STUDY OF PULLOUT RESISTANCE OF SOIL NAILS IN TROPICAL RESIDUAL SOIL

Eng. Ranjan Kumara W.E.P.

(Index No - 118811k)

Supervised By prof: S.A.S Kulathilaka

M.Eng IN FOUNDATION ENGINEERING AND EARTH RETAINING SYSTEMS

DEPARTMENT OF CIVIL ENGINEERING

UNIVERSITY OF MORATUWA

(2011/2012 BATCH)

STUDY OF PULLOUT RESISTANCE OF SOIL NAILS IN TROPICAL RESIDUAL SOIL

Eng. Ranjan Kumara W.E.P.
(Index No - 118811k)

Degree of Master of Engineering

DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF MORATUWA
Sri Lanka

June 2016

STUDY OF PULLOUT RESISTANCE OF SOIL NAILS IN TROPICAL RESIDUAL SOIL

Eng. Ranjan Kumara W.E.P.

(Index No - 118811k)

Thesis Submitted To University Of Moratuwa In Partial Fulfillment Of The Requirements For The Degree of Master of Engineering in Foundation Engineering And Earth Retaining Systems.

DEPARTMENT OF CIVIL ENGINEERING

UNIVERSITY OF MORATUWA

Sri Lanka

June 2016

DECLARATION

I declare that, this is my own work and this thesis does not incorporate without acknowledgement any material previously submitted for a Degree or Diploma in any University or other institute of higher learning and to the best of my knowledge and belief it does not contain any material previously published or written by another person except where the acknowledgement is made in the text.

Also, I hereby grant the University of Moratuwa the non-exclusive right to reproduce and distribute my thesis, in whole or in part in print, electronic or other medium. I retain the right to use this content in whole or part in future works (such as articles or books).

Signature:………………………. Date:…………………

The above candidate has carried out research for the Master Dissertation under my supervision.

Signature of the supervisor:………………………….. Date:…………………….

Professor S A S Kulathilaka

BSc Eng Hons (Moratuwa), PhD (Monash), CEng, MIE (SL)

Professor
Department of Civil Engineering,
University of Moratuwa,
Moratuwa,
Sri Lanka.

ABSTRACT

As a norm, most design engineers typically resort to theoretical and empirical approaches in order to determine the pullout resistance of soils while designing soil nailed retaining walls. The tendency to design based on actual field tests are minimal due to the time and cost involved while implementing such tests. Though results obtained through pull out tests done within the laboratory have been used to perform design calculations, the outcome of such test results are questionable, as such tests do not replicate precise site conditions.

This research primarily juxtaposes and establishes a relationship between the theoretical and on field practical pullout resistance of soil nails in unsaturated conditions with the use of information extracted from an extensive literature review and data obtained through an actual pull out test conducted on a set of soil nails installed in predetermined locations of a 25ft high embankment spanning 70ft.

This research also attempts to explore the effects of over burden pressure on the pull out resistance of the soil nails and the behavior of the actual failure surface of the soil nail, which has also been mentioned as the effective diameter in this report.

ACKNOWLEDGEMENT

It is my pleasure to take this opportunity to thank all those who have been supportive to me to successfully complete this research. First of all I sincerely express my gratitude to my research supervisor Prof. S.A.S. Kulathilaka for offering me the opportunity to carry out the project under his wise guidance.

Next my heartily gratitude goes out to the lectures and the supporting staff of the Geotechnical Engineering division of the Department of Civil Engineering, University of Moratuwa, who have assisted and supported me throughout the research period.

I would like to thank the management of the Central Engineering Consultancy Bureau for encouraging and assisting me to venture on this research, the Central Engineering Consultancy Bureau Laboratory services and the Engineering & Laboratory Services (Pvt) Ltd for rendering invaluable support while performing on field and laboratory tests.

Finally, I would also like to thank the site staff of the Operation Theatre Complex Building, Teaching Hospital, Kegalle for voluntarily involving themselves in the field during various stages of this research.

Eng Ranjan Kumara W.E.P (Index No: 118811K)

Figure No	Description	Page No

List of Symbols

$(u_a - u_w)$	Matric suction
$(\sigma_n - u_a)$	Net normal stress
A_s	Surface area of the nail
D_{eq}	Equivalent width of flat reinforcement
K_s	Coefficient of lateral earth pressure
$Q(u_a-u_w)$	Capacity of soil nails due to the contribution of matric suction
Q_f	Capacity of soil nails installed in saturated soils
C'	Soil cohesion
c_a	Soil adhesion at the grout/soil interface
d_i^d	Minimum grain diameter of the corresponding fraction
d_i^g	Maximum grain diameter of the corresponding fraction
k_θ	Coefficient of lateral earth pressure with respect to soil nail inclination
q_s	Shaft capacity of piles
δ	Interface friction angle
θ_s	Saturated volumetric water content
μ^*	Coefficient of apparent friction of soil ($\mu^* = \tan \phi'$ and $c' = 0$ for granularsoil)
σ'_n	Effective normal stress
σ'_v	Vertical stress calculated at the mid-depth of the nail in the resistance zone
σ'_z	Effective overburden stress
T_{US}	Shear strength of unsaturated soils
ϕ'	Angle of internal friction of soil
ϕ^b	The angle of shearing resistance with respect to matric suction
Δ_{gi}	Fraction weight in parts of the total weight
cu	Coefficient of uniformity
D	Nail diameter (m)
d_e	Dominant particle size diameter, *mm*
e	Void ratio
f_b	Coefficient of roughness
f_c	Coefficient defined by c_a/c'
f_s	Coefficient defined by δ/ϕ
L_s	Embedment depth of soil nail (m)

List of Symbols Abbreviations

m	Soil parameter related to residual water content
n	Soil parameter related to the slope at the inflection point of the SWCC
P	*N*nail perimeter
$\mathbf{S_r}$	Residual degree of saturation
$T_{pull\text{-}out}$	Failure load at which pull-out failure occurs (kN)
$\mathbf{W_r}$	Residual gravimetric water content
$\mathbf{W_s}$	Saturated gravimetric water content
$\mathbf{w_w}$	Gravimetric water content
ψ	Dilation angle.
$C\ (\psi)$	Correction factor that forces the SWCC through a suction of 1,000,000 kPa and zero water content.
S	Degree of saturation
β	Bjerrum –Burland coefficient
θ	Volumetric water content
K	Fitting parameter used for obtaining a best-fit between the measured and predicted values.
λ	Pull-out factor
κ	Parameter dependent on the degree of saturation (varies from 0 to 1)

List of Abbreviations

SW	Well-graded sand
SP	Poorly graded sand
CDG	Completely decomposed granite
CFEM	Canadian Foundation Engineering Manual
CU	Consolidated undrained
DAS	Data Acquisition System
FHWA	Federal Highway Administration
GSD	Grain size distribution
GWT	Ground-water table
HSS	Hollow Steel Section
NATM	New Austrian Tunneling Method
M	Silty sand
SPT	Standard penetration test
SWCC	Soil-water characteristic curve

TYP	Typical
USCS	Unified soil classification system
WWM	Welded wire mesh

1.0 Introduction

Soil nailing is a widely used slope stabilization technique utilizing passive elements (referred to as nails) for retaining soils and enhancing stability. The soil nails are typically subjected to tension when the retained soil moves. The fundamental design principle of soil nails consists of transferring the resisting tensile forces generated in the soil nails into the ground behind the moving mass through friction, mobilized at the grout/soil interface. The load transfer mechanism and the ultimate pull-out capacity of soil nails depends primarily on; strength characteristics of soil, tensile strength of reinforcements, installation technique, geometry of drilled hole and the grouting method. The soil nailing technique has been found to be suitable for supporting excavations, tunnel portals, slope stabilization, bridge abutments and several other civil engineering applications.

Soil nails have been utilized increasingly in recent years due to its technical and economic advantages. The equipment used for soil nailing facilitates quick and easy construction and contribute to significant savings (Powell and Watkins, 1990). Soil nailing applications are best suited for placement above the ground water table, where the soil is in a state of unsaturated condition. Approximately 33% of the earth's surface constitute of arid or semi-arid regions where the soils are typically unsaturated compacted soils and soils in regions other than arid and semi-arid regions are also found in a state of unsaturated condition. When the ground water table is deep, the stresses associated with the constructed infrastructures are distributed in the zone above the ground water table (Vanapalli and Oh, 2010). Shallow foundations, retaining walls and pavement structures are typical examples that fall in to this category. Classical soil mechanics theories applicable to saturated soils are conventionally used in the design of such geotechnical structures, including soil nails without considering the contribution of suction or the negative pore-water pressures in the vadose zone (i.e., the zone above the ground water table) to the capacity. The key reason for this approach can be attributed to the lack of a simple framework for the analysis and design of geotechnical structures using the mechanics of unsaturated soils (Fredlund and Rahardjo, 1993; Vanapalli and Oh, 2010). In most cases, soil nail structures do not become saturated during their design service life and hence it is more appropriate to use the mechanics of unsaturated soils for the design of these structures.

Soil nailing techniques have been widely used to stabilize slopes and retain excavations. The safety of a soil nailing system depends on the pullout shear stress mobilized at the nail-soil interface. Several critical factors affect shear stress on the soil-reinforcement interface including soil type, drilling method, characteristics of grout, overburden pressure, soil density, soil dialatancy, and degree of saturation (Lazarte et al., 2003; Burland, 2002).

Previous theoretical and experimental investigations have indicated that constrained stress due to soil dilation plays a significant role in mobilizing shear stress on the soil-reinforcement interface (Schlosser, 1982; Schlosser et al., 1983; 1993; Tei, 1993; Chai and Hayashi, 2004). In addition, experimental studies have demonstrated that grouting pressure significantly strengthens the soil-nail pullout resistance while the effect of overburden pressure on pullout resistance of a soil-nail is not clearly understood (Pradhan et al., 2006; Su et al., 2008). The influences of these critical parameters on the maximum shear stress at the soil-reinforcement interface are complex and poorly understood. Therefore additional theoretical and experimental investigations are needed.

The pull-out capacity is a key parameter for the design of soil nails. Limit equilibrium methods are typically used to estimate the total soil nail force required to achieve a specified factor of safety (Junaideen et al., 2004). There are no specific design procedures or method of estimation for the pull-out capacity of soil nails. However, the manual (FHWA-SA-96-069R) specifies that the allowable load for soil anchors should be reduced by a factor of 1.35 based on the estimated capacity. The estimated pull-out capacity of soil nails is commonly verified by field pull-out tests during the early construction stage. Several research studies have been conducted to investigate the behavior of the soil/nail interface during pull-out (Chai et al., 2004; Junaideen et al., 2004; Chu et al., 2005; Yin et al., 2006; Pradhan et al., 2006; 2008, Sivakumar and Singh, 2010). It was reported by Zhang et al. (2009) that matric suction is a key factor that contributes to the uncertainties in the estimation of the pull-out capacity of soil nails. Gurpersaud (2010) studied the influence of the matric suction on pullout capacity of soil nailing with laboratory model studies on a compacted sandy soil.

There were no significant experiments done in the past in South Asia or South East Asia to find the pull out capacity for the tropical soils. This research is carried out under the actual site condition with an unsaturated/saturated soil interface. Soil nails were installed in the embankment on a vertical line. Separate undisturbed soil samples were taken from the corresponding location of the soil nails. Actual pullout resistance and grouted perimeters were measured. Meanwhile the pull out resistance was estimated from various theoretical approaches to make comparisons with the observed values.

1.2 Objectives of the thesis

The key objectives of this research study are as follows:

i. To verify the suitability of the proposed theoretical approaches published in literatures to find the pull-out resistance for the local ground conditions and hence establish the most appropriate approach.

ii. To define a basic co-relation for the measured pullout resistance and the calculated value respect to each design methods.

iii. To verify the effect of overburden pressure on pull out resistance.

iv. To identify the behavior of nail (grouted body) /soil interface at failure.

1.3 Scope of the thesis

i. A comprehensive experimental program was undertaken at actual site conditions to evaluate the pull-out capacity of soil nails. The test nails were installed at 15° to the horizontal in the soil surface with a horizontal spacing of 1.5m.

ii. Undisturbed soil samples were taken from the natural ground adjacent test nails, to find the shear strength parameters of the soil under laboratory condition.

iii. The results obtained from the laboratory experimental program were used to compare the pull-out capacity of soil nails with the theoretical estimates based on shear strength parameters.

1.4 Outline of the thesis

The research program undertaken is summarized in this thesis under seven main chapters. These chapters are organized as follows:
Literature review forms the second chapter in which a detailed review of soil nail pull-out capacity, interface behavior and the mechanics of unsaturated soils are succinctly summarized. General background of soil nailing technique, applications, behavior, mechanism and factors influencing the pull-out capacity are also included in this chapter. Additionally, methods used to estimate the pull-out capacity of soil nails and previous research pertaining to the pullout capacity are also summarized.

The third chapter explains the procedures involved to obtain the pullout capacity of nails and present the result of the pull out tests.

The fourth chapter presents the laboratory testing procedures along with the results obtained.

The fifth chapter discusses the pullout test results, where a detailed evaluation of the results is provided. The measured pulls out resistance are compared with the estimated values from the different methods of estimation currently in use.

Chapter six presents the discussion of the results of the research program, conclusion made and recommendations for future research.

2.0 Literature Review – Pullout Resistance of Soil Nails and Design

2.1 Concept of Soil nailing

The basic concept of soil nailing is to reinforce and strengthen the existing ground by installing closely spaced steel bars, called "nails," into a slope or excavation as construction proceeds from the "top down." This process creates a reinforced section that is itself stable and able to retain the ground behind it. The reinforcements are passive and develop their reinforcing action through nail-ground interactions as the ground deforms both during and following construction.

A small movement of the active zone in a soil nail structure will result in both axial and lateral displacement. The bond strength that mobilizes with these movements will result in an axial force (tension force) in the nail. The mobilization of the axial stresses will occur in a progressive manner. Axial stresses in the nail will be limited by the maximum shear capacity which can be developed between the natural soil and the grouted body of the nail.

Nails work predominantly in tension, but are considered by some to also work in bending/shear under certain circumstances. But large deformations are required to mobilize bending and shear. Generally, the soil nails significantly increase the apparent cohesion of the soil through their ability to carry tensile loads. A construction facing is also usually required. Typically shotcrete facing reinforced by welded wire mesh followed by a cast-in-place concrete facing was used in the early development. In recent times more aesthetically pleasing facing types involving vegetation and erosion protection netting and wire mesh of high tensile strength that combines the nail head are developed. Grid beams are also used to combine the nail heads with vegetation cover being used in the space between the beams.

The soil nailing technique was developed as an extension of the New Austrian Tunneling Method (Rabcewicz, 1964, 1965). The first recorded application of soil nailing was completed in France in 1972 . The soil nailing projects were completed in shorter periods of construction compared to conventional methods and proven to be cost-effective. Some of the pioneering work in this research field was conducted in Germany from 1975 to 1981 by the University of Karlsruhe and Bauer Construction Company (Lazarte et al., 2003). The French engineers have also significantly contributed to this field through a major experimental program called "Clouterre" between 1986 and 1990. The main objectives of the Clouterre program were to provide better understanding of the soil nail walls behavior and their limitations in addition to providing elaborate design recommendations including dimensioning (Plumelle et al., 1990).

The first documented application of soil nailing in North America was the support of a 13.7 m deep foundation excavation in dense silty Lacustrine sand for a project in Portland, Oregon, USA in 1976 (Bryne et al., 1998). This project was completed

approximately in half the time while contributing to a 15% of savings in comparison to the cost of conventional support systems.
The main features of soil nails are as follows:

- Provides an increase in the normal force along potential slip surfaces in frictional soils and hence the shear resistance of the soil is also increased.
- The driving force along potential slip surfaces is reduced in both frictional and cohesive soils.

Soil nails are installed horizontally or sub-horizontally in the excavated soil or the slope.

Soil nailing technique has been extensively used in recent years in many geotechnical projects such as excavations support, slopes and retaining walls stabilization and bridge abutments. The success stories of different projects have encouraged several research studies in various parts of the world to explore the use of soil nails for addressing other geotechnical problems (Sivakumar et al., 2010).

2.2 Construction Sequence

In soil nailing the passive reinforcement may be driven into the cut facing or installed in drilled holes and grouted. Figure 2.1 shows typical details of soil nailing. Typical construction sequence is illustrated in Figure 2.2.

Step 1. Excavation.
Initial excavation is carried out to a depth for which the face of the excavation has the ability to remain unsupported for a short period of time, typically in the order of 24 to 48 hours. The depth of the excavation lift is usually between 1 and 2 m and would be slightly below the elevation where nails will be installed. The width of the excavated platform or bench must be sufficient to provide access to the installation equipment.

Step 2. Drilling Nail Holes.
Drill holes are drilled to a specified length, diameter, inclination, and horizontal spacing from this excavated platform. Hole diameter would be in the range 100 mm to 150mm. The length up to 16m is normally used in practice.

Step 3. Nail Installation and Grouting.
Nail bars are placed in the pre-drilled hole. The bars are most commonly solid, Centralizers are placed around the nails prior to insertion to help maintain alignment within the hole and allow sufficient protective grout coverage over the nail bar. A grout pipe (tremie) is also inserted in the drill hole at this time. The drill hole is then filled with cement grout through the tremie pipe. The grout is commonly placed under gravity or low pressure. Prior to Step 4 (facing placement), geocomposite drainage strips are installed on the excavation face approximately midway between each set of adjacent nails. The drainage strips are then unrolled to the next wall lift. The drainage strips extend to the bottom of the excavation where collected water is conveyed via a toe drain away from the soil nail wall. Alternatively, short drains (1.5 long) in the perforated pipes

can be installed in a grid (1.5m X1.5 m) into the shotcrete facing to facilitate drainage of any water trapped behind.

Step 4. Construction of Temporary Shotcrete Facing.

A temporary facing system is then constructed to support the open-cut soil section before the next lift of soil is excavated. The most typical temporary facing consists of a lightly reinforced shotcrete layer commonly 100 mm thick. Following appropriate curing time for the temporary facing, a steel bearing plate is placed over the nail head protruding from the drill hole. The bar is then lightly pressed into the first layer of fresh shotcrete.

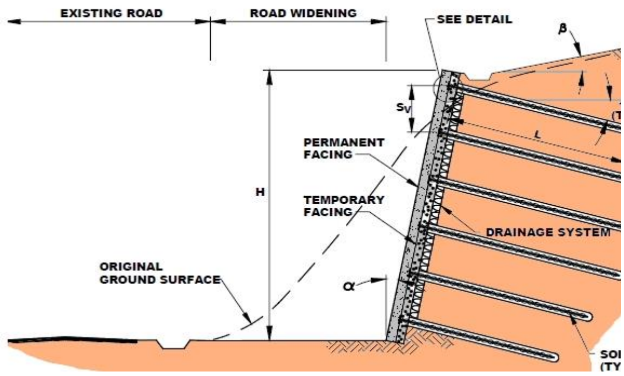

Figure 2.1 – Components of soil nailing
(After Geotechnical Engineering Circular No 7 – Soil Nail Walls-FHWA-2003)

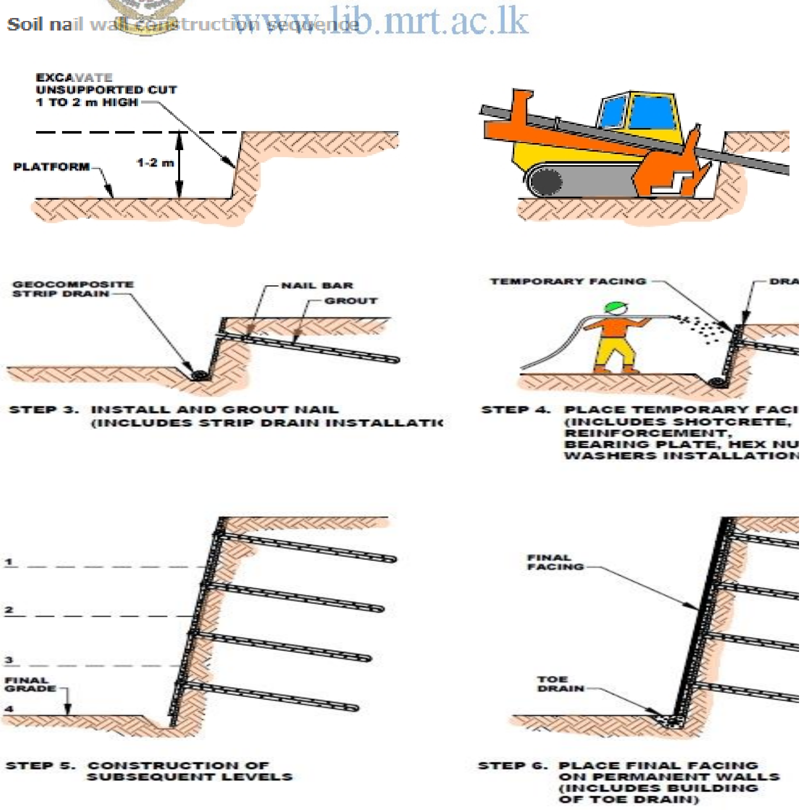

Figure 2.2 – Construction Sequence of Soil Nailing
(After Geotechnical Engineering Circular No 7 – Soil Nail Walls-FHWA-2003)

2.3 Applications of soil nailing

The soil nailing technique is well suited for several applications that require vertical or near vertical cuts. The following are some common applications where soil nail retaining walls have been successfully used.

- Roadway cut excavations
- Widening under an existing bridge
- Tunnel portal cut stabilization
- Repair and construction of existing retaining structures
- Temporary or permanent excavations in an urban environment
- Slope stabilizations
- Bridge abutments

2.4 Ground conditions suitable for soil nails

The technique used for the construction of soil nails is dependent on the existing ground conditions at the site. In certain cases, a conventional method may be more appropriate and economical in comparison to soil nailing technique. For the economical implementation of soil nailing projects, the excavated ground should have the capacity to remain unsupported in a vertical or sloped cut of 1 to 2 m depth for a period of 1 to 2 days (Bryne et al., 1998). Soil nails are generally located above the ground water table to prevent sloughing and have a stable face after excavation. Therefore, the slopes in most soil nailing projects are predominantly placed in a state of unsaturated condition that has apparent cohesion from the contribution of matric suction.

The following ground types are considered favorable for soil nailing applications (FHWA, 1991; Bryne et al., 1998; Lazarte et al., 2003).

- Stiff to hard fine-grained soils: Fine grained soils include stiff to hard clays, clayey silts, sandy clays and sandy silts.
- Dense to very dense granular soils with some apparent cohesion: These soils include sand and gravel with *SPT* - W values greater than 30 with some fines or with weak natural cementation that provide cohesion.
- Residual soils and weathered rock without zones of low strength structure.
- Glacial soils: Glacial outwash and glacial till materials are typically suitable for soil nailing applications as these soils are typically dense, well graded material with a limited amount of fines.
- Soil nailing can also be utilized in the following intermediate soil conditions:
- Engineered fill: Soil nails can be installed in engineered fill consisting of a mixture of well graded granular material and fine grained soil with low plasticity.
- Residual soils: Residual soils can also be considered as acceptable material for soil nailing.

Soil nailing is generally not recommended for areas below the ground water table unless dewatering measures are assured both during construction and for the service life of the structure (FHWA, 1991). Stability problems will occur if soil nailing is performed below the ground water table due to flow of water through the structure. A detailed subsurface investigation is necessary to identify any lenses or pockets of granular soil filled with water, which can also lead to instability.

2.5 Analysis and Design of Soil Nailing

2.5.1 Analysis with limit equilibrium approach

In the analysis and design of soil nailing ideally both limiting conditions; strength limit state and service limit state should be used. However, in most designs done with limit equilibrium approach only the strength limit state is used.

The strength limit state is assessed by considering external failure modes and internal failure modes. External failure modes refer to the development of failure surfaces passing through or behind the nails installed. The failure modes that need to be considered are;

- Global failure modes
- Sliding failure modes (shear at base) and
- Bearing failure modes (basal heave)

External failure modes

The global stability of the soil nail wall is commonly evaluated using two dimensional limit equilibrium analyses. As with traditional slope stability analyses various potential failure surfaces are evaluated to identify the most critical failure surface. Different assumptions and numerical procedures have resulted in different methods of analysis.

Some of the earlier methods include;

- Planer (Sheahan and Oral 2002)
- Bi-linear with a two wedge sliding mass (German method – Stocker et al 1979, Caltrans 1999
- Parabolic (Shen et al 1981)
- Log spiral (Juran et al 1990)
- Circular (Golder 1993)

Comparisons among different methods show that the differences in the geometry of the failure surfaces do not result in significant differences in the factor of safety. (Long et al 1990). Simple methods of analysis consider only force equilibrium. More rigorous analyses consider both force and moment equilibrium simultaneously.

Limit equilibrium methods do not predict deformations. Hence service limit state cannot be considered with that approach. Numerical methods such as finite element method are required to get information on deformations. Semi empirical methods based on previous experience are also used to assess the deformations.

Two commonly used programs in the design of soil nailing are SNAIL and GOLDNAIL. SNAIL considers a two part planer wedge mechanism. GOLDNAIL uses a circular failure mechanism.

The acceptable Factor of Safety values for soil nailing are selected based on the nature of the structure. In general recommended factor of safety values are comparable with those used in the conventional stability analysis.

Sliding failure mechanism along the base of the retained system in response to lateral pressure behind the nails is also to be considered in principle. Bearing capacity failure is not normally a concern when soil nailing walls designs. However, since the wall facing is not extended below the base of the excavation (as in a sheet pile wall), the unbalanced loads may lead to some heaving.

It is noted that if limit equilibrium based computer programs are used in the design of soil nail walls, the explicit consideration of sliding and bearing capacity modes may not be necessary. In selecting the most critical failure surfaces computer programs routinely consider failure surfaces that result from sliding and bearing capacity failure modes.

Internal failure modes

Failure mechanisms in the load transfer mechanism between the soil the nail and the ground are referred to as internal failure mechanisms. The bond strength mobilizes progressively along the entire soil nail as the excavation proceeds. As the bond strength mobilizes, the tensile forces in the nails are developed. Typical internal failure modes are;

- Nail pullout failure – pullout of the nail along the soil-grout interface
- Slippage of the bar at bar-grout interface – use of threaded bars prevents this
- Tensile failure of the soil nail
- Bending and Shear failure of the soil nail- mobilizes only after relatively large deformations. Not normally considered in the designs

2.5.2 Formulation of limit equilibrium analysis with soil nailing

Mettananda and Kulathilaka (1998) developed analytical models based on the limit equilibrium approach extending the Bishop's simplified method and Janbu's method for slope stability analysis. Method based on Bishop's simplified method can be used in analyzing circular failure surfaces and method based on Janbu's simplified method could be used for possible non-circular failure surfaces.

Model based on Bishop's simplified method

Bishop's assumption to neglect inter slice shear forces was used in the derivation, and the model would be applicable to circular failure surfaces only. All the nails crossing a failure arc of a particular slice, and are available within a unit width, are represented by a single nail passing through the center of the failure arc of that particular slice (Figure 2.3). The final equation for factor of safety is given by;

$$F = \frac{\sum\left\{\left[c'\Delta x_i + \left(W_i + Q_i - u_i\Delta x_i + T_N \sin\alpha\right)\tan\phi'\right]\left[1 / M_i(\theta)\right]\right\}}{\sum\left[\left(W_i + Q_i\right)\sin\theta_i - T_N\cos\left(\theta_i + \alpha\right)\right]} \quad ....(1)$$

where,

$$M_i(\theta) = \left(\cos\theta_i + \frac{\tan\phi'\sin\theta_i}{F}\right) \quad ....(2)$$

In these equations Δx_i, W_i, Q_i and u_i represents the slice width, the weight of the slice, surcharge on the particular slice, and the pore water pressure respectively. α denotes the nail angle to the horizontal.

In this model, the mobilized tension in the nail (T_N) is taken into account. Hence,

$$T_N = \text{lesser of}\left[\text{Nail strength, Pullout Resistance}\right]$$

$$= \text{lesser of}\left[n\frac{\pi d^2}{4}f_y \; , \; f_b n\pi dl(c' + \sigma\tan\phi')\right] \quad ....(3)$$

Parameters used above are illustrated in Figure 2.3

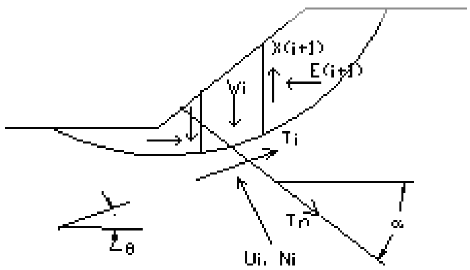

Figure 2.3 Forces acting on a slice (Bishop's method and Janbu's Method)

Model based on Janbu's simplified method

Janbu's assumption was also to neglect inter slice shear forces in the derivation (see Figure 2.3), but the model can be applied to either circular or non-circular failure surfaces. In order to perform a plane stress analysis, all the nails, applicable to a particular slice over a unit width are represented by a single nail, as done in the Bishop's model. The final equation is given by;

$$F_0 = \frac{\sum\left\{\left[c'\Delta x_i + \left(W_i + Q_i + T_N \sin\alpha_i - u_i \Delta x_i\right)\tan\phi'\right]\frac{1}{n_\theta}\right\}}{\sum\left[\left(W_i + Q_i\right)\tan\theta_i - T_N \frac{\cos(\alpha + \theta_i)}{\cos\theta_i}\right]} \quad ....(4)$$

where,

$$n_\theta = \cos\theta_i \left(\cos\theta_i + \frac{\tan\phi' \sin\theta_i}{F}\right) \quad ....(5)$$

Parameters has the usual meanings

After obtaining F_0 from the above equations, the modification factor F_0 is obtained from the charts derived by Janbu, and the final factor of safety is given by,

$F = f_0 F_0$ ………………….(6)

The above equations clearly illustrate the factor of safety increase with the T_N and Soil Nailing is a practical and cost-effective technique to stabilize slopes and excavations through the introduction of reinforcements into the soil mass

Excel spread sheets were developed to perform the analysis for a selected trial failure surface. A procedure was developed to draw the trial failure surface in an AutoCAD drawing and extract the necessary geometric data. This procedure of extracting data relevant to the selected trial failure surface had to be repeated manually.

The experimental studies in this project confirmed to suitability of circular or non circular failure mechanisms rather than the wedge type failures.

2.5.3 Design of Soil Nailing Using GEOSLOPE/SLOPEW Software

GEOSLOPE SLOPE/W software has incorporated the soil nails into the analysis of the stability of a slope. The diameter of the drill hole, the length of the nail and the tensile capacity of the nail has to be inputted. A unit pullout resistance has to be entered for each nail in the system. Appropriate factors of safety for the pullout resistance and tensile strength are also to be entered. The software estimates the resisting force available in each nail, based on the length of the nail outside the failure surface and its tensile capacity. (GEOSLOPE 2004). Analysis is done in the framework of conventional slope stability analysis based on limit equilibrium approach with methods of analysis such as; Bishop's method, Morgenstern and Price method, Spencer's method etc. Morgenstern and Price method and Spencer's method consider both force and moment equilibrium and can be used to analyze both circular and non circular failure surfaces.

2.6 Pull-out behavior of soil nails

Pullout failure at the soil-grouted nail body interface is one of the most critical considerations in a soil nail design. In most cases, the pull-out capacity of a soil nail is estimated based on previous experience with similar soil conditions using appropriate analytical or empirical approaches and verified by pull-out test during the construction phase.

Numerous field and laboratory tests have been performed to investigate the pullout behavior of soil nails by several investigators. These tests were fully instrumented and involved full scale models, modified direct shear box tests or pull-out tests. Pull-out testing studies on grouted soil nails were also conducted to investigate the interface shear strength (Sivakumar and Singh, 2010).

Design charts were proposed to estimate the pull-out capacity of gravity grouted and driven nails in various types of soils based on a number of field pull-out test results performed during the French National Research Project - "Clouterre" (FHWA, 1993). Several researchers also conducted studies to evaluate soil-nail interaction by using a large direct shear box (Chu and Yin, 2005; Sivakumar and Singh, 2010).

Milligan et al. (1997) attempted to study the effects of initial stress in the soil, grouting pressure and stress changes during the pull-out test. Franzen (1998) used a large scale laboratory setup to study the pull-out capacity of driven nails in dry, poorly graded, fine sand. Junaideen (2004) studied the behaviour of different types of embedded steel bars in completely decomposed granite soil and provided a framework for further investigation of grouted soil nails.

2.6.1 Empirical approaches for evaluation of pullout resistance

Several attempts have been made by researchers to correlate the pull-out capacity of soil nails with soil properties obtained from in-situ tests. A correlation between pullout capacity and standard penetration test (N values) was done for soil nails in Brazil. Heymann (1992) contended that the shear stress between nail and residual soil can be limited to *2N* kPa. A correlation with pressure meter tests was done for grouted and driven nails in various soils by Schlosser et al. (1983). Design charts were developed during the Clouterre program to provide preliminary estimates of the pull-out capacity of soil nails (FHWA, 1993).

2.6.2 Analytical approaches for evaluation of pullout resistance

A number of researchers have suggested different approaches to analytically estimate the pull-out capacity of soil nails (Table 2.1). There are differences in the equations outlined in Table 2.1 but they are all based on four main variables: the normal stress acting on the nail surface $\sigma_n^{/}$ coefficient of friction between nail and soil μ, adhesion between nail and soil c_a and nail perimeter.

Table 2.1 the pull-out capacity of soil nails according to various researchers

Reference	**Equations**
Zhang et al. (2009)	$T_L = \pi D\left[c' + (u_a - u_w)\tan\phi_b\right] + \dfrac{2D\sigma'_v \tan\phi'}{1 - \left[\dfrac{2(1+\upsilon)}{(1-2\upsilon)(1+2k_o)}\right]\tan\phi' \tan\psi}$
Chu and Yin (2005)	$T_L = Pc' + 2D\sigma'_v \tan\delta''$
Mecsi (1997)	$T_L = P\sigma'_N f_b \tan\delta$
HA 68/94 (1994)	$T_L = \lambda\left(c' + \sigma'_N \tan\phi'\right)$
Heymann et al. (1992)	$T_L = P\left(c' + \sigma'_N \tan\phi'\right)$
Jewell (1990)	$T_L = P\sigma'_N f_b \tan\phi'$
Schlosser and Guilloux (1982)	$T_L = Pc' + 2D_{eq}\sigma'_v \mu^*$
Potyondy (1961)	$T_L = f_C c' + \sigma'_N \tan(f_b\phi)P$
Hansmann(1992)	$T_L = \pi DC' + 2DK_\alpha \sigma_v \tan\phi'$
Gurpersaud(2010)	$Q_{f(us)} = \left[(c_a + \beta\sigma'_z) + \{(u_a - u_w)(S^K)\tan(\delta+\psi)\}\right]\pi dL$

2.7 Factors affecting pullout resistance

2.7.1 Effect of dilatancy

The normal stress acting on a soil nail is dependent on several factors such as soil properties, nail properties and time factors. Dilation occurs in dense sand during shearing which can result in an increase in normal stress acting on soil nails during pull-out. If dilation is partly restrained by surrounding soils, the effect is referred to as restrained dilatancy and results in normal stress increase up to four times the initial stress.

Pradhan (2003) showed that the soil particles around the nail will dilate when the shear stress is applied on the soil nail interface during pull-out. This phenomenon leads to an increase in the normal stress. Numerical simulation of the effects of dilatancy on soil nail pull-out resistance was performed by Su et al. (2008). The results suggest that soil dilatancy has a significant influence on the soil nail pull-out resistance.

The dilation angle was added to the interface friction angle of the soil according to Coulomb's Model. Figure 2.4 shows the relationship of the average pull-out stress with (a) pull-out displacement and (b) dilation angle (ψ).

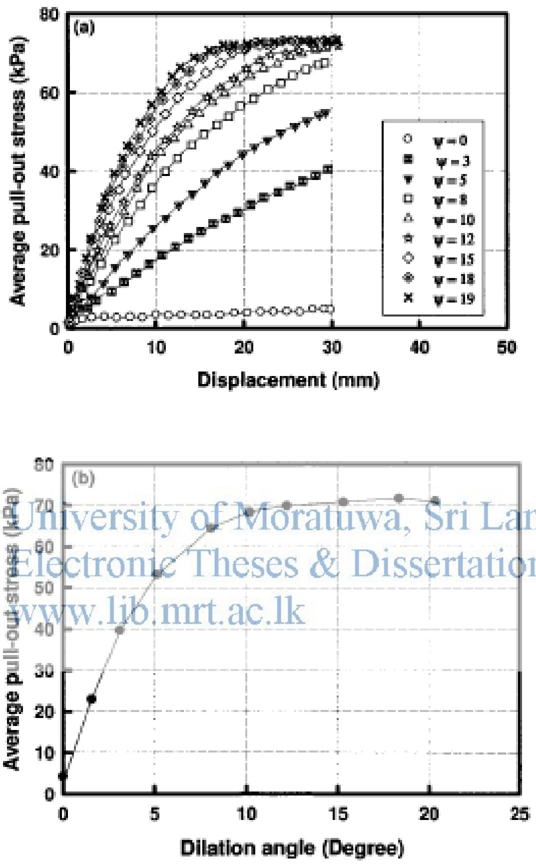

Figure 2.4 Relationship of the average pull-out stress with (a) pull-out displacement and (b) dilation angle (Su et al 2008)

Above results clearly show that the pull-out resistance initially increases quickly with the dilation angle. For dilation angles (ψ) greater than 10°, the pull-out resistance increases and then remains constant.

2.7.2 Effect of matric suction-pore water pressure

Soil nails are typically placed in a zone where the soil is in a state of unsaturated condition. Therefore, the influence of matric suction on the engineering behavior of soil nails is significantly important and has received the attention of researchers in recent times.

Potyondy (1961) showed that interface angle of friction, δ between smooth concrete and sand decreased by about 5° when the water content was increased from completely dry to full saturation. Soil comprising cohesion and friction components was highly influenced by the variation in degree of saturation (i.e. variation in matric suction values). Schlosser et al. (1983) reported a reduction of 50% if the pull-out capacity on ribbed strips in clayey gravel when the water content was increased from optimum water content to full saturation. This increase in the pull-out capacity at the optimum water content can be attributed to the contribution of matric suction.

Some researchers suggest that the pull-out force of a soil nail is not constant over time. The variation of pull-out force with time can be attributed to the changes of pore- water pressure (i.e. variation in matric suction), chemical bonding, stress relaxation, aging and greater normal stress caused by slope movement.

The variation in the degree of saturation associated with the changes in matric suction plays a major role towards the pull-out capacity of soil nails. A series of laboratory pull-out tests were performed by Su et al., 2008 in completely decomposed granite (CDG) at different degrees of saturation. The test results showed that the peak pullout strength of the soil nails was strongly influenced by the degree of saturation of the soil. Peak pull-out shear strength values were obtained between degrees of saturation of 50% and 75% (Su et al., 2008). These results indirectly show a relationship to the shear strength of unsaturated soils where the peak shear strength values typically occurs within the transition zone (Vanapalli et al. 1996). A degree of saturation of approximately 50% typically falls within the transition zone for many unsaturated soils.

The decrease in the pull-out capacity with an increase in the degree of saturation (i.e. associated with a decrease in matric suction) from optimum moisture content to the saturated condition was also observed by Pradhan (2003) and by Chu and Yin (2005) for CDG. The shearing plane also migrated from the nail-soil interface to further into the CDG as the degree of saturation increases. Displacements at peak pull-out shear strength for soils under saturated conditions were higher than that for unsaturated conditions. Figure 2.5 illustrates the relationship between peak pull-out shear resistance and degrees of saturation for CDG with overburden pressure at (a) 40 kPa, (b) 120 kPa, (c) 200 kPa, and (d) 300 kPa. The degrees of saturation at which pull-out tests were performed are 98%, 75%, 50% and 38% with corresponding matric suction values of 0, 6, 68 and 87 kPa respectively.

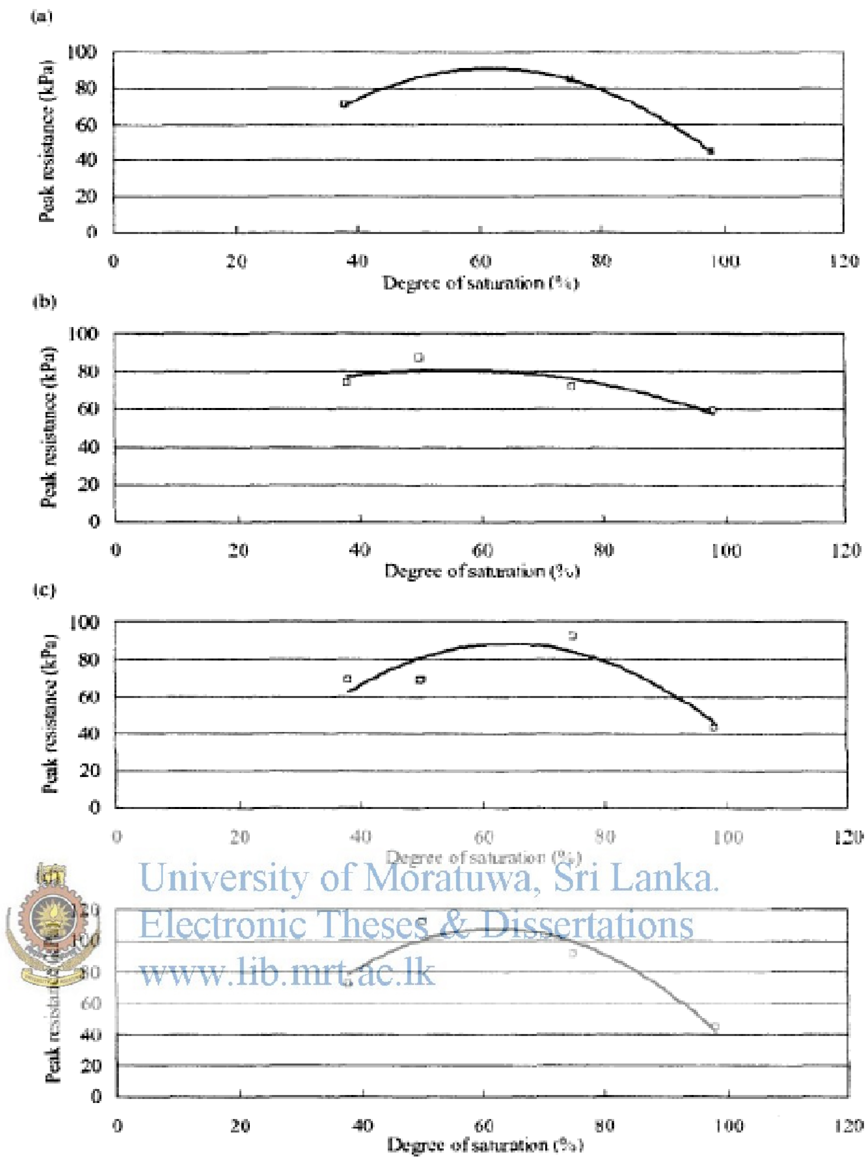

Figure 2.5 Relationship between peak pull-out shear resistance and degree of saturation for CDG with overburden pressure (a) 40kPa (b) 120 kPa (c) 200 kPa (d)300 kPa

In order to study the uncertainties in the measured and actual pull-out capacity of soil nails, Zhang et al. (2009) analyzed a large number of in-situ pull-out tests data in completely decomposed granite (CDG) in Hong Kong. The field measurements were compared with estimated values and the effects of overburden pressure, grout length, soil suction and soil dilatancy. The grouted length of the soil nail was considered as the most important parameter that governs the pull-out capacity. The second most important factor is the matric suction.

Studies performed by Su et al., (2008) indicated that the effect of the degree of saturation (hence matric suction) on soil nail pull-out capacity is significant and should be carefully addressed in the design of soil nailing system. The peak pull-out shear strength for tests at matric suction of 6 kPa was found to be two times that for saturated tests for CDG. Matric suction influences the unsaturated soil interface for numerous civil engineering applications, including piles and soil nails (Vanapalli et al. 2010). All the above discussions support the use of mechanics of unsaturated soils in the design of soil nail.

The parameters governing the pull-out capacity of soil nails are similar to that of friction piles; which are normal stress, surface area and friction parameters. The influence of matric suction on the shaft capacity of piles was investigated by Vanapalli et al. (2010). A test program was performed to evaluate the shaft resistance of jacked open end pipe piles under saturated and unsaturated conditions in sandy soils. The contribution of matric suction was found to be 50% of the shaft capacity of piles installed in silty sand under unsaturated conditions for both compression and tension. Vanapalli et al. (2010) also proposed a method to estimate the shaft resistance of piles in unsaturated soils. This method incorporates the influence of matric suction into the conventional β method.

Following the said study on shaft resistance of piles Gurpersaud (2010) studied the influence of matric suction on the pullout capacity of soil nails installed in compacted sand. The test results indicated that the post-peak pull-out capacity declines at a much faster rate as the degree of saturation of the soil increases. The decrease in the pull-out capacity was found to be a direct result of the reduction in matric suction. The peak pull-out capacity in tests at average suction of 5.3 kPa was approximately 1.7 times that higher than that of the saturated case. The pull-out capacity increased with the increase in matric suction of the soil. The displacement at the peak pull-out capacity in tests at average suction of 5.3 kPa was about 40% less than the saturated case. These results showed similar trends to results presented in FHWA (1993) which showed that the maximum pull-out force was increased by two times when the moisture content was decreased from saturation to the optimum water content and the displacement corresponding to this maximum force was increased by three times.

Gurpersaud (2010) plotted the SWCC on an arithmetic scale together with the variation of the pull-out capacity as shown in Figure 2.6. This relationship demonstrates that there is a linear increase in the pull-out capacity up to the air-entry value, followed by a non-linear increase. There is a significant increase in the pull-out capacity of the nails due to the contribution of matric suction in the range from 1 to 5.3 kPa (i.e., the analysis is based on the average suction value in the proximity of the nail) for the tested coarse-grained soil. A gradual increase in the pull-out capacity is evident from a low suction value (i.e. 1 kPa) up to 5.3 kPa followed by a decline at an average suction value of 7 kPa (i.e. soil approaching residual conditions). The behavior of the pull-out capacity matches the different phases of the SWCC where a gradual increase in strength occurs in the boundary effect zone and the transition zones (i.e. primary and secondary), followed by a decline in the residual zone (i.e. average suction of 7 kPa). The behavior of the pull-out capacity of soil nails with suction resembles the behavior of the shear strength of unsaturated soil during the different phases.

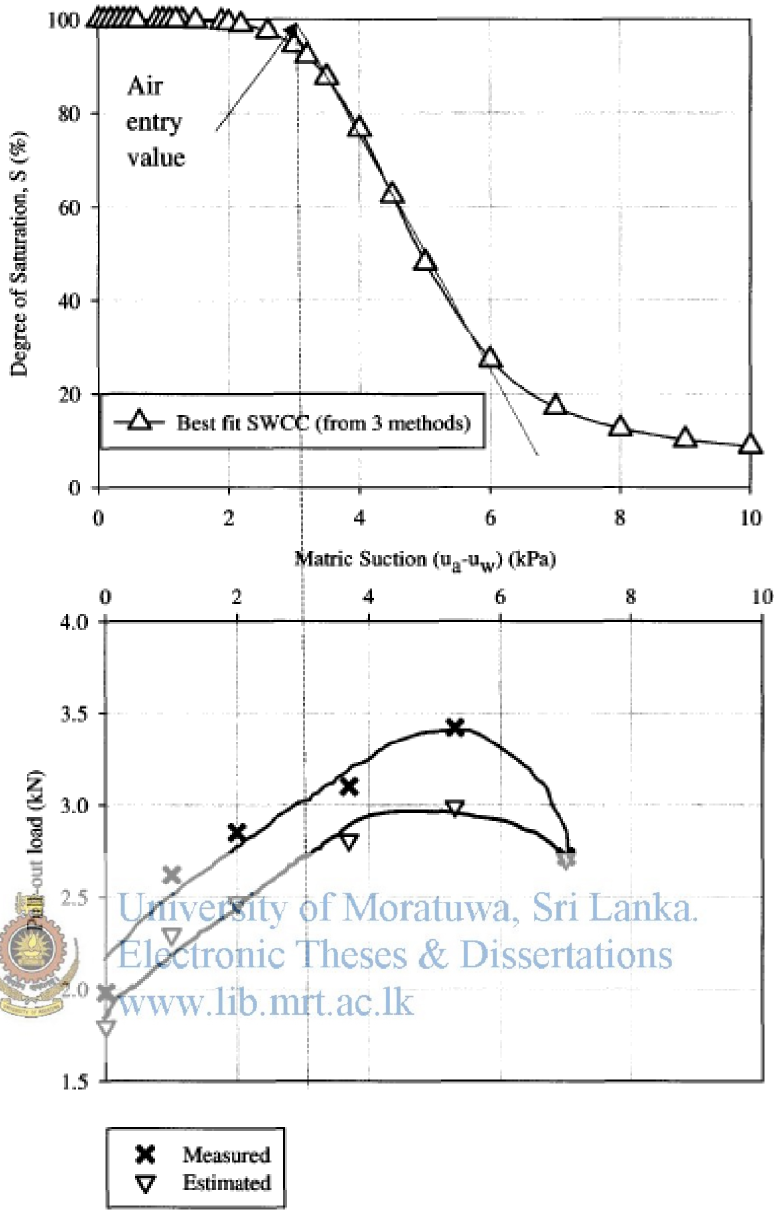

Figure 2.6 Variation of the pull out capacity with matric suction (after Gurpersaud 2010)

2.7.3 Effects of method of installation

The normal stress acting on soil nails is greatly influenced by the method of installation. The profile of the drilled hole for grouted nails will also influence the normal stress acting on the nail. A smooth cylindrical borehole will have normal stress equal to the stress prevailing during drilling (almost zero) and the resulting pull-out capacity will be low. An irregular drilled hole will develop a rib effect during grouting and mobilize restrained dilatancy effect, causing an increase in normal stress (Plumelle et al., 1990). Heymann (1992) showed that the normal stress acting on the nail surface depends on the initial stress and the stress increase is based on the soil stiffness and particle size.

2.7.4 Effect of angle of internal friction of soil

Soil-nail interface coefficient depends on the properties of soil and nail surface characteristics. Franzen (1998) stated that an increase in the angle of internal friction, $\phi^{/}$ of the soil will result in greater mobilized friction between the nail-soil interface and hence results in an increase the normal stress during pull-out. An increase in the coefficient of uniformity of the soil will generally result in an increase in the angle of internal friction $\phi^{/}$. The relative density is another factor affecting the angle of internal friction $\phi^{/}$. Soils with a higher value of relative density have a greater tendency to dilate and contribute to an increase in the angle of internal friction (Franzen, 1998).

Schlosser et al. (1983) showed that ultimate internal friction angle F′*cv* will be obtained from direct shear box test (since no volume change occurs) at failure. However, during pull-out tests some volume change occurs contributing to dilatancy and the mobilized angle of internal friction, F' will be greater than F′*cv*. Studies from direct shear box tests by Jewell and Wroth (1987) shows that the maximum interface angle of friction in direct shear, "**δ**" between a rough reinforcement and sand is limited by the angle of internal friction of soil in direct shear, $\phi^{/}_{ds.}$

The texture of the soil nail surface will also influence the interface friction angle, $\phi^{/}$. An extremely rough surface will fail by pull-out within the soil outside the nail and the angle of the internal friction, $\phi^{/}$ for the soil, will be the governing parameter. A completely smooth nail will fail at the soil-nail interface and the angle of internal friction, $\phi^{/}$ for the soil is governed by soil-nail interface friction, δ. Pull-out failure for most soil nails can be expected to occur partly as soil/soil and partly as soil/nail interface and the actual interface friction angle varies between and tan δ and tan$\phi^{/}$. Potyondy (1961) showed that the interface friction angle is greatly influenced by the type of construction material and the results indicated that the roughness played a major role.

2.7.5 Effect of grout characteristics on pullout resistance

The soil nail surface area is required for the estimation of pull-out capacity of nails. The nail surface area is treated as area of inclusion for driven nails and borehole surface area for grouted nails. Grout characteristic will have a strong influence on the surface area of the nail.

Penetration of grout into the soil depends on the relation between the soil and grout particle sizes. Grout with high water/cement ratio spreads easily and fills all irregularities in boreholes and grout with low water/cement ratio will produce stiff mortar, which will not fill all the voids. The water/cement ratio is often recommended to be 0.4-0.6 to obtain an economical and good quality soil nail.

2.7.6 Influence of overburden pressure on Pull out resistance

Figure 2.7 presents the measured pullout resistance in the tests in which the nails were pulled out. It showed that the measured pullout resistance is essentially independent of the effective overburden pressure. Extensive research studies Cartier and Gigan 1983; Heymann et al. 1992; Byrne et al. 1998; Franzen 1998; Franzen and Jendeby 2001 showed that the pullout resistance was independent of the embedded depth of soil nails. According to a recent discussion by Li and Lo (2007), after a drillhole is formed, the radial stress in the vicinity of the soil face of the nail hole is close to zero. After installation of steel bar and grouting, a small effective radial stress is introduced. When a pullout test is conducted, the pullout resistance is developed mostly because of soil shearing dilatancy, interface dilation from rough drill hole face, and physical bonding at the soil-grout interface (Yeo et al. 2007) investigated the local stresses near the nail-soil interface by a numerical study. In the analysis, the construction procedures including drilling of the nail hole, insertion of the nail, grouting, and pullout of the nail were simulated. The numerical results showed that the radial stress along the nail-soil interface is initially very small due to the nail installation procedure and increases with pullout displacement. The radial stress generated by the pullout displacement is found to be strongly dependent on the dilation angle of the soil.

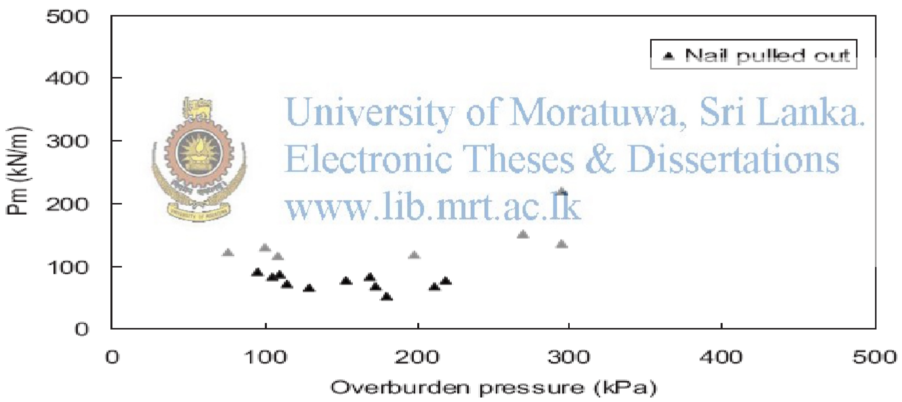

Figure 2.7 Variation of measured pullout resistance for test in which nails were pulled out

2.8 Direct shear test comparison with pull-out test

The shear strength failure envelopes for pull-out tests and interface shear tests show trends similar to soil-soil direct shear test. The peak interface friction angle, δ from the soil-grout interface shear tests is generally close to that of the soil nail pull-out tests. Based on results obtained by Chu and Yin (2005), the interface friction angle δ, of grouted nails can be estimated by using soil-grout interface shear tests. The direct shear box test is considered as a simple and reliable method to measure the interface shear strength parameters. Pradhan et al., 2006 observed that the mobilization of shear stress in the direct shear test is similar to that of the laboratory pull-out test until the first slip occurs for completely decomposed granite (CDG). Figure 2.8 shows a comparison of results obtained from pull-out tests and direct shear test for CDG.

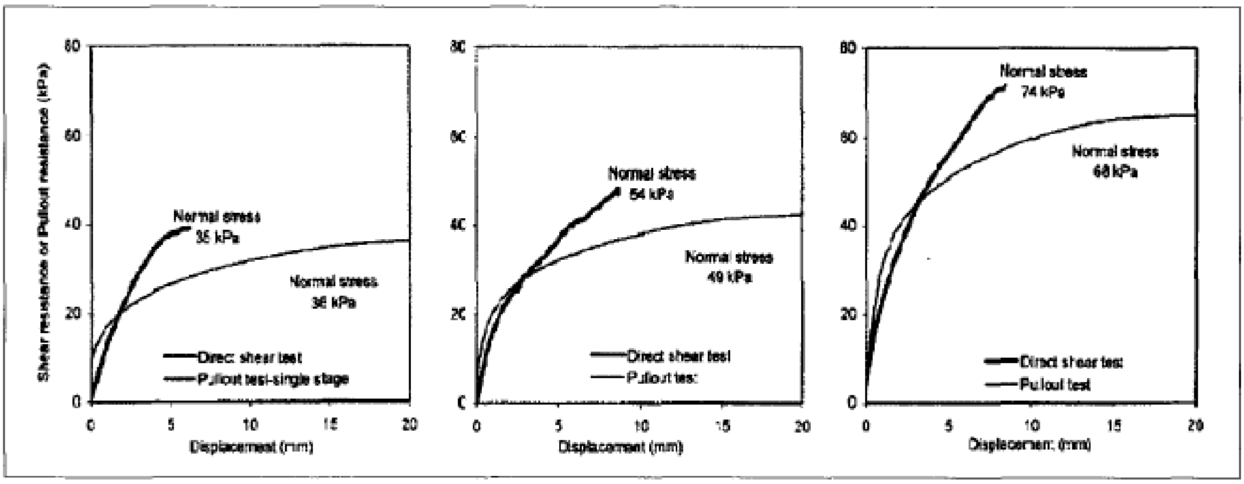

Figure 2.8 Comparison of pull-out test and direct shear box test results

2.9 Different proposed techniques and other equations in the literature

A comparison between the commonly used equations in the literature for the estimation of pullout resistance is presented here.

2.9.1 Equation proposed by Schlosser and Guilloux (1982) and others

The following equation proposed by Schlosser and Guilloux (1982) has been adopted in Hong Kong to estimate the ultimate pull-out resistance of grouted soil nails (Watkins and Powell, 1990).

$$P_{ult} = \pi D c' + 2D\sigma_v' \mu^*$$

Where:

P_{ult} = ultimate pull-out resistance (kN/m)

c' = effective cohesion of the soil

σ_v' = effective vertical stress calculated at the mid-point of the nail in the resistance zone

μ^* = coefficient of apparent friction of the soil (for granular soils, μ^* is usually taken to be equal to tan Φ', the coefficient μ^* takes the effects of dilation into account

This equation does not take the effects of matric suction into account for the evaluation of the pull-out capacity of soil nails.

2.9.2 Equation proposed by Chu and Yin (2005)

The following equation was proposed by Chu (2005) to estimate the pull-out capacity of soil nails:

$$T = \pi D c'_a + 2D\sigma'_v \tan\delta''$$

Where:

c'_a	=	soil adhesion at the interface
δ''	=	interface friction angle for the normal stress on a strip

The equation do not account for the effects of matric suction on the pull-out capacity of soil nails. The above equation proposed by Chu and Yin (2005) is an extension of equation proposed by Schlosser and Guilloux (1982).

2.9.3 Equation proposed by Zhang et al. (2009)

The equation proposed by Schlosser and Guilloux (1982) was also extended by Zhang et al. (2009) to incorporate the effects of soil suction and soil dilatancy. The following equation was proposed by Zhang et al. (2009) to estimate the ultimate pull-out resistance of soil nails by incorporating the effects of soil suction and soil dilatancy.

$$T_L = \pi D\left[C' + (u_a - u_w)\tan\phi_b\right] + \frac{2D\sigma'_v \tan\phi'}{1 - \left[\frac{2(1+\upsilon)}{(1-2\upsilon)(1+2k_o)}\right]\tan\phi' \tan\psi}$$

Where:

D	=	diameter of grouted nail
$(u_a - u_w)$	=	matric suction
Φ^b	=	internal friction angle with respect to soil suction
v	=	Poisson's ratio
ψ	=	dilation angle
k_o	=	coefficient of earth pressure at rest

The contribution due to matric suction in above equation is taken into account by considering an increase of soil shear strength as part of the apparent soil cohesion as shown below:

$$c = c' + (u_a - u_w)\tan\phi^b$$

Where

c	=	apparent cohesion

In this equation, the angle of shearing resistance with respect to suction, Φ^b is required to estimate the contribution due to matric suction. However, Φ^b is a variable for soils with non-linear shear behavior (Vanapalli et al., 1996).

2.9.4 Proposed method by Gurpersaud (2010) and model testing for confirmation

This method to estimate the pull-out capacity of soil nails in unsaturated soils is an extension of the β method used to estimate the shaft capacity of piles (Vanapalli et al. 2010).

$$f_s = c' + \beta\sigma'_z$$

$\beta =$ Bjerrum-Burland coefficient

$$\beta = k_\theta \tan(\delta + \psi)$$

Where:
ke = coefficient of lateral earth pressure with respect to soil nail inclination
d = interface friction angle at residual state
Ψ= dilation angle.

The dilation angle (Ψ) can be defined as a measure of the change in volumetric strain with respect to the change in shear strain.
The ultimate capacity of soil nails placed in saturated condition

$$Q_f = f_s \times A_{surface} = (c_a + \beta\sigma'_z)\pi dL$$

$\sigma'_z = \dfrac{(\gamma' L)}{2}$, In which γ' is effective unit weight

$c_a =$ Soil adhesion at the grout – soil interface

Above equation can be extended to include the contribution of matric suction, and will then yield a general equation for estimating pull-out capacity of grouted soil nails in unsaturated soils (Gurpersaud 2010). The fitting parameter *K* value equal to 1 can be used for non-plastic soils such as sands (Vanapalli & Fredlund, 2000).

$$Q_{f(us)} = \left[(c_a + \beta\sigma'_z) + \{(u_a - u_w)(S^K)\tan(\delta + \psi)\}\right]\pi dL$$

The β used in equation is intended for vertical piles or soil nails, and can be expressed as follows:

b= $Ks \tan \delta$
S= Degree of saturation K= Fitting parameter

The influence of soil nail inclination on the K_θ value is taken into account by using an earth pressure coefficient K_θ which is a function of the inclination of the nail. The coefficient of earth pressure, K_S is influenced by the angle of shearing resistance, the method of installation, the compressibility, degree of over consolidation and original stress in the ground, as well as the material size and shape of the pile. The Canadian Foundation Engineering Manual - CFEM (2006) recommends that the value of K_θ for bored piles can be assumed to be equal to the coefficient of earth pressure at rest K_0. The lateral earth pressure coefficient at rest K_0 is the ratio of the horizontal stress to the vertical stress, and can be substituted for K_θ to yield reasonably accurate results for the case of vertical nails (CFEM).

$Ko = \sigma_h / \sigma_v$

For the case of a vertical soil nail:

$K_\theta = K_O$, where $\theta = 0$, therefore $K_\theta / K_0 = 1$

For the case of inclined soil nail at an angle θ, the coefficient K_0 can be expressed as:

$K_\theta / K_0 = 1 + (1 - K_\theta)/2K_0 \text{ x } (1 - \cos 2\theta)$

The value of interface angle, δ is based on the surface roughness of the nail, the mean particle size of the soil, the normal stress at the gout-soil interface and the method of installation. Direct shear tests are commonly used to obtain the interface friction angle, δ. The value of δ ranges from 0.5 to 1.0 ϕ' as outlined in the Canadian Foundation Engineering Manual (CFEM, 2006).

Gurpersaud (2010) specially designed a test box and a comprehensive test program was undertaken to check the validity and limitations of the proposed approach. The equipment that was specially designed and constructed to determine the pull-out capacity of prototype grouted soil nails placed in inclined, vertical and horizontal orientations under both saturated and unsaturated conditions.

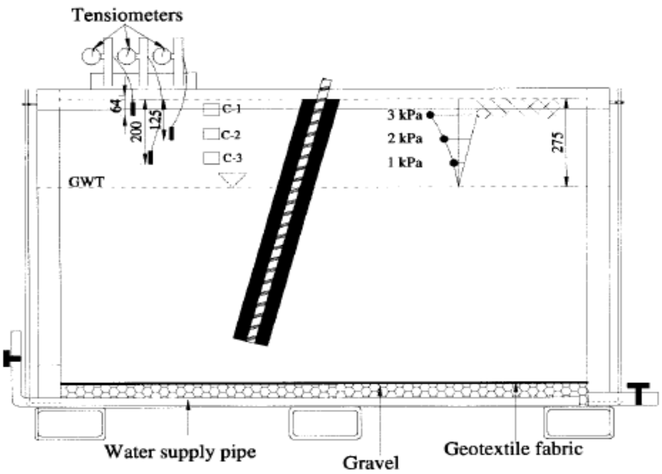

Figure 2.9 Schematic of the test box used (after Gurpersaud (2010))

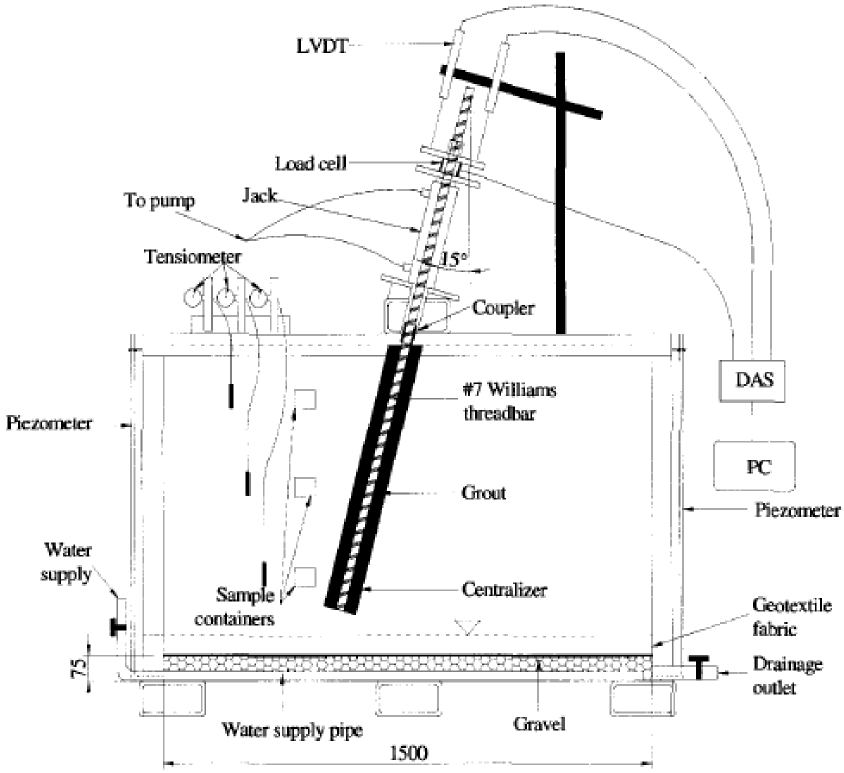

Figure 2.10 Set-up used for testing of nails inclined at 15 degrees

Grout was injected using the tremie method by attaching a grout tube to the bottom of the nail. Grout tubes are typically attached to the central reinforcement and left in placed upon completion of grouting. The grout injection was done in one continuous operation to fill the annular space between the nail and soil without any voids or gaps. The pull-out capacity of the soil nail is heavily dependent on the soil-grout contact surface therefore care was taken in the selection of the grout mix for installation of nail.

The applied force and displacement of the nail were recorded during the pull-out test through a data acquisition system. Matric suction measurements were also taken during each pull-out test at various depths, relative to the location of the water table using tensiometers. The pull-out force was measured with an ANCLO load cell located between the hollow core hydraulic jack and the restraining plate. Two linear variable displacement transducers (LVDT) were installed at the nail head to measure the pull-out displacement.

3 Experimental Evaluation of Pull out Resistance

The pullout resistance of a soil nail could be estimated with the use of different analytical and empirical approaches. It is the normal practice to conduct pullout tests at early stages of a construction to verify these estimates. This chapter extensively discusses the pullout tests conducted and the results obtained.

3.1 Selection of the site and test nails

A slope in the premises of the Kegalle hospital is to be stabilized by the installation of soil nailing. Along with the project, it was decided to have eight test nails for the determination of the pullout resistance. The test nails are not a part of the original design need for stabilization. Therefore it is possible to load the nails up to failure.

The slope under consideration was a cut slope making 75^0 with horizontal and of width about 24m and there were some indications of instability. The natural terrain above this was with a 30^0`slope with the horizontal an was in a stable state. A soil nailing design was done to enhance the stability of the slope. There were eighty four nails in the design. The inclinations of the nails were 15^0 with horizontal. Nails were of lengths 7 m and 9 m and reinforcement bars are of diameter 25 mm. The nominal diameter of the drill hole is 116 mm. The nails locations and the arrangements are present in Figure 3.1 and Figure 3.2.

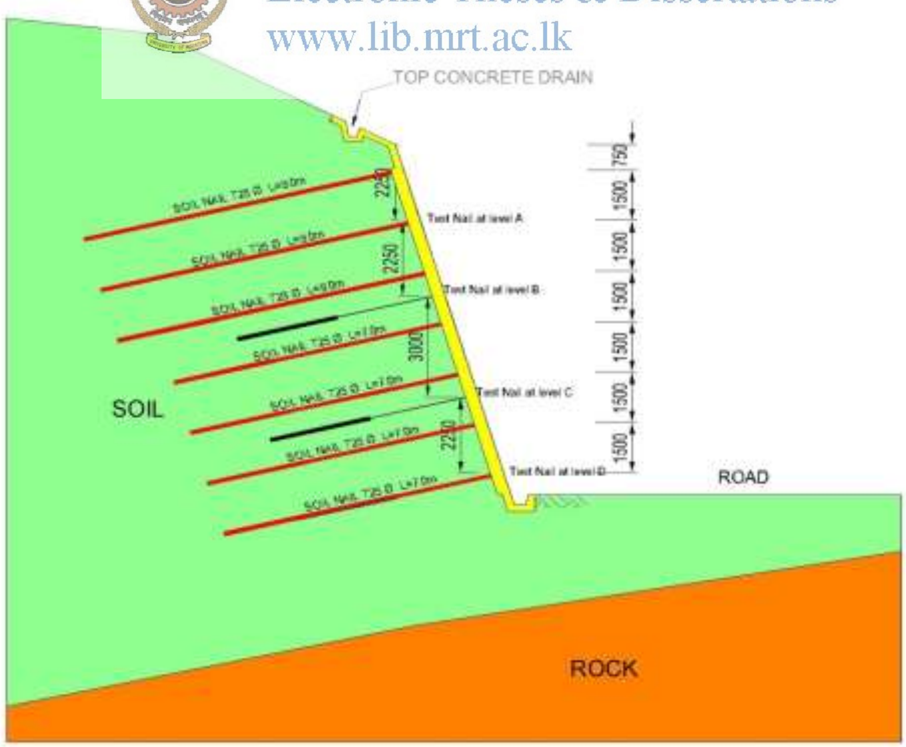

Figure 3.1 Cross Sectional View of the Design Soil Nailing Arrangement

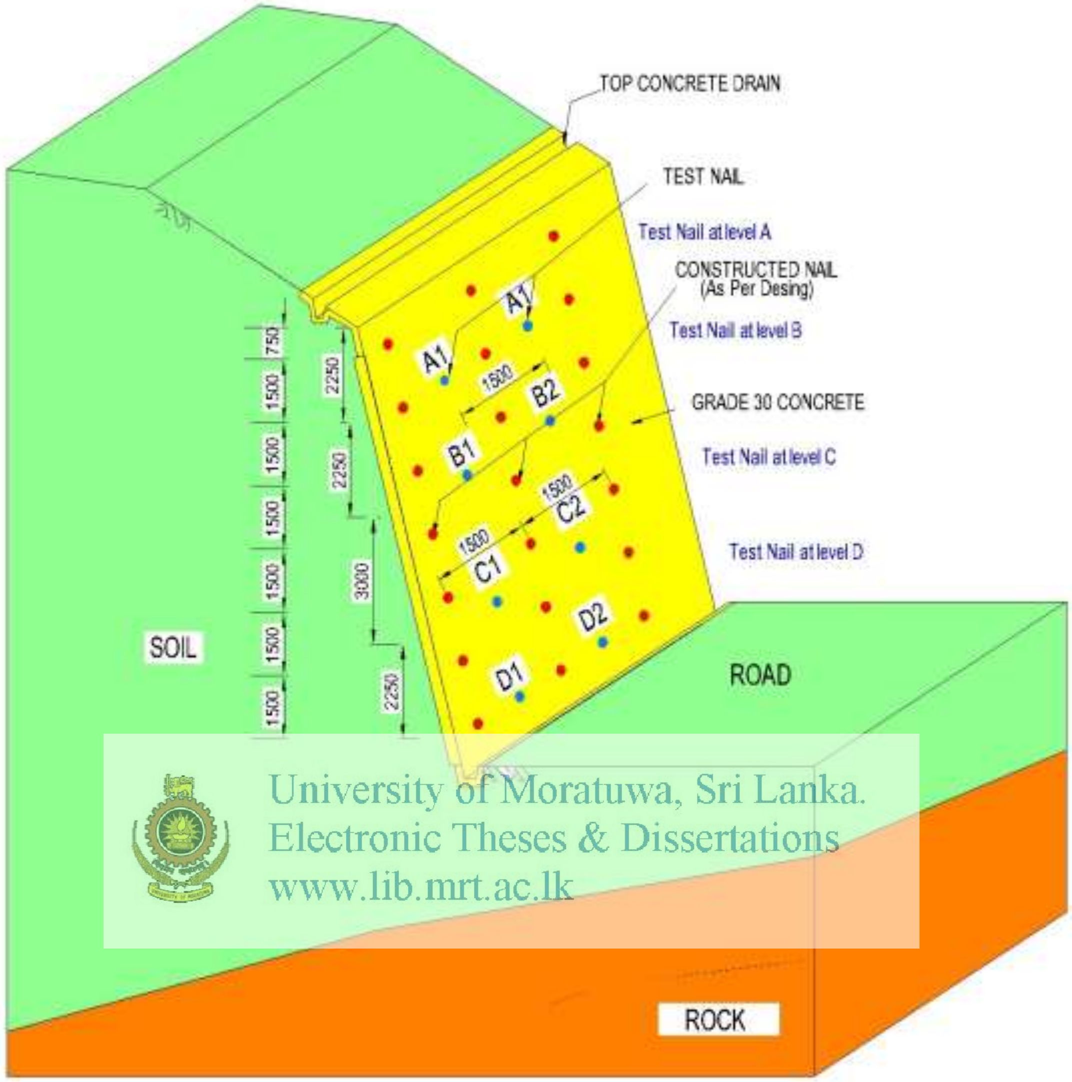

Figure 3.2 Locations and dimensions of design/ test nails

Test nails were installed in drill holes of diameter 116mm done to a length of 5m. The grouting was done only over the last 2.5m. The arrangement is present in Figure 3.3

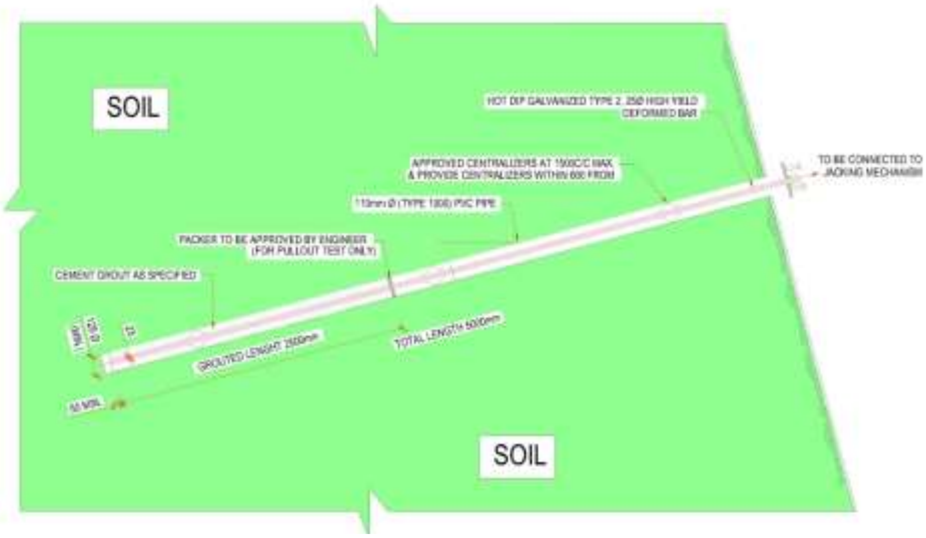

Figure 3.3 Details of a grouted test nail

Since the residual soil formations are generally of highly variable nature, it was decided to take representative samples from each test nail location for the determining of engineering parameters. Locations where box samples are obtained are present in Figure 3.4. Box samples were obtained from all locations.

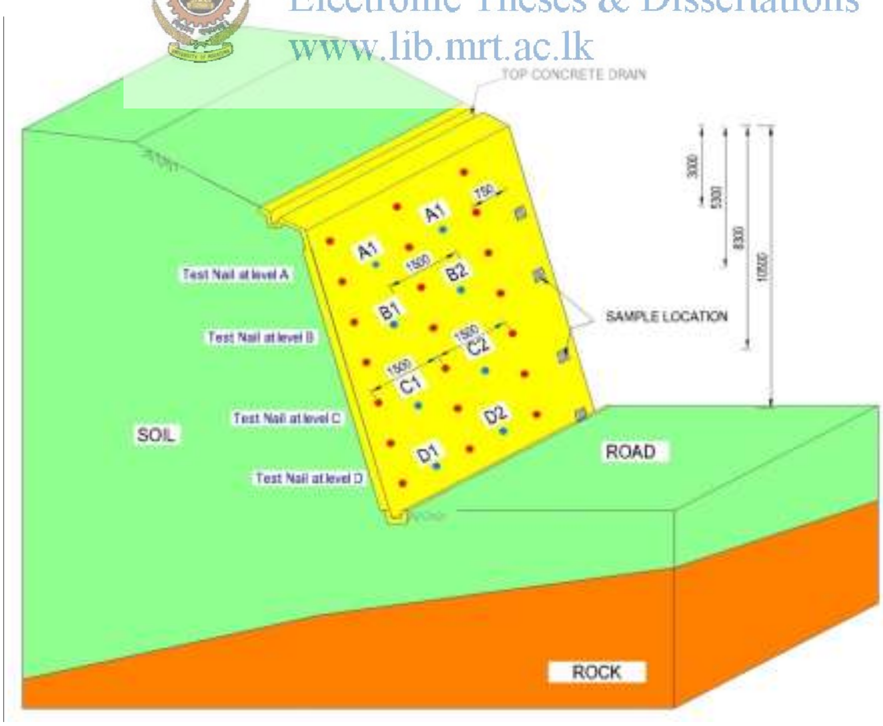

Figure 3.4 Locations of box samples

3.2 Installation of soil nails

Upon selecting a location, a stable platform, in which both machinery and workers could safely carry out the work, was erected with the use of GI pipes connected to each other to form a grid both parallel to the ground and the soil nailing surface. (Figure 3.5)

Figure 3.5 GI pipes connected to each other to form a grid both parallel to the ground and the soil nailing surface

A rotary drilling machine was placed adjacent to the test nail location, and the drilling process commenced by placing the shaft of the drill perpendicular to the surface (Figure 3.6). This procedure was carried out until the shaft reached the desired depth.

Figure 3.6 Shaft of the drilling machine

The soil nails placed within the drilled holes were galvanized (BS729) with a minimum coat thickness of 85microns or 610g/m^2 on 25mm diameter tor steel reinforcement (BS 4449). Grade 500 QST high yield deformed bars of appropriate length with a cold rolled thread of at least 150mm (as such that a significant length would be protruding beyond the soil surface) were used. The soil nail to be grouted required a cover of 50mm. Thus, it was supported by 4$^{//}$ diameter centralizers as shown in Figure 3.7 and Figure 3.8 to ensure 50mm cover from the bored surface.

Figure 3.7 Galvanized soil nail with the PVC

Figure 3.8 Preparing the 50mm cover with the use of 4” dia. centralizer

The grouting procedure initiated, after the soil nail was placed in the bored hole up to the 2.5 m length. The liquidated grout (comply with the provisions of BS12) included ordinary Portland cement mixed with Al powder 0.005 percent by weight of cement to prevent shrinkage (Figure 3.9 to Figure 3.12). Grouting was done from the bottom end of the hole with around 2bar pressure to maintain slow flow rate.

Figure 3.9 Cement and anti shrinkage grout being mixed in a barrel

Figure 3.10 - Grouting machine placed on the platform

Figure 3.11 Grouting the holes

Figure 3.12 Grouted nail

3.3 Pull Out Tests

There were eight test nails installed in 5m long bore holes, but the grouted length was only 2.5m. (Figure 3.3). Grouting of short lengths was done following the guidance in Hong Kong Guide for Soil Nailing to ensure that the nails can be pulled out without causing tensile failure. In the pullout test the resistance would develop over the full length of the nail and with long nails, the pullout capacity would exceed the tensile strength.

The overburden heights at the centre of the grouted length at test locations are illustrated in Figure 3.13 and values are summarized in Table 3.1. The undisturbed samples for the evaluation of properties of the soil are also obtained at close proximity to the test nails. This is to ensure that appropriate parameters can be used in the evaluation of the pullout resistance.

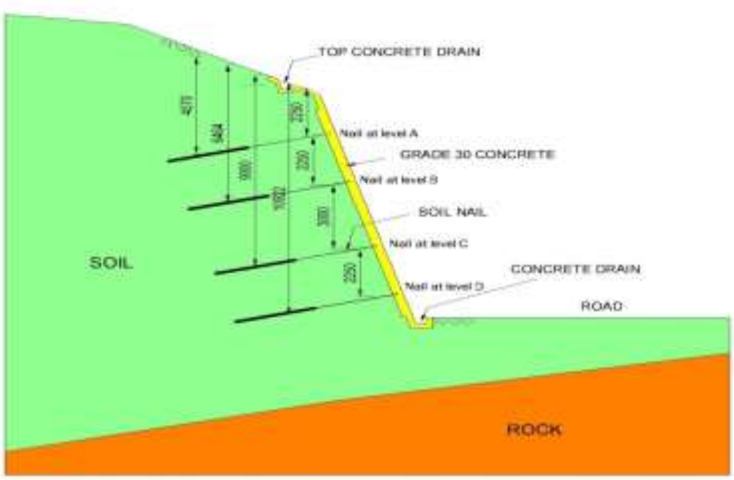

Figure 3.13 Respective test nail locations

Table 3.1 The overburden heights at center of each test nails

Test Nail	Average overburden height
A1 , A2	**4.57m**
B1 , B2	**6.46m**
C1 , C2	**9.00m**
D1 , D2	**10.92m**

3.3.1 Testing procedure of the Pullout Test

The field test involved performing the soil nail pulling out at the selected test locations. A steady platform was constructed adjacent to the test nail locations, with the use of horizontally placed GI pipes. Two thick timber planks were placed at either sides of the soil nail to restrict lateral movement and to support the pulling system that was about to be installed. (Figure 3.14)

Figure 3.14 Establishment of the platform and placing planks beside the soil nail

Two "I" beam sections (or metal plates) were placed above the timber planks (Figure 3.14) and a third and fourth "I" beam (or metal plates) were placed horizontally above the aforesaid "I" beams, thereby isolating the soil nail protruding out of the soil surface and providing a solid platform for the nail pulling apparatus to be fixed.

The jack used to pull the test nail was fixed to the nail as shown in Figure 3.15; the threads cut in the nails were used to tighten the grip of the loads supporting the apparatus, with the use of nuts. A dial gauge was then connected along the axis of nail to the apparatus in order to measure the amount of pullout and the dial gauge mounted to the jack was used to monitor load applied.

Figure 3.15 Apparatus used to pull the test nails

Figure 3.16 Application of pressure to the soil nail through the apparatus

Taking an average pullout resistance of 100kN/m^2 ,the pullout load over the grouted nail length (2.5m) was estimated to be around 100kN (Design Load DL). Pressure was gradually applied on the jack as shown in Figure 3.16, until the dial reached the required load. Test were carried out in four loading cycle (with reference to the Publication No FHWA-CFL/TD-10-001,May 2010 of FHWA) having loading sequence of 0.5DL (51kN),0.75DL(75kN),1.0DL(99kN) and 1.25DL(122kN). Ultimately the nail was loaded to failure. Due to safety reasons, the dial gauge was removed in this final cycle loaded to failure. In this final cycle, the pullout load on the nail was gradually increased as in the previous cycles until the load transferred started to decrease. This release of load signifies that the nail has reached the pullout capacity as illustrated in Figure 3.17.

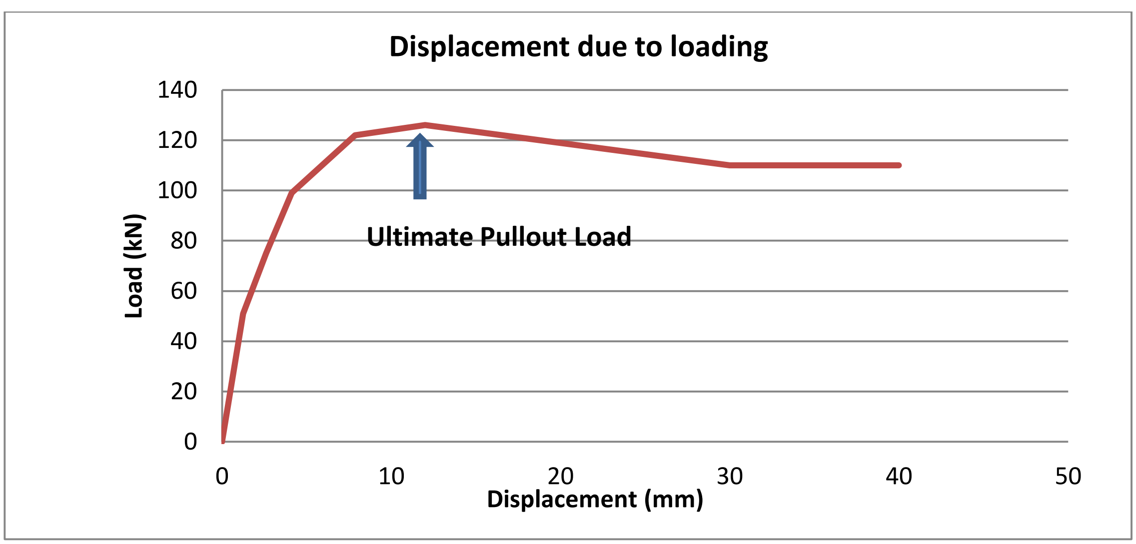

Figure 3.17 General behavior of the ultimate loading cycle.

Displacements at different levels of loading in the initial cycles of the pullout loading are presented in Table 3.2 and are presented graphically in Figure 3.18, Figure 3.19, Figure 3.20 and Figure 3.21.

The measured pullout capacity in the ultimate cycle that was done to determine the ultimate pullout load is summarized in Table 3.3

Table 3.2 Displacement of nails in initial loading cycles

Load (kN)	Displacement Due to Loading & Unloading of Pullout Test (mm)							
	Level A		Level B		Level C		Level D	
	Nail A1	Nail A2	Nail B1	Nail B2	Nail C1	Nail C2	Nail D1	Nail D2
.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
51.00	0.78	1.03	1.03	0.80	0.65	0.82	0.45	0.42
51.00	0.90	1.25	1.03	0.82	0.87	0.83	0.46	0.46
0.00	0.77	0.65	0.01	0.02	0.11	0.14	0.01	0.03
75.00	2.27	2.61	1.65	2.56	1.84	2.49	0.90	1.11
75.00	2.27	2.61	1.65	2.70	1.84	2.49	0.90	1.11
0.00	0.83	0.78	0.20	0.66	0.55	0.43	0.02	0.10
99.00	5.27	4.01	2.76	5.95	3.21	3.04	1.37	1.65
99.00	5.30	4.12	2.76	6.90	3.21	3.04	1.37	1.65
0.00	0.96	1.37	0.54	2.03	0.96	0.84	0.04	0.20
122.00	5.96	6.21	5.18	-	6.71	7.29	1.93	2.28
122.00	5.97	7.87	5.18	-	6.77	7.30	1.99	2.29
0.00	2.01	2.12	2.16	-	1.47	4.78	0.05	0.45

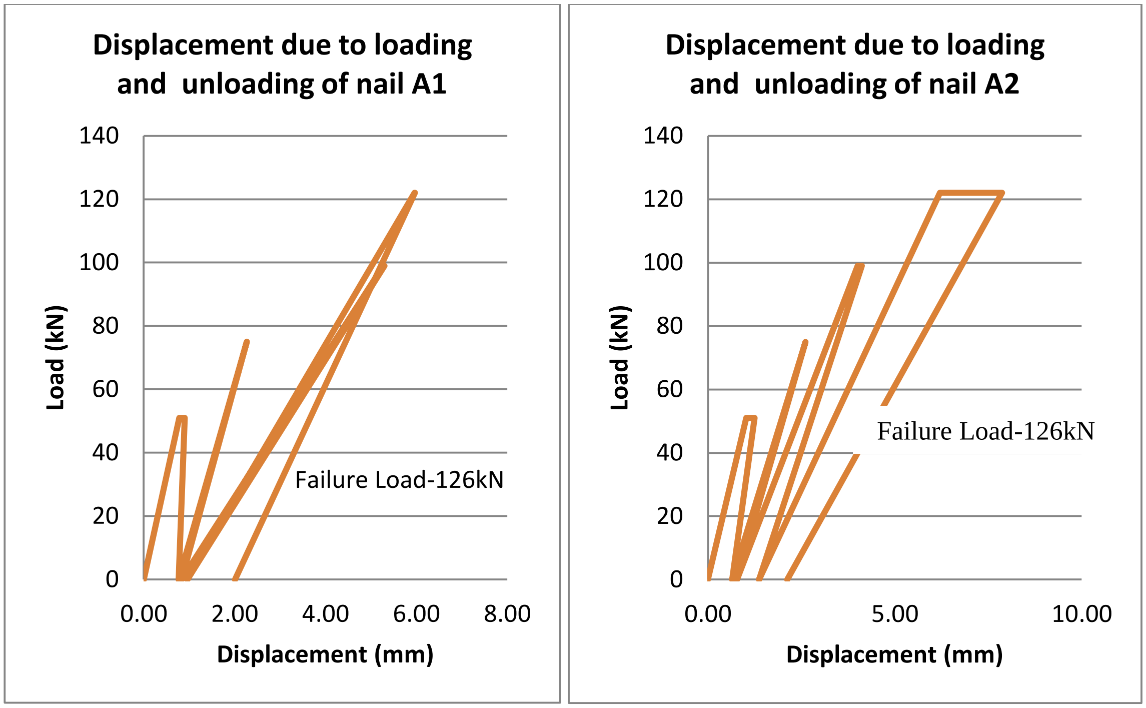

Figure 3.18 Displacement due to loading and unloading in level -A

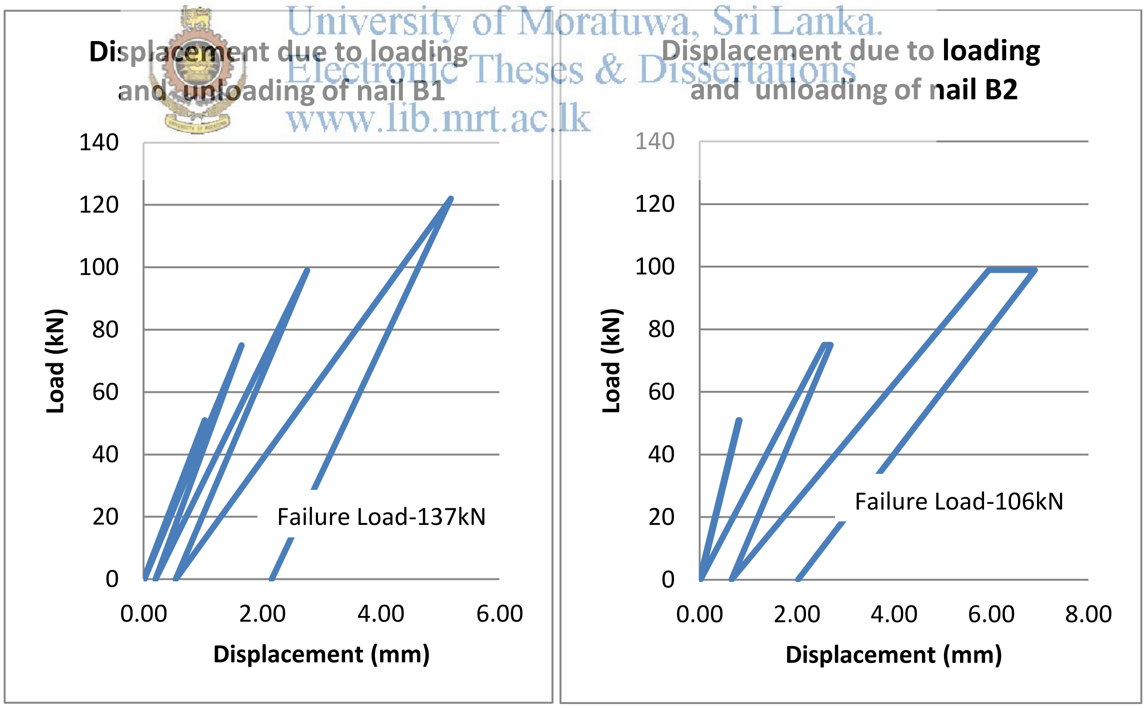

Figure 3.19 Displacement due to loading and unloading in level -B

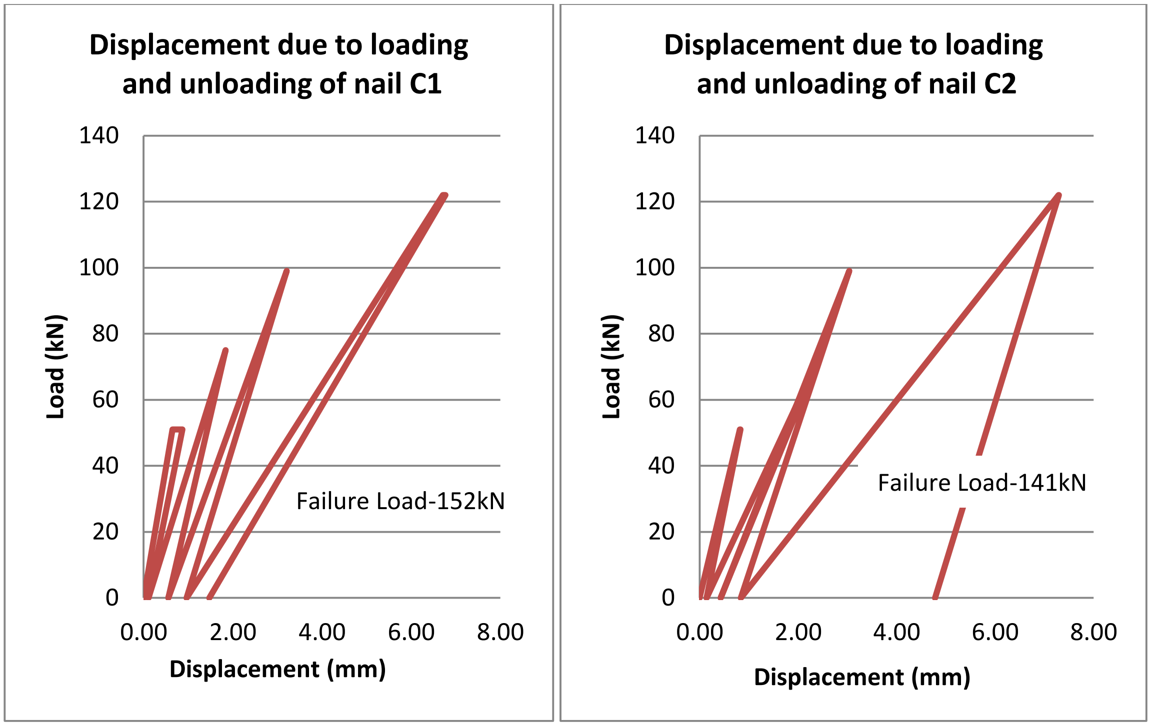

Figure 3.20 Displacement due to loading and unloading in level -C

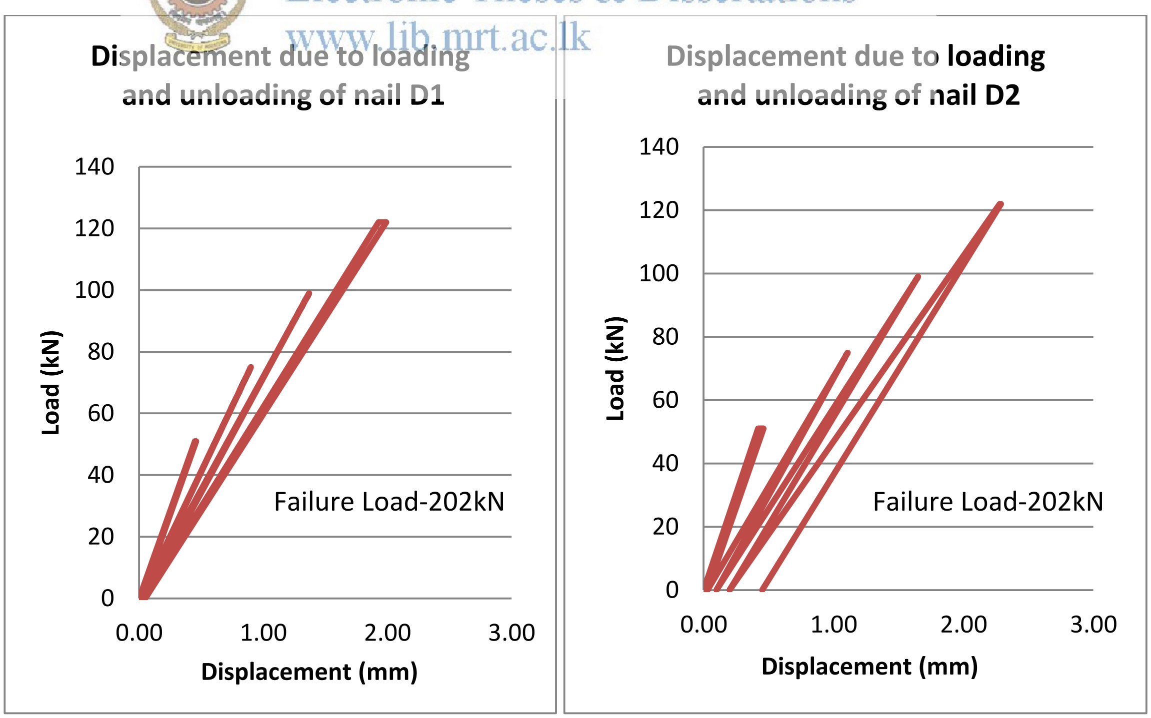

Figure 3.21 Displacement due to loading and unloading in level -D

Table 3.3 Measured pull out capacity in the ultimate cycle

Location	Depth of overburden(m)	Measured Pull Out Capacity(kN)
A1	2.25	126
A2	2.25	126
B1	4.5	106
B2	4.5	137
C1	7.5	152
C2	7.5	141
D1	9.25	202
D2	9.25	202

3.3.2 Complete Pulling out the Test Nail

After the completion of this final cycle, the test nails were completely pullout out of the hole. A 25mm thick MS plate was connected to the tip of the soil nail, with the use of nuts on both sides and cable of the tractor (Portland Major) and a mobile crane was used to pull the nail. A thick timber plank was supported by jacks, was used to ensure that the tension exerted by the vehicle acted perpendicular to the soil nail surface. After that force was exerted both in a gradually increasing manner as well as in an impulsive manner to remove the grouted nail. After extraction of the nails, the perimeter of the grouted body was obtained by direct measurements.

Figure 3.22 Extracted soil nails

The perimeters of the nails pulled out were measured to estimate the diameter. The results are summarized in Table 3.5. During the drilling operation the intended drill hole diameter was 116mm.Thus the perimeter would be 364.4mm. The measured perimeters are much larger and as summarized in Table 3.5, there is a general increase of the order of 22%.

Table 3.5 Measured perimeter / calculated diameter of the soil nail

Diameter of the grouted nail (mm)					% Increase
Nail No		Diameter of drill hall	Average measured perimeter of grouted nail	Estimated diameter of grouted nail	
A	A1	116	440	140	21
	A2	116	446	142	22
B	B1	116	450	143	23
	B2	116	452	144	24
C	C1	116	447	142	22
	C2	116	454	144	24
D	D1	-	-	-	
	D2	-	-	-	
Average					22.6

4 Determination of engineering properties of the soil forming the slope

4.1 Obtaining Representative Samples

The experimentally determined pullout resistances should be compared with the theoretical estimates made with currently used methods of estimation. Therefore it is necessary to obtain relevant shear strength parameters. Considering the variability of residual soils attempts were made to obtain undisturbed samples from all four test nail heights. Under these conditions the box sampling would be the most appropriate technique. Hence 300mmX300mmX300mm box samples were obtained from all four test height. At each height two pullout tests had been performed. Sampling locations A,B,C& D and test nail locations A1,A2,B1,B2,C1,C2,D1and D2 are illustrated in Figure 4.1. The process of sampling is illustrated in Figure 4.2.

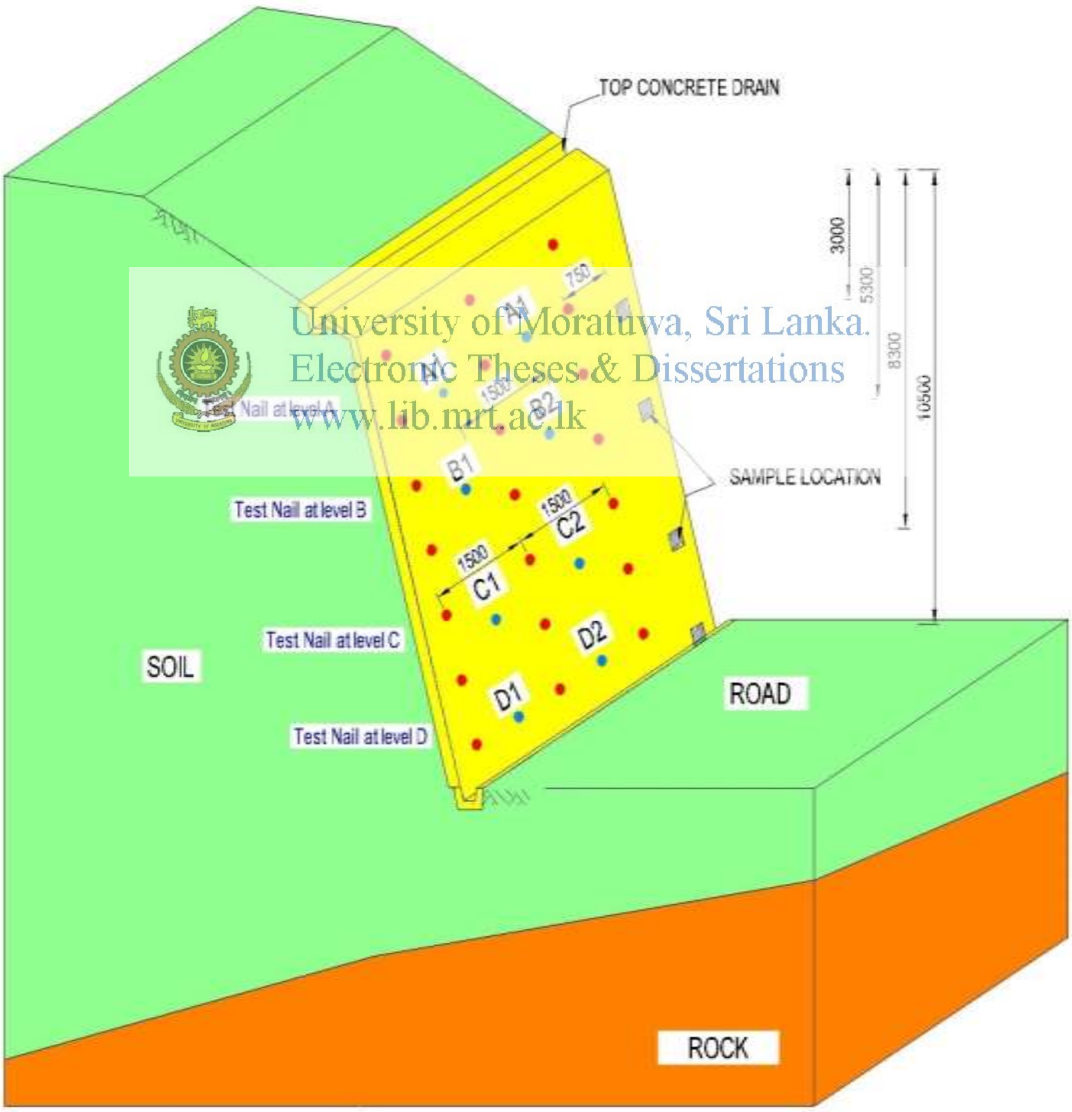

Figure 4.1 – Locations of test nails and undisturbed sampling

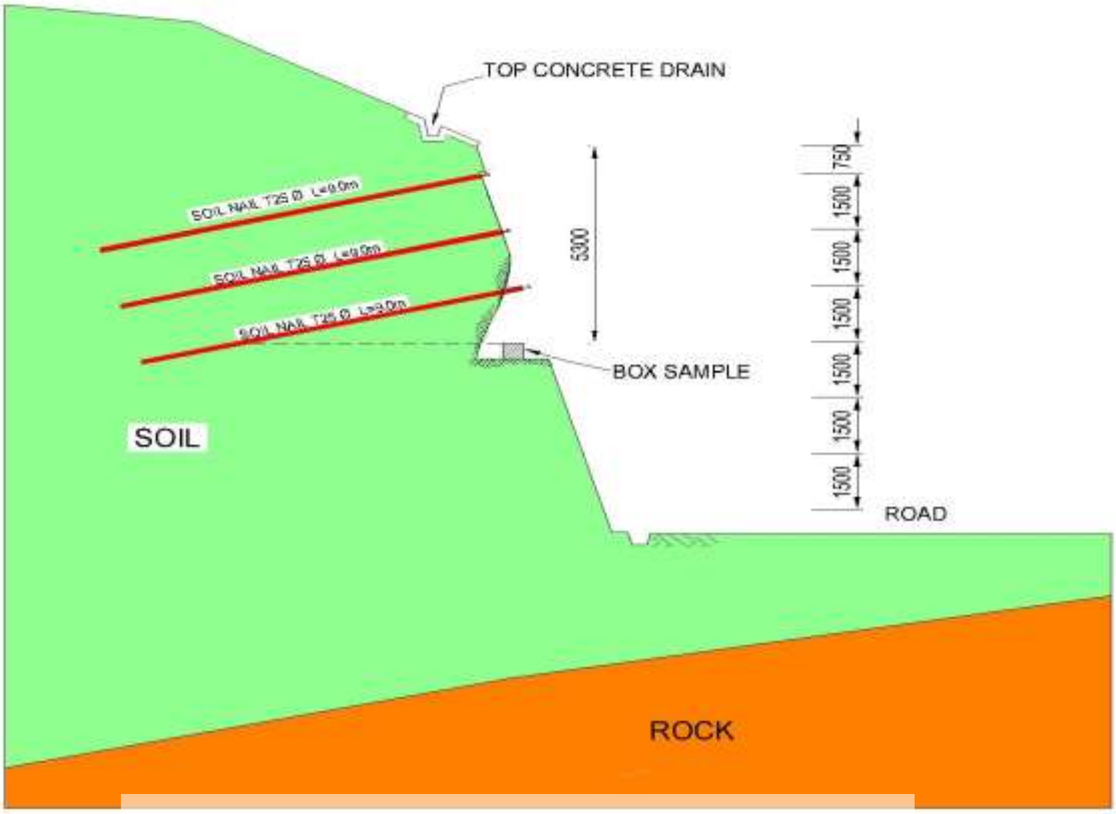

Figure 4.2 The process of obtaining undisturbed sample

Procedure followed to acquire an undisturbed soil sample:

A location adjacent to the test nail location - assuring it also coincided with the effective height of the test nail (Figure 4.2), was selected. The external hard earth surface above the selected location was then excavated to form working space in order to obtain the undisturbed box sample. The inner dimensions of plywood boxes, custom made to obtain the samples were measured, and marked on the soil surface. The surrounding earth was trimmed with the use of tools such as crow bars for excavation and table knives for fine cuts to form sharp edges.

The sample boxes were steadily shoved down the partially cut soil surface, upon extracting five faces of the sample cube (Figure. 4.3)

Figure 4.3 Five faces of the soil sample extracted

The final face of the sample was extracted by trimming the surface beyond the required limit and tilting the box sideways in order to remove the sample box from the location with only one face exposed with an irregular surface (due to excess earth).

The sample box was then placed in such a way that the irregular surface faced upwards. Then the irregular surface was trimmed with the use of table knives to obtain a smooth top layer. The top layer of the sample was subsequently coated with a layer of malted wax in order to seal the surface from the surrounding atmosphere (Figure 4.4).

Figure 4.4 Sealing the surface of the undisturbed soil sample

The sealed surface was enclosed by riveting a plywood lid to the edges of the exposed face, as the heated wax solidified. The undisturbed soil sample was eventually tagged, packed, loaded and delicately transported to the laboratory (Figure 4.5). However, the box sample A was found have got disturbed and was not used for the laboratory testing.

Figure 4.5 Tagged, packed undisturbed soil sample ready to be transported to the lab

4.2 Laboratory Tests.

After receiving box samples at the laboratory a series of tests were conducted. There were tests to determine the basic index properties and the effective shear strength parameters. Tests conducted are summarized along with the relevant testing standards

Atterberg Limit

Hydrometer Test and Sieve Analysis

Moisture Content

Disturbed Sample

Bulk Density

Direct Shear Test-Natural Condition

Undisturbed Sample

The experiments compiled with the following standards

ASTM C900-15

ASTM D 4318

ASTM D 421

ASTM D 422

4.2.1 Results of Basic Index Tests

Atterberg limit

The Atterberg limits are a basic measure of the critical water contents showing changes of consistency of a fine-grained soil, namely; shrinkage limit, plastic limit, and liquid limit. As a dry clayey soil takes on increasing amounts of water, it undergoes dramatic and distinct changes in behavior and consistency. Depending on the water content of the soil, it may appear in four states: solid, semi-solid, plastic and liquid. In each state, the consistency and behavior of a soil is different and consequently so are its engineering properties. Thus, the boundary between each state can be defined based on a change in the soil's behavior. The Atterberg limits can be used to distinguish between silt and clay, and it can distinguish between different types of silts and clays. The results of the Atterberg limit tests are summarized in Table 4.1

Table 4.1 Result of the Atterberg limits tests

Test	Sample-B	Sample-C	Sample-D
Natural Moisture Content (MC)%	28.9	47.8	14.1
Liquid Limit (LL) %	65	77	Non Plastic
Plastic Limit (PL) %	39	40	Non Plastic
Plasticity Index (PI) %	26	37	-

Sieve-analysis test and Hydrometer test

Sieve-analysis is used to determine the size distribution of large particles. Hydrometer test is used to determine the particle size distribution of very fine particles such as silt and clay. As the soils contained a significant amount of fine particles both tests were performed and the results are presented in Table 4.2 and Figure 4.6.

Table 4.2 Results of Sieve-analysis/Hydrometer analysis

Sieve Size or Particle diameter (mm)	Total Percentage of passing		
	sample of location -B	sample of location -C	sample of location -D
12.50	-	-	100.00
9.50	100.00	-	98.98
4.75	99.93	100.00	97.70
2.36	98.84	99.82	95.62
1.18	94.81	98.40	88.93
0.60	90.31	96.71	71.86
0.30	84.43	93.07	47.62
0.15	78.04	91.68	27.04
0.08	73.78	87.54	16.84
0.03	46.48	46.02	4.09
0.02	37.97	39.02	3.28
0.01	33.04	31.76	2.72
0.008	28.89	27.08	2.17
0.006	25.02	24.27	1.89
0.003	20.14	18.45	0.70
0.001	14.18	13.3	0.01

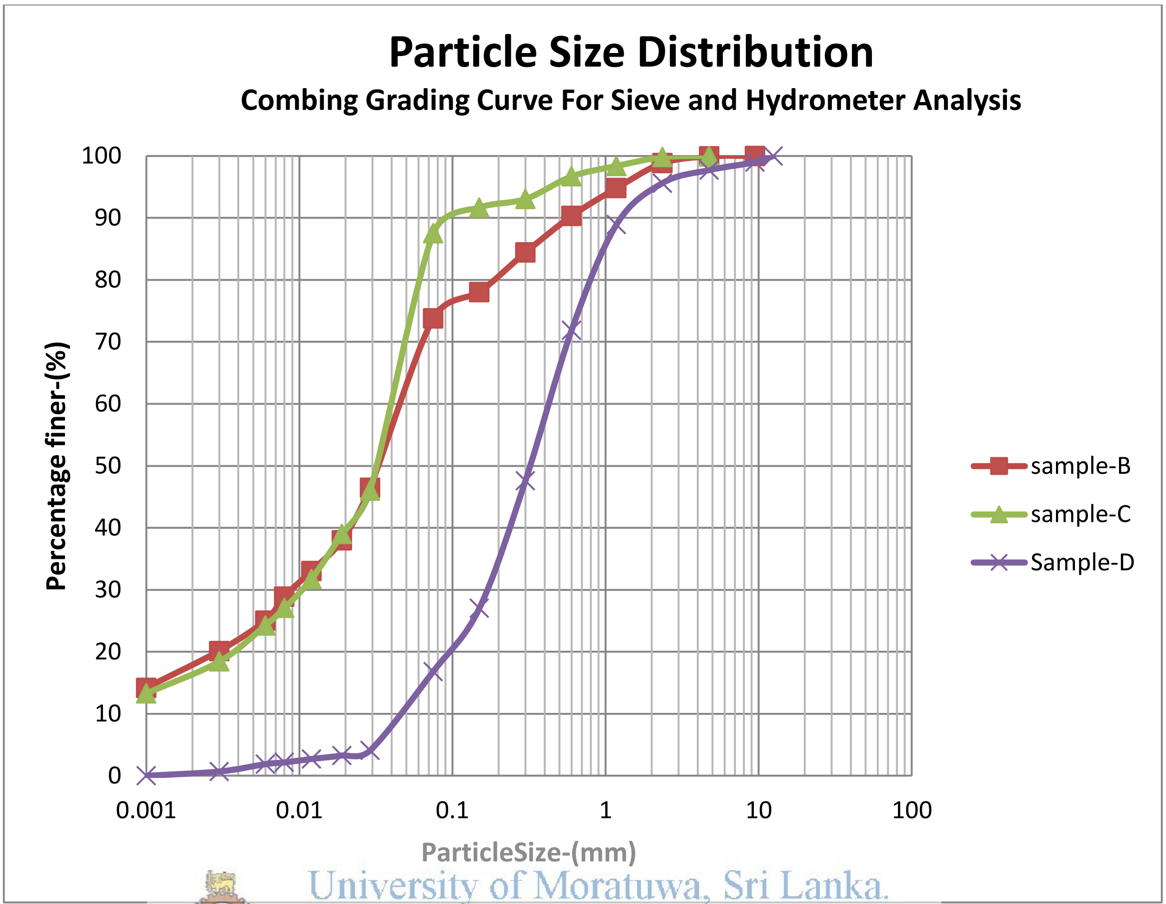

Figure 4.6 Particle size distribution curves for sample B, C and D

Table 4.3 presents percentages of different soil fractions and classification symbol assigned according to the Unified Classification System. Sample B and C are silts of high plasticity and Sample D was a silty sand.

Table 4.3 Percentages of different soil types and Classification of the Samples

Name of Index	Sample-B	Sample-C	Sample-D
Percentage fines %	73.78	87.54	16.84
Percentage Sand %	26.22	12.46	80.16
Percentage Gravel %	0.00	0.00	3.00
Classification Symbol	MH	MH	SM

Bulk density

Bulk density is obtained by dividing the mass of the soil by the volume it occupies. It can be used to estimate the dry density of the soil which can be correlated to strength and stiffness characteristics of the soil. From each box sample B, C and D three tests of bulk density was performed. Test procedure is illustrated in Figure 4.7 and test results are summarized in Table 4.4.

Figure 4.7 Bulk density test

Table - 4.4 Results of Bulk density test

Bulk density data of samples (Mg/m^3)								
Sample	Results for unsaturated samples				Results for saturated samples (before saturated)			
	specimen-1	specimen-2	specimen-3	Average	specimen-1	specimen-2	specimen-3	Average
Sample -B	1.58	1.56	1.58	1.57	1.64	1.63	1.63	1.63
Sample -C	1.54	1.51	1.49	1.51	1.52	1.57	1.53	1.54
Sample-D	1.96	2.01	1.82	1.93	1.98	1.84	1.92	1.91

4.2.2 Direct shear test

To evaluate the theoretical pull out resistance of the nail it is essential to have the effective parameters of cohesion $c^{/}$ and Friction angle $\phi^{/}$ of the relevant soil. The direct shear test is one of the laboratory test used by geotechnical engineers to measure the shear strength properties of soil .

The test is performed on three or four identical specimens from an undisturbed soil sample. A specimen is placed in a shear box which has two stacked rings to hold the sample; the contact between the two rings is at approximately the mid-height of the sample. A confining stress is applied vertically to the specimen, and the upper ring is pulled laterally until the sample fails, or through a specified strain. The load applied and the strain induced is recorded at frequent intervals to determine a stress–strain curve for each confining stress. Several specimens are tested at varying confining stresses to determine the shear strength parameters, the soil cohesion ($c^{/}$) and the angle of internal friction ($\phi^{/}$).

Direct shear test was selected over the more sophisticated triaxial test in this research due to the difficulties in extruding undisturbed test specimen from the box samples of hard residual soils. The direct shear test specimens are much smaller and easy to extrude. Also, due to the smaller size the drainage paths are shorter and duration of the tests would be shorter. In order to obtain the effective shear strength parameters the test has to done slowly ensuing drained conditions. (pore pressures are fully dissipated during the test).

Out of the four box samples obtained Sample “A” was damaged while transporting and could not be used in direct shear tests. Six sets of direct shear tests were done from box samples B, C, and D; three sets at natural moisture content and three sets under saturated conditions. The natural sample was brought to a saturated by adding water (submerge in water body) and keeping for 24hrs to get the saturation condition before testing.

For each box sample tests were done at three different confining normal stresses and stress strain curves for all three tests were obtained. Using the peak shear stress of each test the shear strength Vs normal stress graphs were plotted to obtain the shear strength parameters.

Test were done under both unsaturated (the natural existing) condition and the saturated condition. The F' values obtained from the saturated test were used in the result of the unsaturated samples to obtain the cohesion intercept.

As the sample at location A was disturbed it was not used for Direct Shear tests. A slow shearing rate range of 0.5mm/min was to ensure that shearing is done under drained conditions.

Evaluation of Effective Shear Strength Parameters $c^{/}$ and $\phi^{/}$ of sample B (Saturated Condition)

The basic characteristics of the three test specimens obtained from Sample B before saturation are presented in Table 4.5. The bulk density of the saturated soil was estimated using the relationship between bulk density (g), specific gravity of soil particle (Gs), degree of saturation (Sr), void ratio (e) and unit weight of water (g_w). Assuming sample was fully saturated, the degree of saturation (Sr) was taken as 1. The stress strain curves obtained from the three specimens are presented in Figure 4.8. The plots of shear stress Vs normal stress at failure for the three specimens are presented in Figure 4.9. The shear strength parameters are summarized in Table 4.6.

Table 4.5 Specimen data of sample B (saturated)

Specimen data of sample B (saturated)					
Description		specimen-1	specimen-2	specimen-3	Average
Moisture content Measured	(%)	28.90	28.90	28.90	28.90
Bulk Density	(Mg/m^3)	1.64	1.63	1.63	1.63
Dry Density	(Mg/m^3)	1.27	1.26	1.26	1.26
Void Ratio		1.15	1.16	1.16	1.16
Initial Degree of Saturation	(%)	68.78	67.74	68.04	68.19
Specific gravity of soil particle					2.73
Estimated average Bulk Density At 100% saturated	(kN/m^3)				17.68

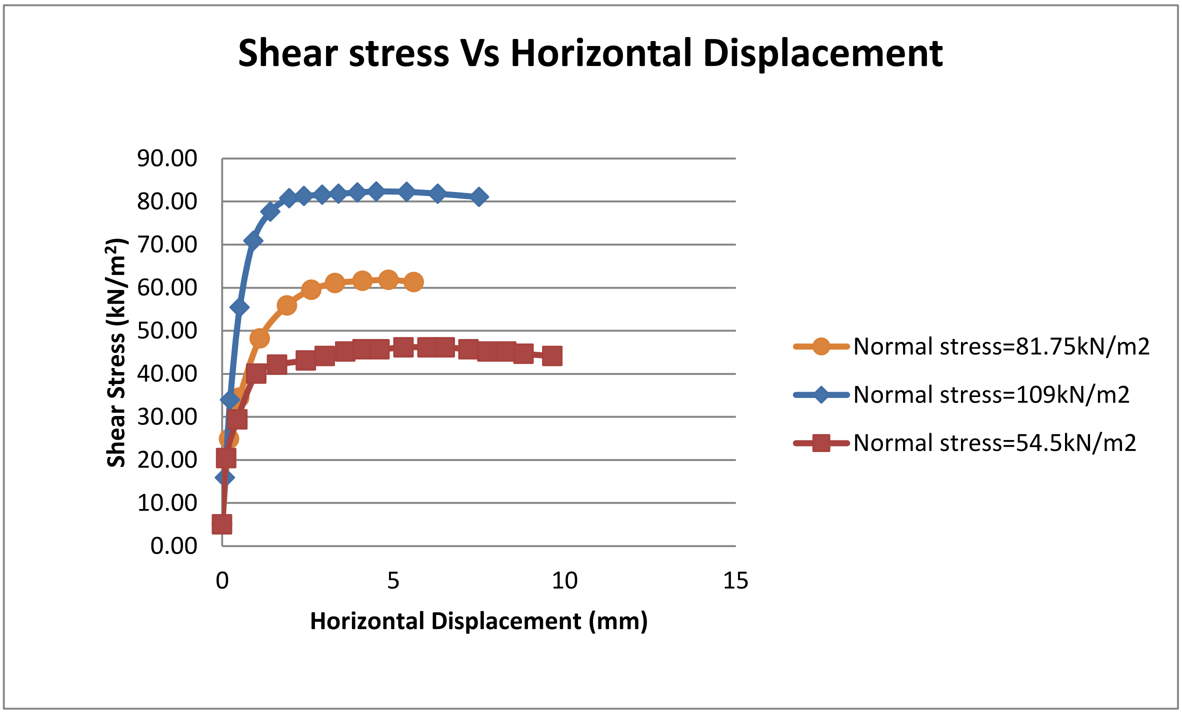

Figure 4.8 Shear stress Vs horizontal displacement- sample B (saturated)

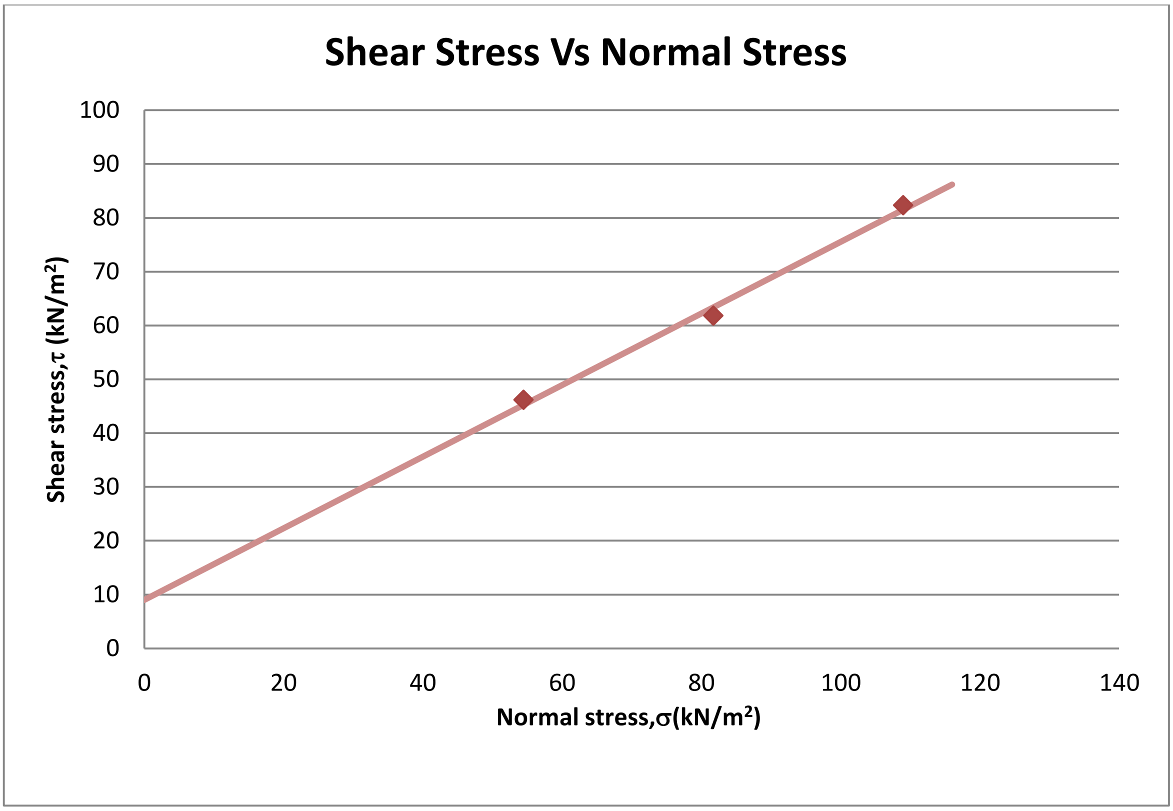

Figure 4.9 Shear stress Vs Normal Stress- sample B (saturated)

Table 4.6 Specimen results of sample B (saturated)

Test No		1	2	3
Normal Stress	kN/m^2	81.75	109.00	54.50
Peak Shear Stress	kN/m^2	61.83	82.36	46.17
Rate of Stain	mm/min	0.06	0.05	0.04
Stain of Peak Shear Stress	%	8.10	7.50	8.80
Sample Preparation		Undisturbed		
Peak Shear Strength Parameters				
Angle of Internal Friction (in degree) ϕ'		33^o		
Cohesion Intercept (C′) kN/m^2		9		

Evaluation of Effective Shear Strength Parameters c' and ϕ' of sample B

(Natural Unsaturated Condition)

The basic characteristics of the three test specimen obtained from Sample B of natural conditions (unsaturated) are presented in Table 4.7. The stress strain curves obtained from the three specimens are presented in Figure 4.10. The plots of shear stress Vs normal stress at failure for the three specimens are presented in Figure 4.11. The shear strength parameters are summarized in Table 4.8.

Table 4.7 Specimen data of sample B (un-saturated)

Specimen data of sample B (un-saturated)					
Description		specimen-1	specimen-2	specimen-3	Average
Moisture content Measured	(%)	28.90	28.90	28.90	28.90
Bulk Density	(Mg/m^3)	1.58	1.56	1.58	1.57
Dry Density	(Mg/m^3)	1.23	1.21	1.23	1.22
Void Ratio		1.23	1.25	1.23	1.236
Degree of Saturation	(%)	64.41	63.08	64.33	63.94

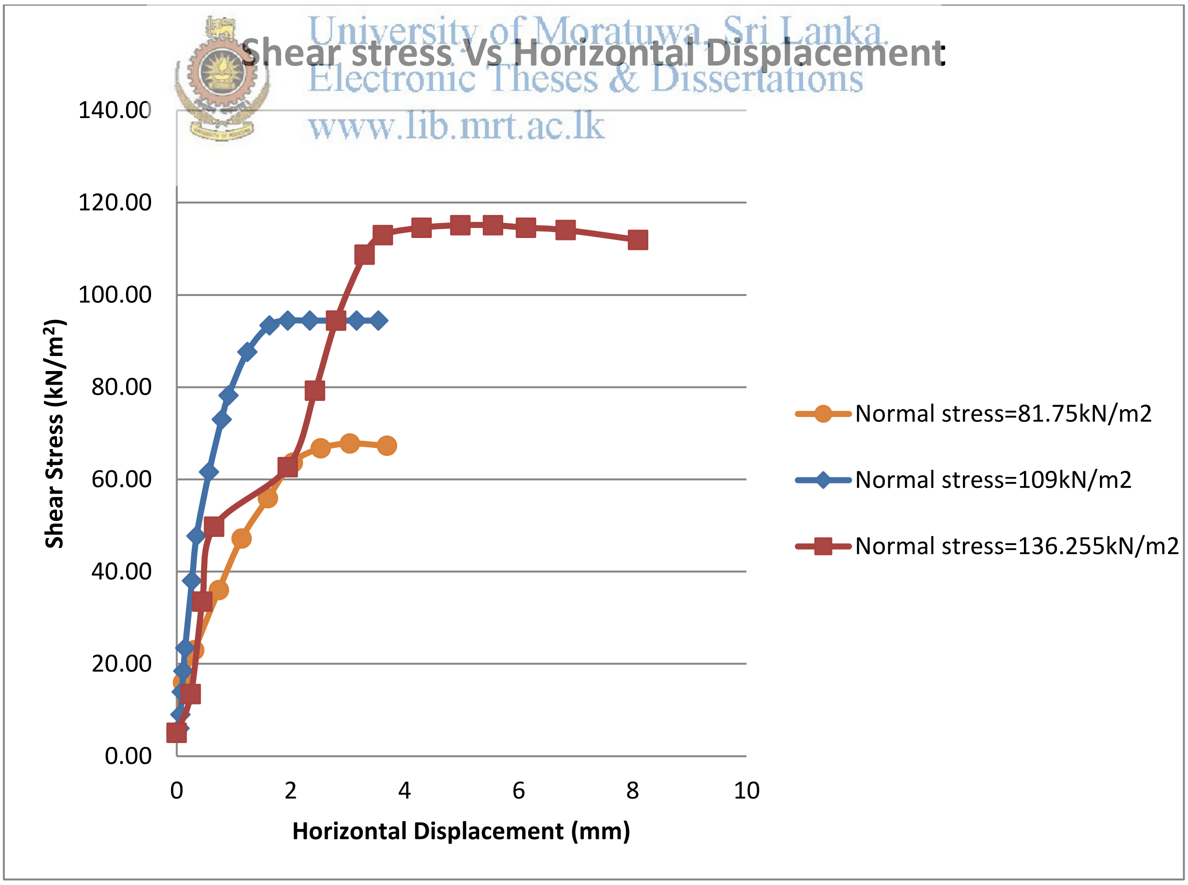

Figure 4.10 Shear stress Vs horizontal displacement- sample B (un-saturated)

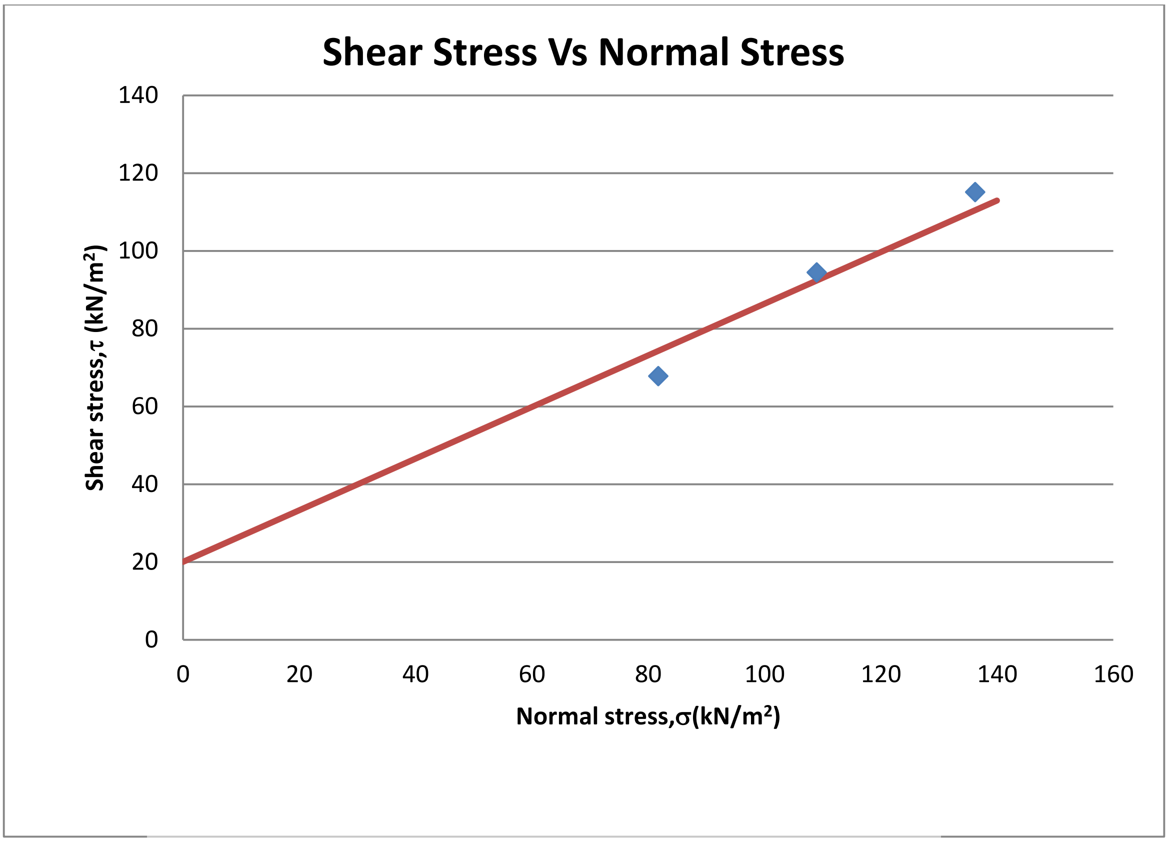

Figure 4.11 Shear stress (τ) Vs Normal Stress (σ)-sample B (un-saturated)

Table 4.8 Specimen results of sample B (Unsaturated)

Test No		1	2	3
Normal Stress	kN/m^2	81.75	109.00	136.25
Peak Shear Stress	kN/m^2	67.78	94.44	115.10
Rate of Stain	mm/min	0.04	0.03	0.05
Stain of Peak Shear Stress	%	6.03	3.25	8.30
Sample Preparation		Undisturbed		
Peak Shear Strength Parameters				
Angle of Internal Friction (in degree) ϕ'		33^o		
Cohesion Intercept (C′) kN/m^2		20		

Evaluation of Effective Shear Strength Parameters c' and ϕ' of sample C (Saturated Condition)

The basic characteristics of the three test specimen obtained from Sample C after saturation are presented in Table 4.9. The bulk density of saturated soil was estimated using Table 4.9 results and relationship between bulk density (γ), specific gravity of soil particle (G_s), degree of saturation (S_r), void ratio (e) and unit weight of water (γ_w). Assuming sample was fully saturated, the degree of saturation (S_r) was taken as 1.The stress strain curves obtained from the three specimens are presented in Figure 4.12. The plots of shear stress Vs normal stress at failure for the three specimens are presented in Figure 4.13. The shear strength parameters are summarized in Table 4.10.

Table 4.9 Specimen data of sample C (Saturated)

Specimen data of sample C (saturated)					
Description		specimen-1	specimen-2	specimen-3	Average
Moisture content Measured	(%)	47.78	47.78	47.78	47.78
Bulk Density	(Mg/m^3)	1.52	1.57	1.53	1.54
Dry Density	(Mg/m^3)	1.03	1.06	1.04	1.04
Void Ratio		1.58	1.51	1.57	1.55
Initial Degree of Saturation	(%)	80.36	84.43	80.97	81.92
Specific gravity of soil particle					2.66
Estimated average Bulk Density At 100% saturated	(kN/m^3)				16.20

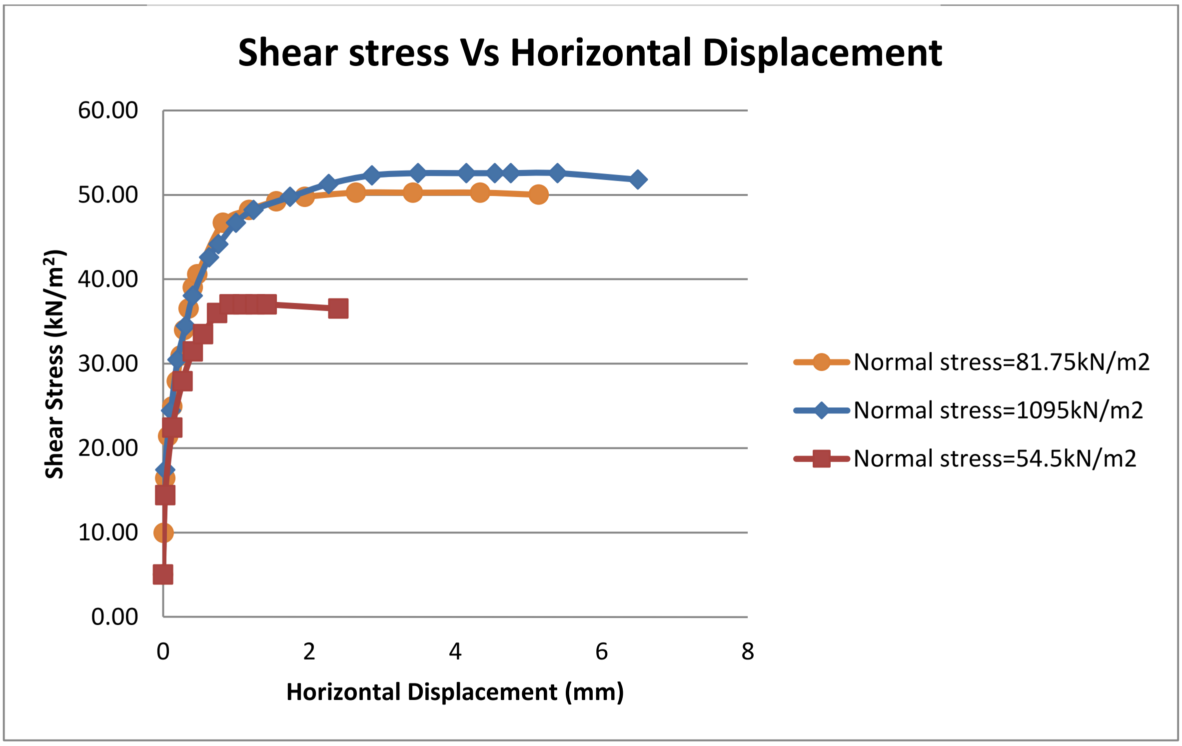

Figure 4.12 Shear stress Vs horizontal displacement- sample C (Saturated)

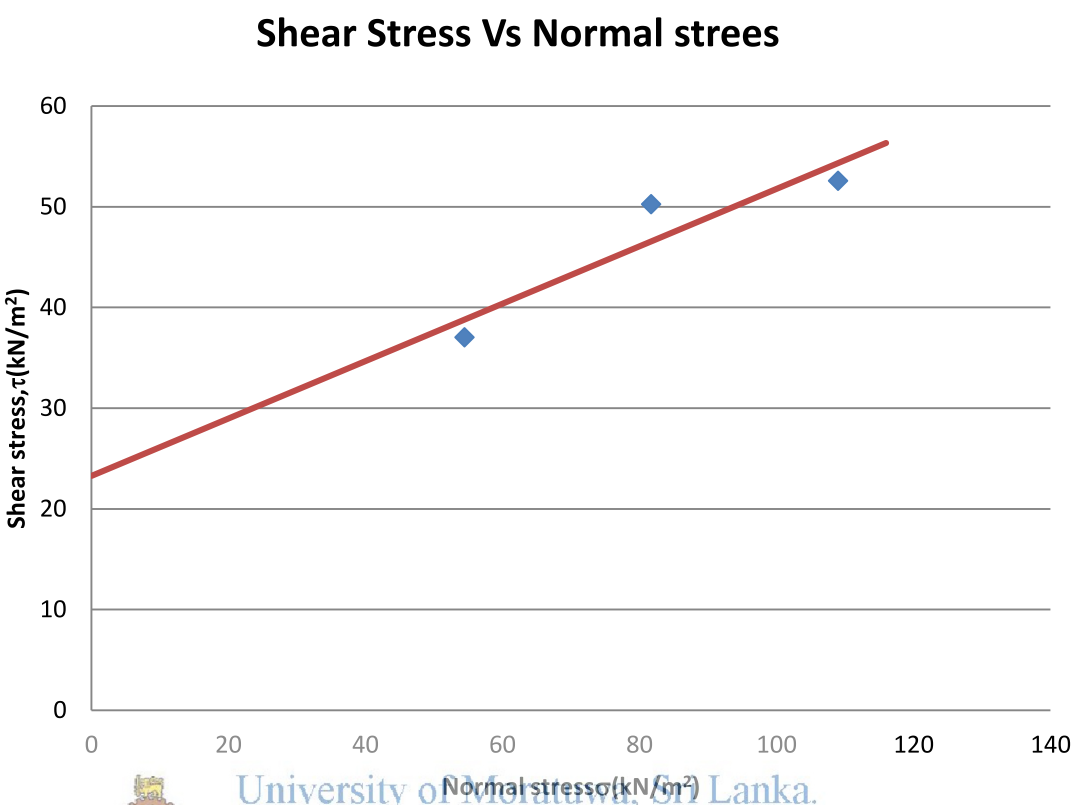

Figure 4.13 Shear stress Vs Normal Stress- sample C (Saturated)

Table 4.10 Specimen results of sample C (saturated)

Test No		1	2	3
Normal Stress	kN/m^2	81.75	109.00	54.50
Peak Shear Stress	kN/m^2	50.26	52.57	37.02
Rate of Stain	mm/min	0.03	0.05	0.07
Stain of Peak Shear Stress	%	4.40	5.82	1.52
Sample Preparation		Undisturbed		
Peak Shear Strength Parameters				
Angle of Internal Friction (in degree) ϕ'		16^o		
Cohesion Intercept (C′) kN/m^2		23		

Evaluation of Effective Shear Strength Parameters $c^{/}$ and $\phi^{/}$ of sample C (Natural Unsaturated Condition)

The basic characteristics of the three test specimen obtained from Sample C of natural conditions (unsaturated) are presented in Table 4.11. The stress strain curves obtained from the three specimens are presented in Figure 4.14. The plots of shear stress Vs normal stress at failure for the three specimens are presented in Figure 4.15. The shear strength parameters are summarized in Table 4.12.

Table 4.11 Specimen data of sample C (un-saturated)

Specimen data of sample C (un saturated)					
Description		specimen-1	specimen-2	specimen-3	Average
moisture content Measured	(%)	47.78	47.78	47.78	47.78
Bulk Density	(Mg/m^3)	1.54	1.51	1.49	1.51
Dry Density	(Mg/m^3)	1.04	1.02	1.01	1.02
Void Ratio		1.56	1.61	1.64	1.60
Degree of Saturation	(%)	81.70	78.95	77.44	79.36

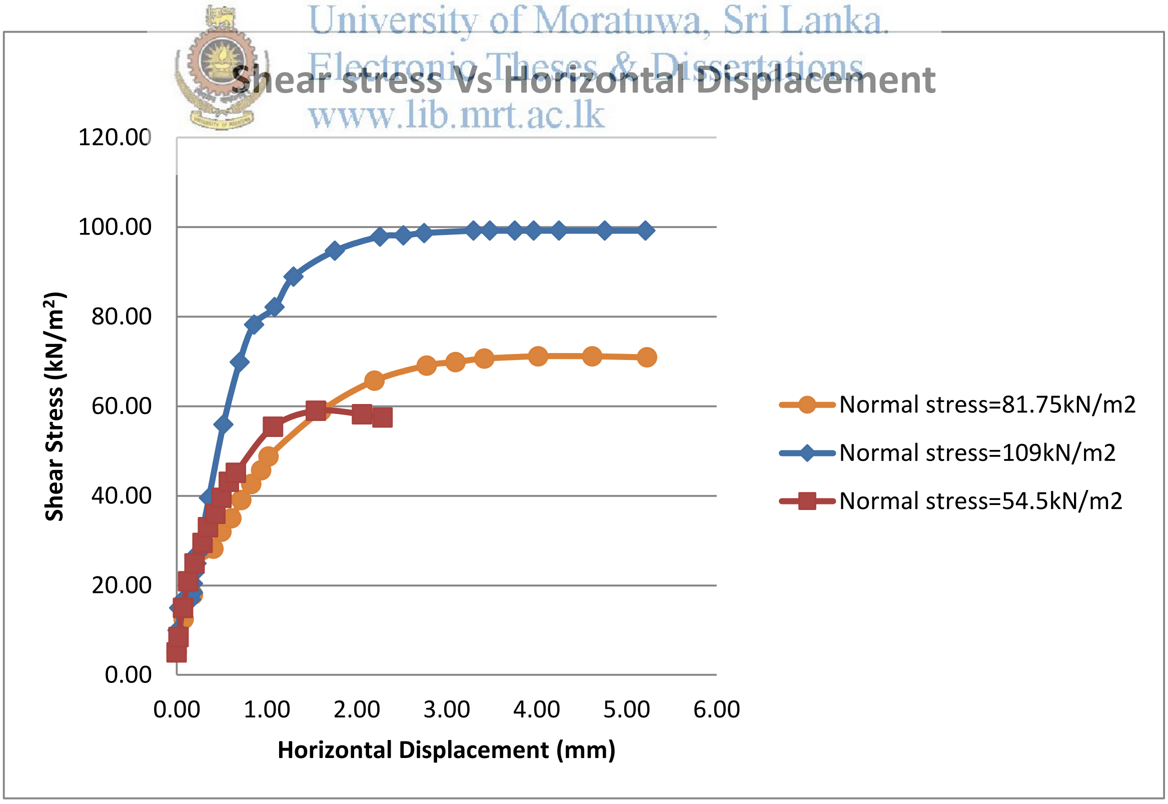

Figure 4.14 Shear stress Vs horizontal displacement- sample C (un-saturated)

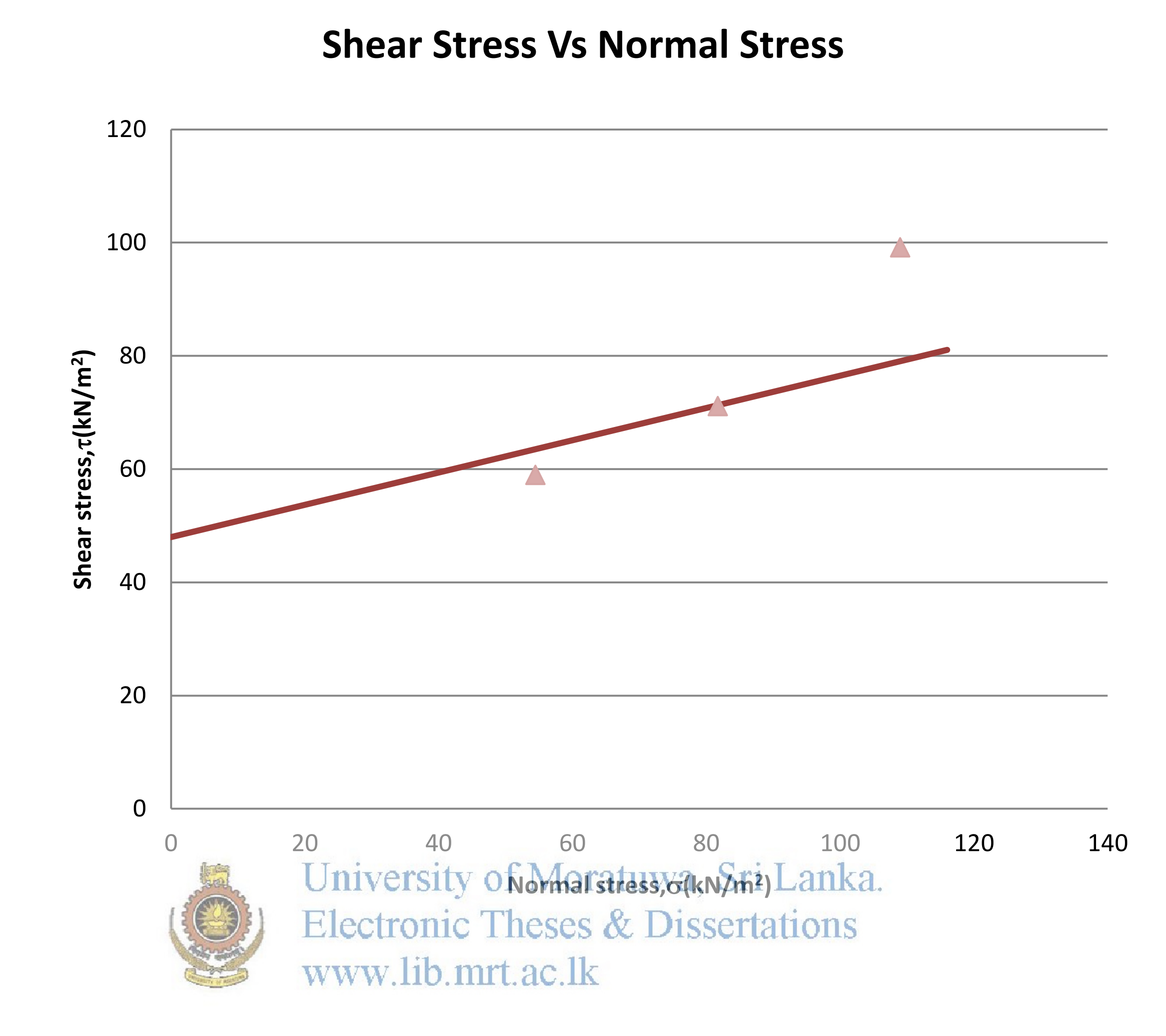

Figure 4.15 Shear stress (τ) Vs Normal Stress (σ)-sample C (un-saturated)

Table 4.12 Specimen results of sample C (Unsaturated)

Test No		1	2	3
Normal Stress	kN/m^2	54.50	81.75	109.00
Peak Shear Stress	kN/m^2	59.00	71.15	99.19
Rate of Stain	mm/min	0.04	0.05	0.02
Stain of Peak Shear Stress	%	2.58	6.70	5.50
Sample Preparation		Undisturbed		
Peak Shear Strength Parameters				
Angle of Internal Friction (in degree) ϕ'		16^o		
Cohesion Intercept (C′) kN/m^2		48		

Evaluation of Effective Shear Strength Parameters c' and ϕ' of sample D (Saturated Condition)

The basic characteristics of the three test specimens obtained from Sample D after saturation are presented in Table 4.13. The bulk density of saturated soil was estimated using Table 4.13 results and relationship between bulk density (γ), specific gravity of soil particle (Gs), degree of saturation (Sr), void ratio (e) and unit weight of water (γ_w). Assuming sample was fully saturated, the degree of saturation (Sr) was taken as 1.The stress strain curves obtained from the three specimens are presented in Figure 4.16. The plots of shear stress Vs normal stress at failure for the three specimens are presented in Figure 4.17. The shear strength parameters are summarized in Table 4.14.

Table 4.13 Specimen data of sample D (saturated)

Specimen data of sample D (saturated)					
Description		specimen-1	specimen-2	specimen-3	Average
Moisture content Measured	(%)	14.07	14.07	14.07	14.07
Bulk Density	(Mg/m^3)	1.98	1.84	1.92	1.91
Dry Density	(Mg/m^3)	1.74	1.61	1.68	1.68
Void Ratio		0.64	0.76	0.69	0.70
Degree of Saturation	(%)	62.87	52.62	57.82	57.77
Specific gravity of soil particle					2.84
Estimated average Bulk Density At 100% saturated	(kN/m^3)				20.44

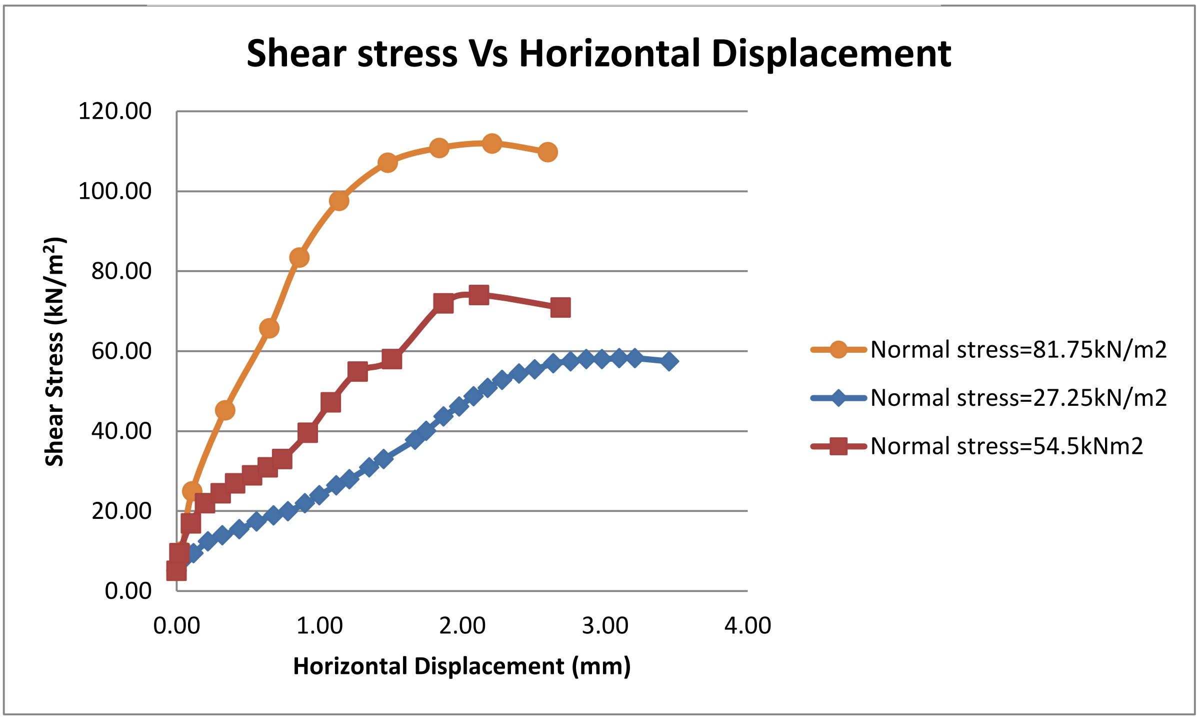

Figure 4.16 Shear stress Vs horizontal displacement- sample D (saturated)

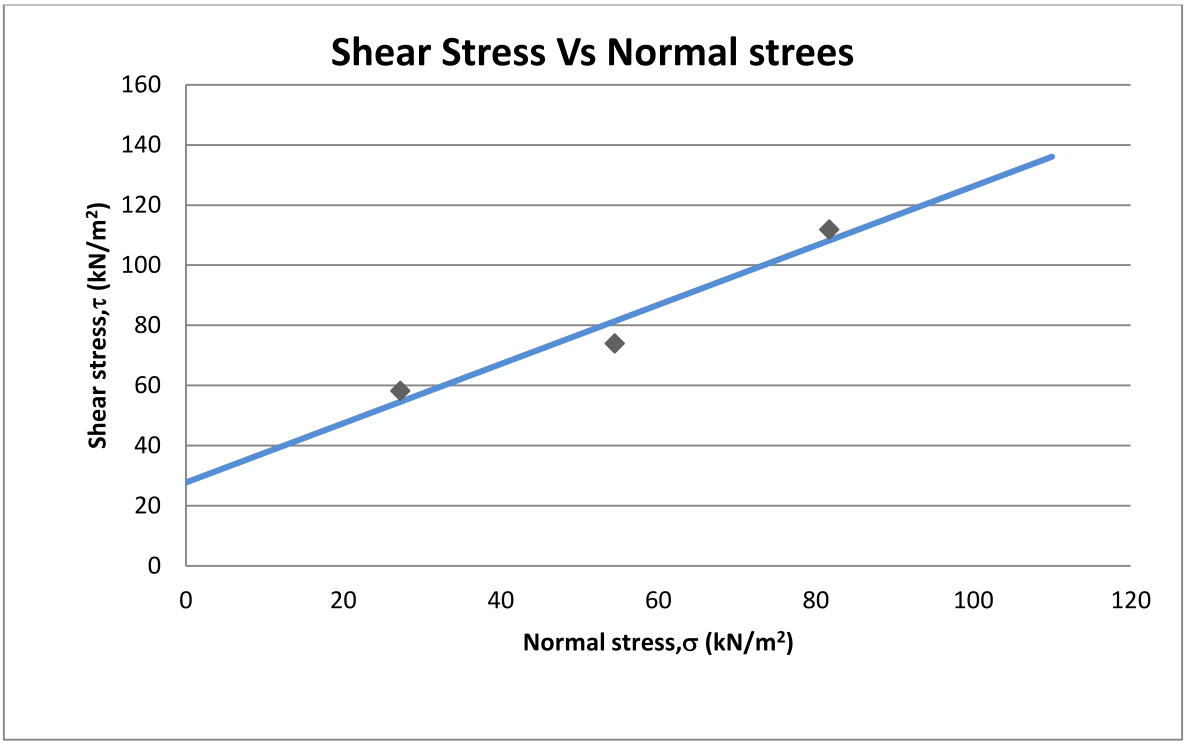

Figure 4.17 Shear stress Vs Normal Stress- sample D (saturated)

Table 4.14 Specimen results of sample D (saturated)

Test No		1	2	3
Normal Stress	kN/m^2	27.25	54.5	81.75
Peak Shear Stress	kN/m^2	58.22	74.01	111.91
Rate of Stain	mm/min	0.01	0.01	0.03
Stain of Peak Shear Stress	%	5.17	3.53	3.68
Sample Preparation		Undisturbed		
Peak Shear Strength Parameters				
Angle of Internal Friction (in degree) ϕ'		44^o		
Cohesion Intercept (C′) kN/m^2		27		

Evaluation of Effective Shear Strength Parameters c' and ϕ' of sample D

(Natural Unsaturated Condition)

The basic characteristics of the three test specimen obtained from Sample D of natural conditions (unsaturated) are presented in Table 4.15. The stress strain curves obtained from the three specimens are presented in Figure 4.18. The plots of shear stress Vs normal stress at failure for the three specimens are presented in Figure 4.19. The shear strength parameters are summarized in Table 4.16.

Table 4.15 Specimen data of sample D (un-saturated)

Specimen data of sample D (un-saturated)					
Description		specimen-1	specimen-2	specimen-3	Average
Moisture content Measured	(%)	14.07	14.07	14.07	14.07
Bulk Density	(Mg/m^3)	1.96	2.01	1.82	1.93
Dry Density	(Mg/m^3)	1.72	1.76	1.6	1.69
Void Ratio		0.65	0.61	0.78	0.68
Degree of Saturation	(%)	61.52	64.98	51.40	59.30

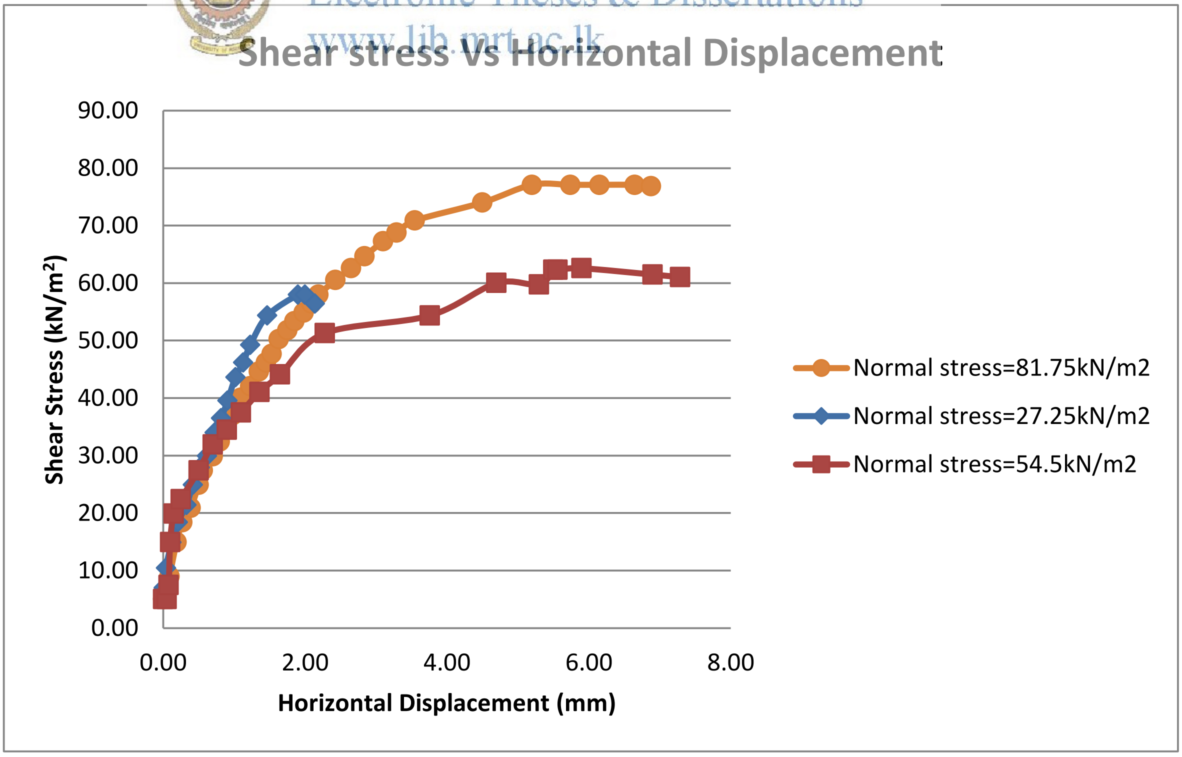

Figure 4.18 Shear stress Vs horizontal displacement- sample D (un-saturated)

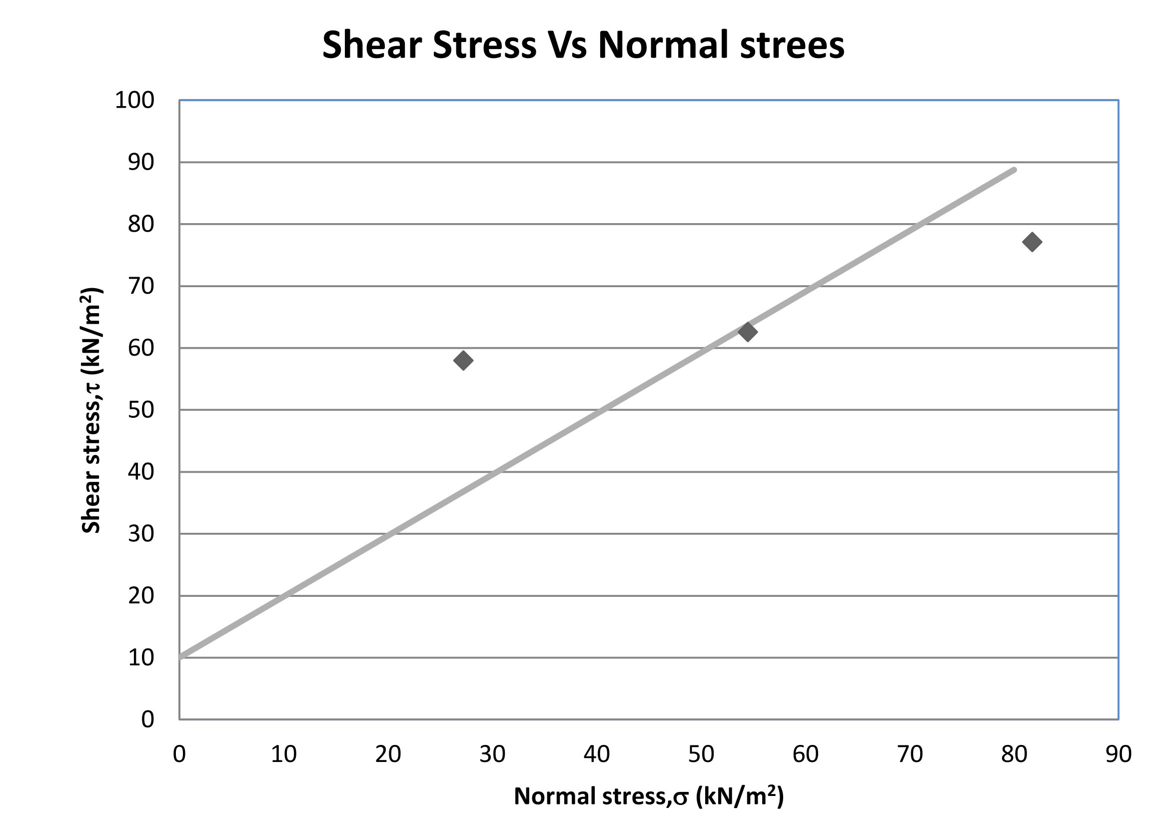

Figure 4.19 Shear stress (τ) Vs Normal Stress(σ)-sample D (un-saturated)

Table 4.16 Specimen results of sample D (Unsaturated)

Test No		1	2	3
Normal Stress	kN/m^2	27.25	54.50	81.75
Peak Shear Stress	kN/m^2	57.96	62.61	77.14
Rate of Stain	mm/min	0.01	0.01	0.01
Stain of Peak Shear Stress	%	3.17	9.83	8.67
Sample Preparation		Undisturbed		
Peak Shear Strength Parameters				
Angle of Internal Friction (in degree) ϕ'		44^o		
Cohesion Intercept (C′) kN/m^2		10		

The Table 4.17 shows the summarized Shear Strength parameters which required to estimation of pullout resistance

Table 4.17 Summery of test result were obtained from direct shear test

Nail Location	**A**	**B**	**C**	**D**
C' (kN/m^2)—Unsaturated(Natural)	20	20	48	10
C' (kN/m^2)--Saturated	9	9	23	27
F'---Unsaturated(Natural)	33	33	16	44
F'---Saturated	33	33	16	44

5. Estimate of pullout resistance and comparison with field test results.

5.1 Data and Formulae for estimation of pullout resistance

The literature survey conducted provided information on number of different methods used at present by design engineers for the estimation of pullout resistance in soil nailing designs. Five of those methods were used in this study to estimate the pullout resistance. The methods are summarized in Table 5.1. The mathematical formulae used for the estimation of pullout resistance under different methods are given in the table.

Methods 1 to 4 did not account for the presence of matric suction. They did not made any specific comment on the nature of shear strength parameters, i.e. whether they are saturated parameters or unsaturated parameters. Method 5, based on more recent research has accounted for the matric suctions and allowed for dilation effects as well.

During the laboratory testing shear strength parameters were obtained both under the insitu unsaturated condition and under the saturated conditions from three undisturbed box samples. The next task is to assign an appropriate set of parameters to different test locations. From the four box samples obtained tests could be conducted only on three; B, C and D.

It is necessary to have an estimate of matric suction to be used with Method 5. In the absence of any measured matric suction values a profile was assumed taking a maximum negative value of 100 kN/m^2. The assumed profile is presented in Figure 5.1. The matric suction values for the different test nail locations are presented in Table 5.2 also.

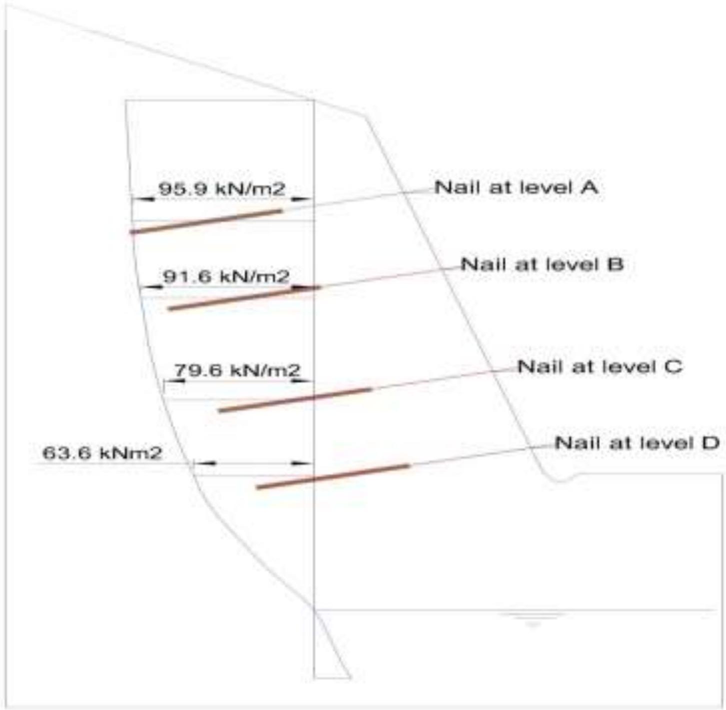

Figure 5.1 Assumed matric Suction variations with ground elevation

Table 5.1 Different methods for Estimation of pullout resistance

Method 01	Schlosser and Guilloux $T_L = P_C' + 2D_{eq}\sigma'_v\mu^*$ Where: T_L = ultimate pull-out resistance (kN/m) c' = effective cohesion of the soil σ_v= effective vertical stress calculated at the mid-point of the nail in the resistance zone μ^* = coefficient of apparent friction of the soil (for granular soils, μ^* is usually taken to be equal to tan Φ' P=perimeter of the soil nail , D=diameter of the nail
Method 02	Heymann et al.(1992) $T_L = P(C' + \sigma'_N \tan\phi')$ where T_L=ultimate pullout resistance (kN/m) , P=perimeter of nail ,C= cohesion σ_N=normal stree (assumed $\sigma_N=\sigma_v$ for the calculations)
Method 03	Hansmann (1992) $T_L = \pi DC' + 2DK_\alpha\sigma_v \tan\phi'$ $K_\alpha=(1-\alpha/90)(1-k_0)$ $K_\alpha=(1-\alpha/90)(\sin\phi')$ where D=diameter of the nail, C' effective cohesion , α=cutting slope (15 degree) σ_v=effective verticle stress T_L=ultimste pullout resistance (kN/m)
Method 04	According to HA 6894 $T_p = \pi D(C + \sigma_n \tan\phi)$ where $\sigma_n=(1+K_L)\sigma_z/2$, $K_L=(1+Ka)/2$, $Ka=(1-\sin\phi)/=(1+\sin\phi)$ D=diameter of the nail , T_p=ultimate pullout resistance kN/m , C=cohesion of the soil
Method 05	Gurpersaud(2010) $Q_{f(us)} = [(c_a + \beta\sigma'_z) + \{(u_a - u_w)(S^K)\tan(\delta+\psi)\}]\pi dL$ $\beta=k_\theta\tan(\delta+\psi)$, $k_\theta/k_0=1+(1-k_0)/2k_0\ x(1/\cos2\theta)$ Where: $Q_{f(us)}$ = ultimate pull-out resistance (kN/m) , β=Bjerrum-burland coefficient c_a = aperent cohesion of the soil , S=degree of saturation , K=fitting parameter (=1) σ_v= effective vertical stress calculated at the mid-point of the nail in the resistance zone ψ = dilation angle (taken 10 and 20 degree) , δ= Interface friction angle L=effective length of the soil nail , d=diameter of the nail ,θ=cutting slope (75 degree)

Table 5.2 Computed matric suction values for each nail level

Location	Depth(m)	Matric suction (kN/m^2)
A	4.6	95.9
B	6.5	91.6
C	9.0	79.6
D	10.9	63.6

The undisturbed box samples were located at same depth of each test nail centre. The Table 5.3 is presented required measurement of test nails and sample locations.

Table 5.3 Location of test nails and sampling

Test Nail	depth to nail at slope (m)	depth to nail centre (m)	depth to sample location (m)
A1	2.25	4.6	4.6
A2	2.25	4.6	4.6
B1	4.50	6.5	6.5
B2	4.50	6.5	6.5
C1	7.50	9.0	9.0
C2	7.50	9.0	9.0
D1	9.75	10.9	10.9
D2	9.75	10.9	10.9

As methods 1 to 4 did not made any reference to matric suction values, the pullout resistance values were estimated with two sets; unsaturated parameters and saturated parameters. The numerical values of parameters used with reference do different nail locations are presented in Table 5.4.

Table 5.4 Shear Strength Parameters Used for Estimation of pullout resistance

Test Nail Location	Lab test	Saturated parameters used	Unsaturated parameters used
A1	Test-B	$c' = 9$ kN/m^2, $\phi' = 33^o$	$c' = 20$ kN/m^2, $\phi' = 33^o$
A2	Test-B	$c' = 9$ kN/m^2, $\phi' = 33^o$	$c' = 20$ kN/m^2, $\phi' = 33^o$
B1	Test-B	$c' = 9$ kN/m^2, $\phi' = 33^o$	$c' = 20$ kN/m^2, $\phi' = 33^o$
B2	Test-B	$c' = 9$ kN/m^2, $\phi' = 33^o$	$c' = 20$ kN/m^2, $\phi' = 33^o$
C1	Test-C	$c' = 23$ kN/m^2, $\phi' = 16^o$	$c' = 48$ kN/m^2, $\phi' = 16^o$
C2	Test-C	$c' = 23$ kN/m^2, $\phi' = 16^o$	$c' = 48$ kN/m^2, $\phi' = 16^o$
D1	Test-D	$c' = 27$ kN/m^2, $\phi' = 44^o$	$c' = 10$ kN/m^2, $\phi' = 44^o$
D2	Test-D	$c' = 27$ kN/m^2, $\phi' = 44^o$	$c' = 10$ kN/m^2, $\phi' = 44^o$

With the different method, separate calculations were done with both saturated and unsaturated parameters as applicable. Average pullout capacity over 1m length was calculated from the measured pullout capacities obtained over 2.5m grouted length. The pullout resistances estimated under different conditions were compared with the experimental values.

5.2 Estimation of pullout capacity with unsaturated and saturated shear strength parameters

Method 1 to Method 4 for the estimation of pullout resistance used shear strength parameters without specifying whether they are saturated or unsaturated parameters. However, according to the prevailing site conditions all the test nails were under unsaturated conditions. As such, the pullout resistance was estimated using Methods 1 to Method 4 using unsaturated shear strength parameters in the equations. Method 5 accounts for matric suction as well as dilation effects. The shear strength parameters C and ϕ corresponding the expression of Method 5 are therefore saturated parameters. Hence, pullout resistance values were estimated by method 5 using the saturated parameters and estimated matric suction and angle of dilation. The values obtained are summarized in Table 5.5 and graphically presented in Figure 5.2.

The estimated values as a percentage of corresponding experimentally observed values are presented in Table 5.6.

If saturated shear strength parameters were used in the estimate of pullout resistance, Method 1 to Method 4 would result in values presented in Table 5.7. The estimated values as a percentage of corresponding experimentally observed values are presented in Table 5.8. Obviously, these values are much lower than the values computed under unsaturated condition and are much lower than the actual observations.

Table 5.5 Measured and Estimated Pullout capacity

Pull out capacity kN/m							
Overburden height (m)	Measured	Method 01	Method 02	Method 03	Method 04	Method 05	
						for Ψ =10°	for Ψ =20°
4.57	50.40	17.91	23.98	16.95	21.03	31.24	37.38
4.57	50.40	17.91	23.98	16.95	21.03	31.24	37.38
6.46	54.80	22.32	30.89	20.95	26.73	34.76	40.62
6.46	42.40	22.32	30.89	20.95	26.73	34.76	40.62
9.00	60.80	26.38	31.45	25.97	29.94	34.15	40.05
9.00	56.40	26.38	31.45	25.97	29.94	34.15	40.05
10.92	80.80	49.96	76.40	44.59	54.53	57.76	64.10
10.92	80.80	49.96	76.40	44.59	54.53	57.76	64.10

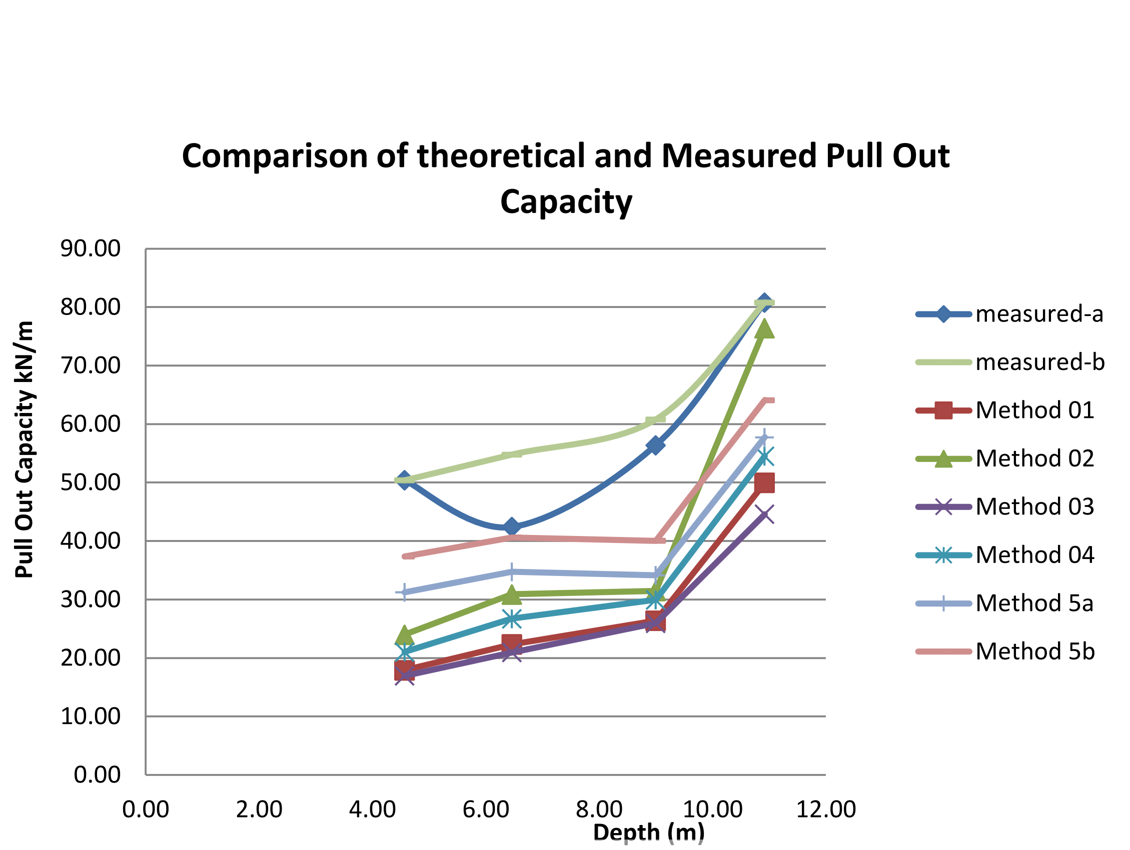

Figure 5.2 Comparison of theoretical and measured pullout capacity.

Table 5.6 Percentage of Deviation –Theoretical & Measured Pullout Capacity

Percentage of estimated pull out capacity as a measured capacity						
Overburden height (m)	Method 01	Method 02	Method 03	Method 04	Method 05	
					for Ψ =10⁰	for Ψ=20⁰
4.57	35.54	47.57	33.62	41.73	61.98	74.17
4.57	35.54	47.58	33.62	41.73	61.98	74.17
6.46	40.72	56.37	38.23	48.77	63.43	74.12
6.46	52.64	72.85	49.41	63.04	81.98	95.80
9.00	43.39	51.73	42.71	49.25	56.17	65.87
9.00	46.77	55.76	46.04	53.09	60.55	71.01
10.92	61.83	94.55	55.19	67.49	71.49	79.33
10.92	61.83	94.55	55.19	67.49	71.49	79.33

Table 5.7 Estimates Pullout capacity for saturated condition

Pull out capacity for saturated condition kN/m					
Location	Depth (m)	Method 01	Method 02	Method 03	Method 04
A1	4.57	15.45	22.40	14.35	19.02
A2	4.57	15.45	22.40	14.35	19.02
B1	6.46	20.50	30.33	18.93	25.55
B2	6.46	20.50	30.33	18.93	25.55
C1	9.00	18.08	23.62	17.64	21.97
C2	9.00	18.08	23.62	17.64	21.97
D1	10.92	59.86	88.41	54.05	66.25
D2	10.92	59.86	88.41	54.05	66.25

Table 5.8 Percentage of estimates pullout capacity for saturated condition as a measured capacity

Percentage of estimated pull out capacity as a measured capacity					
Location	Depth (m)	Method 01	Method 02	Method 03	Method 04
A1	4.57	30.7	44.4	28.5	37.7
A2	4.57	30.7	44.4	28.5	37.7
B1	6.46	37.4	55.3	34.5	46.6
B2	6.46	48.3	71.5	44.6	60.3
C1	9.00	29.7	38.8	29.0	36.1
C2	9.00	32.1	41.9	31.3	39.0
D1	10.92	74.1	109.0	66.9	82.0
D2	10.92	74.1	109.0	66.9	82.0

5.3 Variation of measured pullout capacity with overburden height

The ultimate pull out capacity obtained by testing was tabulated with respect to overburden pressure to study behavior of pullout capacity with the depth

Table 5.9 Measured pull out capacity

Depth(m)	Measured Pull Out Capacity(kN)
2.25	126
2.25	126
4.5	106
4.5	137
7.5	141
7.5	152
9.25	202
9.25	202

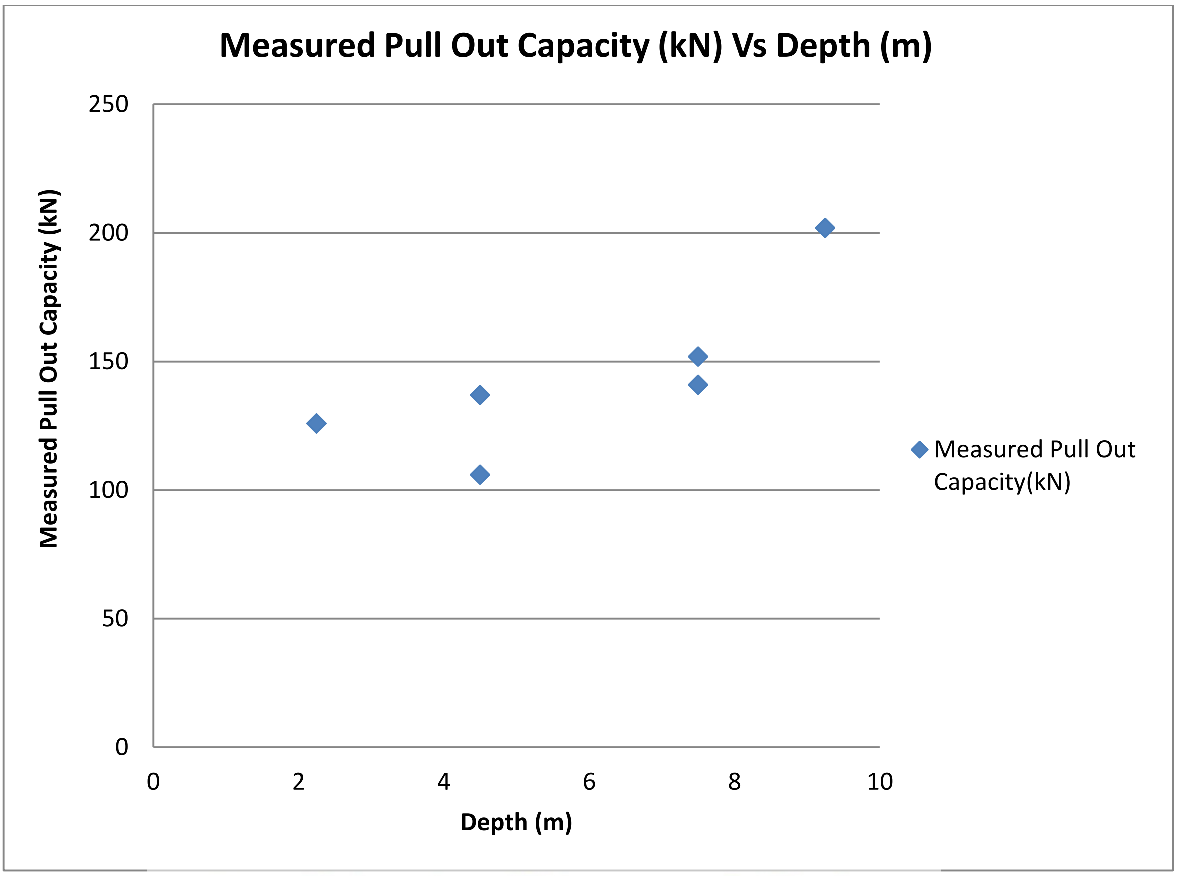

Figure 5.3 Measured pullout capacity with overburden height

5.4 Effective diameter of the soil nail.

Nominal drill hall diameter used in the drilling process was lower than the diameter calculated using measured perimeter. The comparison is presented in Table 5.10.

Table 5.10 Effective diameter of the soil nail

Diameter of the grouted nail /(mm)					
Nail No		Drill hole diameter	Average measured perimeter of grouted nail	Average diameter of grouted nail	% of increase
	A1	116	440	140	21
A	A2	116	446	142	22
	B1	116	450	143	23
B	B2	116	452	144	24
	C1	116	447	142	22
C	C2	116	454	144	24
	D1	116	-	-	
D	D2	116	Average percentage of increase		**22.6**

Therefore, the general increase of the perimeter from the normal vales is 22.6%. If all the estimates made under the unsaturated conditions were increased by 22.6%, it would provide values as presented in Table 5.11. The estimated values as a percentage of corresponding experimentally observed values are presented in Table 5.12. However, with the comparison it is evident that even these values are lower than the measured pullout resistance except last nail level of method 02.

Table 5.11 Estimated pullout capacities considering measured Effective diameter of nails

Pull out capacity kN/m							
Overburden height (m)	Measured	Method 01	Method 02	Method 03	Method 04	Method 05	
						for Ψ =10°	for Ψ =20°
4.57	50.40	21.96	29.40	20.78	25.78	38.30	45.83
4.57	50.40	21.96	29.40	20.78	25.78	38.30	45.83
6.46	54.80	27.36	37.87	25.68	32.77	42.62	49.80
6.46	42.40	27.36	37.87	25.68	32.77	42.62	49.80
9.00	60.80	32.34	38.56	31.83	36.71	41.87	49.10
9.00	56.40	32.34	38.56	31.83	36.71	41.87	49.10
10.92	80.80	61.25	93.67	54.67	66.85	70.81	78.59
10.92	80.80	61.25	93.67	54.67	66.85	70.81	78.59

Table 5.12 Percentage of Deviation Theoretical & Measured Pullout Capacity

Percentage of estimated pull out capacity as a measured capacity							
Overburden height (m)	Measured	Method 01	Method 02	Method 03	Method 04	Method 05	
						for Ψ =10°	for Ψ =20°
4.57	50.4	43.6	58.3	41.2	51.2	76.0	90.9
4.57	50.4	43.6	58.3	41.2	51.2	76.0	90.9
6.46	54.8	49.9	69.1	46.9	59.8	77.8	90.9
6.46	42.4	64.5	89.3	60.6	77.3	100.5	117.5
9.00	60.8	53.2	63.4	52.4	60.4	68.9	80.8
9.00	56.4	57.3	68.4	56.4	65.1	74.2	87.1
10.92	80.8	75.8	115.9	67.7	82.7	87.6	97.3
10.92	80.8	75.8	115.9	67.7	82.7	87.6	97.3

5.5 Concluding Comment on the comparison of pullout resistance

The comparison of the measured pullout resistance with the predictions done with different methods currently in use clearly indicated that all the current methods underestimate the pullout capacity. By accounting for the;

- Unsaturated condition that prevail
- Possible dilation during pullout
- Possible increase of drill hole diameter due to grout pressure

Values much closer to the experimentally determined values can be obtained. In here the matric suction values were assumed as it could not be measured. If much higher matric suction values have prevailed and used in the analysis the computed pullout resistance values would be increased.

5.6 Comparison of pullout test and direct shear test results

Both direct shear test and pullout test mechanisms are having similar behavior where shear resistance was mobilizing gradually with the shear displacement. Hence a direct comparison of shear stress Vs displacement behavior during tests could provide some insight into the pullout mechanisms. As such, it was decided plot the two test results together. Since it was not obtained dial gauge reading at the failure load, 10mm displacement was assumed to plot aforesaid graphs. Similar comparison was done by Chu and Yin (2005). The loads at different stages in the pullout tests were converted to shear stresses dividing by the effective nail surface area. The converted pullout test results are presented in Table 5.13a and Table 5.13b.

Table 5.13a– Pullout Test Results (converted to unit shear resistance)

Pull Out Test Result							
Pull Out Resistance- Normal stress 73.0 kN/m^2 (nail level A)				Pull Out Resistance- Normal stress 103.35 kN/m^2 (nail level B)			
Nail A1		Nail A2		Nail B1		Nail B2	
Displacement (mm)	pullout stress at loading (kN/m^2)	Displacement (mm)	pullout stress at loading (kN/m^2)	Displacement (mm)	pullout stress at loading (kN/m^2)	Displacement (mm)	pullout stress at loading (kN/m^2)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.90	55.98	1.25	55.98	1.03	55.98	0.82	55.98
2.27	82.32	2.61	82.32	1.65	82.32	2.70	82.32
5.30	108.66	4.12	108.66	2.76	108.66	6.90	108.66
5.97	133.91	7.87	133.91	5.18	133.91	8.65	133.91
10.00	138.30	10.00	138.30	10.00	116.35	10.00	150.37

Table 5.13b– Pullout Test Results (converted to unit shear resistance)

Pull Out Test Result							
Pull Out Resistance- Normal stress 135.9 kN/m^2 (nail level C)				Pull Out Resistance- Normal stress 164.9 kN/m^2 (nail level D)			
Nail C1		Nail C2		Nail D1		Nail D2	
Displace ment (mm)	pullout stress at loading (kN/m^2)	Displace ment (mm)	pullout stress at loading (kN/m^2)	Displace ment (mm)	pullout stress at loading (kN/m^2)	Displace ment (mm)	pullout stress at loading (kN/m^2)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.87	55.98	0.83	55.98	0.10	55.98	0.46	55.98
1.84	82.32	2.49	82.32	0.90	82.32	1.11	82.32
3.21	108.66	3.04	108.66	1.37	108.66	1.65	108.66
6.75	133.91	7.30	133.91	1.99	133.91	2.29	133.91
10.00	166.84	10.00	154.76	10.00	221.72	10.00	221.72

5.6.1 Comparison of pullout test and direct shear test results of sample test B

The test results presented in Table 5.13a and Table 5.13b were compared with direct shear test B (saturated) test results presented in Table 5.14 and graphically presented in Figure 5.4. Thereafter the pullout test results were compared with the results of Test B (unsaturated) in Table 5.15. The results are compared graphically in Figure 5.5.

Table 5.14– Direct Shear Test Results – Test B Saturated

Direct Shear Test Result					
Direct Shear- Normal stress 54.5 kN/m^2		Direct Shear- Normal stress 81.75 kN/m^2		Direct Shear -Normal stress 109 kN/m^2	
Displacement (mm)	shear stress (kN/m^2)	Displacement (mm)	shear stress (kN/m^2)	Displacement (mm)	shear stress (kN/m^2)
0.00	5.03	0.00	5.03	0.00	5.03
0.12	20.41	0.20	24.92	0.08	15.92
0.44	29.44	0.50	34.49	0.23	33.98
1.00	40.06	1.10	48.22	0.51	55.39
1.60	42.10	1.89	55.91	0.91	70.89
2.45	43.12	2.60	59.51	1.41	77.66
3.00	44.13	3.30	61.06	1.96	80.79
3.60	45.15	4.10	61.57	2.39	81.32
4.10	45.66	4.86	61.83	2.92	81.58
4.60	45.66	5.60	61.32	3.40	81.84
5.30	46.17			3.95	82.10
6.00	46.17			4.50	82.36
6.50	46.17			5.39	82.26
7.20	45.66			6.30	81.84
7.75	45.15			7.50	81.06
8.30	45.15				
8.80	44.64				
9.65	44.13				

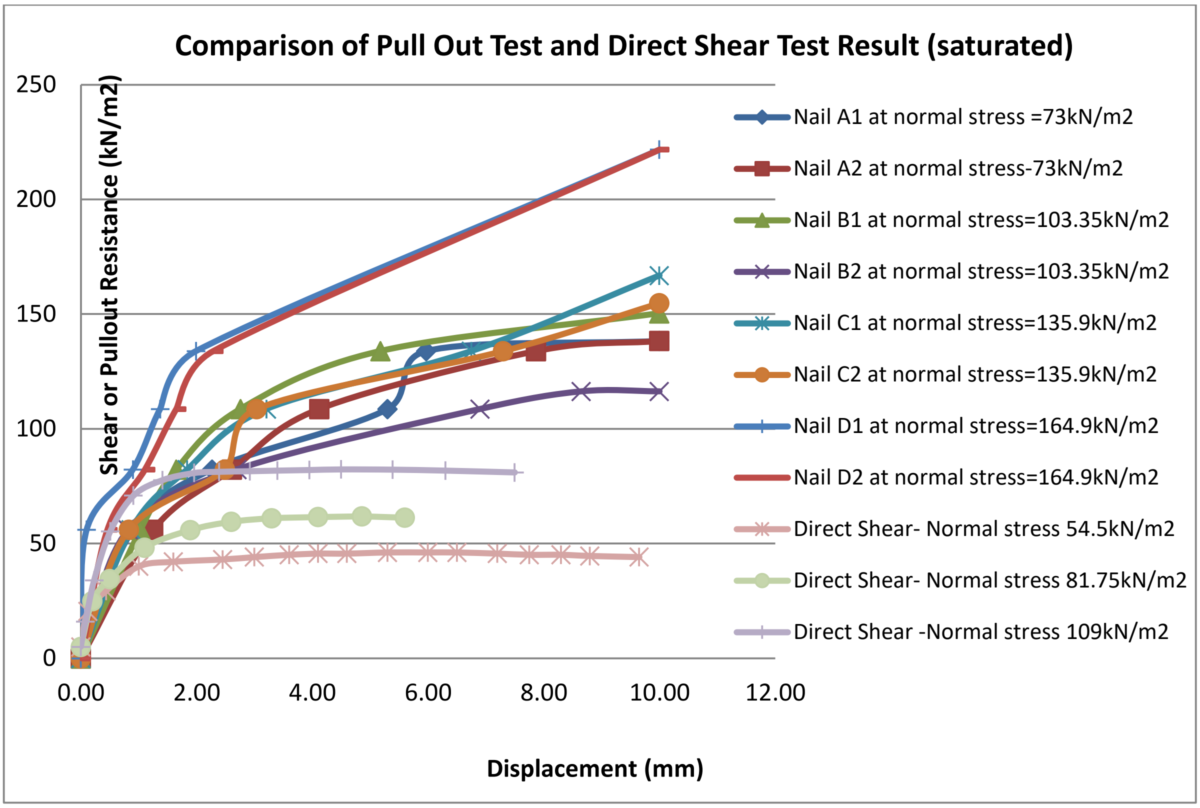

Figure 5.4 Comparison of pullout test and direct shear test result –Test B (saturated)

Table 5.15– Direct Shear Test Results – Test B Unsaturated (natural condition)

Direct Shear Test Result					
Direct Shear- Normal stress 54.5 kN/m²		Direct Shear- Normal stress 81.75 kN/m²		Direct Shear -Normal stress 109 kN/m²	
Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)
0.00	5.03	0.00	5.03	0.00	5.03
0.11	15.923	0.05	6.015	0.25	13.44
0.31	22.91	0.07	8.978	0.45	33.48
0.74	36.00	0.09	13.934	0.66	49.75
1.14	47.20	0.12	18.414	1.95	62.61
1.6	55.91	0.15	23.413	2.43	79.23
2.04	63.64	0.27	38.033	2.80	94.44
2.53	66.74	0.35	47.706	3.30	108.72
3.04	67.78	0.57	61.574	3.62	112.97
3.69	67.26	0.79	72.973	4.30	114.57
		0.91	78.183	4.98	115.10
		1.24	87.602	5.55	115.10
		1.63	93.385	6.13	114.57
		1.95	94.438	6.83	114.04
		2.34	94.438	8.10	111.91
		3.16	94.438		
		3.54	94.438		

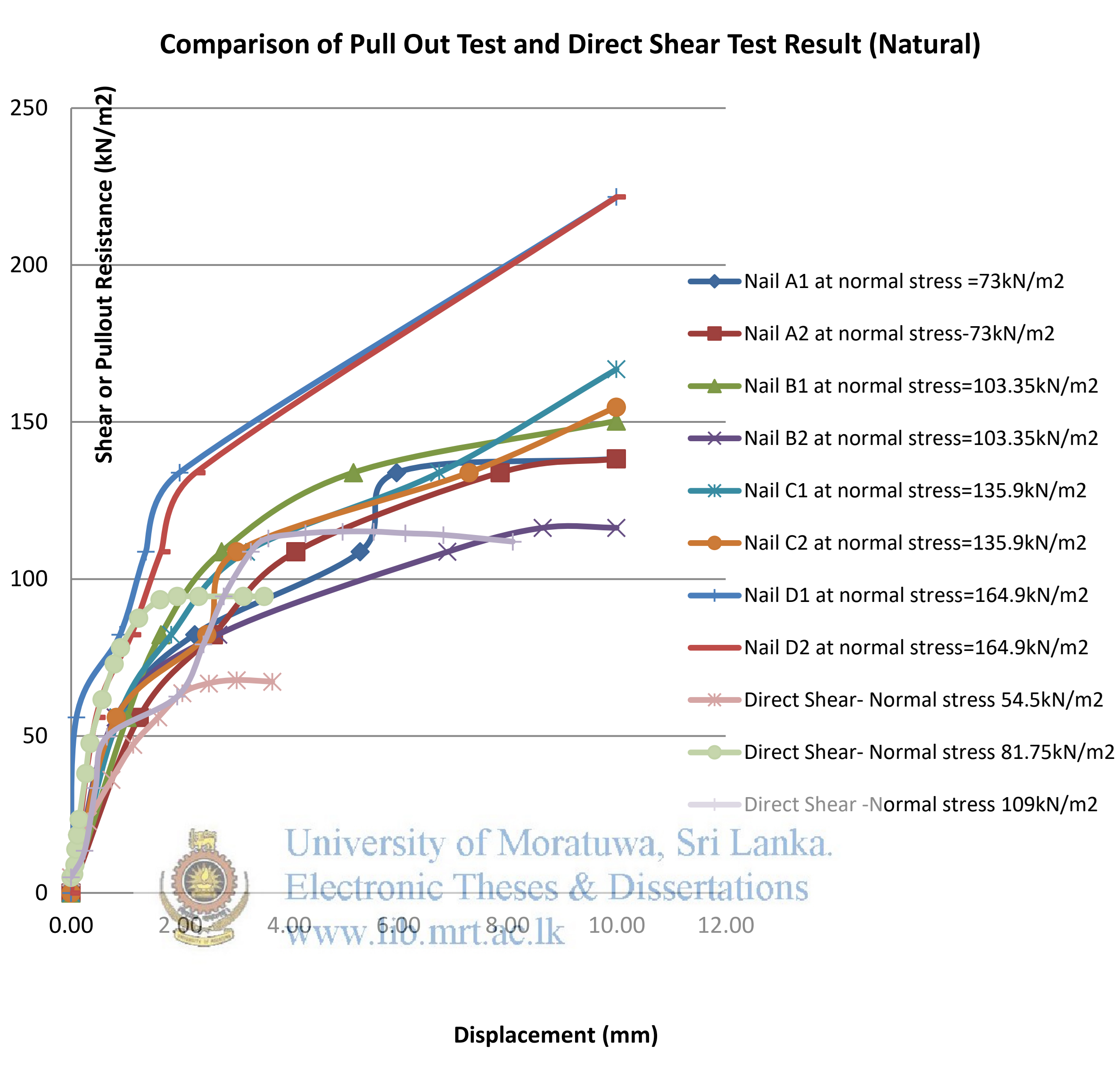

Figure 5.5 Comparison of pullout test and direct shear test result (Test B-unsaturated)

The shear stress-displacement graphs presented in the figures are drawn for different normal stresses or different overburden stresses. However, even under comparable normal stresses and overburden stresses, the pullout resistance values generally plotted above the direct shear test values.

In order to obtain a direct comparison of two types of plots, both the direct shear test results and the pullout resistance test results were normalized by the dividing by the applied normal stress or dividing by the overburden stress. The values obtained are presented in Table 5.16, Table 5.17, Table 5.18a and Table 5.18b.

The values are compared graphically in Figure 5.6 and Figure 5.7.

Table 5.16– Normalized Direct Shear Test Results – Test B (Saturated)

Direct Shear Test Result					
Direct Shear- Normal stress 54.5 kN/m²		Direct Shear- Normal stress 81.75 kN/m²		Direct Shear -Normal stress 109 kN/m²	
Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)
0.00	0.09	0.00	0.06	0.00	0.05
0.12	0.37	0.20	0.30	0.08	0.15
0.44	0.54	0.50	0.42	0.23	0.31
1.00	0.74	1.10	0.59	0.51	0.51
1.60	0.77	1.89	0.68	0.91	0.65
2.45	0.79	2.60	0.73	1.41	0.71
3.00	0.81	3.30	0.75	1.96	0.74
3.60	0.83	4.10	0.75	2.39	0.75
4.10	0.84	4.86	0.76	2.92	0.75
4.60	0.84	5.60	0.75	3.40	0.75
5.30	0.85			3.95	0.75
6.00	0.85			4.50	0.76
6.50	0.85			5.39	0.75
7.20	0.84			6.30	0.75
7.75	0.83			7.50	0.74
8.30	0.83				
8.80	0.82				
9.65	0.81				

Table 5.17– Normalized Direct Shear Test Results – Test B (Unsaturated)

Direct Shear Test Result					
Direct Shear- Normal stress 54.5 kN/m²		Direct Shear- Normal stress 81.75 kN/m²		Direct Shear -Normal stress 109 kN/m²	
Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)
0.00	0.09	0.00	0.06	0.00	0.05
0.11	0.29	0.05	0.07	0.25	0.12
0.31	0.42	0.07	0.11	0.45	0.31
0.74	0.66	0.09	0.17	0.66	0.46
1.14	0.87	0.12	0.23	1.95	0.57
1.6	1.03	0.15	0.29	2.43	0.73
2.04	1.17	0.27	0.47	2.80	0.87
2.53	1.22	0.35	0.58	3.30	1.00
3.04	1.24	0.57	0.75	3.62	1.04
3.69	1.23	0.79	0.89	4.30	1.05
		0.91	0.96	4.98	1.06
		1.24	1.07	5.55	1.06
		1.63	1.14	6.13	1.05
		1.95	1.16	6.83	1.05
		2.34	1.16	8.10	1.03
		3.16	1.16		
		3.54	1.16		

Table 5.18a– Normalized Pullout Test Results (unit shear stress)

Pull Out Test Result							
Pull Out Resistance- Normal stress 73.0 kN/m² (nail level A)				Pull Out Resistance- Normal stress 103.35 kN/m² (nail level B)			
Nail A1		Nail A2		Nail B1		Nail B2	
Displacement (mm)	pullout stress at loading (kN/m²)	Displacement (mm)	pullout stress at loading (kN/m²)	Displacement (mm)	pullout stress at loading (kN/m²)	Displacement (mm)	pullout stress at loading (kN/m²)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.90	0.77	1.25	0.77	1.03	0.54	0.82	0.54
2.27	1.13	2.61	1.13	1.65	0.79	2.70	0.79
5.30	1.49	4.12	1.49	2.76	1.05	6.90	1.05
5.97	1.83	7.87	1.83	5.18	1.29	8.65	1.29
10.00	1.89	10.00	1.89	10.00	1.13	10.00	1.45

Table 5.18b– Normalized Pullout Test Results (unit shear stress)

Pull Out Test Result							
Pull Out Resistance- Normal stress 135.9 kN/m² (nail level C)				Pull Out Resistance- Normal stress 164.9 kN/m² (nail level D)			
Nail C1		Nail C2		Nail D1		Nail D2	
Displacement (mm)	pullout stress at loading (kN/m²)	Displacement (mm)	pullout stress at loading (kN/m²)	Displacement (mm)	pullout stress at loading (kN/m²)	Displacement (mm)	pullout stress at loading (kN/m²)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.87	0.41	0.83	0.41	0.10	0.34	0.46	0.34
1.84	0.61	2.49	0.61	0.90	0.50	1.11	0.50
3.21	0.80	3.04	0.80	1.37	0.66	1.65	0.66
6.75	0.98	7.30	0.98	1.99	0.81	2.29	0.81
10.00	1.23	10.00	1.14	10.00	1.34	10.00	1.34

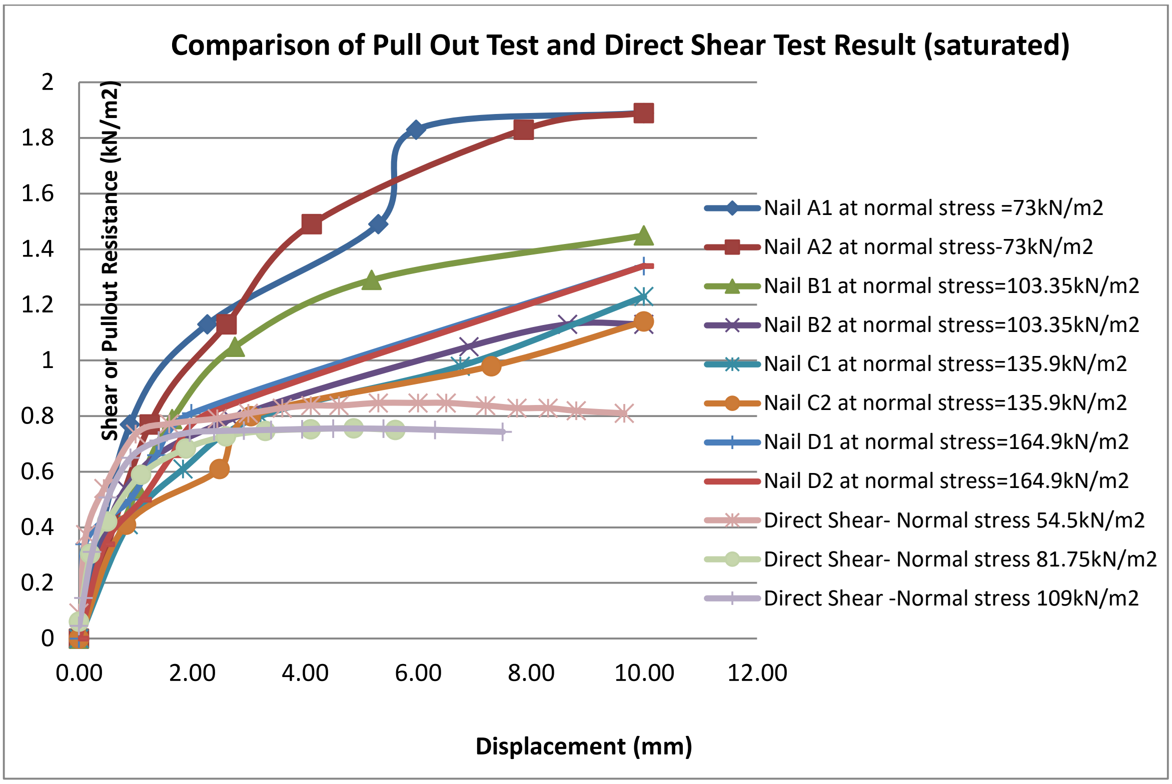

Figure 5.6 Comparison of normalized stresses of pullout and direct shear test result Test B - Saturated

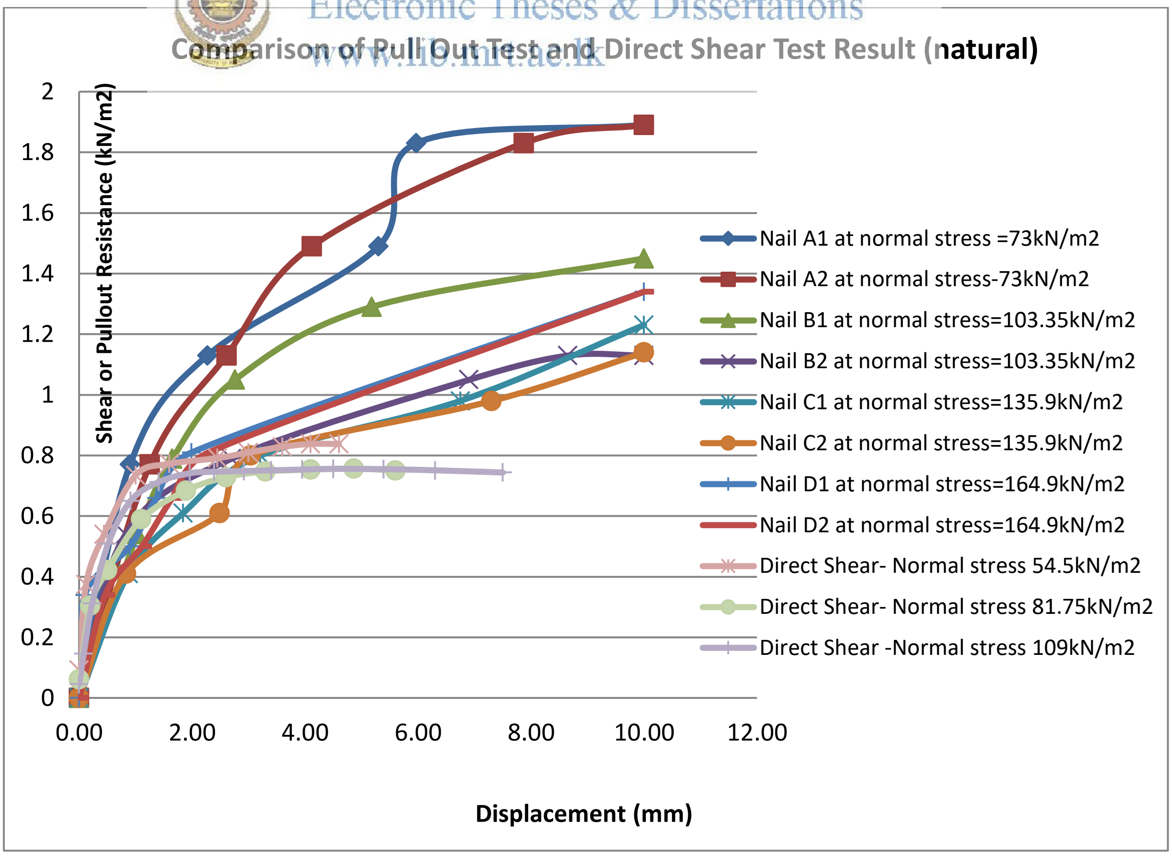

Figure 5.7 Comparison of normalized stresses of pullout and direct shear test result – Test B unsaturated

Even after the normalizing all the pullout test results plotted above the direct shear test results. Most of the pullout test results are showing a strain hardening effect. In contrast, the direct shear test results show a flatter shape after reaching the peak value. This strain hardening effect indicates that at the pullout of a nail it may not be subjected to a simple interface failure. There may be other effects such as dilation that contributes to the pullout resistance.

5.6.2 Comparison of pullout test and direct shear test results of sample test C

The test results presented in above Table 5.13a and Table 5.13b were compared with Test C (saturated) test results presented in Table 5.19 and graphically presented in Figure 5.8. Thereafter the pullout test results were compared with Test C (unsaturated) test results presented in Table 5.20. The results are compared graphically in Figure 5.9.

Table 5.19– Direct Shear Test Results – Test C Saturated

Direct Shear Test Result					
Direct Shear- Normal stress 54.5 kN/m²		**Direct Shear- Normal stress 81.75** kN/m²		**Direct Shear -Normal stress 109** kN/m²	
Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)
0.00	5.03	0.00	5.03	0.00	5.03
0.03	14.43	0.01	9.97	0.03	17.42
0.13	22.41	0.03	16.42	0.11	24.42
0.27	27.93	0.07	21.41	0.20	30.45
0.41	31.46	0.13	24.92	0.31	34.49
0.55	33.48	0.19	27.93	0.41	38.04
0.74	36.01	0.24	30.95	0.63	42.61
0.91	37.02	0.29	33.98	0.76	44.13
1.09	37.02	0.35	36.51	1.00	46.68
1.26	37.02	0.41	39.05	1.24	48.22
1.42	37.02	0.47	40.57	1.74	49.75
2.40	36.51	0.82	46.69	2.27	51.29
		1.18	48.22	2.86	52.31
		1.55	49.24	3.49	52.57
		1.94	49.75	4.15	52.57
		2.64	50.26	4.54	52.57
		3.42	50.26	4.76	52.57
		4.34	50.26	5.40	52.57
		5.14	50.01	6.50	51.80

The following figure represents the behavior of pull out test and direct shear test results respect to the different normal stress conditions.

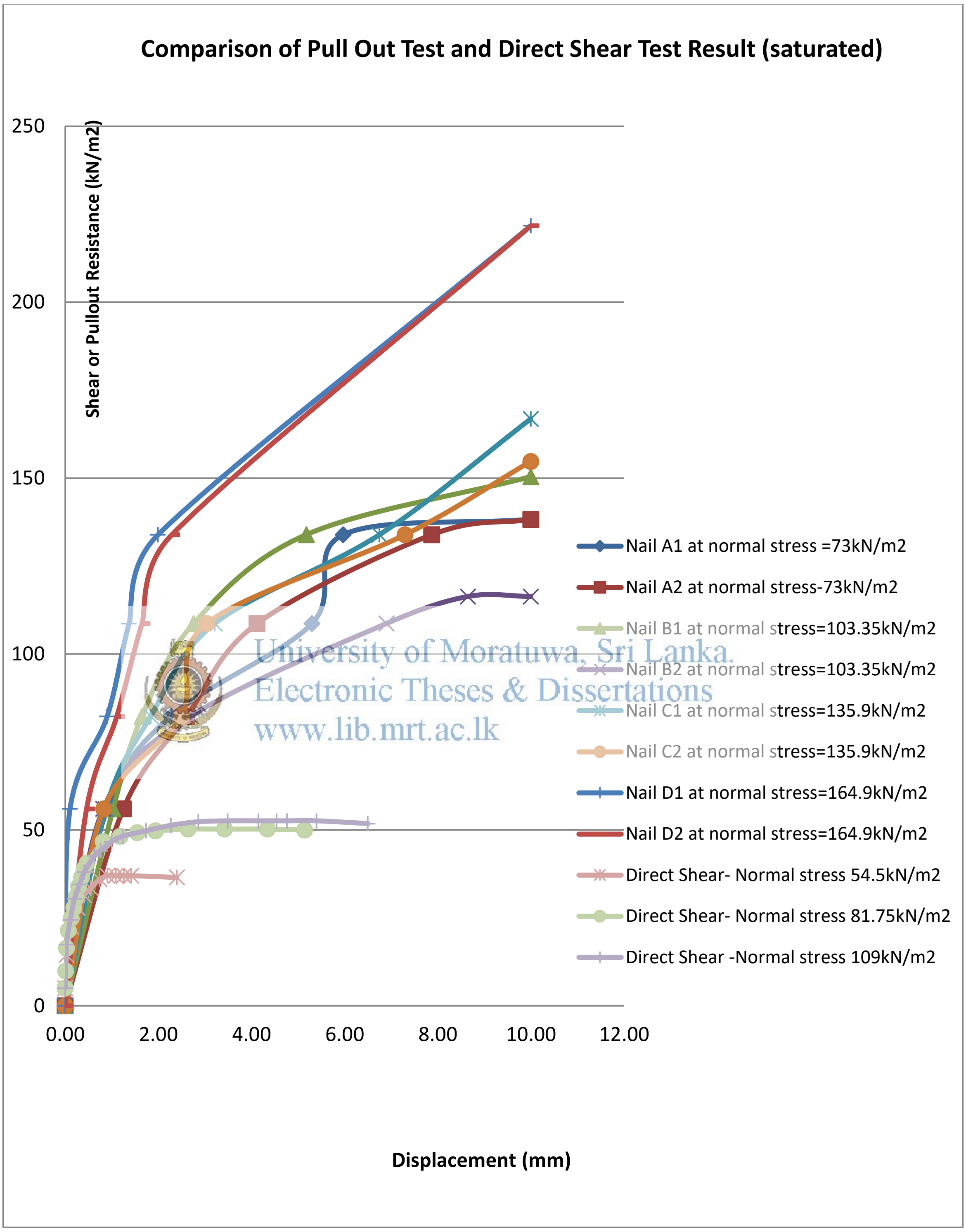

Figure 5.8 Comparison of pullout test and direct shear test result (Test C - saturated)

Table 5.20– Direct Shear Test Results – Test C Unsaturated (natural condition)

Direct Shear Test Result					
Direct Shear- Normal stress 54.5 kN/m²		**Direct Shear- Normal stress 81.75** kN/m²		**Direct Shear -Normal stress 109** kN/m²	
Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)
0.00	5.03	0.00	5.03	0.00	5.03
0.02	8.48	0.08	12.44	0.00	5.52
0.07	14.93	0.18	17.91	0.01	9.97
0.13	20.91	0.29	27.93	0.03	14.93
0.20	24.92	0.41	28.18	0.08	16.92
0.29	29.44	0.50	31.96	0.16	16.92
0.35	32.97	0.61	34.99	0.18	18.41
0.43	36.01	0.72	39.05	0.18	20.41
0.50	39.56	0.83	42.61	0.20	22.91
0.58	43.12	0.94	45.66	0.22	24.92
0.66	45.15	1.02	48.73	0.24	27.18
1.07	55.39	1.61	58.99	0.36	39.56
1.55	59.00	2.20	65.71	0.52	55.91
2.06	58.22	2.78	69.08	0.70	69.86
2.29	57.45	3.10	69.86	0.86	78.18
		3.42	70.63	1.09	82.10
		4.02	71.15	1.30	88.92
		4.62	71.15	1.76	94.70
		5.23	70.89	2.26	97.87
				2.52	98.13
				2.75	98.66
				3.30	99.19
				3.48	99.19

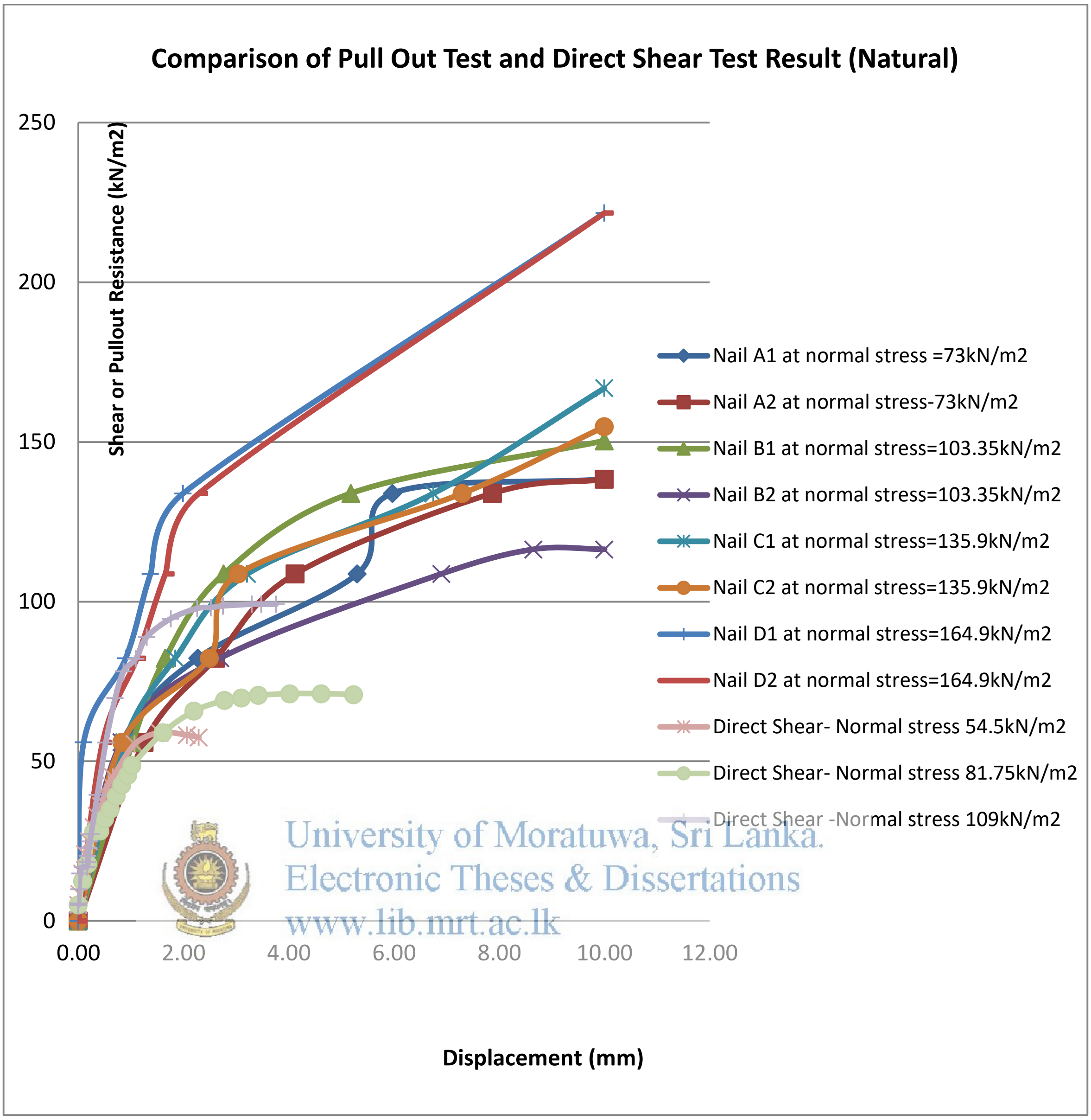

Figure 5.9 Comparison of pullout test and direct shear test result (Test C - unsaturated)

The shear stress-displacement graphs presented in the figures are drawn for different normal stresses or different overburden stresses. However, even under comparable normal stresses and overburden stresses, the pullout resistance values generally plotted above the direct shear test values.

In order to obtain a direct comparison of two types of plots, both the direct shear test results and the pullout resistance test results were normalized by the dividing by the applied normal stress or dividing by the overburden stress. The values obtained are presented in Table 5.21, Table 5.22, and above Table 5.18a and Table 5.18b.

The values are compared graphically in Figure 5.10 and Figure 5.11.

Table 5.21– Normalized Direct Shear Test Results – Test C Saturated

Direct Shear Test Result					
Direct Shear- Normal stress 54.5 kN/m²		**Direct Shear- Normal stress 81.75** kN/m²		**Direct Shear -Normal stress 109** kN/m²	
Displacement (mm)	Normalized shear stress (kN/m²)	Displacement (mm)	Normalized shear stress (kN/m²)	Displacement (mm)	Normalized shear stress (kN/m²)
0.00	0.09	0.00	0.06	0.00	0.05
0.03	0.26	0.01	0.12	0.03	0.16
0.13	0.41	0.03	0.20	0.11	0.22
0.27	0.51	0.07	0.26	0.20	0.28
0.41	0.58	0.13	0.30	0.31	0.32
0.55	0.61	0.19	0.34	0.41	0.35
0.74	0.66	0.24	0.38	0.63	0.39
0.91	0.68	0.29	0.42	0.76	0.40
1.09	0.68	0.35	0.45	1.00	0.43
1.26	0.68	0.41	0.48	1.24	0.44
1.42	0.68	0.47	0.50	1.74	0.46
2.40	0.67	0.82	0.57	2.27	0.47
		1.18	0.59	2.86	0.48
		1.55	0.60	3.49	0.48
		1.94	0.61	4.15	0.48
		2.64	0.61	4.54	0.48
		3.42	0.61	4.76	0.48
		4.34	0.61	5.40	0.48
		5.14	0.61	6.50	0.48

Table 5.22– Normalized Direct Shear Test Results – Test C Unsaturated

Direct Shear Test Result					
Direct Shear- Normal stress 54.5 kN/m²		**Direct Shear- Normal stress 81.75** kN/m²		**Direct Shear -Normal stress 109** kN/m²	
Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)
0.00	0.09	0.00	0.06	0.00	0.05
0.02	0.16	0.08	0.15	0.00	0.05
0.07	0.27	0.18	0.22	0.01	0.09
0.13	0.38	0.29	0.34	0.03	0.14
0.20	0.46	0.41	0.34	0.08	0.16
0.29	0.54	0.50	0.39	0.16	0.16
0.35	0.60	0.61	0.43	0.18	0.17
0.43	0.66	0.72	0.48	0.18	0.19
0.50	0.73	0.83	0.52	0.20	0.21
0.58	0.79	0.94	0.56	0.22	0.23
0.66	0.83	1.02	0.60	0.24	0.25
1.07	1.02	1.61	0.72	0.36	0.36
1.55	1.08	2.20	0.80	0.52	0.51
2.06	1.07	2.78	0.84	0.70	0.64
2.29	1.05	3.10	0.85	0.86	0.72
		3.42	0.86	1.09	0.75
		4.02	0.87	1.30	0.82
		4.62	0.87	1.76	0.87
		5.23	0.87	2.26	0.90
				2.52	0.90
				2.75	0.91
				3.30	0.91
				3.48	0.91

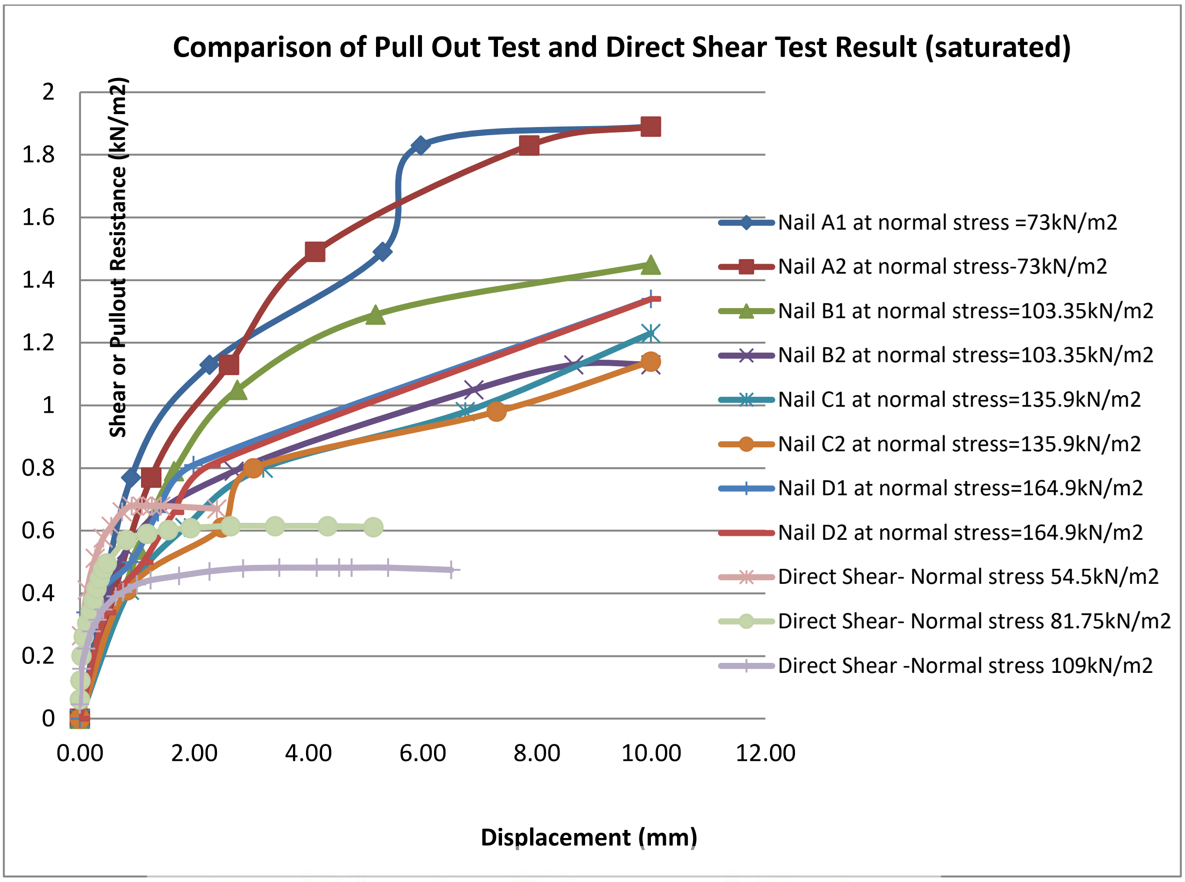

Figure 5.10 Comparison of normalized stresses of pullout and direct shear test result -Test C -Saturated

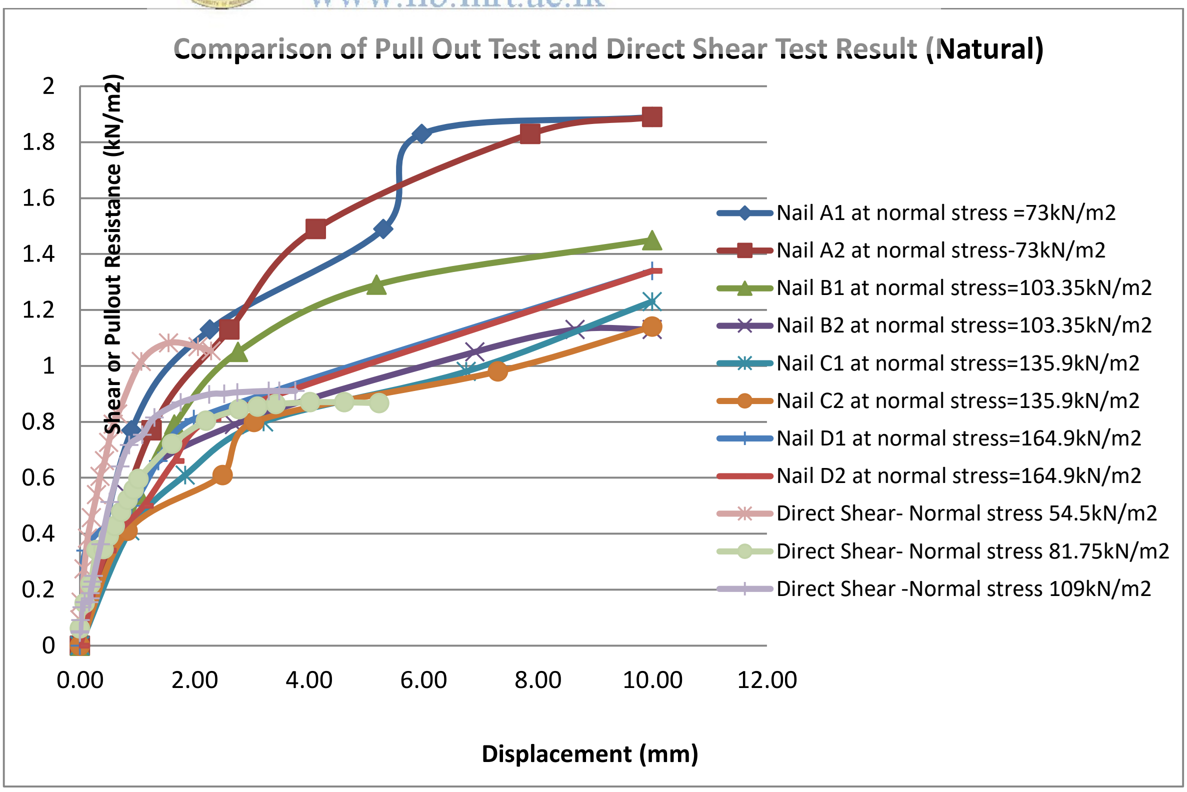

Figure 5.11 Comparison of normalized stresses of pullout and direct shear test result - Test C - unsaturated

Even after the normalizing all the pullout test results plotted above the direct shear test results. All the pullout test results are showing a strain hardening effect. In contrast, the direct shear test results show a flatter shape after reaching the peak value. This strain hardening effect indicates that at the pullout of a nail it may not be subjected to a simple interface failure. There may be other effects such as dilation that contributes to the pullout resistance.

5.6.3 Comparison of pullout test and direct shear test results of sample test D

The test results presented in above Table 5.13a and Table 5.13b were compared with Test D (saturated) test results presented in Table 5.23 and graphically presented in Figure 5.12. Thereafter the pullout test results were compared with Test D (unsaturated) test results presented in Table 5.24. The results are compared graphically in Figure 5.22.

Table 5.23– Direct Shear Test Results – Test D Saturated

Direct Shear Test Result					
Direct Shear- Normal stress 27.25 kN/m²		Direct Shear- Normal stress 54.5 kN/m²		Direct Shear -Normal stress 81.75 kN/m²	
Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)
0.00	5.03	0.00	5.03	0.00	5.03
0.04	7.50	0.02	9.47	0.04	9.97
0.12	9.47	0.10	16.92	0.11	24.92
0.22	12.45	0.20	21.91	0.34	45.15
0.32	13.93	0.31	24.42	0.65	65.70
0.44	15.43	0.41	26.93	0.86	83.41
0.56	17.42	0.53	28.94	1.14	97.60
0.68	18.91	0.64	30.95	1.48	107.10
0.78	19.91	0.74	32.97	1.84	110.80
0.90	21.91	0.92	39.56	2.21	111.90
1.00	23.92	1.08	47.20	2.60	109.78
1.12	26.42	1.27	54.88		
1.21	27.93	1.51	57.97		
1.35	30.95	1.87	71.93		
1.45	32.97	2.12	74.01		
1.67	37.78	2.69	70.89		
1.75	40.06				
1.87	43.62				
1.98	46.17				
2.08	48.73				
2.18	50.78				
2.28	52.83				
2.40	54.37				
2.51	55.39				
2.64	56.94				
2.76	57.45				
2.87	57.97				
2.98	57.97				
3.10	58.22				
3.21	58.22				
3.45	57.45				

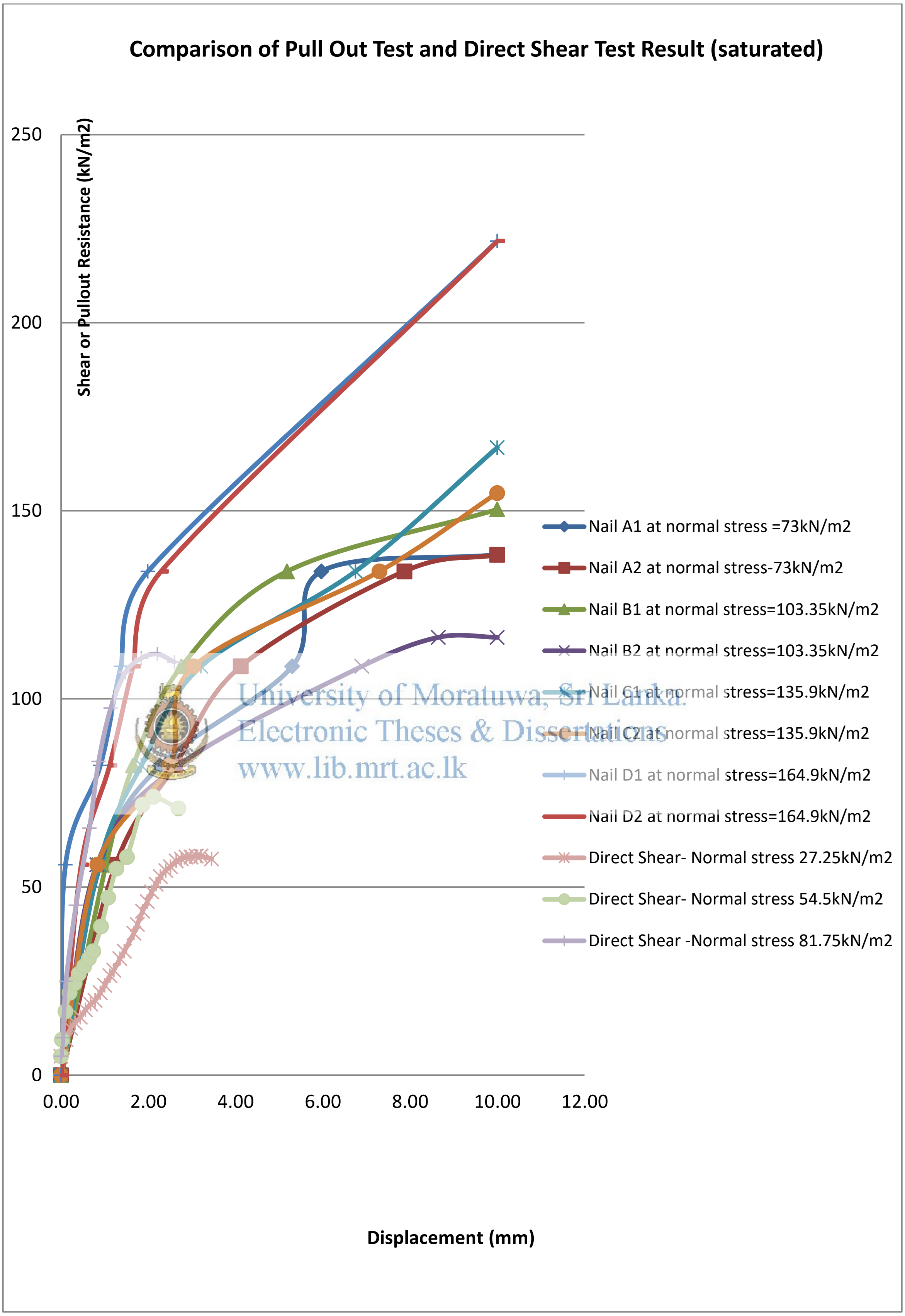

Figure 5.12 Comparison of pullout test and direct shear test result (Test D - saturated)

Table 5.24– Direct Shear Test Results – Test D Unsaturated (natural condition)

Direct Shear Test Result					
Direct Shear- Normal stress 27.25 kN/m²		**Direct Shear- Normal stress 54.5** kN/m²		**Direct Shear -Normal stress 81.75** kN/m²	
Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)
0.00	5.03	0.00	5.03	0.00	5.03
0.01	7.00	0.05	5.03	0.01	5.03
0.04	10.46	0.08	7.49	0.05	6.51
0.12	14.93	0.10	14.93	0.09	8.98
0.21	18.41	0.15	19.90	0.19	14.93
0.33	21.41	0.25	22.40	0.27	18.41
0.42	24.92	0.50	27.43	0.39	20.91
0.52	27.93	0.70	31.96	0.50	24.91
0.62	29.95	0.90	34.49	0.56	27.43
0.73	33.98	1.10	37.50	0.70	29.90
0.82	36.51	1.36	41.08	0.80	32.46
0.91	39.55	1.65	44.10	0.85	34.99
1.02	43.62	2.28	51.30	0.98	37.02
1.13	46.17	3.76	54.37	1.10	40.06
1.23	49.24	4.70	60.02	1.23	42.09
1.47	54.36	5.30	59.76	1.35	44.64
1.90	57.96	5.50	62.35	1.45	46.17
2.00	57.96	5.56	62.35	1.53	47.70
2.14	56.42	5.90	62.60	1.63	50.26
		6.90	61.50	1.75	51.80
		7.29	61.05	1.85	53.34
				1.98	54.88
				2.09	56.42
				2.19	57.96
				2.43	60.54
				2.65	62.60
				2.84	64.67
				3.10	67.26
				3.29	68.80
				3.55	70.89
				4.50	74.01
				5.20	77.10
				5.74	77.10
				6.15	77.10
				6.65	77.10
				6.88	76.88

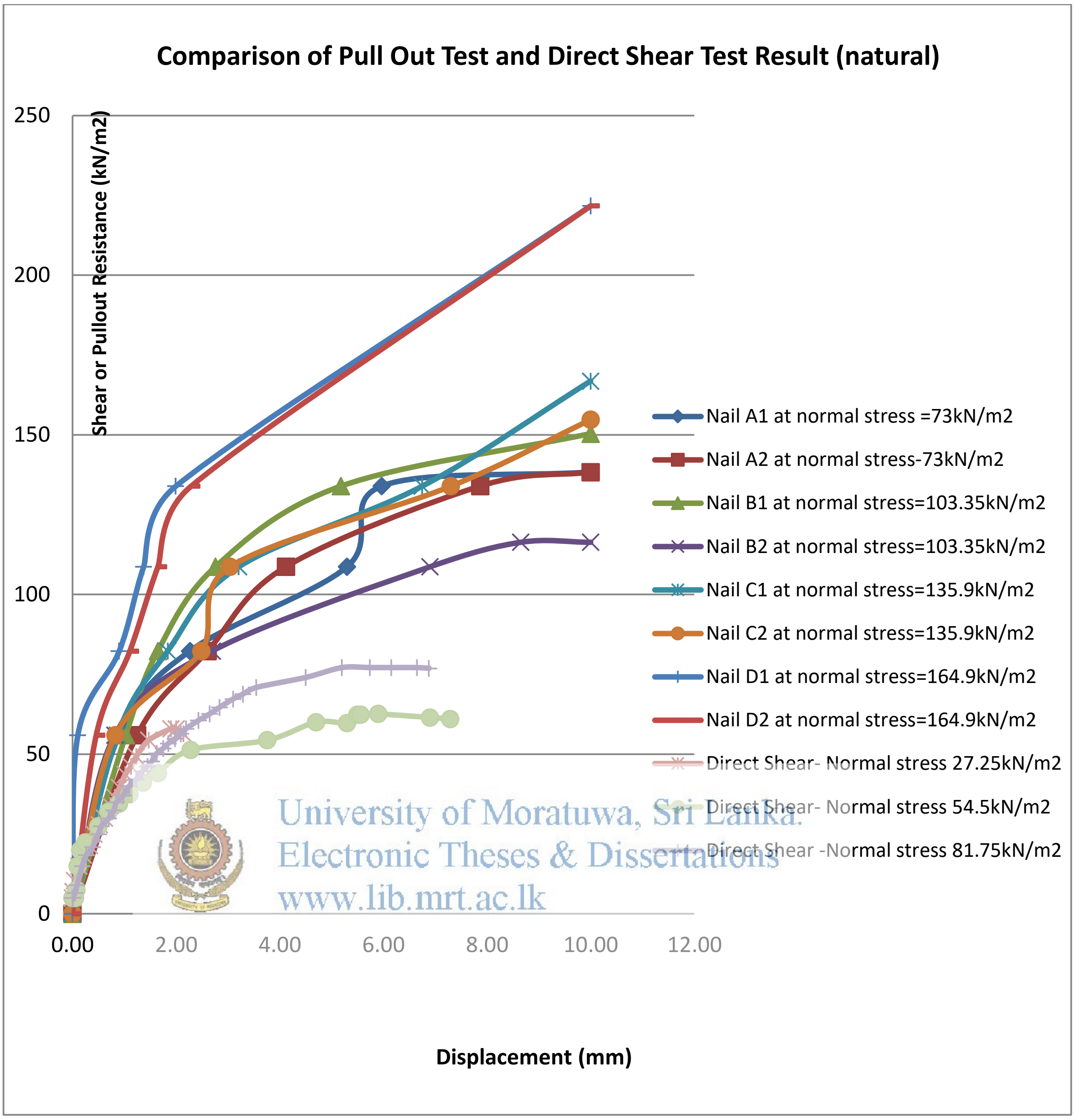

Figure 5.13 Comparison of pullout test and direct shear test result (Test D - unsaturated)

The shear stress-displacement graphs presented in the figures are drawn for different normal stresses or different overburden stresses. However, even under comparable normal stresses and overburden stresses, the pullout resistance values generally plotted above the direct shear test values.

In order to obtain a direct comparison of two types of plots, both the direct shear test results and the pullout resistance test results were normalized by the dividing by the applied normal stress or dividing by the overburden stress. The values obtained are presented in Table 5.25, Table 5.26, and above Table 5.18a and Table 5.18b.

The values are compared graphically in Figure 5.14 and Figure 5.15. In the normalized plots the direct shear test results plotted above the pullout test results. The sample at location D had a very high friction angle.

Table 5.25– Normalized Direct Shear Test Results – Test D Saturated

Direct Shear Test Result					
Direct Shear- Normal stress 27.25 kN/m²		**Direct Shear- Normal stress 54.5** kN/m²		**Direct Shear -Normal stress 81.75** kN/m²	
Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)	Displacemen t (mm)	shear stress (kN/m²)
0.00	0.18	0.00	0.09	0.00	0.06
0.04	0.28	0.02	0.17	0.04	0.12
0.12	0.35	0.10	0.31	0.11	0.30
0.22	0.46	0.20	0.40	0.34	0.55
0.32	0.51	0.31	0.45	0.65	0.80
0.44	0.57	0.41	0.49	0.86	1.02
0.56	0.64	0.53	0.53	1.14	1.19
0.68	0.69	0.64	0.57	1.48	1.31
0.78	0.73	0.74	0.60	1.84	1.36
0.90	0.80	0.92	0.73	2.21	1.37
1.00	0.88	1.08	0.87	2.60	1.34
1.12	0.97	1.27	1.01		
1.21	1.02	1.51	1.06		
1.35	1.14	1.87	1.32		
1.45	1.21	2.12	1.36		
1.67	1.39	2.69	1.30		
1.75	1.47				
1.87	1.60				
1.98	1.69				
2.08	1.79				
2.18	1.86				
2.28	1.94				
2.40	2.00				
2.51	2.03				
2.64	2.09				
2.76	2.11				
2.87	2.13				
2.98	2.13				
3.10	2.14				
3.21	2.14				
3.45	2.11				

Table 5.26– Normalized Direct Shear Test Results – Test D Unsaturated

Direct Shear Test Result					
Direct Shear- Normal stress 27.25 kN/m²		**Direct Shear- Normal stress 54.5** kN/m²		**Direct Shear -Normal stress 81.75** kN/m²	
Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)	Displacement (mm)	shear stress (kN/m²)
0.00	0.18	0.00	0.09	0.00	0.06
0.01	0.26	0.05	0.09	0.01	0.06
0.04	0.38	0.08	0.14	0.05	0.08
0.12	0.55	0.10	0.27	0.09	0.11
0.21	0.68	0.15	0.37	0.19	0.18
0.33	0.79	0.25	0.41	0.27	0.23
0.42	0.91	0.50	0.50	0.39	0.26
0.52	1.02	0.70	0.59	0.50	0.30
0.62	1.10	0.90	0.63	0.56	0.34
0.73	1.25	1.10	0.69	0.70	0.37
0.82	1.34	1.36	0.75	0.80	0.40
0.91	1.45	1.65	0.81	0.85	0.43
1.02	1.60	2.28	0.94	0.98	0.45
1.13	1.69	3.76	1.00	1.10	0.49
1.23	1.81	4.70	1.10	1.23	0.51
1.47	1.99	5.30	1.10	1.35	0.55
1.90	2.13	5.50	1.14	1.45	0.56
2.00	2.13	5.56	1.14	1.53	0.58
2.14	2.07	5.90	1.15	1.63	0.61
		6.90	1.13	1.75	0.63
		7.29	1.12	1.85	0.65
				1.98	0.67
				2.09	0.69
				2.19	0.71
				2.43	0.74
				2.65	0.77
				2.84	0.79
				3.10	0.82
				3.29	0.84
				3.55	0.87
				4.50	0.91
				5.20	0.94
				5.74	0.94
				6.15	0.94
				6.65	0.94
				6.88	0.94

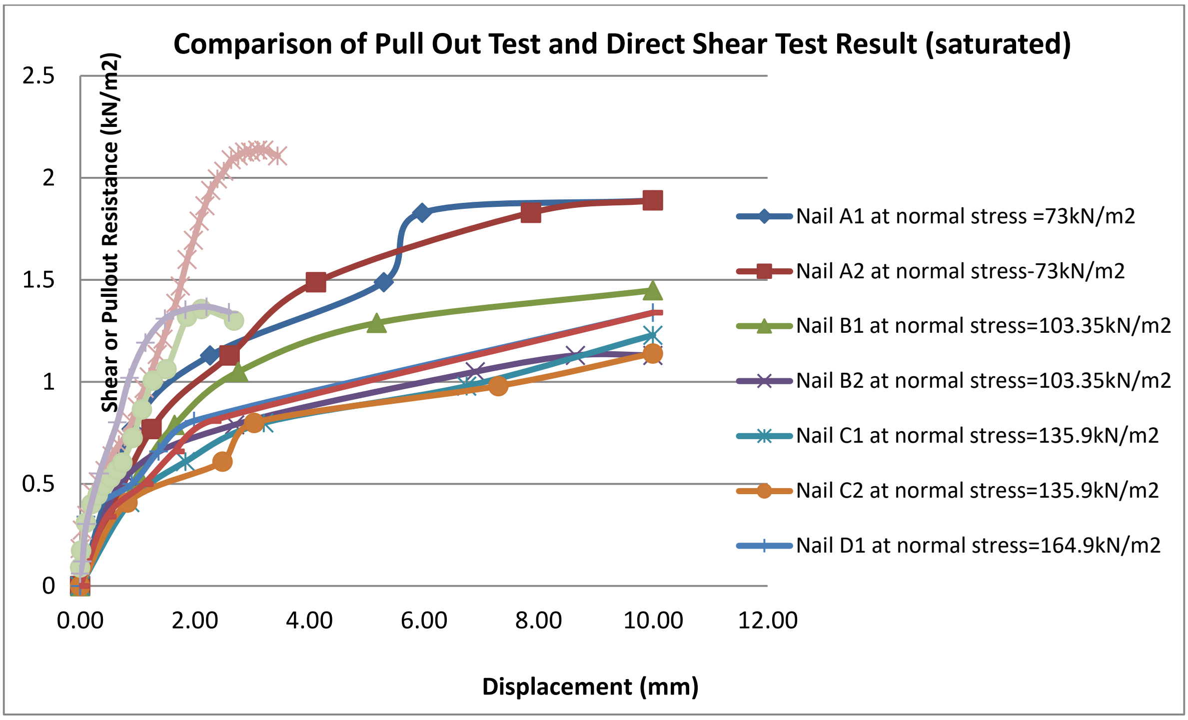

Figure 5.14 Comparison of normalized stresses of pullout and direct shear test result Test D -saturated

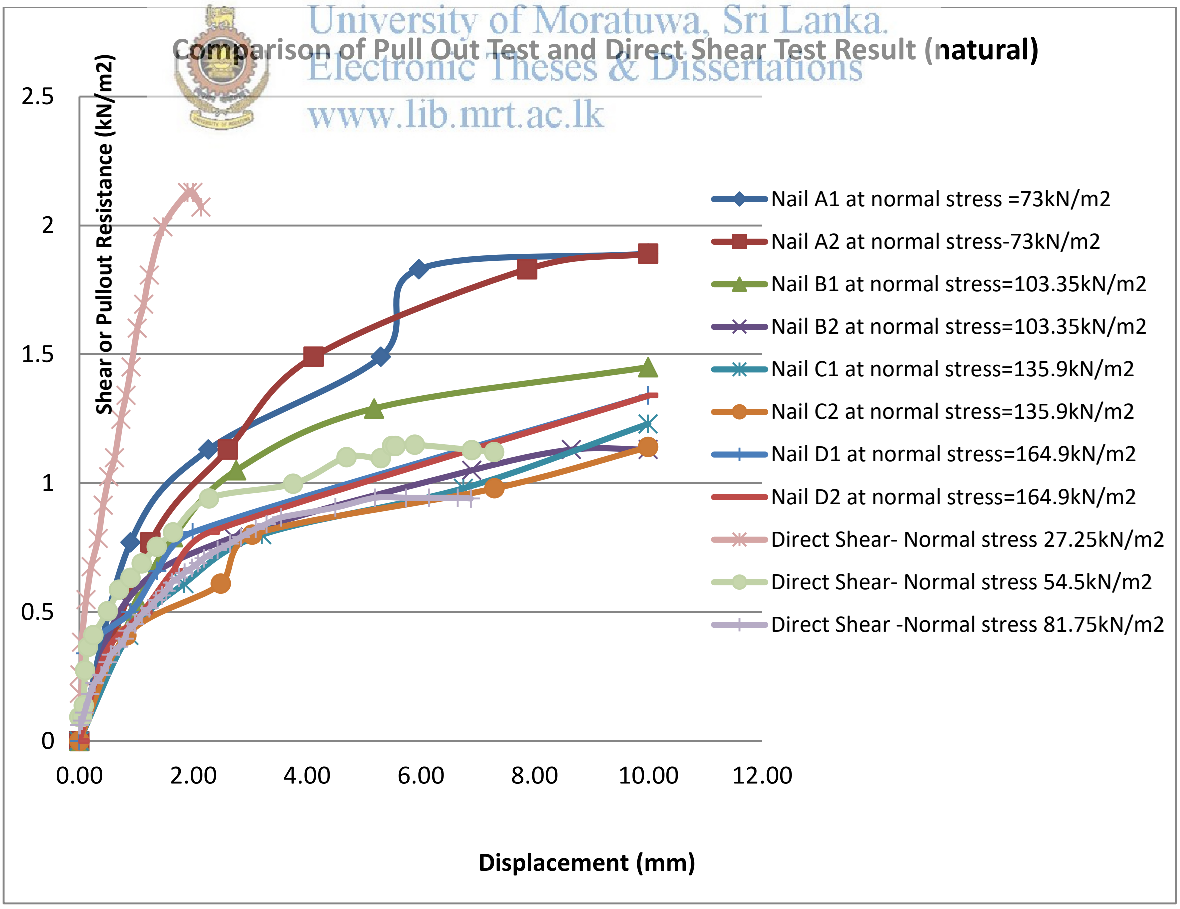

Figure 5.15 Comparison of normalized stresses of pullout and direct shear test result Test D unsaturated

5.7 Concluding comments on the comparison of stress-strain curves of direct shear tests and pullout tests

In both the direct shear test and the pullout test shear resistance is mobilized along a failure surface as the induced deformation increases. As such, the stress strain curves of the two tests are comparable.

The test results were initially compared directly and thereafter compared after normalizing with the normal stress or overburden stress. Pullout tests were compared separately with the direct shear tests done with undisturbed samples obtained from locations B, C and D.

When the comparisons were done with direct shear test results of samples B and C all the normalized pullout test were above the direct shear tests. The stress-strain curves of pullout test showed a strain hardening type effect whereas the direct shear test results showed a reduction in stress after reaching the peak value.

The undisturbed sample D was more granular with a high angle on internal friction. When the normalized pullout test results were compared with the normalized direct shear test results of sample D, the direct shear test results were plotted above the pullout test results.

The undisturbed samples B, C and D were obtained at the face of the nailed slope. The test nails were installed over the last 2.5 m length of the hole drilled for a 5m length.

The change in the soil properties further into the slope over this 5m perhaps to a less weathered condition could cause an increase in the pullout resistance from the computed values. Therefore, to make an accurate prediction of the pullout resistance it is necessary to test samples obtained further inside the slope closer to the resistant zones in the nails.

6 Summary, Conclusion & Recommendations for Further Research

6.1 Summary and Conclusions

The surface of the selected embankment was divided in to four levels, namely A, B, C, and D. Eight soil nails (A1, A2, B1, B2, C1, C2, D1 and D2) were installed in to the soil surface by ensuring each level had two nailed locations.

Separate undisturbed soil samples were obtained from levels A, B, C and D with the use of a 300mm X 300mm X 300mm custom made sample box, in order to find important soil parameters to calculate the theoretical pullout capacity of soil nails. The sample A was disturbed and could not be used. Shear strength parameters, both under saturated and unsaturated conditions were obtained from other sample B, C and D.

Pullout resistance values were obtained experimentally and compared with the predictions made with five methods that are currently in use. The relevant shear strength parameters for the estimation of pullout resistance at each test nail location were obtained from the closest undisturbed sample.

Variation of pull out capacity with depth

It is evident that the measured pull out capacity that it increases with the depth. Nevertheless, some researchers have commented that pullout resistance is independent with the depth.

All equations used for the estimation of pull out capacity suggest that it is directly proportional to the overburden pressure (which is a dependent variable of depth).

Effective diameter of soil nails

The diameters of the cross sections of the grouted areas deduced by measuring their respective circumferences after complete physical pullout indicate that the diameters of the failed sections are greater than nominal diameter of the drilled hole shaft. The average increase is about 22.6%. This could be due to anomalies in drilling or penetration of grout into the soil under the grouting pressure. Nevertheless very high grouting pressures are not normally used in soil nailing. This is an area that needs further studies.

Comparison of the estimated and measured pullout resistance

Five different methods were used to estimate the pullout resistance. Method 1 to method 4 has shear strength parameters and overburden stress as main parameters in addition to the nail dimensions. These four methods do not specify whether the shear strength parameters correspond to saturated or unsaturated conditions. In the field all the test nails were under unsaturated conditions.

When the pullout resistance was estimated with the saturated shear strength parameters the predictions were much lower than the observations. Even when the pullout resistance was estimated with the unsaturated parameters, the predictions were lower than the observations.

The fifth method by Gurpersaud (2010) accounts for matric suction and dilation. When the predictions were done accounting for both matric suction (using an assumed suction profile) and dilation the predictions were much closer although still lower than the experimental observations.

For the predictions at respective test nail locations shear strength parameters obtained from the closest undisturbed sample were used. Sample D had a greater sand content and incidentally a higher angle of internal friction. When comparisons were done for the deeper nails (nails D 1 and D 2) using shear strength parameters of sample D, the values were much closer.

Comparison of stress- strain curves of pull out and the direct shear tests

In the comparison of stress-stain curves for the direct shear tests and pullout tests after normalizing, the pullout test results were lying above the direct shear test results of undisturbed samples B and C. But when compared with the test results of sample D (which is highly frictional) direct shear test results were above the pullout test results.

The stress –strain curves of pullout tests showed a strain hardening effect. The direct shear test results showed a lowered or constant resistance after reaching the peak.

Concluding comment on the comparisons

All the comparisons indicate that the matric suction and dilation have a contribution to the pullout resistance.

The variability of the soil should also be accounted. The test samples used for the laboratory tests were at the face of the slope and much less weathered (or more frictional) soils could be encountered further into the slope where pullout resistances of the nails are mobilized.

As such, it is necessary to get sufficient samples to get a good assessment of special variability of the shear strength. However, there are practical limitations on this.

6. 2 Recommendations for further research

The influence of matric suction on the pullout resistance should be established. In a identified test slope, pullout resistance measurements should be under highly unsaturated conditions as well as under induced saturated conditions. (may be with an artificial rainfall or sprinkling of water). These tests should be done along with the matric suction measurements in the resistant zone closer to the nail locations.

The variability of the soil conditions over the length of the nail should be well established by taking sufficient number of samples. This could be done initially as a large laboratory model to be followed by a instrumented field study.

With such studies, the contribution of matric suction to the pullout resistance and the reduction of the pullout resistance with the loss of matric suction could be established. After studying the rainfall pattern in a given location a design rainfall (say 100 year) could be established and incorporating that into an infiltration analysis a design matric suction profile could be derived. This matric suction profile could be used in the estimation of pullout resistance. The formula proposed by Gurpersaud (2010) could be further developed with these data.

REFERENCES

1. American Petroleum Institute (API). 1990. 13B-1Recommended standard procedure for field testing water-based drilling fluids, API, Dallas.Canadian foundation engineering manual (CFEM). 2006.
2. Burland, J. B. _2002_. *Reliability of soil nailed slopes in Hong Kong*, Geotechnical Engineering Office, Hong Kong Government, Hong Kong.
3. Byrne, R. J., Cotton, D., Porterfield, J., Wolschlag, C., and Ueblacker, G._1998_. "Manual for design and construction monitoring of soil nail walls." *Rep. No. FHWA-SA-96-069*.
4. Canadian Foundation Engineering Manual - CFEM (2006) .
5. Cartier, G., and Gigan, J. P. _1983_. "Experiments and observations on soil nailing structures." *Proc., European Conf. on Soil Mechanics and Foundation Engineering*, 473–476.
6. Chai, X.J. and Hayashi, S. and Du, Y.J. 2004.Contribution of dilatancy to pullout capacity of nails in sandy clay. In Soil Nailing and Stability of Soil and Rock Engineering, Nanjing, China, pp. 73-80.
7. Chu, L.-M., and Yin, J.-H. _2005_. "Comparison of interface shear strength of soil nails measured by both direct shear box tests and pullout tests." *J. Geotech. Geoenviron. Eng.*, 131_9_, 1097–1107.
8. FHWA. 1993. Recommendations clouterre 1991 (English translation). Report on the French national research project clouterre, FHWA-SA-93-026. Federal Highway Administration, Washington, DC.
9. FHWA (2003). Geotechnical Engineering Circular No. 7 - Soil Nail Walls. Federal Highway Administration, US Department of Transportation, Washington, D.C., USA, Report No. FHWA0-IF-03-017, 210 p.
10. Franzen, G. _1998_. "Soil nailing a laboratory and field study of pull-out capacity." Ph.D. thesis, Chalmers Univ. of Technology, Göteborg, Sweden.
11. Franzen, G., and Jendeby, L. _2001_. "Prediction of pullout capacity of soil nails." *Proc., 15th Int. Conf. on Soil Mechanics and Geotechnical Engineering*, Vol. 3, Balkema, Rotterdam, The Netherlands, 1743–1748.
12. Franzen, G. 1998. Soil nailing - a laboratory and field study of pull-out capacity. PhD dissertation,Department of Geotechnical Engineering, ChalmersUniversity of Technology, Goteborg, Sweden.
13. Fredlund, D. G., and Rahardjo, H. _1993_. *Soil mechanics for unsaturated soils*, Wiley, New York.
14. Fredlund, D.G., Xing, A., Fredlund, M.D. and Barbour, S.L.1996. Relationship of the unsaturated soil shear strength to the soil-water characteristic curve. Canadian Geotechnical Journal 33(3): 440-448.

15. French National Project CLOUTERRE. _1991_. “Recommendations CLOUTERRE 1991: Soil nailing recommendations for designing, calculating, constructing and inspecting earth support systems using soil nailing.” *Rep. No. FHWA-SA-93-026*, Washington, D.C. _English translation_.
16. Geotechnical Engineering Office _GEO_. _2007_. “Good practice in design of steel soil nails for soil cut slopes.” *GEO technical guidance note No. 23*, Hong Kong
17. Geotechnical Engineering Circular No 7 – Soil Nail Walls-FHWA-2003
18. Golder (1993). “GOLDNAIL Soil Nailing Design Program,” Golder Associates, Seattle, Washington
19. HA68/94. 1994. Design methods for the reinforcement of highway slopes by reinforced soil and soil nailing technique. Road Authorities in England, Welsh, Scotland and Northern Ireland.
20. M.R.HAUSMANN. (1992). “ Soil and rock anchorage, rock bolting, soil nailing & dowelling “.
21. Heymann, G., Rhode, A. W., Schwartz, K., and Friedlaender, E. _1992_.“Soil nail pullout resistance in residual soils.” *Proc., Int. Symp. On Earth Reinforcement Practice*, Vol. 1, Balkema, Rotterdam, The Netherlands, Kyushu, Japan, 487–492.
22. Jewell, R.A. and Wroth, C.P. 1987. Direct shear tests on reinforced sand. Géotechnique, 37:53-68
23. Jewell, R.A. 1990. General Report – Soil Nailing. Proceedings of the International Reinforced Soil Conference, Glasgow, 197-202.
24. Junaideen, S.M., Tham, L.G., Law, K.T., Lee, C.F. and Yue, Z.Q. 2004. Laboratory study of soil-nail interaction in loose completely decomposed granite, Canadian Geotechnical Journal, 41(2): 274-286.
25. Juran, I., Baudrand, G., Farrag, K., and Elias, V. (1990). “Kinematical limit analysis for design of nailed structures,” Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 116, No. 1, pp. 54-72.
26. Lazarte, C. A., Baecher, G. B., and Withiam, J. L. _2003_. “New directions in LRFD for soil nailing design and specifications.” *Proc., LSD 2003: Int. Workshop on Limit State Design in Geotechnical Engineering Practice*, K. K. Phoon, Y. Honjo, and R. B. Gilbert, eds., World Scientific.
27. Li, J., Tham, L. G., Junaideen, S. M., Yue, Z. Q., and Lee, C. F. _2007_.“Loose fill slope stabilization with soil nails: Full-scale test.” *J. Geotech.Geoenviron. Eng.*, 134_3_, 277–288.
28. Li, K. S., and Lo, S. R. _2007_. “Discussion of comparison of interface shear strength of soil nails measured by both direct shear box tests and pullout tests by Lok-Man Chu and Jian-hua Yin.” *J. Geotech. Geoenviron.Eng.*, 133_3_, 344–346.
29. Luo, S. Q., Tan, S. A., and Yong, K. Y. _2000_. “Pull-out mechanism of a soil nail reinforcement in dilative soils.” *Soils Found.*, 40_1_, 47–56.

30. Long, J.H., Chow, E., Cording, E.T., and Sieczkowski, W.J. (1990). “Stability Analysis for Soil Nailed Walls,” Geotechnical Special Publication No. 25, American Society of Civil Engineers, pp. 676-691.
31. Mecsi, J. 1997. The load bearing capacity and the load elongation diagram of soil anchors. Proceedings of the European Conference on Soil Mechanics and Foundation Engineering, Hamburg, 1327-1330.
32. Milligan, G. W. E., and Tei, K. _1997_. “The pullout resistance of model soil nails.” *Soils Found.*, 38_2_, 179–190.
33. Pradhan, B. 2003. Study of the pull-out behaviour of soil nails in completely decomposed granite fill. M.Pill thesis, The University of Hong Kong.
34. Pradhan, B., Tham, L. G., Yue, Z. Q., Junaideen, S. M., and Lee, C. F._2006_. “Soil-nail pullout interaction in loose fill materials.” *Int. J.Geomech.*, 6_4_, 238–247.
35. Plumelle, C. Schlosser, F., Oclage, P., and Knochenmus, G. (1990). “French National Research Project on Soil Nailing: CLOUTERRE,” Geotechnical Special Publication No. 25, American Society of Civil Engineers, pp. 660-675.
36. Potyondy, J.G. 1961. Skin friction between various soils and construction materials. Géotechnique, 11(4): 339.
37. Powell, G.E. and Watkins, A.T. 1990. Improvement of marginally stable existing slopes by soil nailing in Hong Kong. In Proceedings of the International 16. Reinforced Soil Conference, Glasgow, U.K., pp. 241-247.
38. Prof.S.A.S. Kulathilaka and D.C.A. Mettananda, 1998,Use of Soil Nailing Technique for Stabilization of Slopes.
39. Rabcewicz, L.V. (1964b). “The New Austrian Tunneling Method,” Part 2, Water Power, London, Vol. 16, December, pp 511-515.
40. Rabcewicz, L.V. (1965). “The New Austrian Tunneling Method,” Part 3, Water Power, London, Vol. 17, January, pp 19-24.
41. Schlosser, F. and Guilloux, A. 1982. “Le frottement dens les sols. Revue Francaise de.” Géotechnique. 16:65-77(1981)
42. Schlosser, F. (1983). “Analogies et différences dans le comportement et le calcul des ouvrages de soutènement en Terre Armée et par clouage des sols,” (Trad.: Similarities and differences in the behavior and design of retaining structures of reinforced earth and soil nailing). Annales de L’Institut Technique de Batiment et des Travaux Publics, No. 418, Series: Sols et Fondations, Paris, France. Schlosser.
43. Sheahan, T.C., T. Oral, and C.L. Ho (2002). “A Simplified Trial Wedge Method for Soil Nailed Wall Analysis,” Journal of Geotechnical and Geoenvironmental Engineering, American Society of Civil Engineers, Vol. 128, No.:TBD. Shen.
44. Shen, C.K., Herrmann, L.R., Romstand, K.M., Bang, S., Kim, Y.S., and Denatale, J.S. (1981a). “In situ Earth Reinforcement Lateral Support System,” Report No. 81-03, Department of Civil Engineering, University of California, Davis, California.

45. Sivakumar,. and Singh, V.P. 2010. Soil nails field pullout testing: evaluation and applications. International Journal of Geotechnical Engineering, 4: 13-21.
46. /Stocker, M.F., Korber, G.W., Gässler, G., and Gudehus, G. (1979). "Soil Nailing," in International Conference on Soil Reinforcement I, Paris, France, Vol. 2, pp. 469–474.
47. Su, L.J., Chan, T.C.F., Shiu, Y.K., and Cheung, T. and Yin, J.H. 2008. Influence of degree of saturation on soil nail pullout resistance in compacted completely decomposed granite fill. Canadian Geotechnical Journal, 44(11): 1314-1328.
48. Tei, K., Taylor, R. N., and Milligan, G. W. E. _1998_. "Centrifuge model tests of nailed soil slopes." Soils Found., 38_2_, 165–177.
49. The influence of matric suction on the pull-out capacity of Grouted soil nails Naresh Gurpersaud B.Sc., University of Guyana (2010)
50. Vanapalli, S. K., Fredlund, D. G., Pufahl, D. E., and Clifton, A. W._1996_. "Model for the prediction of shear strength with respect to soil suction." Can. Geotech. J., 33_3_, 379–392.
51. Vanapalli, S.K. and Fredlund, D.G. 2000. Comparison of empirical procedures to predict the shear strength of unsaturated soils uses the soil-water characteristic curve. Geo-Denver 2000, American Society of Civil Engineers, Special Publication, 99:195-209.
52. Vanapalli, S.K. and Catana, M.C. 2005. Estimation of the soil-water characteristic curve of coarse grained soils using one point measurement and simple properties. In International Symposium on Advanced Experimental Unsaturated Soil Mechanics, Trento, pp.401-407.
53. Vanapalli, S.K., Eigenbrod, K.D., Catana, C., Taylan, Oh, W.T. and Garven, E. 2010. A technique for estimating the shaft resistance of test piles in unsaturated soils. 5th International Conference on Unsaturated Soils, Barcelona, Spain. September 6-8, 2010. Paper accepted for publication.
54. Vanapalli, S.K. 2009. Shear strength of unsaturated soils and its applications in geotechnical engineering practice. Keynote Address. In Proceedings of the 4 Asia-Pacific Conference on Unsaturated Soils. New Castle, Australia. Nov. 23-25. Edited by O. Buzzi, S. Fityus and D. Sheng. 2010 Taylor & Francis Group, London , ISBN 978-0-415-80480-6. pp. 579-598.
55. Yeo, K. C., Lo, S. R., and Yin, J. H. _2007_. "Installation method and overburden pressure on soil nail pullout test." New Horizons in Earth Reinforcement. Proc., 5th Int.Symp. on Earth Reinforcement, J. Otani, Y. Miyata, and T. Mukunoki, eds., Taylor and Francis, Kyushu, Japan,321–327.
56. Yin, J.H. and Su, L.J., 2006. An innovative laboratory box for testing nail pull-out resistance in soil, ASTM Geotechnical Testing Journal, 29: 1 - 11
57. Zhang, L.L., Zhang, L.M., and Tang, W.H. 2009. Uncertainties of field pullout resistance of soil nails, Journal of Geotechnical and Geo-environmental Engineering, 135(7): 966(7)-973.