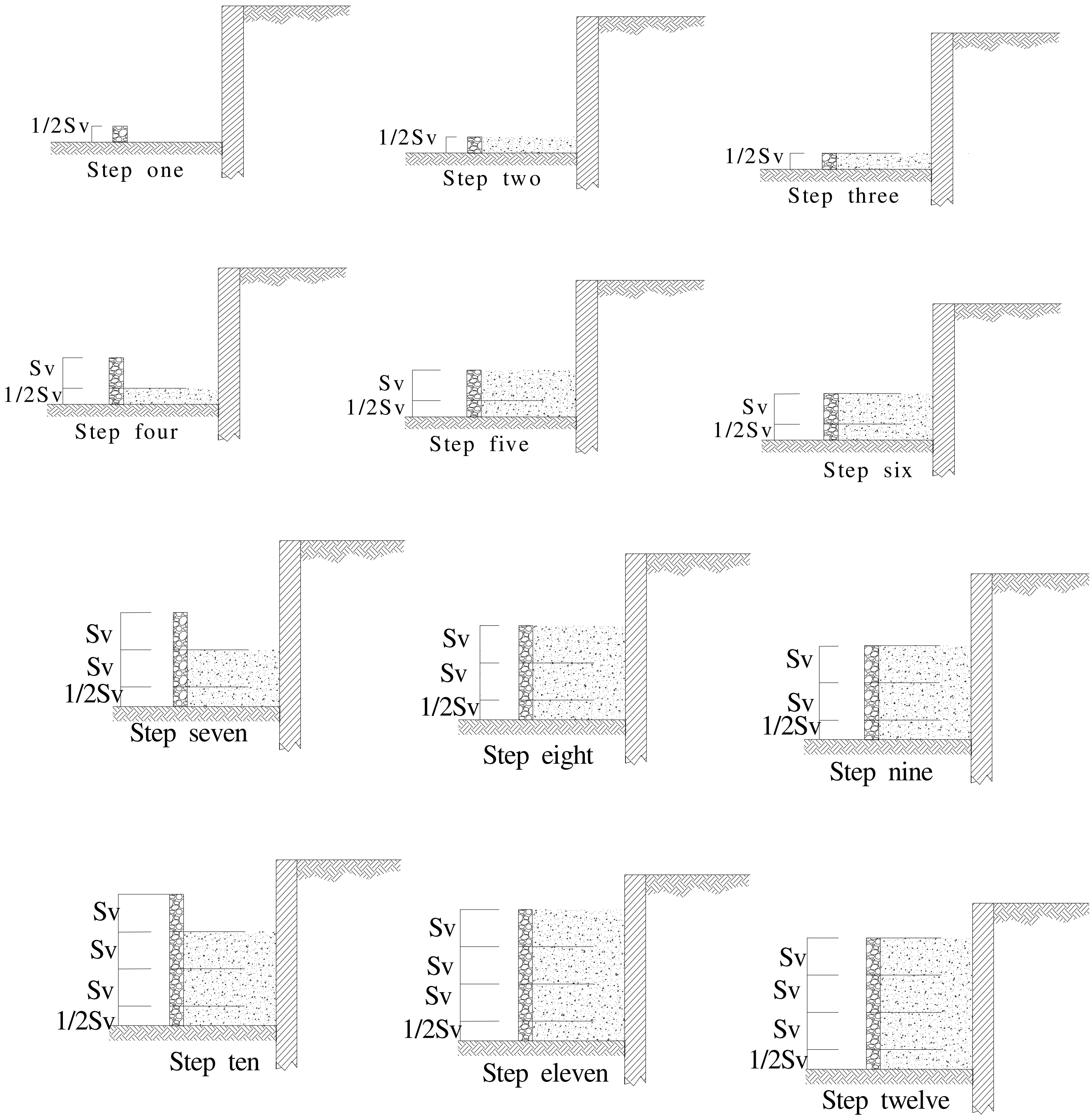
Figure 8: Construction details of the side wall

5.2 Reinforcing Element

The size and length, horizontal spacing (s_h) and vertical spacing (s_v) was determined in preliminary design of the retaining wall from assumed values of the shear strength parameters of the backfill material. Based on assumed values, reinforcement with a total length of three meters, a horizontal spacing (s_h) of thirty six centimeters and vertical spacing (s_v) of forty centimeters were selected uniformly for all layers. The total length of the reinforcement used is the sum of embedment length (l_r) and effective length (l_e). For each layer, a total of four reinforcements were used. All reinforcement used for the first three model wall was uncoated. But in the fourth model wall, the reinforcements in the forth layer were coated with 2.5 cm concrete and the remaining reinforcement used uncoated. One may equally use mortar and conduct the investigation. Here a concrete coating was used. The thickness of the coating was determined based on standard concrete cover for reinforced concrete structures.

5.3 Gabion Facing and Model Retaining Wall Construction

The gabion facing was prepared from rock boulders greater than five centimeters in size and a five centimeters opening wire mesh. The wire mesh was sewed like a sack opened only from the top. The rock boulders were filled in to the prepared wire mesh sack to the selected depth. In selecting the thickness of the facing, a compromise is made between the workability and flexibility of the gabion facing. Thinner wall is more flexible but it is difficult to work with. In this model test, wall thickness of twenty five centimeters was selected. This selected wall thickness is assumed to result in a flexible skin that the supporting resistance of the gabion wall is neglected. The gabion was constructed in step by step process, simultaneously with back filling and reinforcement placing (Fig. 9). Since the bulk densities were not determined at stage of the backfilling process, it was the backfill thickness which was directly measured and later on converted to equivalent surcharge load after the respective bulk densities were determined during dismantling the wall. The back fill soil was compacted manually with hand tamper. The compaction process should be carefully done with out disturbing the gabion facing and uniformly with the selected lift thickness. Due to variation in densities with depth, bulk densities were taken at different level while dismantling the wall and average value was used for calculation of surcharge load. To give the reinforcement sufficient bond length with in the gabion, a bent hook was provided which was anchored in the gabion facing with mortar. The gabion was constructed together with backfilling and reinforcement placing step by step as indicated in Fig. 9.


1/2Sv
Step one
1/2Sv
Step two
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Step three
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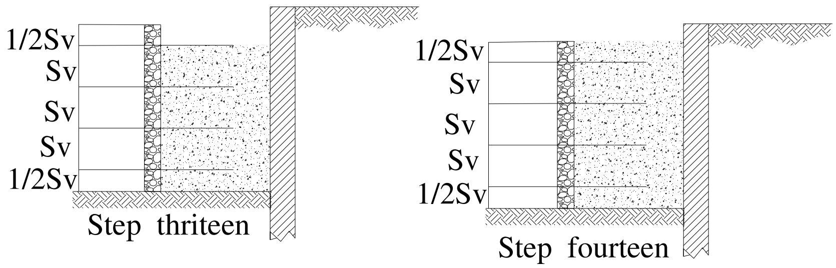
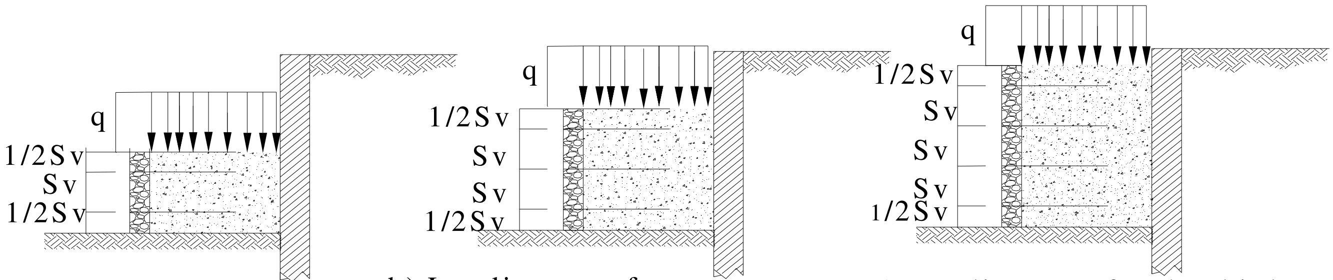


Figure 9: Sequencial Steps in reinfoeced earth model wall construction.

5.4 Application of Surcharge Load

After construction of each wall was completed, the measuring mechanism was suitably fixed in front of the wall and the initial reading for the selected points was taken using the measuring gauge. During the time of surcharge load application, the total overburden thickness and reinforcement displacement at each level of reinforcement was directly measured at each step of surcharge load application. Later on, after the determination of the bulk unit weight of the surcharge material during dismantling the model wall, this overburden thickness was converted to equivalent total vertical pressure. This total vertical pressure was plotted against reinforcement displacement The Plot of the whole group of reinforcement was done in this manner. Surcharge was applied step by step and simultaneously the measurement was taken at each step for the selected points until the required level of loading was reached.



a) Loading step for the firts model wall

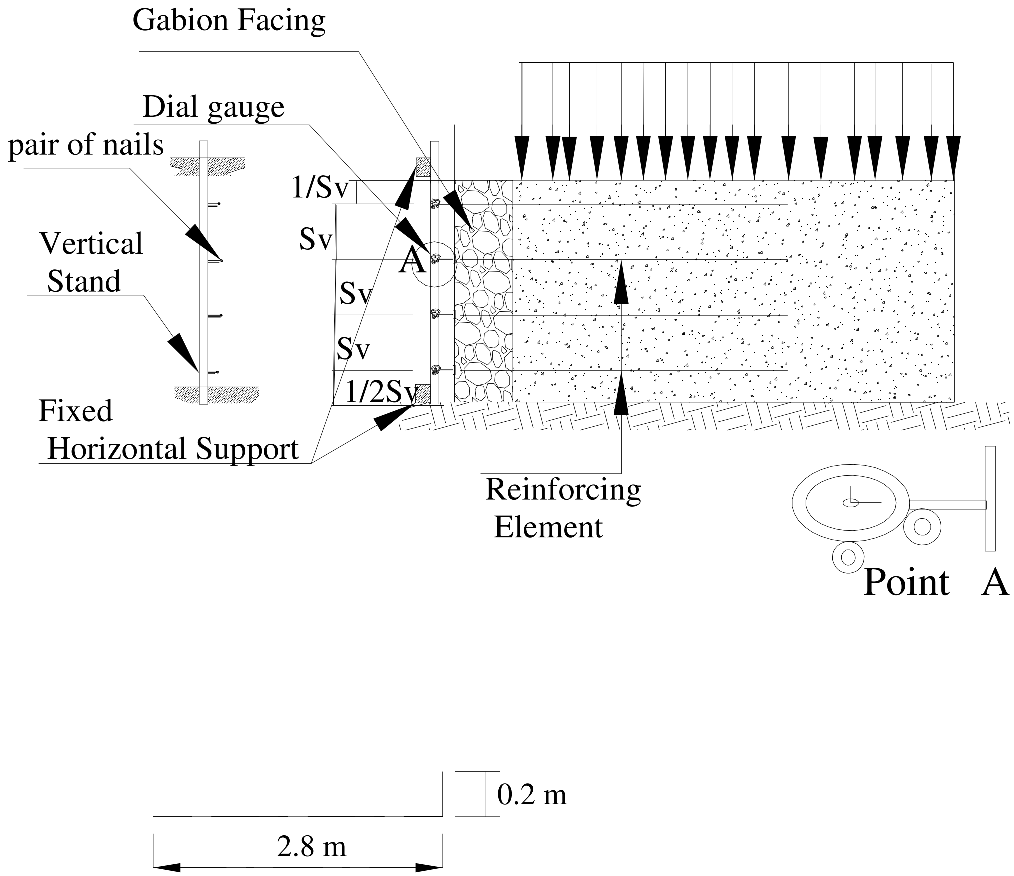
b) Loading step for the second model wall

c) Loading step for the third and fourth model wall

Figure 10: Surcharge Load Application Step

5.5 Measuring Mechanism

The Measuring Mechanism was prepared from wood (lumber), nails and the measuring gauge. Four straight lumbers were prepared. A pair of nail was nailed on the selected lumber such that the point of nailing was pre-determined to match the point where the reinforcement was laid (Fig. 11).The prepared measuring mechanism was suitably fixed in front of the retaining wall so that the pairs of nails on the lumber will match the position of the reinforcement laid in every layer. The gap between the measuring mechanism and the face of the retaining wall was suitably adjusted to measure horizontal displacement of the reinforcement using the measuring gauge.



Reinforcement details used in reiforced earth wall construction

Figure 11: Typical section of the model retaining wall

6. THE MODEL WALL STUDY

6.1 The first Model Wall Test

The construction of the first model wall was started after preparation of all necessary material for the construction of the model wall (Fig. 9, step one to six).In the first step, the gabion facing was constructed to a depth of twenty centimeters from the bottom of the wall. The facing was supported from out side to keep vertical position until the construction process of the wall to a selected height was reached. The first layer of the back fill soil was laid to a level of twenty centimeters in the second step. Then, the first layer of reinforcement was laid over the back fill soil in the third step. For each layer, a total of four reinforcement pieces were used with a horizontal spacing (s_h) of thirty six centimeters and vertical spacing (s_v) of forty centimeters. To give the reinforcement sufficient bond with the gabion, a twenty centimeters bent hook was provided which was anchored in the gabion facing with mortar. In the fourth step, the gabion facing was constructed to a depth of forty centimeters from the level of the first layer. In the fifth step, the back fill soil was filled to a level of the constructed gabion facing in the same way as the first layer. The second layer of reinforcement was laid in similar way using same length and spacing as the first reinforcement layer in the sixth step. In step seven, the gabion facing was again constructed to a depth of twenty centimeters. And finally in step eight, the back fill soil was filled to a level equal to the constructed gabion facing .With this, the first model wall construction was over. The external support previously provided to the gabion facing was removed. The stand prepared for measuring purpose was suitably fixed in front of the wall. Using the advantage of symmetry, four points (A-1, A-2, B-1 and B-2) from the labeled points (Fig. 12) were selected. Using the measuring gauge, the initial reading for the selected points was taken. First, a surcharge of 2.923 kPa was applied. The applied surcharge soil was supported with soil filled sacks in the front direction. The first reading was taken just after its application. The reading was continuously taken until the change in displacement ceased with the applied surcharge (Fig.10a). Secondly additional surcharge of 3.654 kPa was applied over the first surcharge layer. Again, reading was taken in similar way as the first layer. Similarly, a third surcharge of 5.116 kPa was applied and reading was taken. Finally, a surcharge of 3.654 kPa was applied and reading was taken. The plot of the applied load versus displacement of reinforcement steel placed at point B-1 and B-2 were plotted as shown in Fig. A-1 of appendix A. With this the first model test was concluded and the wall was dismantled. During dismantling of the model wall, the bulk density and moisture content tests were taken at different layers

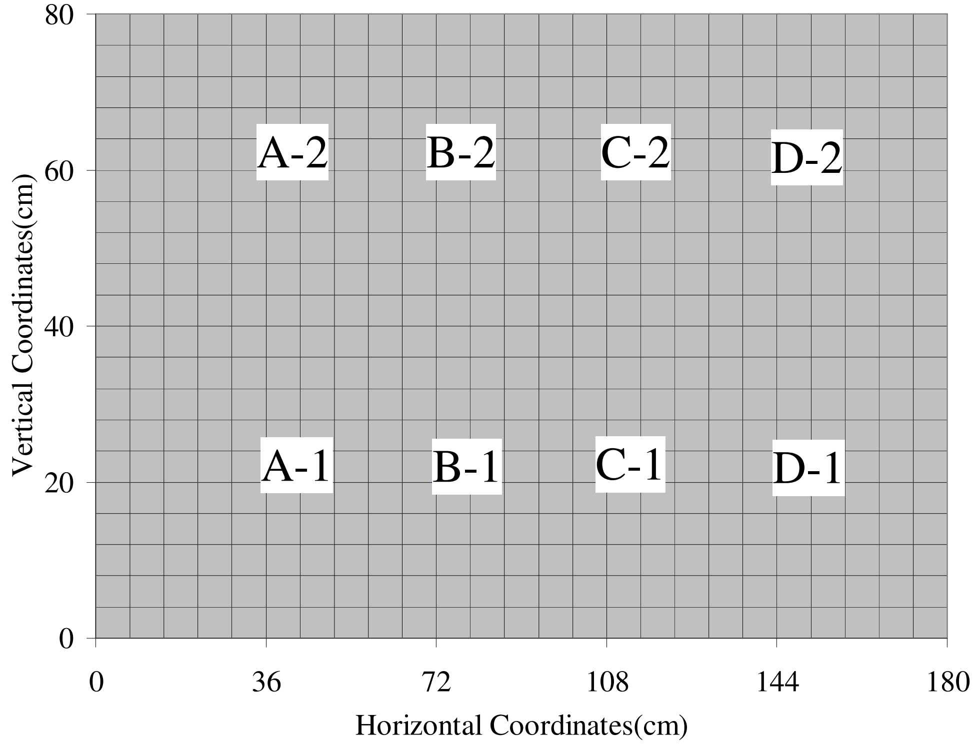


Figure 12: The first Model Wall Test (front face view)

6.2 The Second Model Wall Test

The second model wall was erected after the first model wall was dismantled and removed (Fig. 9, Step one to nine) Similar to the first model wall. In the tenth step, gabion facing of twenty centimeter depth was extended and finally the back fill soil was filled to level equal to the constructed gabion facing. At this point, the second model wall construction was complete. Again, the external support previously provided to the gabion facing was removed. The stand prepared for measuring mechanism was suitably fixed in front of the wall facing. Similar to the first model wall using the advantage of symmetry, nine points (A-1, A-2, A-3, B-1, B-2, B-3, C-1, C-2 and C-3) from the labeled points (Fig. 13) were selected. Using the measuring gauge, the initial readings of the selected points were taken. The surcharge load was applied step by step (Fig. 10b). First, a surcharge of 3.08kPa was applied. The surcharge was supported with soil filled sacks in the front direction. The first readings were taken just after application of the surcharge. The reading was continuously taken until the change in displacement ceased with the applied surcharge. In the second step, an additional surcharge of 4.62 kPa was applied over the first surcharge layer. Again, rereading was taken in similar way, with the first layer. Similarly, in the third step, a surcharge of 4.62 kPa was applied and reading was taken. Finally in the fourth step, a surcharge of 3.85 kPa was applied and reading was taken. The plot of the applied load versus reinforcement displacement for selected reinforcements placed at points C-1, C-2 and C-3 were plotted as shown in Fig. A-2 of appendix A. With this the second model test was concluded and the wall was dismantled. During dismantling the model wall, the bulk density and moisture content tests were taken at different layers

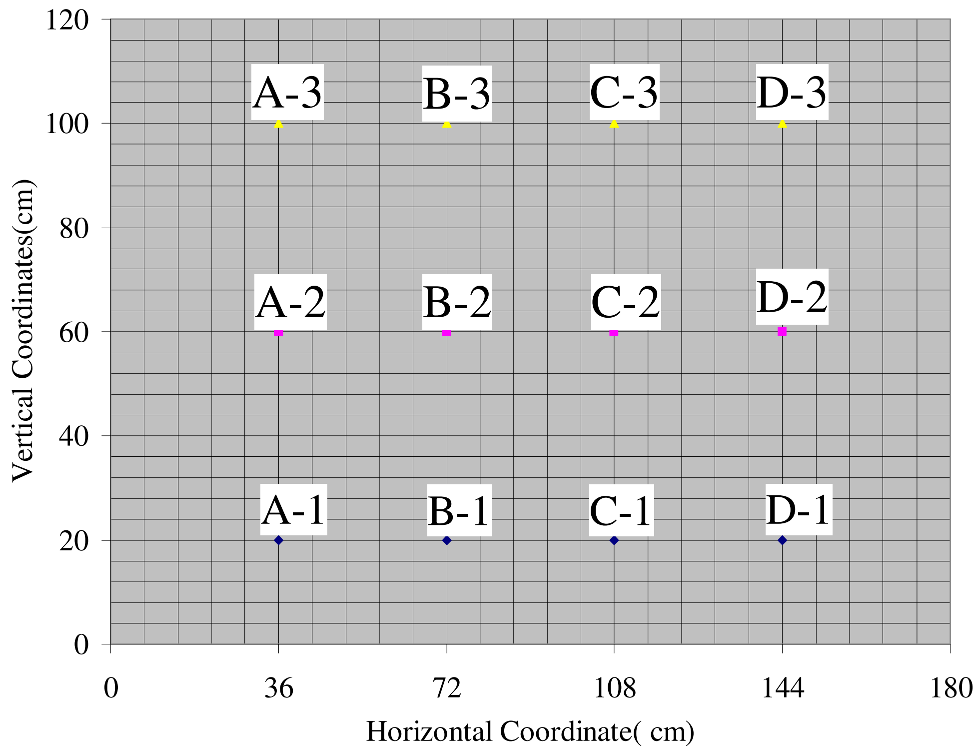


Figure 13: The Second Model Wall Test (front face view)

6.3 The Third Model Wall Test

After dismantling the second model wall, the third model wall was erected (Fig. 9, step one to fourteen) in similar way as the first and second model wall. The initial reading for the selected points (A-1, A-2, A-3, A-4, B-1, B-2, B-3, B-4, C-1, C-2, C-3, C-4, D-1, D-2, D-3 and D-4) was taken. The surcharge load was applied step by step (Fig. 10c). First, a surcharge of 3.22kPa was applied. The first reading was taken just after application of the first surcharge. In the second step, an additional surcharge of 4.83kPa was applied over the first surcharge layer. Again, reading was taken in similar way, with the first layer. Similarly, in the third step, a surcharge of 3.22kPa was applied and reading was taken. In the fourth step, a surcharge of 3.22kPa was applied and reading was taken. Finally in the fifth step, a surcharge of 4.83 kPa was applied and reading was taken. The plots of the applied load versus reinforcement displacement of the selected reinforcement placed at points B-1, B-2, B-3 and B-4 were plotted as shown in Fig. A-3 of appendix A. With this the third model test was concluded and the wall was dismantled. During dismantling the model wall, the bulk density and moisture content was taken at different layers

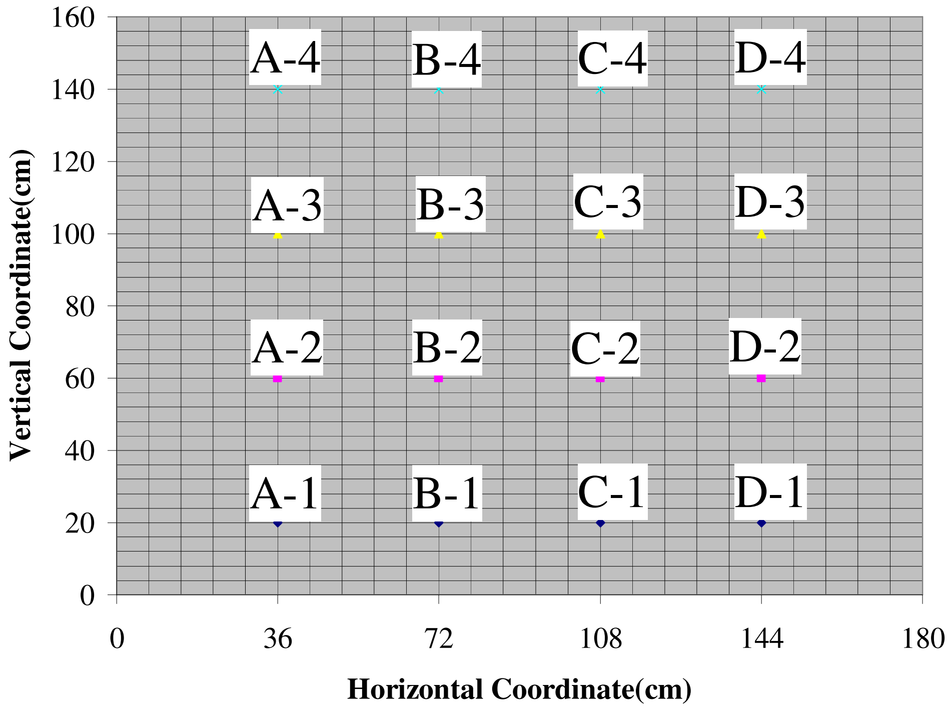


Figure 14: The Third Model Wall Test (front face view)

6.4 The Fourth Model Wall Test

The erection of the fourth model wall was similar to the third model wall test except that in the first three model wall, uncoated reinforcement was used. To compare the relative performance of concrete coated reinforcement with uncoated reinforcement for practical applications, reinforcements placed at the fourth layer (A-4, B-4, C-4 and D-4) were coated with 2.5cm concrete cover. The thickness of concrete cover could meet the standard values for concrete structures. To make this concrete cover, first 2.5cmx2.5 cmx280cm concrete mold is prepared from wood lumber. Then, reinforcement steel which was already prepared was suitably placed in the mold. Finally the concrete is poured in to the mold with steel and allow it to cure. When the concrete cover sufficiently cured, the mold is removed and the reinforcement was ready to use. In the first three model walls uncoated reinforcement was used because of the fact that the dimension of coated reinforcement will be more irregular which makes more difficult to analyze. In addition to its irregularity, coated reinforcement would have larger surface area (about six times larger) that needs larger surcharge load (not less than 120kPa) to mobilize its frictional resistance. As it can be clearly seen from the graph of reinforcement displacement versus surcharge load of Fig.A-4 appendix A, the displacement of the reinforcement at fourth layer is very small relative to the second and the third reinforcement layer. This is because the surface area of reinforcement at fourth layer increased due to concrete coat which in turn increased the frictional resistance of the reinforcement. It may be this increase in frictional resistance that hinders the reinforcement displacement. It was also noted that the roughness of concrete surface could affect the frictional resistance. This could have done in detail to determine its effect on frictional resistance. However, due to the absence of certain required facilities (pull out facilities) and time limitation, the effect of reinforcement roughness, spacing and size was not determined in this research work.

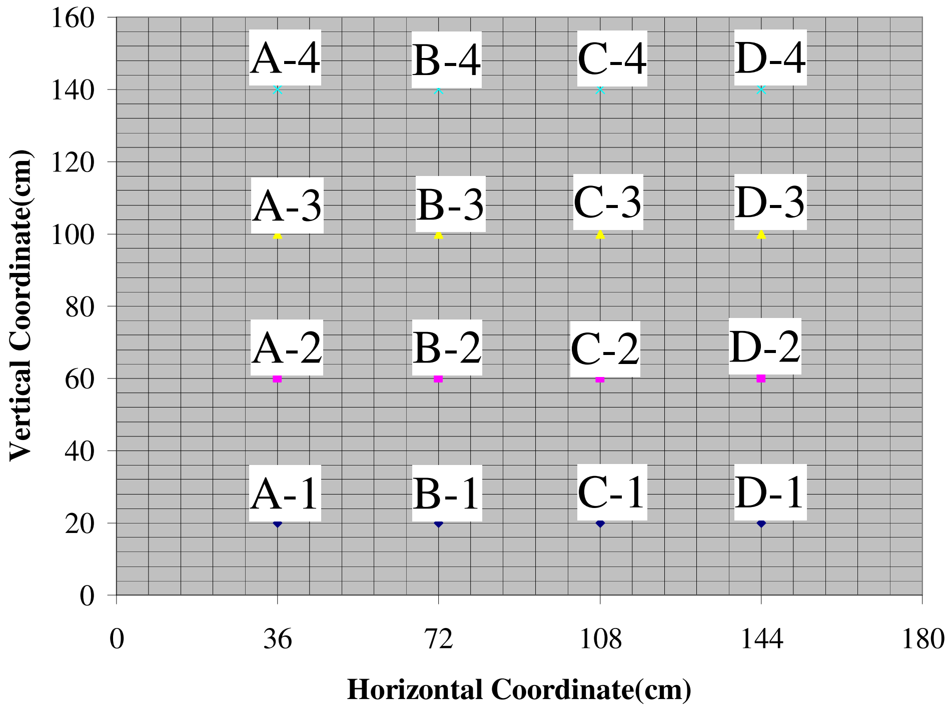


Figure 15: The fourth Model Wall Test (front face view)

7. DISCUSSION OF THE TEST RESULT

As we can clearly observe from plotted graph of selected reinforcement for all the tests (Fig.A-1, A-2, and A-3 of appendix A), all graphs can be sub divide in to three categories. These are reinforcement group, whose frictional resistance was fully mobilized, a reinforcement group, whose frictional resistance was not fully mobilized and coated reinforcement group.

7.1 Reinforcement groups whose frictional resistance was fully mobilized.

These groups include those reinforcements which were found at the top layer of the model wall. From the first model wall, the selected reinforcement laid at point A-2 and B-2, from the second model wall, the reinforcement laid at A-3, B-3, C-3 and D-3 and from the third model wall, the reinforcement laid at A-4, B-4 C- 4 and D-4 are categorized in this group. The effective length (L_e) of the reinforcement in this group is relatively shorter than the remaining group. Hence, the frictional resistance of these groups is smaller. This is why the top reinforcement was displaced more and mobilizes its frictional resistance first. Similar to the other group of reinforcement, to Plot the total vertical stress versus displacement for this group of reinforcement, first the total overburden thickness and reinforcement displacement at each level of reinforcement was directly measured at each step of surcharge load application. Later on, this overburden thickness was converted to equivalent total vertical stress after the determination of the bulk unit weight of the backfill material during dismantling the model wall. Then, the total vertical stress was plotted against reinforcement displacement as shown in (Fig.16). It is known that in the elastic range, the displacement is proportional to the applied loading as shown in segment AB of the graph. In segment BD, the displacement increases at a decreasing rate with load. This is because of the fact that additional shear strength is gained from frictional resistance developed between the back fill soil and reinforcement. But on further increasing the applied surcharge load, the displacement increases with increasing rate. In other words the shear resistance of the reinforcement increases at a decreasing rate with increasing applied load. This shows that at point D on the Fig., the frictional resistance between the back fill and reinforcement is fully mobilized and further loading will make the reinforcement pulling out from the fill and finally a pullout failure is likely to occur.

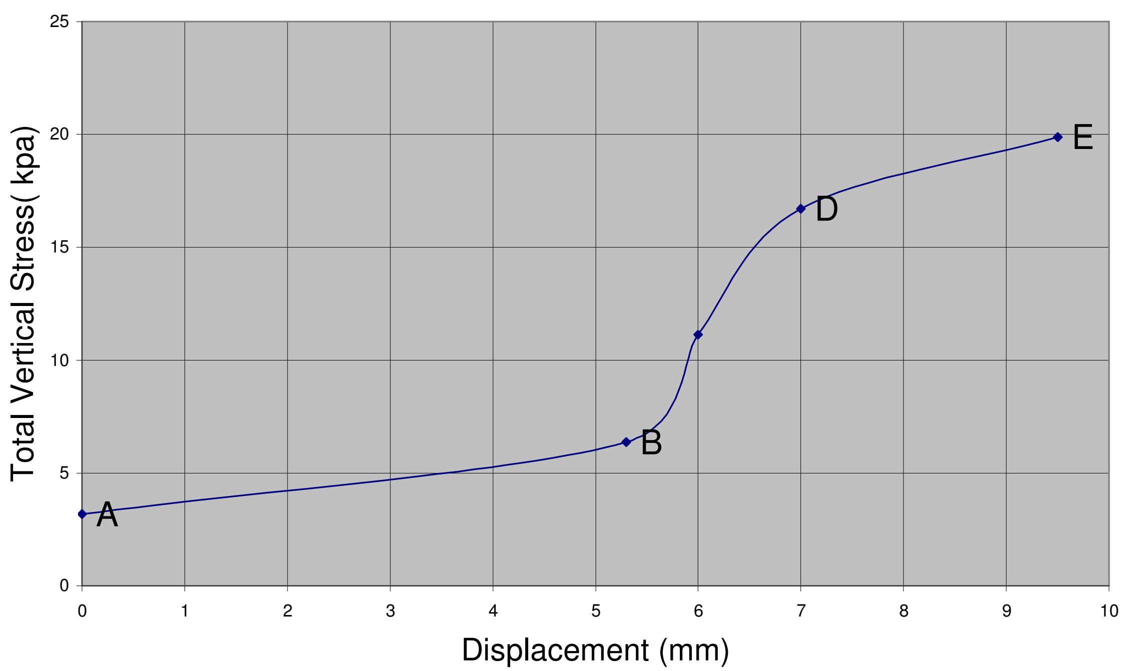


Figure16: Total vertical stress versus displacement for reinforcement groups whose frictional resistance was fully mobilized. (From model two of C- 3)

7.2 Reinforcement groups, whose frictional resistance was not fully mobilized.

These groups of reinforcements include all reinforcement used except those reinforcements which were found at the top layer of the reinforcement. The effective length ($l_{e)}$ of the reinforcement increases with depth. Hence, the frictional resistance of these groups is also increases with depth. This is why the reinforcement is less displaced and did not fully mobilize its frictional resistance at the level of loading at which the top reinforcements reach its full resisting capacity. The Plot of this group of reinforcement can be represented with the following graph (Fig. 17). Similar to the first group of reinforcement, in the elastic range, the displacement is proportional to the applied loading as shown in segment ABC of the graph. In segment CE the displacement increases at a decreasing rate with load. This is also due to additional shear strength gained from frictional resistance developed between the back fill soil and reinforcement. After point E, this graph seems to continue in similar manner with further additional loading. This shows that the applied surcharge at loading level E is not sufficient to fully mobilize the frictional resistance. And hence, additional load should be applied to this reinforcement group for full mobilization of their frictional resistance

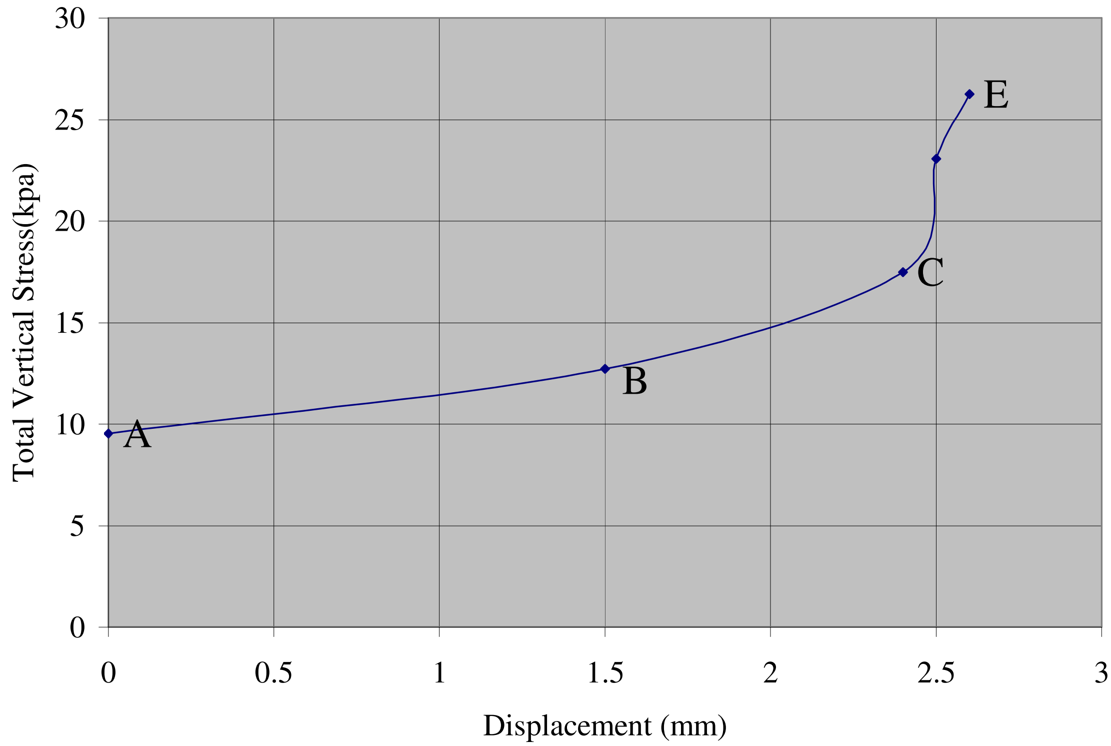


Figure17: Total vertical stress Versus Displacement for reinforcement groups whose frictional resistance was not fully mobilized. (From model two of C- 2)

7.3 Coated Reinforcement groups

These groups of reinforcements include reinforcements placed at the fourth layer (A-4, B-4, C-4 and D-4) which was found at the top layer of the forth model wall. The surface area of reinforcements of these groups increased due to concrete coat which in turn increased the frictional resistance of the reinforcement. The increase in frictional resistance hinders the reinforcement displacement The Plot of this group of reinforcement can be represented with the following graph (Fig. 18). Similar to the first and second group of reinforcement, in the elastic range, the displacement is proportional to the applied loading as shown in segment AB of the graph. In segment CDE the displacement totally ceased to take place. This is due to additional high shear strength gained from frictional resistance developed between the back fill soil and the coated reinforcement. After point E, this graph seems to continue in similar manner with further additional loading. This shows that very large surcharge load should be applied to fully mobilize the frictional resistance.

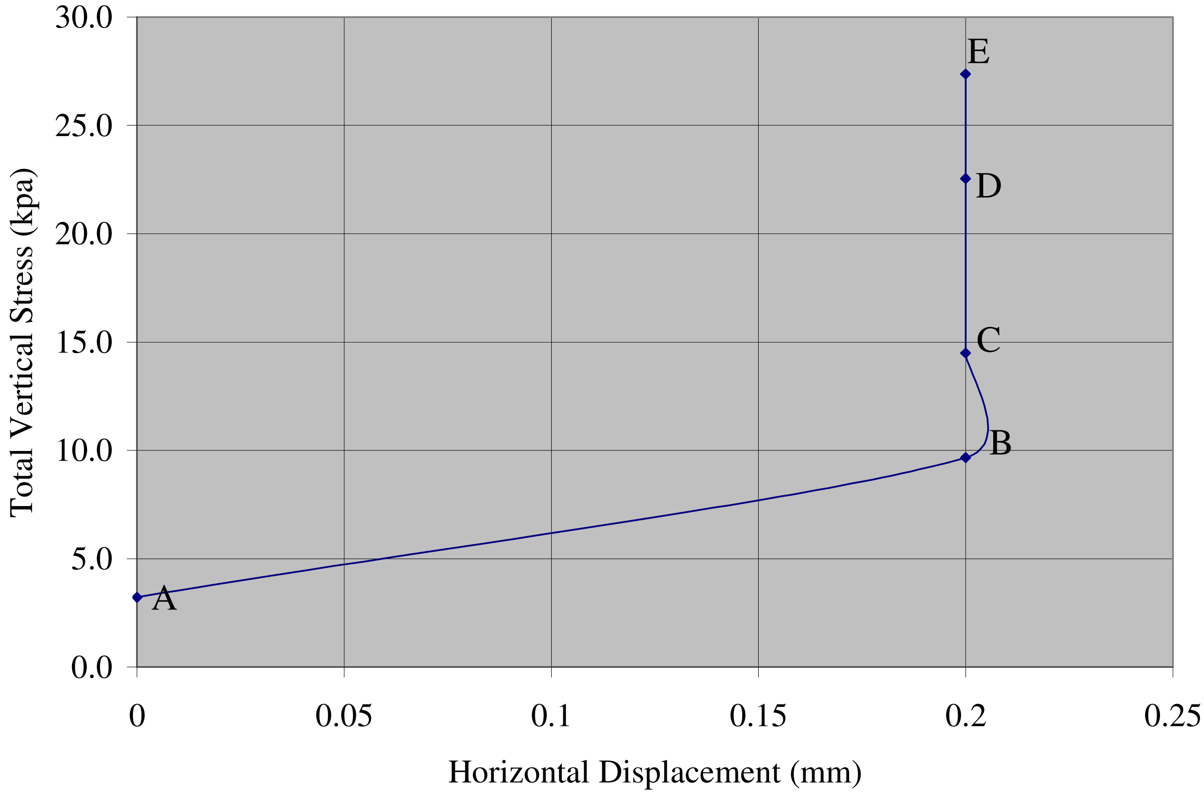


Figure18: Total vertical stress Versus Displacement for coated reinforcement groups (From model four of B- 4)

8. SUMMARIZED DATA OF THE MODEL WALL AND DIRECT SHEAR TEST.

From the result of the analysis of the model and direct shear test (Equation 1.15), the coefficient of friction (μ) can be determined, $\mu = \tan\phi_f$ and was found to be 0.457, 0.462 and 0.48 for the first, second and third model test respectively. Again, from the value obtained for coefficient of friction(μ), the mobilized friction angle (ϕ_f) of the backfill material can be determined and was found to be 24.56, 24.80 and 25.64 for the first, second and third model test respectively (Table 3). Hence, according to the test result, the average mobilized friction angle is about 61% of the actual internal friction angle of the backfill material. As it can be observed from the result of the analysis of Getu's and Nuriye's work, the material used as a back fill material has similar friction angle (ϕ) to the backfill material of this research work. But, there is large difference between the values of mobilized friction angle (ϕ_f) obtained in this research and modified direct shear test of both Getu's and Nuriye's work. However, there is only little difference between the value of mobilize friction angle (ϕ_f) obtained in this research and Nuriye's pullout test.

Table 3. The Summarized Data of the model wall.

No	Description	The First Model Wall Test	The Second Model Wall Test	The Third Model Wall Test	The Fourth Model Wall Test
1	Average Moisture Content w (%)	8.8	15.85	13.84	14.05
2	Average Bulk Density ρ_b (gm/cm$^{3)}$	1.49	1.59	1.61	1.60
3	Angle of internal friction of the backfill(ϕ)	39.69	41.24	42.52	–
4	Rankine Active Earth pressure Coefficient(k_a)	0.22	0.21	0.19	–
5	(Vertical spacing)x (Horizontal spacing)k_a $= s_v s_h k_a$ (m^2)	0.032	0.030	0.027	–
6	Effective length of reinforcement(l_e)(m)	2.27	2.10	1.85	–
7	Perimeter of reinforcement (p_r) (m)	0.031	0.031	0.031	–
8	$P_r l_e$ (m^2)	0.070	0.065	0.057	–
9	Coefficient of friction between the reinforcement and backfill $\mu = \tan\phi_f = s_v s_h k_a / p_r l_e$	0.457	0.462	0.48	–
10	Mobilized angle of internal friction of backfill(ϕ_f)	24.56	24.80	25.64	–
11	Ratio of mobilized friction to actual angle of friction angle(ϕ_f/ϕ)	0.619	0.601	0.603	–

9. CONCLUSION AND RECOMMENDATIONS

The use of reinforced earth retaining wall as an alternative to masonry and reinforced cantilever retaining wall has been studied. Literature review, experimental model wall studies and Laboratory investigations were employed at various levels of the research. Based on the result of the study on reinforced earth retaining wall, the following conclusions and recommendation are provided:

1. The main contribution of this research work is to determine average values for the ratio of mobilized friction to actual angle of friction ($\phi_f/\phi = 0.60$) which is important to apply in designing reinforced earth retaining wall using reinforcement steel and backfill materials having similar properties.
2. The difference between unreinforced and reinforced soil is the addition of the term $\Sigma (T_i (\cos (45+\phi/2) + \sin (45+\phi/2) \tan\phi))$ which increases the shear resisting force in reinforced earth structures.
3. There is an effect of deformation restrictions resulting from the facing resistance. However, due to complexity in determining this resisting effect, the role of facing resistance is neglected in designing the reinforced earth retaining wall.
4. Since at large strain values, only the constant lower limit shear resistance is available for both dense and loose sand, it is recommended to use the large strain shear resistance value for designing reinforced earth.

Suggestion for Future Work

With in the frame work of this thesis, an attempt has been made to use reinforcement steel as longitudinal reinforcement with gabion facing in reinforced earth retaining wall design. However, it is necessary to emphasize that the scope of this research work does not exhaust all aspects of reinforced earth structures with reinforcement steel. Therefore, it is recommended that further study may be made using reinforcement steel as grid reinforcement with reinforced concrete plate facing in the design of reinforced earth retaining walls.

APPENDIX

I. APPENDIX A: The Model Wall Test Data

Note:

$(B\text{-}1)_1$, $(B\text{-}2)_1$: First model

$(C\text{-}1)_2$ - $(C\text{-}3)_2$: Second model

$(B\text{-}1)_3$ - $(B\text{-}3)_3$: Third model

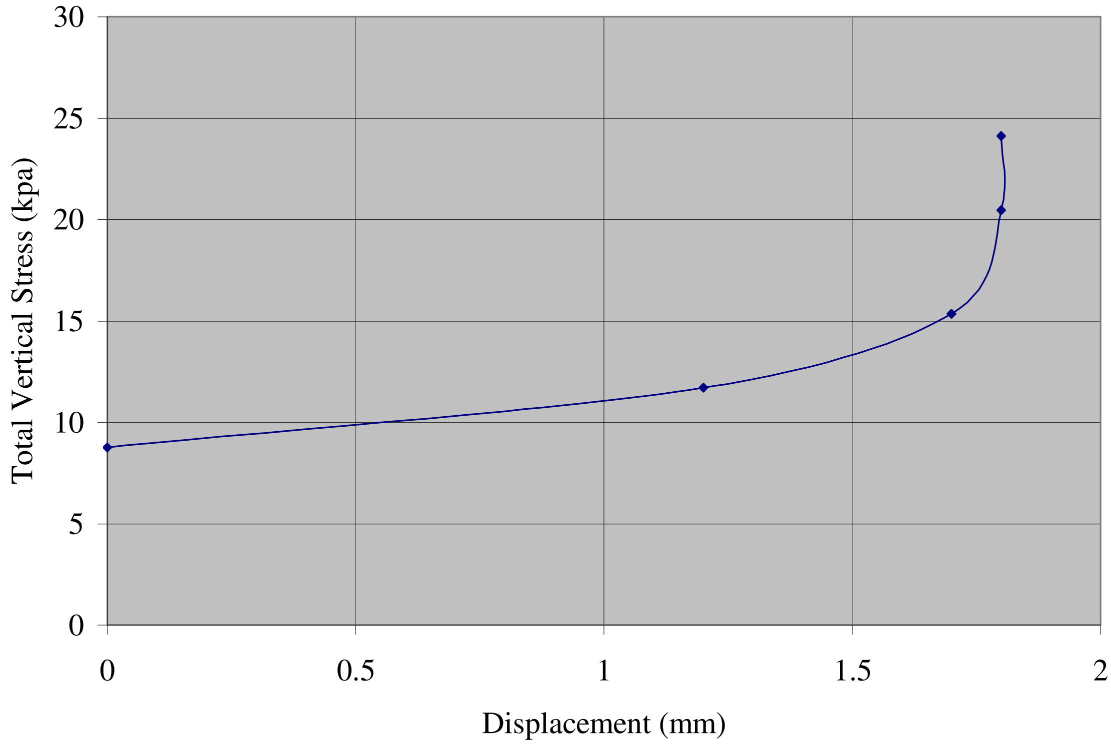
$(B\text{-}1)_4$ - $(B\text{-}4)_4$: Fourth model

First Model Test

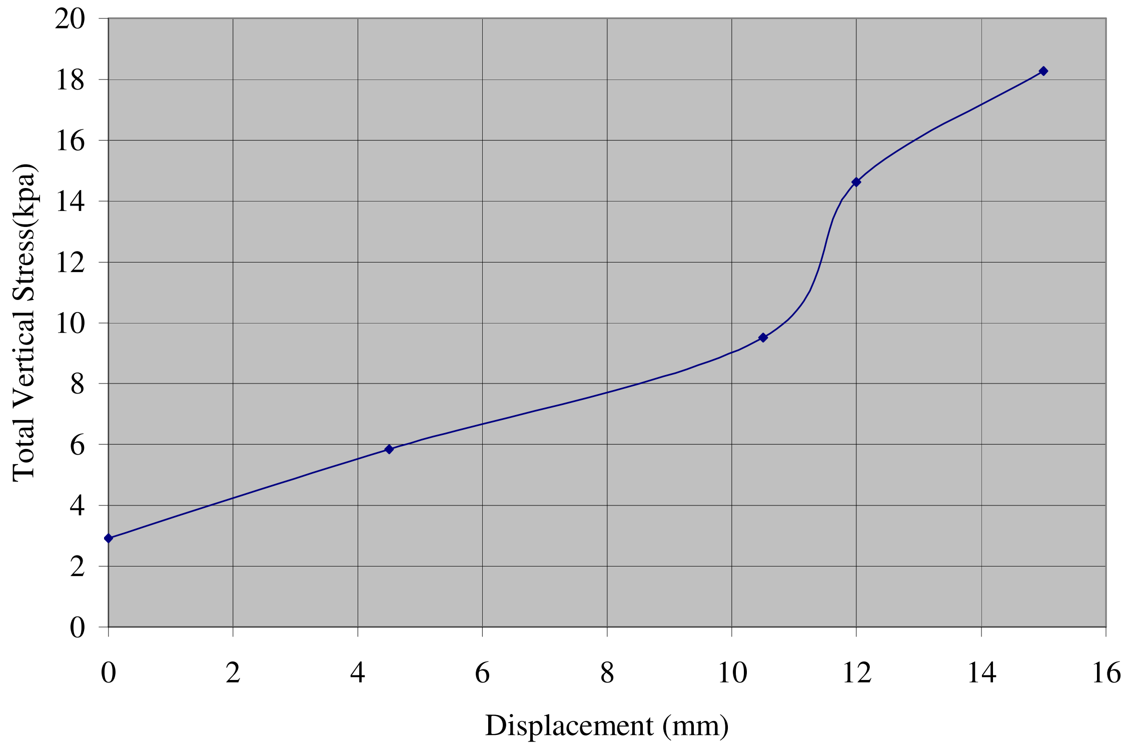
Thickness of over burden(m)	Dial Gauge Reading on points				Remark
	A-1	A-2	B-1	B-2	
0.2	0	0	0	0	Initial reading
0.4	1.1	4.2	1.2	4.5	
0.65	1.7	10.5	2	10.5	
1.0	1.8	11.4	2.1	12	
1.25	1.9	14.5	2.2	15	

A Series				B Series			
A-1		A-2		B-1		B-2	
Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)
0	8.94	0	2.98	0	8.94	0	2.98
1.1	11.92	4.2	5.96	1.2	11.92	4.5	5.96
1.7	15.65	10.5	9.69	1.7	15.65	10.5	9.69
1.8	20.86	11.4	14.90	1.8	20.86	12	14.90
1.9	24.59	14.5	18.63	1.8	24.59	15	18.63

Wall Face Displacement with Height (mm)			
A		B	
0	0	0	0
1.9	200	1.8	200
14.5	600	15	600



a. Total vertical stress versus displacement for (B- 1)1



b) Total vertical stress versus displacement for (B- 2)1

Figure A-1: Selected reinforcement steel displacement against total vertical stress for the first model wall.

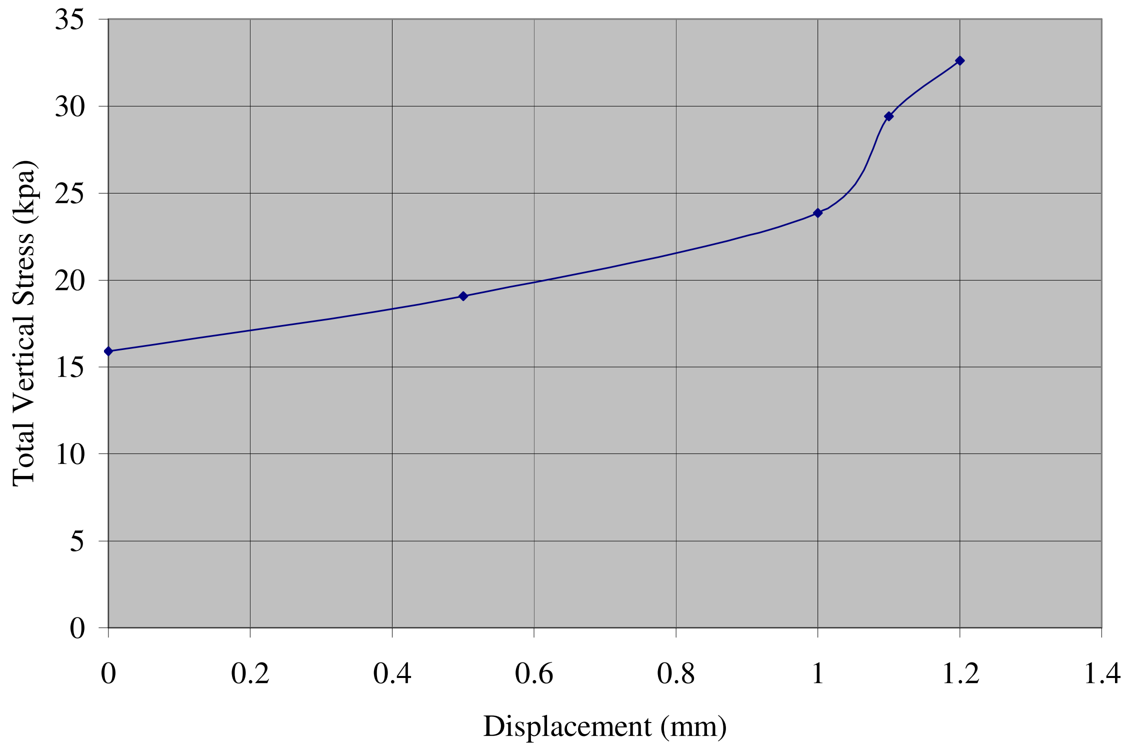
Second Model Test

	A Series					
	A-1		A-2		A-3	
	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)
0.2	5	0	0	0	0	0
0.4	5.5	0.5	1.2	1.2	2.5	2.5
0.7	6	1	2	2	7	7
1.05	6.1	1.1	2.5	2.5	7.4	7.4
1.25	6.1	1.1	2.5	2.5	8.5	8.5
B Series						
	B-1		B-2		B-3	
Thickness of over burden(m)	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)	Dial Reading	
0.2	0	0	5	0	1	0
0.4	0.5	0.5	6.4	1.4	5	4
0.7	0.9	0.9	7	2	8.5	7.5
1.05	1	1	7.7	2.7	10	9
1.25	1	1	7.7	2.7	12	11
C Series						
	C-1		C-2		C- 3	
Thickness of over burden(m)	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)
0.2	0	0	3.5	0	5.5	0
0.4	0.5	0.5	5	1.5	10.8	5.3
0.7	1	1	5.9	2.4	11.5	6
1.05	1.1	1.1	6	2.5	12.5	7
1.25	1.2	1.2	6.1	2.6	15	9.5

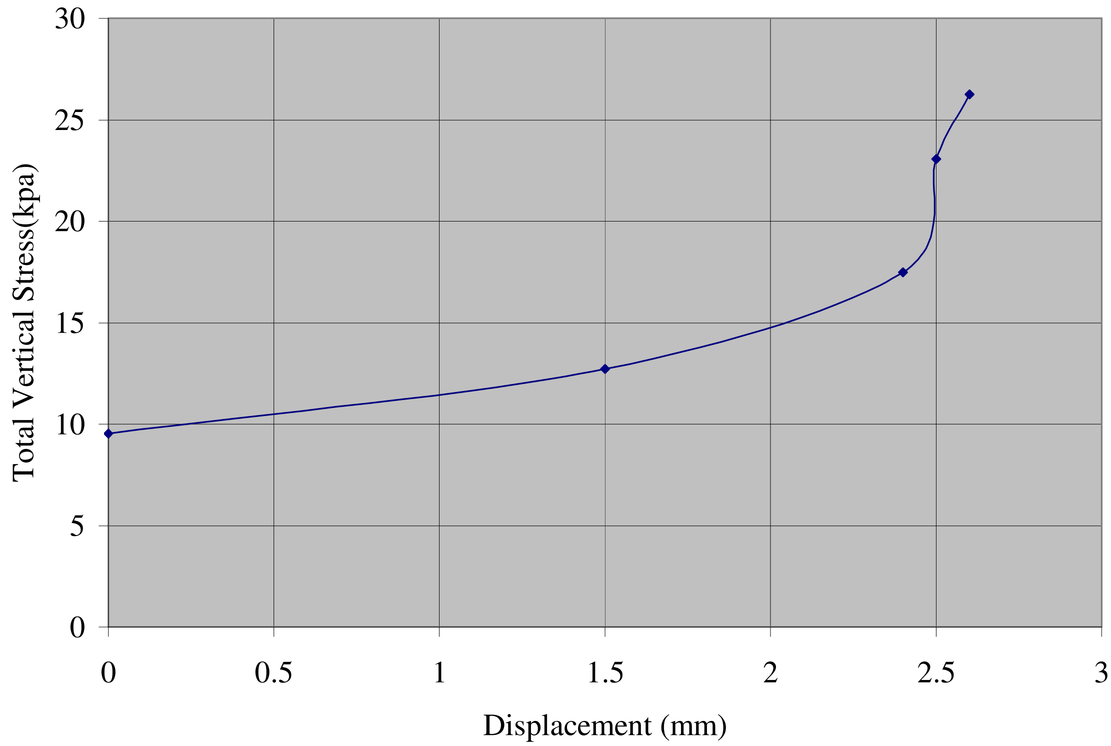
A series					
A-1		A-2		A-3	
Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm))	Total Vertical Stress (kpa)
0	15.90	0	9.54	0	3.18
0.5	19.08	1.2	12.72	2.5	6.36
1	23.85	2	17.49	7	11.13
1.1	29.42	2.5	23.06	7.4	16.70
1.1	32.60	2.5	26.24	8.5	19.88
B Series					
B-1		B-2		B-3	
Disp. of reinfor. (mm))	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)
0	15.90	0	9.54	0	3.18
0.5	19.08	1.4	12.72	4	6.36
0.9	23.85	2	17.49	7.5	11.13
1	29.42	2.7	23.06	10	16.70
1	32.60	2.7	26.24	11	19.88
C Series					
C-1		C-2		C-3	
Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)
0	15.90	0	9.54	0	3.18
0.5	19.08	1.5	12.72	5.3	6.36
1	23.85	2.4	17.49	6	11.13
1.1	29.42	2.5	23.06	7	16.70
1.2	32.60	2.6	26.24	9.5	19.88

Wall Face Displacement with Height (mm)

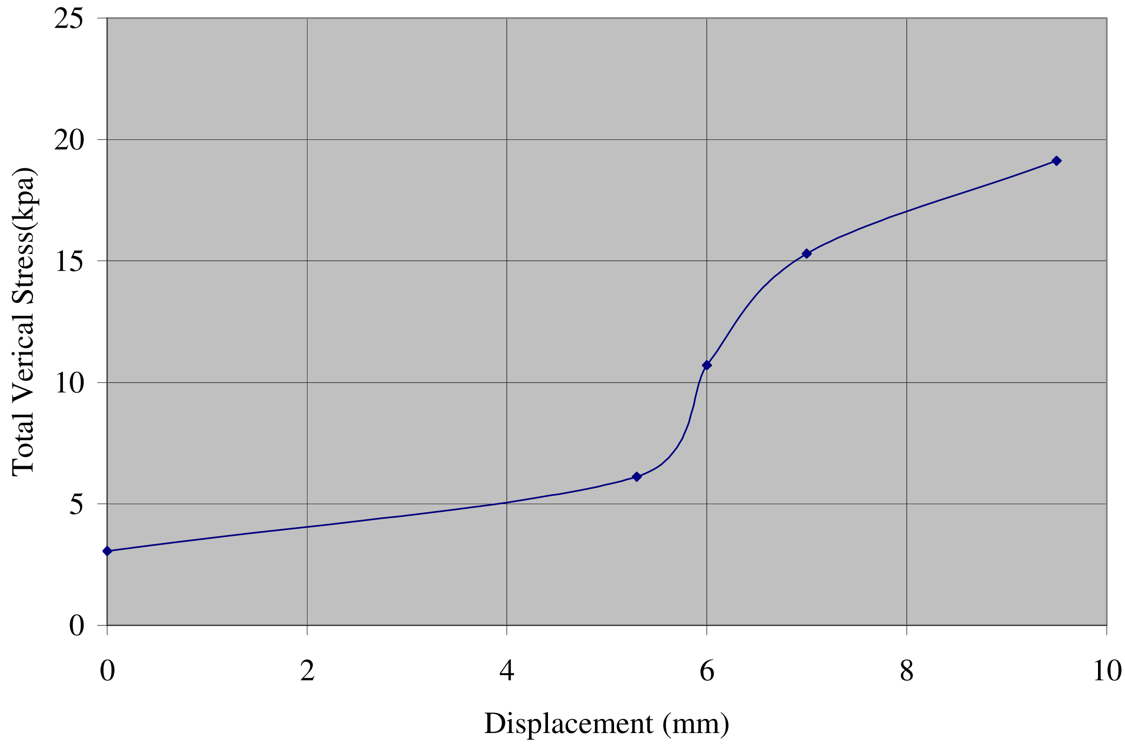
A		B		C	
0	0	0	0	0	0
1.1	200	1	200	1.2	200
2.5	600	2.7	600	2.6	600
8.5	1000	11	1000	9.5	1000



a) Total vertical stress versus displacement for (C- 1)2



b) Total vertical stress versus displacement for (C- 2)2



c) Total vertical stress versus displacement for (C- 3)2

Figure A-2: Selected reinforcement steel displacement against total vertical stress for the second model wall.

The Third model wall test

	A Series							
	A-1		A-2		A-3		A-4	
	Dial Reading	Disp. (mm)	Dial Readi ng	Disp. (mm)	Dial Readin g	Disp. (mm)	Dial Reading	Disp. (mm)
0.2	3.5	0	4.5	0	2.5	0	18.5	0
0.7	3.8	0.3	5.3	0.8	4.3	1.8	20.5	2
0.9	4.2	0.7	6.4	1.9	4.7	2.2	23.1	4.6
1.1	4.3	0.8	6.6	2.1	4.8	2.3	23.4	4.9
1.4	4.4	0.82	6.6	2.1	7	4.5	25.5	7
B Series								
	B-1		B-2		B-3		B-4	
Thickness of over burden(m)	Dial Reading	Disp. (mm)	Dia l Rea din g	Disp. (mm)	Dial Readin g	Disp. (mm)	Dial Reading	Disp. (mm)
0.2	2.5	0	5.5	0	9.7	0	8.6	0
0.7	2.8	0.3	6.2	0.7	11.9	2.2	10.3	1.7
0.9	3.2	0.7	7.1	1.6	13.8	4.1	12.4	3.8
1.1	3.47	0.97	7.2	1.7	14	4.3	12.4	3.8
1.4	3.5	1	7.3	1.8	16	6.3	15.3	6.7
C Series								
	C-1		C-2		C-3		C-4	
Thickness of over burden(m)	Dial Reading	Disp. (mm)	Dia l Rea din g	Disp. (mm)	Dial Readin g	Disp. (mm)	Dial Reading	Disp. (mm)
0.2	4	0	4.8	0	2	0	12	0
0.7	4.2	0.2	5.3	0.5	4.9	2.9	13.8	1.8
0.9	4.5	0.5	5.8	1	6.8	4.8	14.9	2.9
1.1	4.6	0.6	6.1	1.3	7	5	15.1	3.1
1.4	4.6	0.6	6.1	1.3	9	7	18.3	6.3

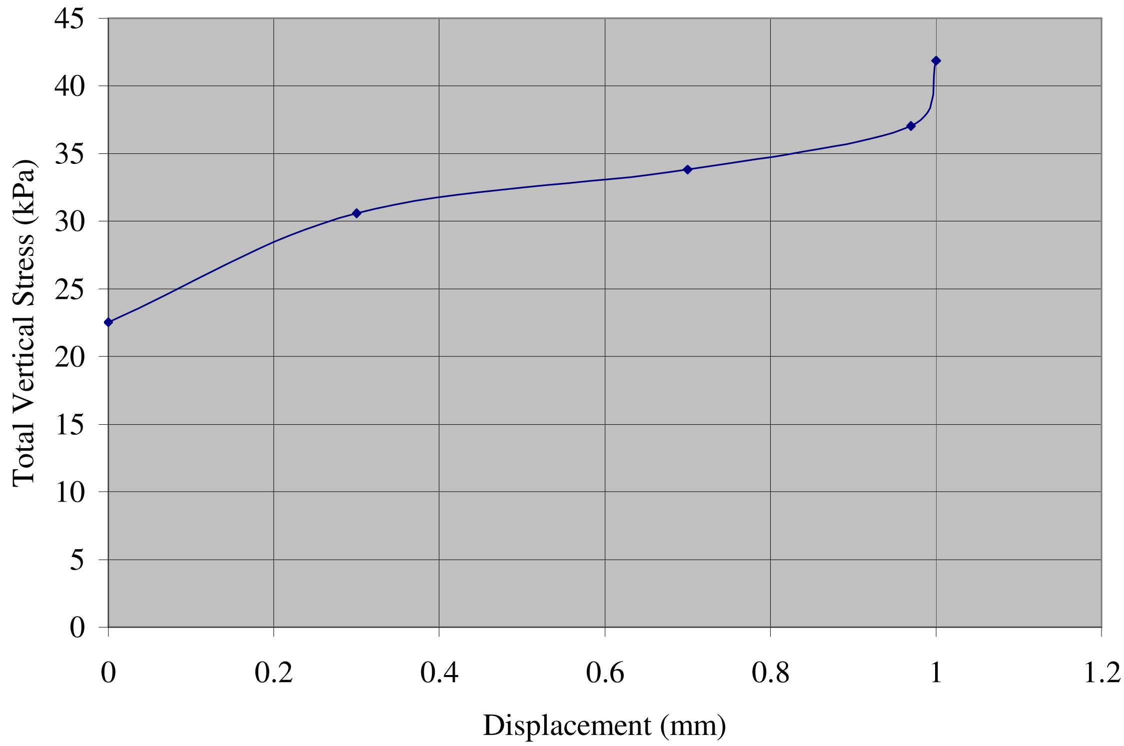
D Series								
	D-1		D-2		D-3		D-4	
Thickness of over burden(m)	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)
0.2	3.6	0	2.2	0	8.5	0	8.5	0
0.7	3.9	0.3	3.1	0.9	10.3	1.8	10.7	2.2
0.9	4.4	0.8	3.8	1.6	12	3.5	12.1	3.6
1.1	4.5	0.9	3.9	1.7	12.2	3.7	12.3	3.8
1.4	4.5	0.9	3.9	1.7	13.8	5.3	13.8	5.3

A Series							
A-1		A-2		A-3		A-4	
Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)
0	22.54	0	16.1	0	9.66	0	3.22
0.3	30.59	0.8	24.15	1.8	17.71	2	11.27
0.7	33.81	1.9	27.37	2.2	20.93	4.6	14.49
0.8	37.03	2.1	30.59	2.3	24.15	4.9	17.71
0.82	41.86	2.1	35.42	4.5	28.98	7	22.54
B Series							
B-1		B-2		B-3		B-4	
Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)
0	22.54	0	16.1	0	9.66	0	3.22
0.3	30.59	0.7	24.15	2.2	17.71	1.7	11.27
0.7	33.81	1.6	27.37	4.1	20.93	3.8	14.49
0.97	37.03	1.7	30.59	4.3	24.15	3.8	17.71
1	41.86	1.8	35.42	6.3	28.98	6.7	22.54

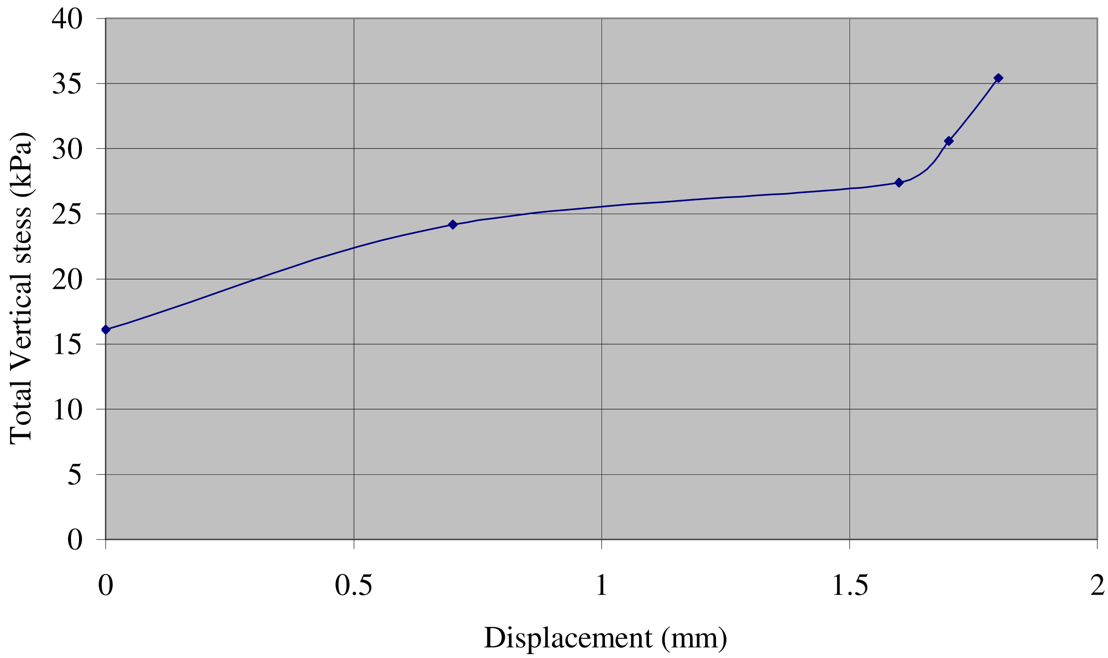
C Series							
C-1		C-2		C-3		C-4	
Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)
0	22.54	0	16.1	0	9.66	0	3.22
0.2	30.59	0.5	24.15	2.9	17.71	1.8	11.27
0.5	33.81	1	27.37	4.8	20.93	2.9	14.49
0.6	37.03	1.3	30.59	5	24.15	3.1	17.71
0.6	41.86	1.3	35.42	7	28.98	6.3	22.54
D Series							
D-1		D-2		D-3		D-4	
Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)
0	22.54	0	16.1	0	9.7	0	3.22
0.3	30.59	0.9	24.15	1.8	17.71	2.2	11.27
0.8	33.81	1.6	27.37	3.5	20.93	3.6	14.49
0.9	37.03	1.7	30.59	3.7	24.15	3.8	17.71
0.9	41.86	1.7	35.42	5.3	28.98	5.3	22.54

Wall Face Displacement with Height (mm

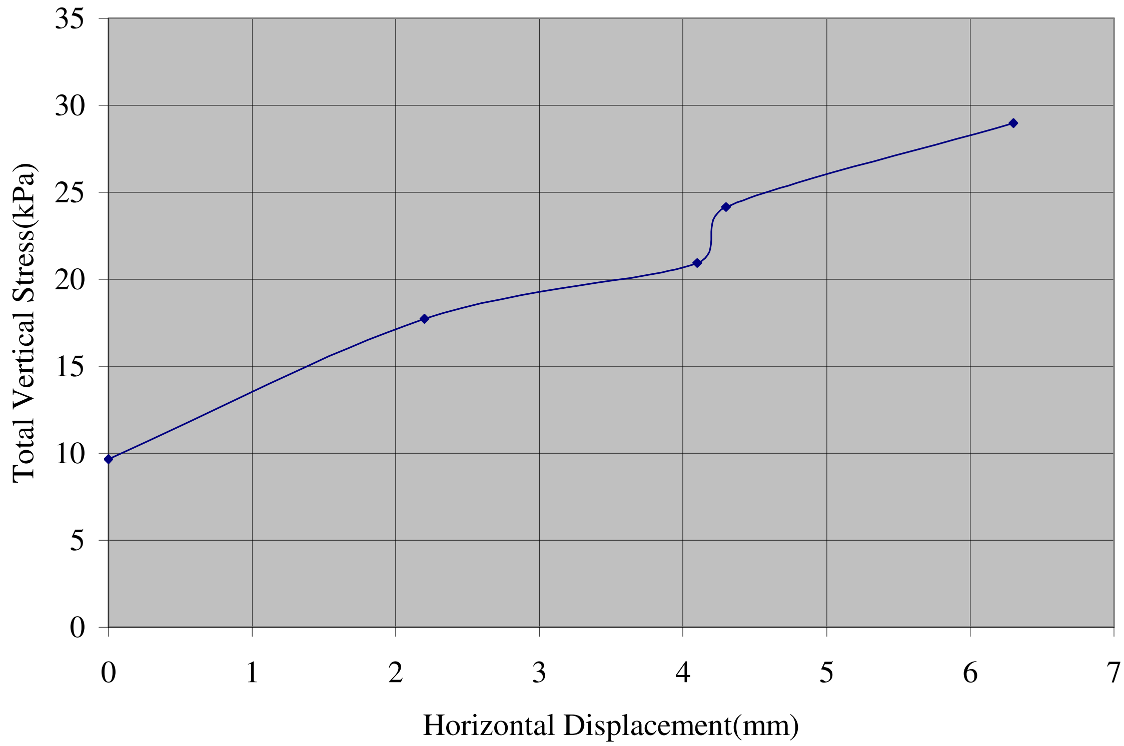
A		B		C		D	
0	0	0	0	0	0	0	0
0.82	200	1	200	0.6	200	0.9	200
2.1	600	1.8	600	1.3	600	1.7	600
4.5	1000	6.3	1000	7	1000	5.3	1000
7	1400	6.7	1400	6.3	1400	5.3	1400



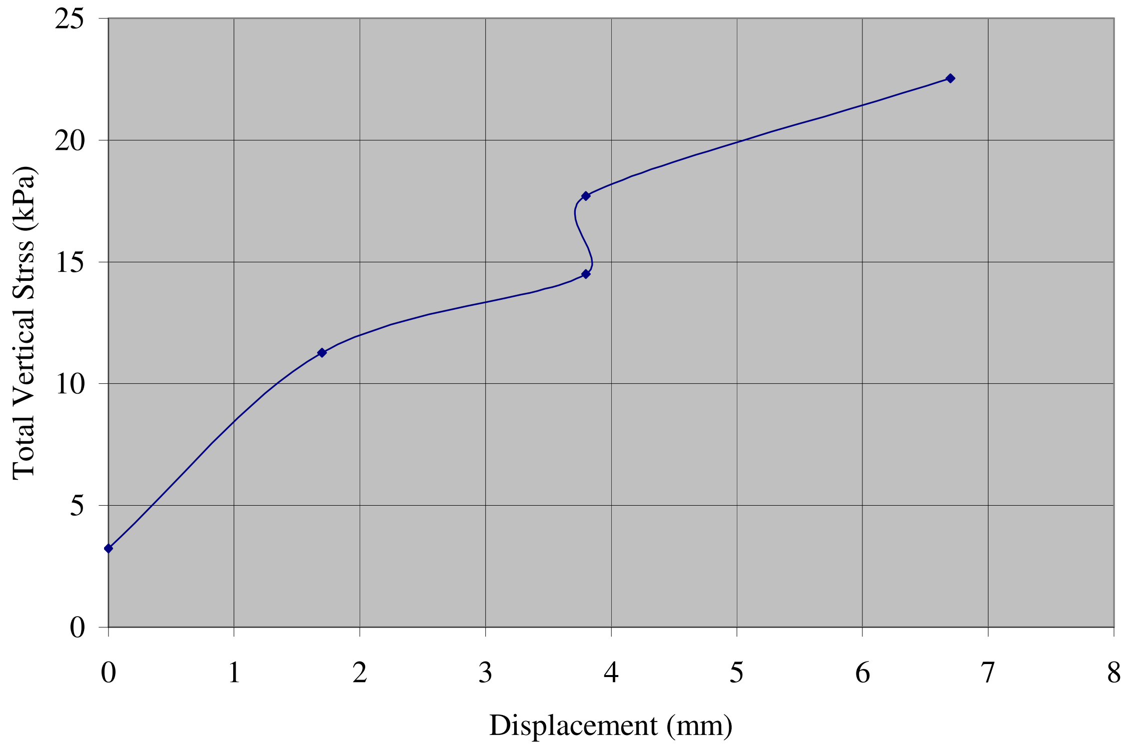
a) Total vertical stress versus displacement for (B- 1)3



b) Total vertical stress versus displacement for (B- 2)3



c) Total vertical stress versus displacement for (B- 3)3



d) Total vertical stress versus displacement for (B- 4)3

Figure A-3: Selected reinforcement steel displacement against total vertical stress for the third model wall.

The Forth model wall test

	A Series							
	A-1		A-2		A-3		A-4	
	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)	Dial Readi ng	Disp. (mm)
0.2	4.5	0	3.8	0	5.5	0	17.3	0.00
0.6	4.7	0.2	4.2	0.4	5.9	0.4	17.45	0.15
0.9	5	0.5	4.9	1.1	6.2	0.7	17.45	0.15
1.4	5.1	0.6	5	1.2	6.3	0.8	17.45	0.15
1.7	5.1	0.6	5	1.2	6.3	0.8	17.45	0.15
	B Series							
	B-1		B-2		B-3		B-4	
Thickness of over burden(m)	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)	Dial Readi ng	Disp. (mm)
0.2	2.1	0	4.5	0	3.4	0	3.3	0
0.6	2.4	0.3	4.9	0.4	3.8	0.4	3.5	0.2
0.9	2.7	0.6	5.5	1	4.1	0.7	3.5	0.2
1.4	2.8	0.7	5.7	1.2	4.3	0.9	3.5	0.2
1.7	2.8	0.7	5.7	1.2	4.3	0.9	3.5	0.2
	C Series							
	C-1		C-2		C-3		C-4	
Thickness of over burden(m)	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)	Dial Readi ng	Disp. (mm)
0.2	3	0	6.8	0	4	0	3.6	0
0.6	3.1	0.1	7.2	0.4	4.3	0.3	3.9	0.3
0.9	3.4	0.4	7.6	0.8	4.5	0.5	3.9	0.3
1.4	3.5	0.5	7.8	1	4.7	0.7	3.9	0.3
1.7	3.5	0.5	7.8	1	4.7	0.7	3.9	0.3

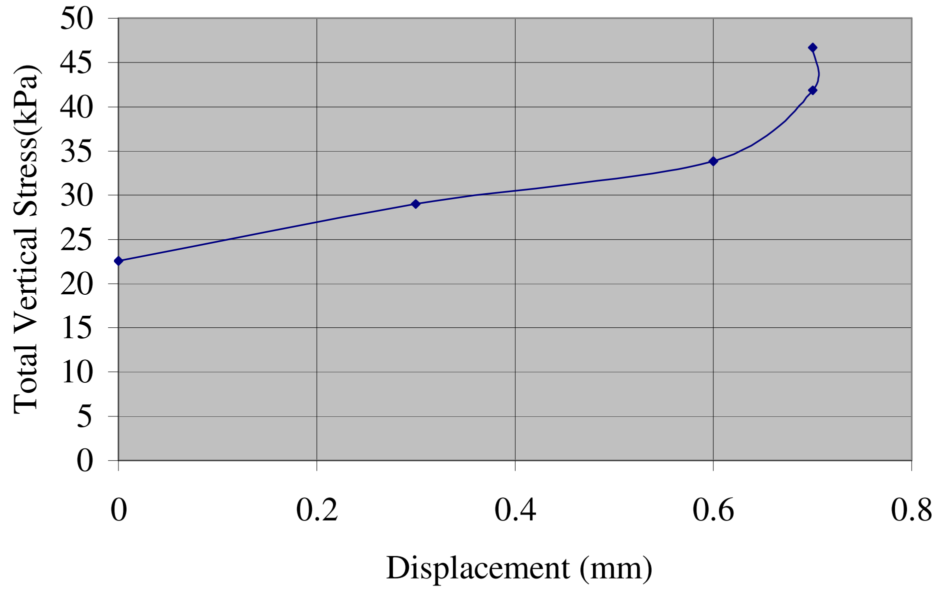
D Series								
	D-1		D-2		D-3		D-4	
Thickness of over burden(m)	Dial Reading	Disp. (mm)	Dial Reading	Disp. (mm)	Dial Readin g	Disp. (mm)	Dial Reading	Disp. (mm)
0.2	4	0	5.8	0	6	0	8.5	0
0.6	4.1	0.1	6.2	0.4	6.4	0.4	8.6	0.1
0.9	4.4	0.4	6.5	0.7	6.6	0.6	8.7	0.2
1.4	4.5	0.5	6.8	1	6.8	0.8	8.7	0.2
1.7	4.5	0.5	6.8	1	6.8	0.8	8.7	0.2

A series							
A-1		A-2		A-3		A-4	
Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)
0	22.4	0	16.0	0	9.6	0.00	3.2
0.2	28.8	0.4	22.4	0.4	16.0	0.15	9.6
0.5	33.6	1.1	27.2	0.7	20.8	0.15	14.4
0.6	41.6	1.2	35.2	0.8	28.8	0.15	22.4
0.6	46.4	1.2	40.0	0.8	33.6	0.15	27.2
B Series							
B-1		B-2		B-3		B-4	
Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)
0	22.4	0	16.0	0	9.6	0	3.2
0.3	28.8	0.4	22.4	0.4	16.0	0.2	9.6
0.6	33.6	1	27.2	0.7	20.8	0.2	14.4
0.7	41.6	1.2	35.2	0.9	28.8	0.2	22.4
0.7	46.4	1.2	40.0	0.9	33.6	0.2	27.2

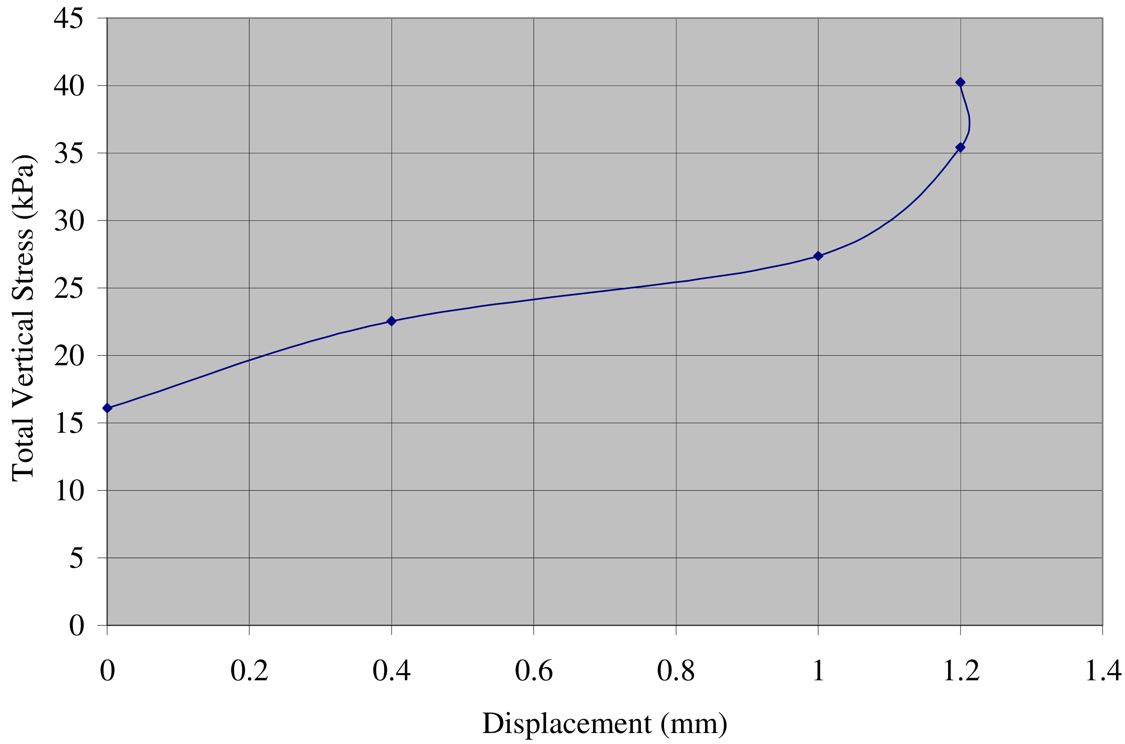
C Series							
C-1		C-2		C-3		C-4	
Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)
0	22.4	0	16.0	0	9.6	0	3.2
0.1	28.8	0.4	22.4	0.3	16.0	0.3	9.6
0.4	33.6	0.8	27.2	0.5	20.8	0.3	14.4
0.5	41.6	1	35.2	0.7	28.8	0.3	22.4
0.5	46.4	1	40.0	0.7	33.6	0.3	27.2
D Series							
D-1		D-2		D-3		D-4	
Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)	Disp. of reinfor. (mm)	Total Vertical Stress (kpa)
0	22.4	0	16.0	0	9.6	0	3.2
0.1	28.8	0.4	22.4	0.4	16.0	0.1	9.6
0.4	33.6	0.7	27.2	0.6	20.8	0.2	14.4
0.5	41.6	1	35.2	0.8	28.8	0.2	22.4
0.5	46.4	1	40.0	0.8	33.6	0.2	27.2

Wall Face Displacement with Height (mm)

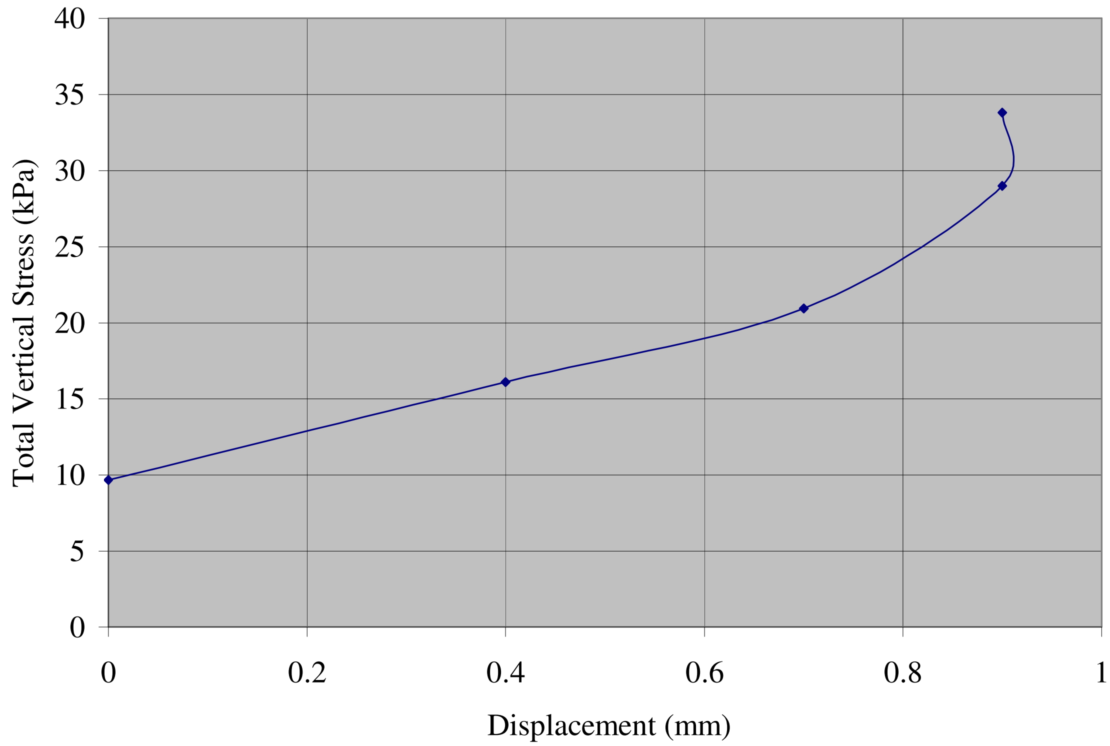
A		B		C		D	
0	0	0	0	0	0	0	0
0.6	200	0.7	200	0.5	200	0.5	200
1.2	600	1.2	600	1	600	1	600
0.8	1000	0.9	1000	0.7	1000	0.8	1000
0.15	1400	0.2	1400	0.3	1400	0.2	1400



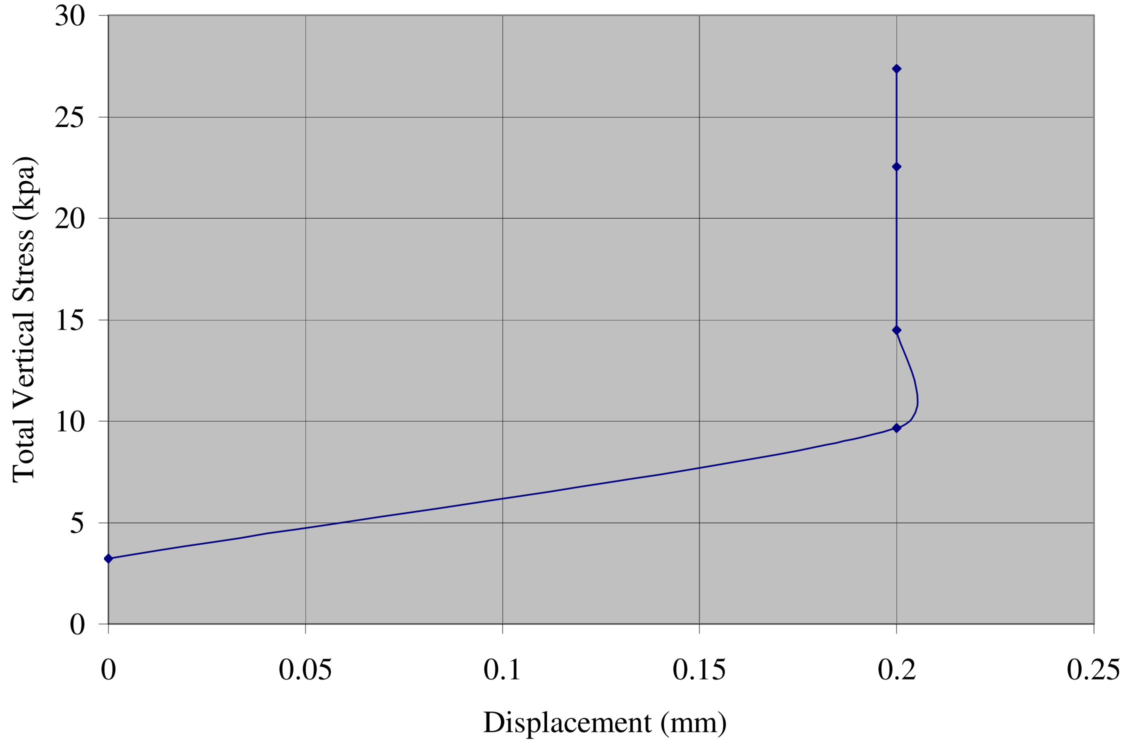
a) Total vertical stress versus displacement for (B- 1)4



b. Total vertical stress versus displacement for (B- 2)4



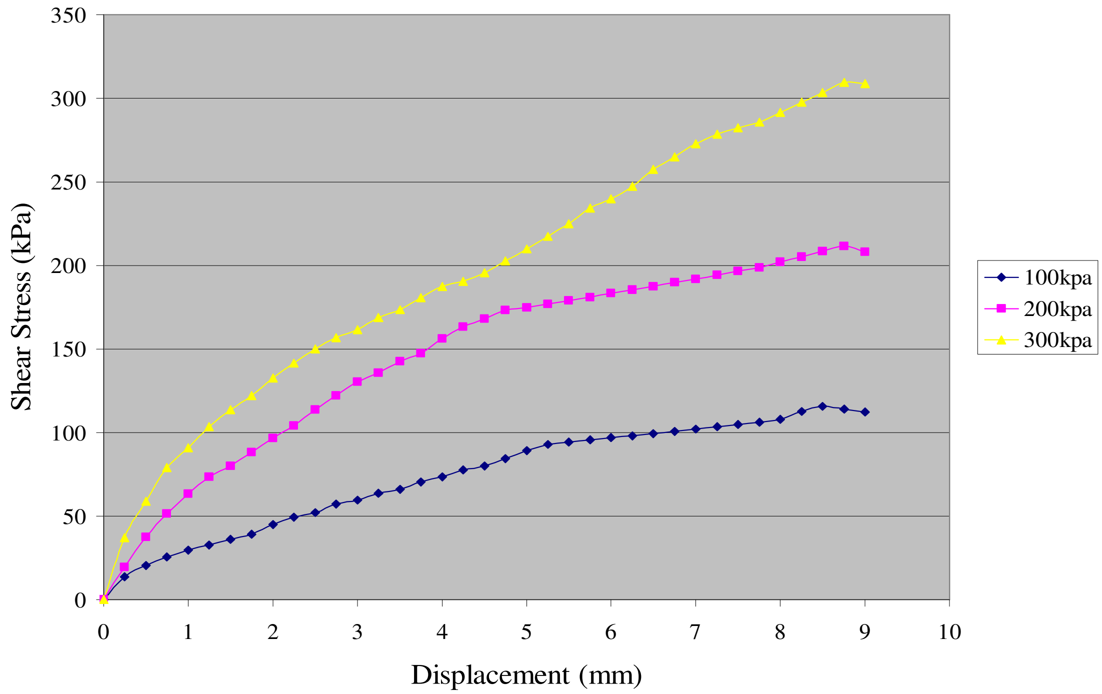
c. Total vertical stress versus displacement for (B- 3)4



d) Total vertical stress versus displacement for (B- 4)4

Figure A-4: Selected reinforcement steel displacement against total vertical stress for the fourth model wall.

II. **APPENDIX B:** The Laboratory Test Data



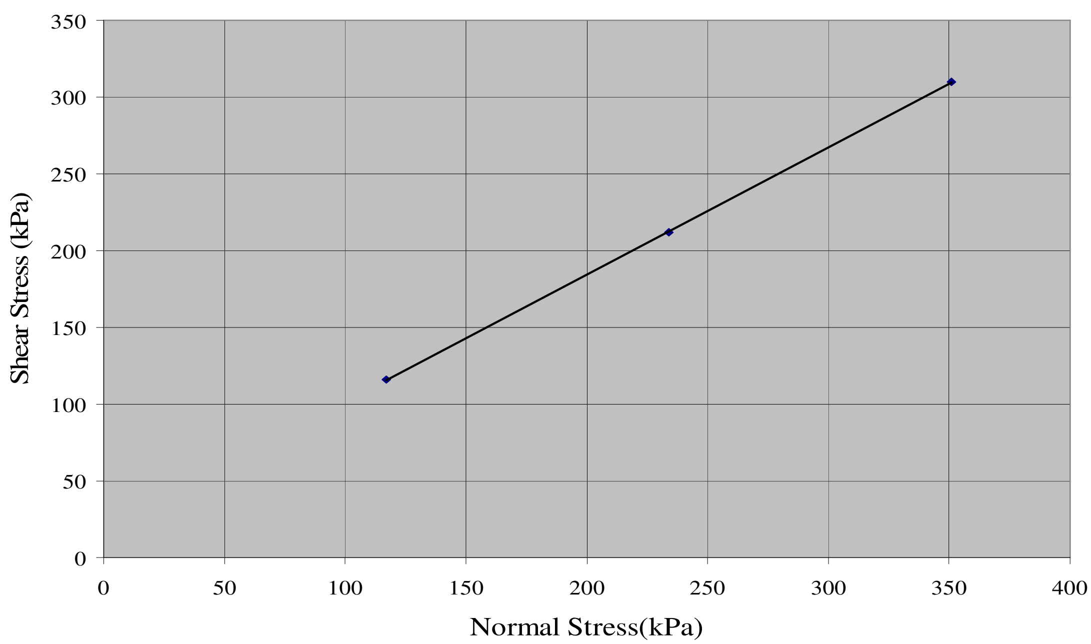
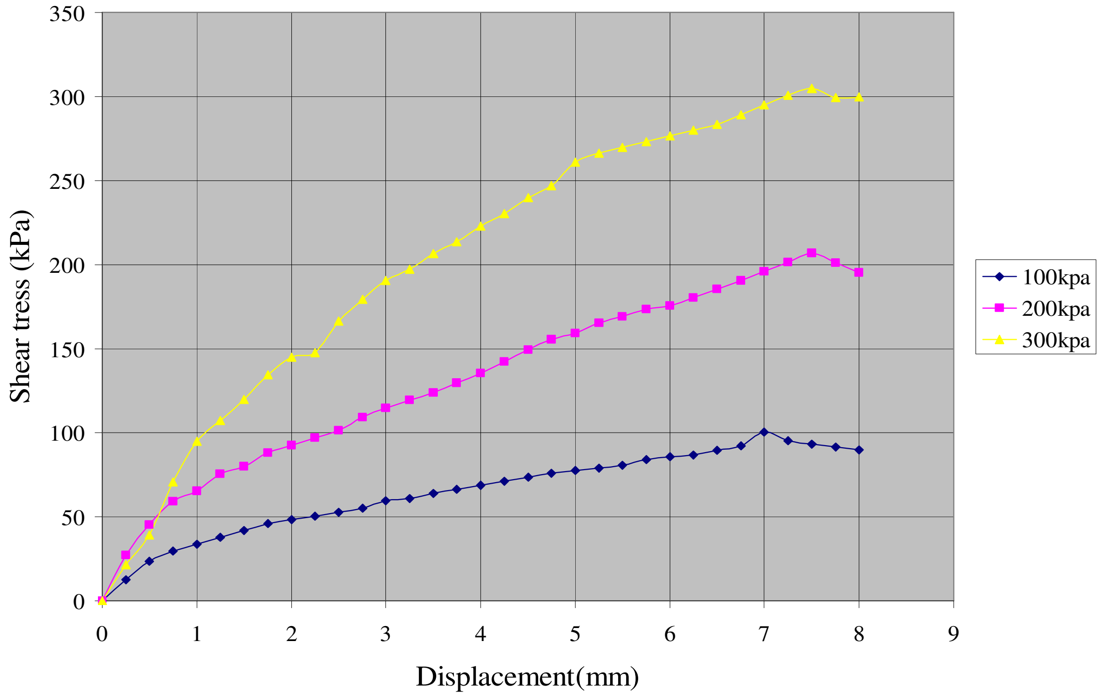


Figure B-1: Direct Shear Test Result for the First Model Wall



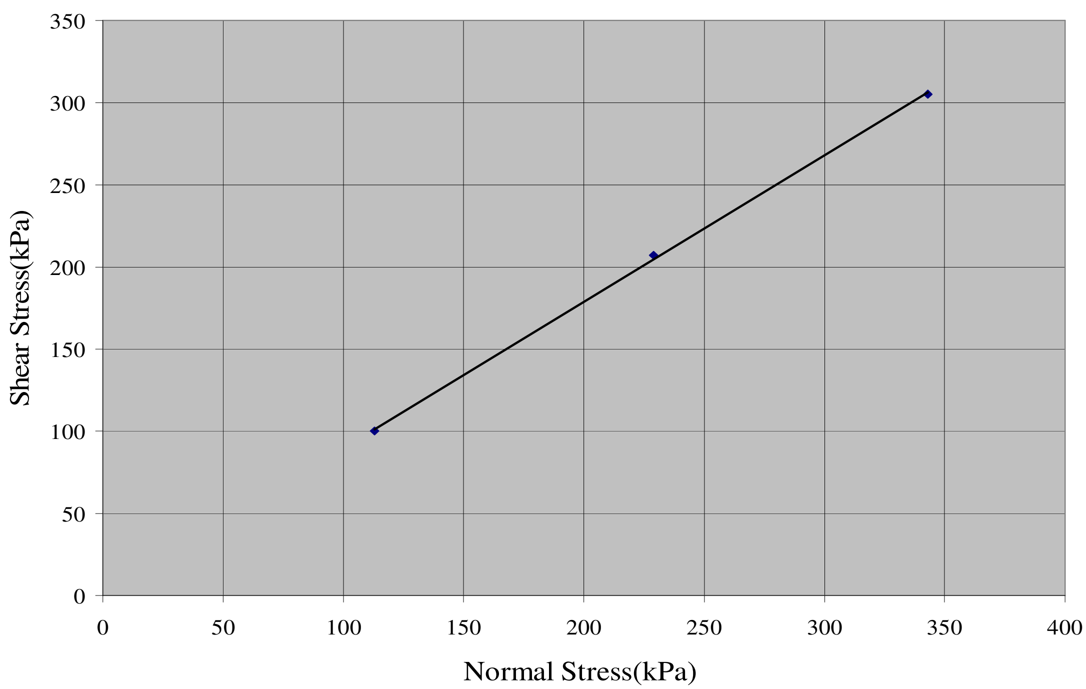
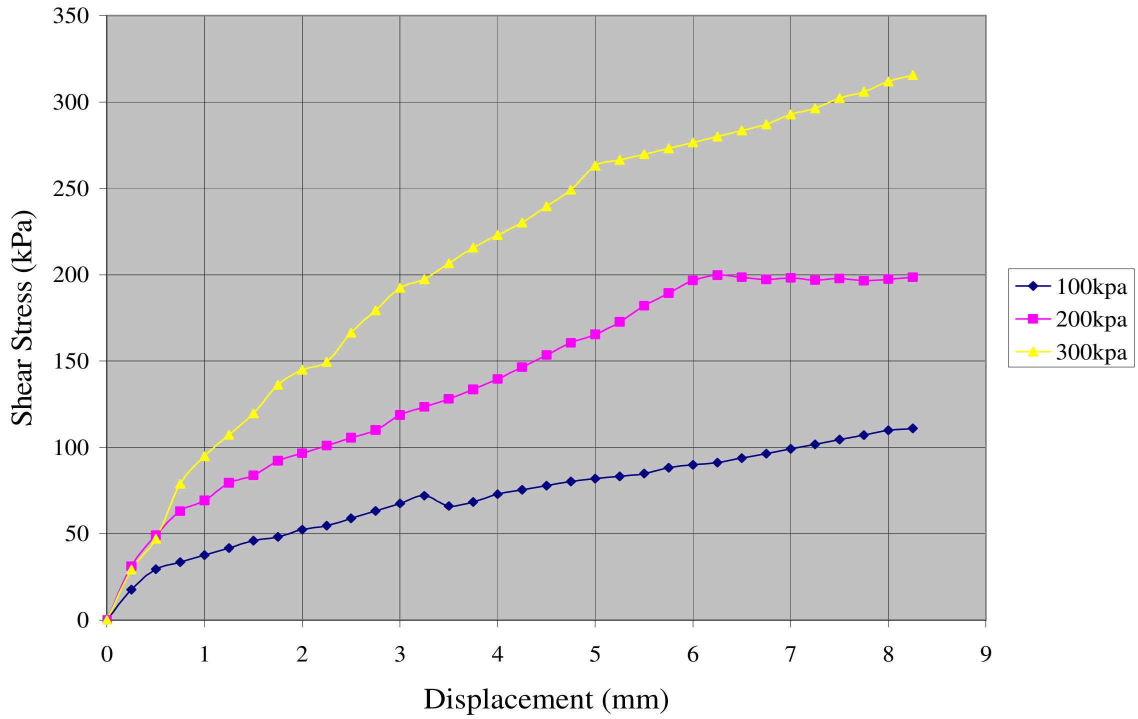


Figure B-2: Direct Shear Test Result for the Second Model Wall



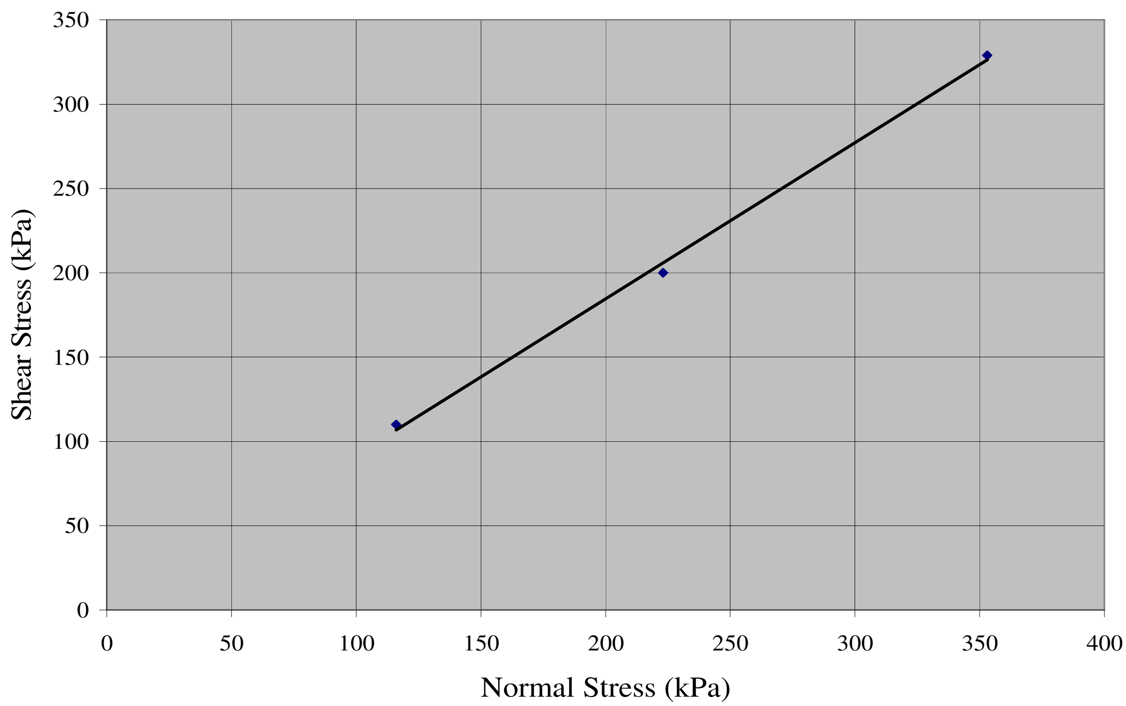


Figure B-3: Direct Shear Test Result for the Third Model Wall

III. **APPENDIX C:** The Test Model

Figure C-1: Side walls under construction

Figure C-2: The First model retaining wall, measuring the horizontal displacement of reinforcement steel by dial gauge

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Declaration

I, the undersigned, declare that this thesis is my original my original work has not been presented for a degree in any other universities and that all sources of material used for this thesis have been duly acknowledged.

Name: **Tilahun Tadesse**

Signature:

Place and date of submission: Addis Ababa University
Faculty of technology
School of Graduate Studies
October, 2006