INFLUENCE OF MATRIC SUCTION ON PULLOUT RESISTANCE OF SOIL NAILS

Paranthaman Rajeevkaran

178048G

Degree of Master of Science

Department of Civil Engineering

University of Moratuwa
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ABSTRACT

Soil nailing is a widely used slope stabilisation technique using the passive intrusions-soil nails. Soil nail is a reinforcement bar encased within a grouted borehole without any pre-tensioning. When the soil mass attempts to move down, a tensile forces mobilized in the intercepted nails will enhance the shear resistance by increasing the normal stress along the potential failure surface while reducing the shear stress to be mobilized for equilibrium. This enhances the factor of safety.

The tensile force developed on reinforced bar depends on tensile capacity of the bar and pull-out resistance at the soil-grout interface. The contribution of matric suction in pull-out capacity of soil nails is often neglected in conventional design formulae to make it conservative. But most of the soil nails are installed in unsaturated region of soil and significant matric suctions would prevail.

This paper investigates the influence of matric suction on pull-out resistance of soil nails. To ensure uniform conditions the nails were installed in a soil mass compacted under controlled laboratory condition in a test box with dimensions of 1.30 m × 1.08 m × 0.90 m. In this study, four soil nails were installed at an inclination of 5^0 to the horizontal. The pull-out capacity of soil nails measured under different matric suctions and overburden pressures were compared with the values estimated using the design formulae. The matric suctions varied by controlled wetting were monitored by tensiometers. A good agreement is found between the estimated and measured pull-out capacities and the influence of matric suction on the pull-out resistance was found to be very significant. The numerical analysis performed with Plaxis 2D justified the results obtained.

Keywords: Pull-out resistance, tensiometer, soil nail, matric suction, unsaturated soil, Plaxis 2D

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Abbreviations	Description
FSR	Force Sensitive Resistor
FHWA	Federal Highway Administration
CDG	Completely Decomposed Granite
GFRP	Glass Fibre Reinforced Polymer
FEM	Finite Element Model
VP	Vertical Pressure
SWCC	Soil Water Characteristic Curve
MEMs	Micro Electro Mechanical Systems
ASTM	American Society for Testing and Materials
DL	Design Load
GWT	Ground Water Table
USCS	Unified Soil Classification System

CHAPTER 1 INTRODUCTION

1.1 Background

In the soil nailing technique, reinforcement bars are encased in grouted drill holes oriented in a downward direction. When the soil mass attempts to move downward a tensile force is mobilized in the reinforcements intercepted by the potential failure surface. This will increase the normal stress mobilized along the failure surface and reduce the shear resistance necessary to be mobilized for equilibrium and thus enhance the safety margin. The tensile force that can be mobilized depend on the tensile strength of the reinforcement and pullout resistance of the nail.

The soil nailing technique has been found to be suitable for supporting excavations, tunnel portals, slope stabilization, bridge abutments and many other civil engineering applications. Soil nails have been utilized increasingly in recent years due to its technical and economic advantages. Due to the advantages of simplicity, speed of construction, and economical efficiency, soil nailing has been increasingly used and become one of the most common slope stabilization method.

Correct estimation of pullout capacity is critically important in the design of soil nails as over estimation of capacity leads to design failures while under estimation lead to increase in construction cost. A practice adopted widely is to estimate the pullout capacity using an existing formula and to verify it by field pullout tests during the early stages of construction.

Most slopes in Sri Lanka are formed by residual soils and rocks at different degrees of weathering. Colluvial soils are also encountered. The ground water table is quite low during periods of dry weather and high matric suctions will prevail at the upper levels of slope. These matric suctions would be reduced or completely lost due to uncontrolled infiltration during prolonged rainfall if attention is not paid to surface drainage.

Slope failures in Sri Lanka are triggered by such excessive rainfalls and improvement of surface and subsurface drainage is the primary step to guarantee the stability. When slopes are to be cut to steep gradients strengthening by reinforcement would also be necessary to ensure stability. Soil nailing is the most widely used reinforcing technique used in such instances. With the implemented drainage measures it would be possible to maintain significant matric suctions in slopes even under the conditions of prolong rainfall. Hence the study of the effect of matric suction on pull-out resistance is significant considering the practical importance.

1.2 Objective

Through a well instrumented experimental study, establish the influence of matric suction on the pullout resistance of soil nails and determine the suitability of existing design formulae.

The detailed objectives of the study are as follows.

1. Investigation of influence of matric suction on pullout resistance of soil nails for compacted residual soil under well controlled laboratory conditions.
2. Verification of the suitability of the proposed theoretical approaches published in literatures to estimate the pull-out capacity of soil and hence select the most appropriate approach.
3. Identification of the behavior of nail (grouted body) /soil interface at failure through a direct shear test under saturated conditions.
4. Experimentation on the shear strength parameters at saturated state for soil-soil interface and soil-grout interface.
5. Comparison of experimental results with results obtained from the model developed in finite element software for the same experiment.

1.3 Scope of the Research

i. A comprehensive experimental program was undertaken under laboratory conditions to find the pullout capacity of soil nails in the compacted residual soil. The test nails were installed at an angle of 5° downward to the horizontal in the soil surface.

ii. The soil samples compacted at maximum dry density are to be used to find the shear strength parameters under laboratory condition with fully saturated level.

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 at different matric suction.

iv. The results obtained from the laboratory experimental program are to be used to model the present study numerically based on shear strength parameters at different matric suction.

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, which is a detailed review of soil nail pull-out capacity, interface behavior and the mechanics of unsaturated soils are summarized. General background of soil nailing technique, 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 researches performed with numerical estimation of soil nail pullout capacity are also summarized.

The third chapter explains the procedures followed in design of pullout testing apparatus, calibration of sensors and equipment design.

The fourth chapter presents the laboratory testing procedures along with the results obtained and the pullout testing procedures where a detailed evaluation of the results is provided.

The fifth chapter discusses the finite element modelling of the pullout study along with the results.

Chapter six discusses the results obtained from the pullout test results .The measured pullout resistance are compared with the estimated values from the different methods of estimation currently in use.

Chapter seven presents the conclusion and the recommendation for the future research.

CHAPTER 2 LITERATURE REVIEW

2.1 Development of Soil Nailing Technique

The soil nailing technique started its development in the early 1960s, as the techniques used for rock bolting and multi-anchorage systems and reinforced fill (Clouterre, 1991; FHWA, 1993). The New Austrian Tunneling Method, for the first time used steel bars and shotcrete to stabilize the ground. With use of different techniques, semi-empirical designs for soil nailing was introduced in the early 1970s. The first systematic research on soil nailing, which involved both model and field tests, was carried out in Germany in the mid-1970s. With these evolutions, development in the soil nailing techniques was initiated in France and the United States in the early 1990s.

From theoretical studies, experimental analyses and observations, several classical design methods were established by different researchers and those are German Method, French method, Davis and Modified Davis Methods, Federal Highway Administration (FHWA) Design Method and the UK Method. Many other countries or regions such as Hong Kong, Brazil, and Japan, and etc. also have contributions to the development of soil nailing technique (Ortigao et al. 1995; Barley et al. 1996). Those are summarized in Table 2.1.

Table 2.1 Design assumptions of soil nailing methodologies

Soil nailing design method	Failure mechanism	Resisting force	Failure surface	Ultimate pullout capacity
German Method (Bodenvernagelung)	Two-wedges failure	Tensile resistance	Bi-linear	Constant
French Method (Clouterre, FHWA)	One-wedge failure	Tensile and shear resistance	Circular	Constant
US Method (FHWA-SA-96-096R)	Two-wedges failure	Tensile resistance	Bi-linear, Circular	Constant
Modified Davis Method (FHWA-RD-89-193)	Two-wedges failure	Tensile resistance	Parabolic	Constant
UK Method	Two-wedges failure	Tensile resistance	Bi-linear	Increase with depth
Hong Kong Method	Slices failure	Tensile resistance	Any shape	Increase with depth

2.2 Soil Nailing Construction Procedure

The Figure 2.1 shows the general construction sequence adapted in the construction of a soil nail wall. This the general procedure followed in the construction site. The depth of excavation without having support will be decided based on the soil properties, angle of side slope etc. The dimensions of soil nails, angle of installation, and spacing of soil nails are based on the design calculation and numerical analysis.

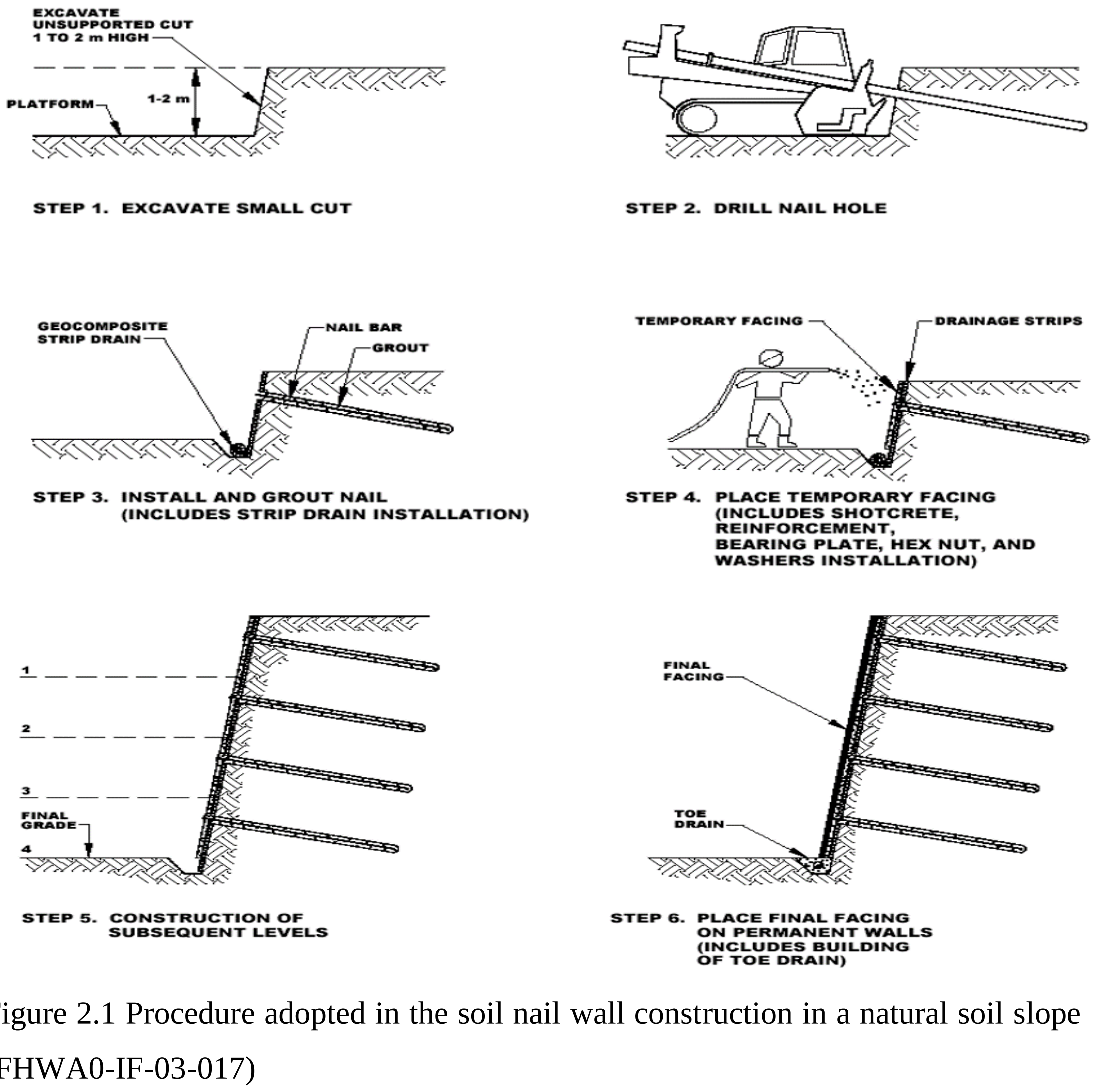


Figure 2.1 Procedure adopted in the soil nail wall construction in a natural soil slope (FHWA0-IF-03-017)

2.3 Pull-out Behavior of Soil Nails

In soil nail design, the failure that occurs at the soil grout interface is considered as the most critical. Most of the soil nail designs are based on the previous experience with the same soil conditions using appropriate analytical or empirical approaches and the designed pullout capacity is verified in the construction stage by pullout test of test nails.

Proper design of soil nailing requires an understanding of the interaction mechanism between the nails and soil being reinforced. Bond resistance is developed by relative displacement of nail and soil and may be investigated using pullout tests. Palmeria and Milligan (1989) stated that soil nail interaction is influenced by several factors such as properties of the soil, the roughness and stiffness of the nail and the boundary conditions of the test. Furthermore, the change in stress on the nail due to dilatancy of the soil, when nail is pulled out, makes interaction mechanism very complicated and difficult to analyse (Schlosser and Guilloux, 1979).

Many research studies have been performed in field and laboratory to investigate the pullout behaviour of soil nails. These tests were fully instrumented and involved full scale models, modified direct shear box tests or pull-out tests (Hossain et al, 2012; Gurpersaud et al, 2010; Yin et al, 2009). Based on the field pullout test results, design charts were proposed to estimate the pullout capacity of gravity grouted and driven nails in different type of soils (FHWA, 1993). Last five decades, different researchers have verified that conditions under which the pullout testing were performed influence the pullout behaviour of soil nails.

2.4 Factors Affecting the Pullout Resistance of Soil Nails

Many researchers have pointed out that the soil nail pullout resistance is influenced by soil dilatancy, matric suction, overburden pressure, soil type, method of installation, grouting pressure etc. The matric suction and soil dilatancy were taken into consideration in recent research as those are the reasons behind the uncertainties in the soil nail pullout resistance. The brief key findings of the research studies done on the factors that affect the pullout resistance are summarized below.

2.4.1 Effect of dilatancy

Extensive studies have proved that the interface behaviour between soil and different construction materials are influenced by many factors. The behaviour of partially saturated soil is different from saturated or fully dried soil due to matric suction. This matric suction significantly influences the dilatancy of soil, thus soil dilation affects the apparent friction angle and shear strength of soil (Hossain and Yin 2010).

Further, Hossain et al (2014) experimented the dilatancy effects in cement soil interface in a direct shear tests and observed no dilative behaviour at fully saturated state. While dilation effect was greater under high matric suction with lower net normal stress and lower under lower matric suction and higher net normal stress. His experiment concluded that dilative behaviour of unsaturated compacted Completely Decomposed Granite (CDG) soil-cement grout interface is significantly influenced by matric suction and net normal stress. Moreover, the dilatancy of unsaturated soil-structure interfaces should be considered to determine the interface strength, which is important for safety and design.

The soil particles around the nail will dilate when the shear stress is applied on the soil nail interface during pull-out (Pradhan, 2003). This increases the normal stress applied on the soil nail. Su et al. (2007) conducted a numerical simulation of the effects of dilatancy on soil nail pull-out resistance. 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 2.2 shows the relationship of the average pull-out stress with (a) pull-out displacement and (b) dilation angle **(ψ)**.The results clearly show that the pull-out resistance initially increases quickly with the dilation angle up to a dilation angle of 10^0 and remains constant thereafter.

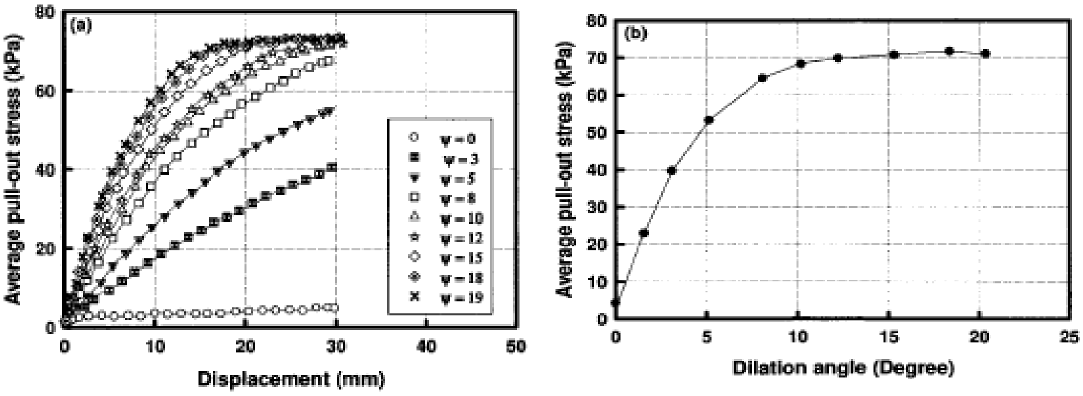


Figure 2.2 Relationship of the average pull-out stress with (a) pull-out displacement and (b) dilation angle (Su et al., 2007)

Yin et al. (2009) conducted a study on the influence of grouting pressure on a CDG fill and concluded that the increase in pressures in the earth pressure cells are attributable to the constrained dilation of the soil at or near the interface between the soil nail and soil. Further, Yin et al. (2009) pointed out the factors that contributed to the constrained dilation as soil suction, the interface roughness and soil density. Ng and Zhou (2005) observed that the suction can cause an increase in dilatancy. These studies help to understand the influence of dilatancy on peak pullout and the different factors that cause soil dilatancy.

2.4.2 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 lower. 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).

2.4.3 Effect of soil type

Pullout resistance of soil nail depends on the properties of soil and nail surface characteristics (bond surface). The bond strength along grout and soil interface contribute to the shear strength of nail. Bond strength is rarely measured in laboratory and there is no standard for tests that evaluate the bond strength (Su, 2006; Yin and Su, 2006). But the pullout tests that are conducted in field provide an approximate estimation of the bond strength. It is obvious that increase in angle of internal friction of soil will affect the mobilized friction along the bond area and increases the normal stress during the pullout. Similarly factors like relative density, coefficient of uniformity, texture of soil surface affect the shear resistance of soil.

2.4.4 Influence of overburden pressure

There were differences in the findings of different researchers on the influence of overburden pressure. Franzen and Jendeby (2001) found that pullout resistance was independent of the embedded depth of the soil nails. Schlosser (1983) found that normal pressure is close to the overburden pressure for driven nails, while for grouted nails, the pressure can be very low, due to the stress release during hole drilling. A study on the influence of the hole, drilling process and overburden pressure on the soil nail pullout resistance in a compacted completely decomposed granite (CDG) fill was conducted by Su (2006) and Su et al. (2008). Su et al. (2008) found that the hole-drilling process caused significant soil stress reduction around the drill hole and the nail pullout resistance was hardly dependent on the overburden pressures.

A study on the influence of overburden pressure by Zhou et al. (2010) illustrated that the pullout resistance increases with overburden pressure. The Figure 2.3 shows finite element analysis and the corresponding test results of pullout resistance under different overburden pressures. Both measured and stimulated pullout resistance did not vary much with the overburden pressure. The peak pullout shear stress was scattered and not directly related to the applied overburden pressure. The decrease or increase in pull-out capacity was accompanied by decrease or increase in the vertical stress increment during pull-out.

A series of field pullout testing carried out by Ranjan Kumara and Kulathilaka (2016) proved that the measured pull out capacity increases with the depth and further stated that 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). Figure 2.4 shows the variation of measured pullout capacity with depth.

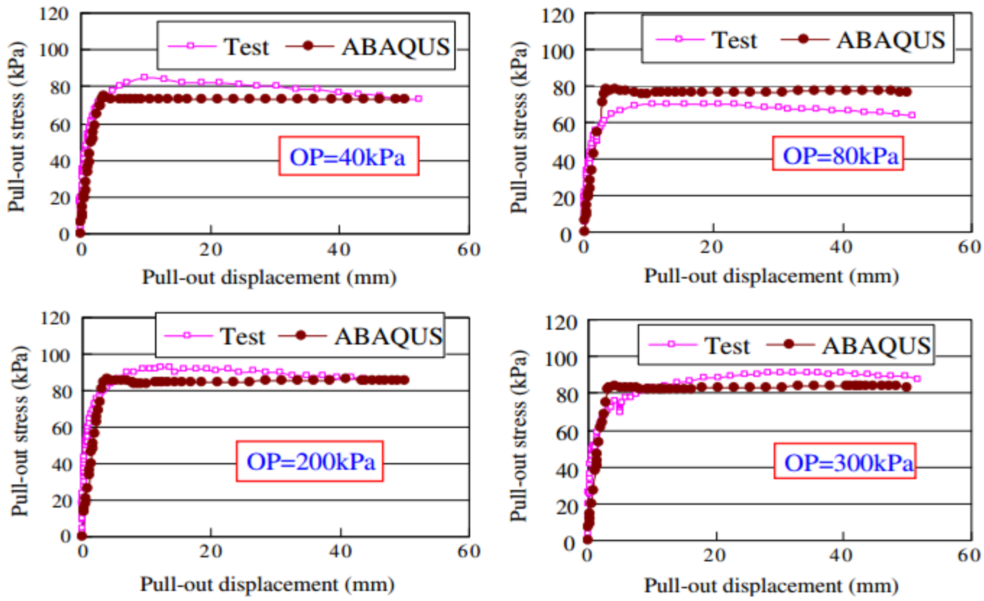


Figure 2.3 Comparison between the measured and stimulated relationships of average pullout shear stress and pullout displacement under different overburden pressures (Zhou et al., 2010)

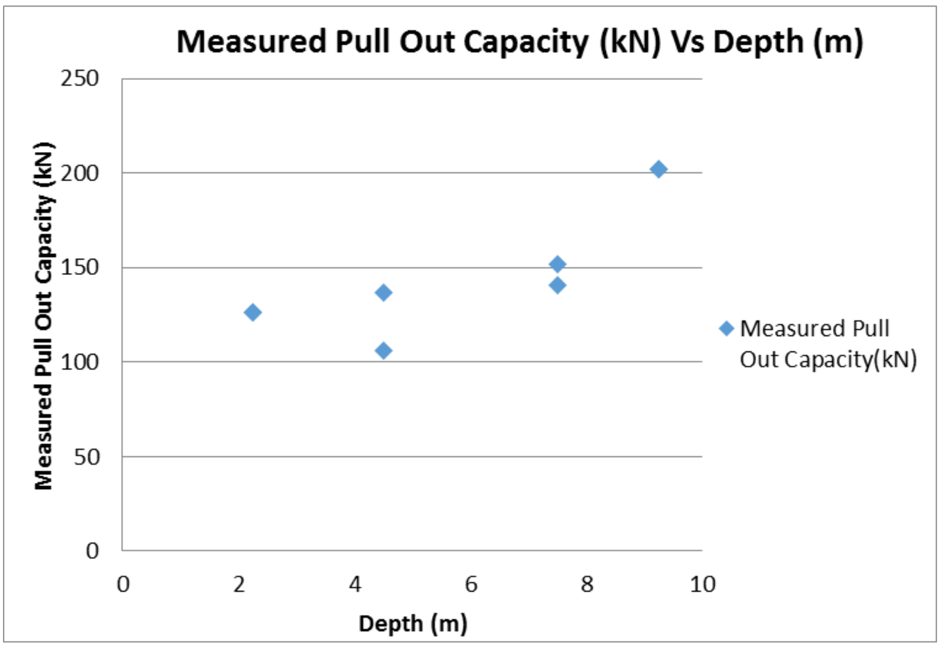


Figure 2.4 The variation of measured pullout capacity with depth (Ranjan Kumara and Kulathilaka, 2016)

2.4.5 Effect of grouting pressure

The interface behaviour of the soil-cement grout in both saturated and unsaturated states is influenced by the grouting pressure. The pressure grouted soil nails provide better interface strength than gravity grouted soil-nails. The use of grouting pressure is an effective way to increase the interface strength. But the common practice followed in soil nail construction is to adopt gravity or low pressure grouting. Studies on grouting pressure are very significant in the common application such as anchors, piles and ground improvement, but limited to soil nails. Yeung et al. (2005) carried out field pullout tests on glass fibre reinforced polymer (GFRP) pipes in CDG soil slope in Hong Kong and observed a significant increase in the pullout resistance with grouting pressure. The laboratory tests done by Yin et al. (2009) and Yin and Zhou (2009) showed that the soil nail pullout resistance increases with grouting pressure.

Yin et al. (2009) observed that the pullout peak shear stress of the soil nail was less dependant on the overburden pressure when the grouting pressure was low, but the peak shear stress increased with the increasing overburden pressure when the grouting pressure was high. Yin et al. (2009) explained this observation as when the grouting pressure was low, the disturbed soil around the drill hole was still loose and soil arching effect existed to a certain extent and when the grouting pressure was high the loosened soil was well compacted by the grout and as a result the strength of the soil increased back to or even higher than that before drilling. Thus this led to the change in the stress condition in the soil around the hole and the soil arching effect no longer existed. The Figure 2.5 shows the variation of shear stress with grouting pressure.

Akhtar H. and Jian H.Y (2012) determined the influence of the grouting pressure in the behaviour of soil grout interface in a direct shear apparatus in a compacted decomposed gravel and stated that the influence of pressure grouting is significant in the interface shear strength. The Figure 2.6 shows how the interface shear strength increased with the grouting pressure. The numerical analysis performed by Zhou et al. (2011) on the effect of grouting pressure concluded that the grouting pressure influence the peak pullout force of soil nails.

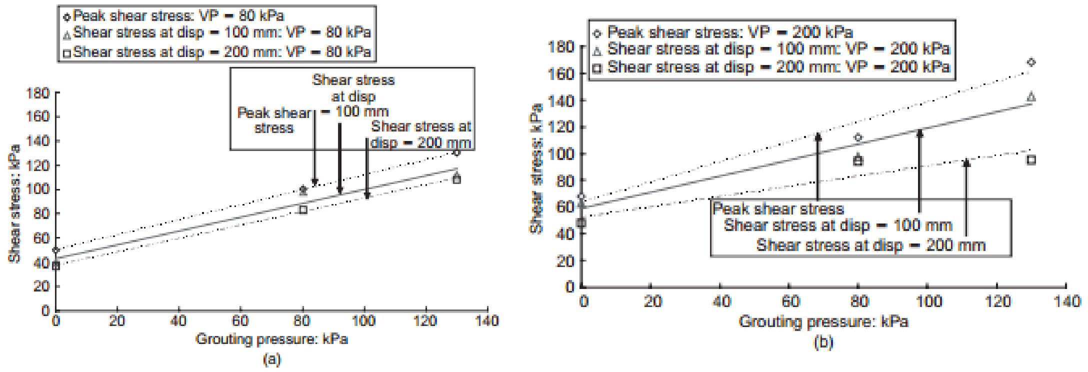


Figure 2.5 (a) Peak shear stress, shear stress at displacement of 100 mm, and shear stress at displacement of 200 mm plotted against grouting pressure with the same VP =80 kPa and S_r =50% and (b) peak shear stress, at displacement of 100 mm, and shear stress at displacement of 200 mm plotted against grouting pressure with the same VP=200 kPa and S_r =50% (Yin et al., 2009).

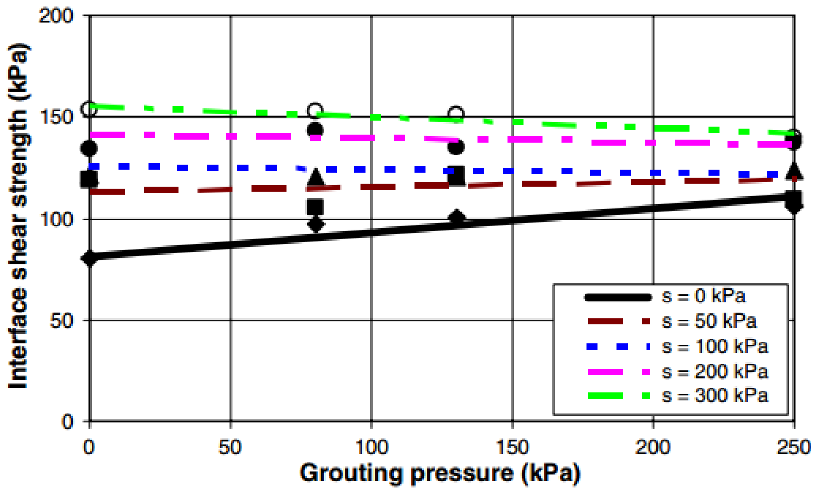


Figure 2.6 Interface shear strength corresponding to different grouting pressures under different suctions (Akhtar H. and Jian H.Y, 2012)

2.4.6 Effect of matric suction

The changes in the matric suction is associated with the variation in the degree of saturation. This matric suction plays an important role in the interface strength of soil nails. Thus, affecting the pullout capacity of the soil nails. Unlike in saturated soils which need only σ-u_w to interpret the mechanical behavior, net normal stress, (σ- u_a) and matric suction (u_a - u_w) are required for interpreting the engineering behavior of unsaturated soils (Fredlund & Rahardjo 1993). However, the influence of suction is generally ignored in conventional geotechnical engineering practice extending the conservative assumption that the soils are saturated.

Soil nails are classically positioned in a region where the soil is in unsaturated condition state. The soil nails are influenced by the matric suction in the soil mass. The influence of matric suction on the pullout capacity didn't receive much attention in the design of soil nails (Su et al., 2008 and Zhang et al., 2009). Matric suction is the major parameter that contributes to the uncertainties in the estimation of the pullout capacity of soil nails (Zhang et al., 2009).

Classical saturated soil mechanics theories are used in the design of geotechnical applications, including soil nails by neglecting the contribution of matric suction or in other words, the negative pore-water pressures in the vadose zone (i.e., the zone above the ground water table) to the capacity. The main reason for this is the absence of a simple technique for the analysis and design of geotechnical structures (Fredlund and Rahardjo, 1993; Vanapalli and Oh, 2010). Figure 2.7 shows relationships between shear strength and matric suction.

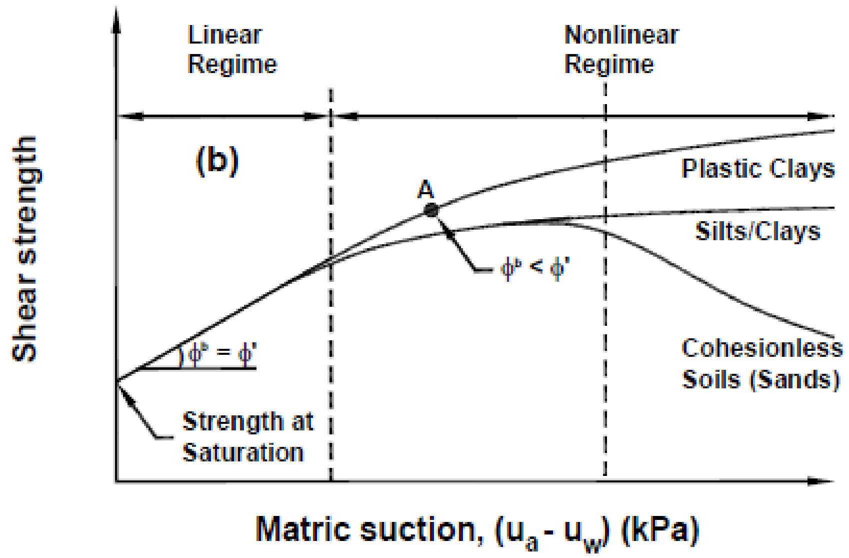


Figure 2.7 Relationships between shear strength with matric suction (Vanapalli, 2009)

$(u_a - u_w)$ - Matric suction in unsaturated soil,

Where u_a - pore air pressure

u_w - pore water pressure

Soil has cohesion component which are highly influenced by variation of degree of saturation. Pullout capacity of clayey gravel reduced to 50% when the water content was increased from optimum water content to full saturation (Schlosser et al., 1983). The test results obtained by Su et al. (2008) showed that the peak pullout strength of soil nails was strongly influenced by the degree of saturation of soil. Hence the influence of matric suction should be properly addressed in the design of the soil nailing system.

Soil water characteristic curve (SWCC) shows the relationship between matric suction with the degree of saturation. The variation of soil, water, air phase change and its stresses when the soil moves from saturated to unsaturated can be studied through SWCC curve (Barbour, 1999). Figure 2.8 shows the relationship between degree of saturation and suction (SWCC curve). At fully saturated state the matric suction was zero and it can be observed with a small decrease in level of saturation the matric suction increase was very significant.

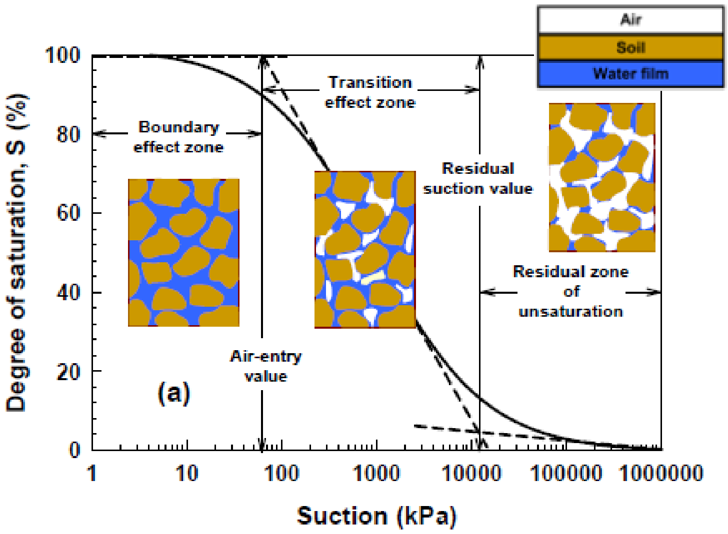


Figure 2.8 Relationships between degrees of saturation with matric suction (Vanapalli, 2009)

The saturation in the soil depends on the dry density of soil, moisture content, specific gravity of soil, void ratio, etc. The variation in level of saturation in the sites is induced by wetting and drying of soil mainly by rainfall infiltration and evaporation. A typical matric suction profile is present in Figure 2.9. With rain water infiltration the high matric suction at the upper levels will be dissipated. The infiltration of rainfall on unsaturated slopes depends on the saturated coefficient of permeability, permeability function and the water storage capacity of the soil (Zhang et al., 2004).

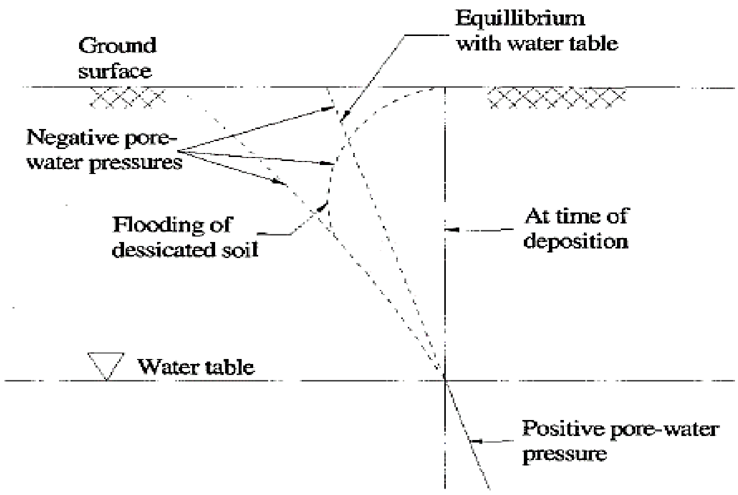


Figure 2.9 Typical pore-water pressure profile (Modified after Fredlund & Rahardjo, 1993)

The influence of matric suction on the shaft capacity of jacked piles in coarse-grained soils was investigated by Vanapalli et al. (2010). His experimental outcome illustrated that the matric suction contribution towards shaft capacity. It contributed to 35-40 % of the total shaft capacity of silty sand. Further they established a relationship between the Soil Water Characteristic Curve (SWCC) and shaft capacity of piles in unsaturated soils. This approach was used by Gurpersaud to estimate the influence of matric suction on pullout capacity of soil nails.

Gurpersaud (2010) conducted experiments to understand the influence of matric suction on the pullout capacity of compacted sand under both unsaturated and saturated conditions. A strong relationship was obtained between pullout resistance and soil water characteristic curve. There is a linear increase in pullout value up to the air entry value and nonlinear increase thereafter. Moreover, he matched the pullout behaviour to the different stages of the SWCC where a steady increase in strength occurred in the boundary effect zone (Primary) and transition zones (secondary), thereafter declined in the residual zone. This behaviour of the pullout capacity of soil nails with matric suction resembles the shear strength of unsaturated soil during the different phases. The Figure 2.10 shows variation of pullout capacity with matric suction compared with SWCC curve.

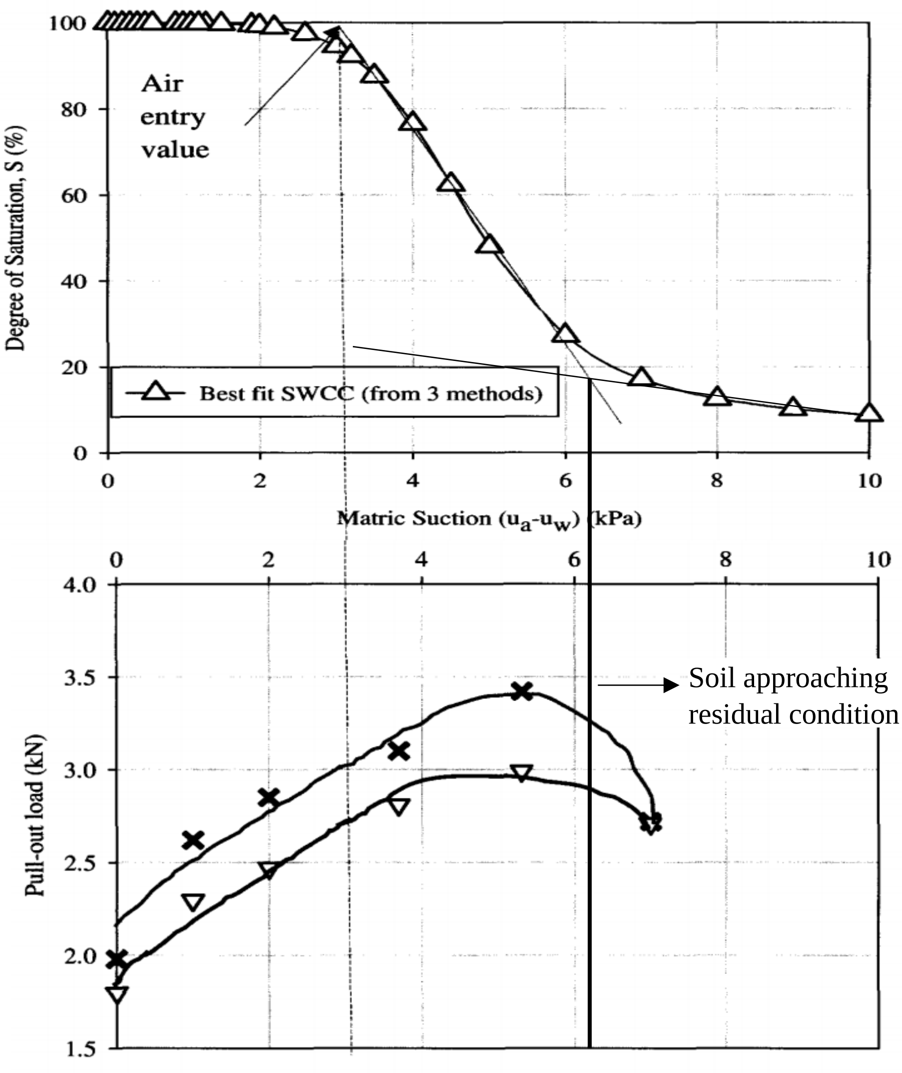


Figure 2.10 Variation of pullout capacity with matric suction compared with SWCC curve (Gurpersaud, 2010)

Gurpersaud (2010) observed that a gradual increase in the pullout capacity is evident from a low suction value up to 5.3 kPa followed by a decline at an average suction value of 7 kPa. The decline in capacity is due to the soil approaching residual conditions. Praden (2003) and Chu and Yin (2005) also observed a decrease in pullout capacity with degree of saturation and this observation is due to the decreasing matric suction from optimum moisture content to the saturated condition. Further Praden (2003) related that behaviour in the pullout capacity to the decrease in apparent soil cohesion. The decrease in the soil cohesion is also directly related to the reduction in matric suction. In addition to the above, the measured pullout capacity of the soil nails used by Gurpersaud (2010) found to be 1.3-1.7 times higher than the pullout capacity under saturated state. Further, Figure 2.6 in section 2.4.5 shows the work done by Akhtar H. and Jian H.Y (2012) on the influence of the grouting pressure in the behaviour of soil grout interface where the influence of matric suction can be observed clearly. The interface shear strength increased with the matric suction at a constant grouting pressure.

Jason and chang (2012) studied the effect of wetting on the soil nail pullout resistance, showed that the peak pullout resistance decreases upon wetting, with a reduction of about 60 % after soaking for 28 days. Initially the peak pullout capacity dropped about 15 % after 1 day soaking. Further they added that there is a threshold water content beyond which the effect of infiltration on the pullout resistance is reduced. From this relationships they concluded that the water content played a significant role in the mobilization of the pullout shear resistance and the effect of wetting led to the reduction in pullout resistance due to the loss of matric suction by the process of infiltration. The wetting (or saturation) decreased the constrained dilatancy of the soil-nail interface.

Ranjan Kumara and Kulathilaka (2016) reported results of pull-out tests on 8 short nails installed in a slope in Kegalle which was stabilized by soil nailing. The results showed that the methods currently used for designing of soil nails underestimate the pull-out capacity. By accounting for the unsaturated condition that prevail, possible dilation during pull-out a value much closer to the field value was estimated. Matric suctions were not measured but some estimates were done based on the known ground water table level. Shear strength parameters obtained from two undisturbed samples obtained close to the test nail locations were used to estimate the pullout resistance. The variability of the natural soil profile imposed considerable difficulties in the analysis. The strength parameters obtained from the direct shear test under saturated state and unsaturated state showed an increase in values under unsaturated state which was attributed to the influence of matric suction.

2.5 Estimation of Pullout Capacity

The two key factors that should be considered in the design of soil nails are the pullout capacity of the nails generated by interface friction and tensile capacity of the nails. The pullout capacity generated by the interface of soil nail depends on many factors like strength characteristics of soil, overburden pressure, grouting pressure, dilatancy of soil, matric suction etc. But tensile capacity depends on the material property of the reinforcement bar. Different formulae developed by researchers over the year are summarized in Table 2.2.

Table 2.2 The proposed formulae to estimate the pull-out capacity of soil nails

Reference	Equations
Schlosser and Guilloux (1981)	$T_L = Pc' + 2D_{eq}\sigma'_v \mu^*$
Jewell (1990)	$T_L = P\sigma'_N f_b \tan\phi'$
Heymann et al. (1992)	$T_L = P\left(c' + \sigma'_N \tan\phi'\right)$
HA 68/94 (1994)	$T_L = \lambda\left(c' + \sigma'_N \tan\phi'\right)$
Chu and Yin (2005)	$T_L = Pc' + 2D\sigma'_v \tan\delta''$
Zhang et al. (2009)	$T_L = \pi D\left[c' + (u_a - u_w)\tan\phi_b\right] + \dfrac{2D\sigma'_v \tan\phi'}{1 - \left[\dfrac{2(1+\upsilon)}{(1-2\upsilon)(1+2k_o)}\right]\tan\phi' \tan\psi}$
Gurpersaud (2011)	$Q_{f(us)} = \left[\left(c_a + \beta\sigma'_z\right) + \left\{(u_a - u_w)\left(S^{K}\right)\tan(\delta + \psi)\right\}\right]\pi dL$

Where:
c' = soil cohesion
P = nail perimeter
D_{eq} = equivalent width of flat reinforcement
σ_v'= effective overburden stress at mid-depth of reinforcing element
μ* = coefficient of apparent friction
λ = pull-out factor
● '= angle of internal friction of soil
σ_N' = effective normal stress
f_b = coefficient of roughness
δ = soil/nail interface friction angle
C_a = soil/nail interface adhesion
f_c = coefficient defined by c_a/c'
f_s = coefficient defined by δ/●
υ = coefficient defined by δ/●
ψ = dilation angle
K_0 = coefficient of earth pressure at rest
● ^{b} = internal friction angle with respect to soil suction
δ '' = interface friction angle for the normal stress on a strip
(U_a- U_w) = matric suction

2.6 Numerical Modelling of Soil Nail Pullout Resistance

Simulation of interaction between the grouted body and soil is the key issue in modelling a soil-nailed problem. Liu (2003) and Akis (2009) studied this soil structure interaction in terms of pullout resistance or bond strength of the soil nails. Many soil nailed structure are being erected each year. But the understanding of soil nail pullout behaviour under different conditions remains uncertain. Numerical modelling have been successfully used in the stimulation of the soil nailing system. However, the stimulation of laboratory soil nail pullout tests is limited. The fully controlled instrumented laboratory pullout tests are effective and economical in the study of factors that influences the pullout resistance.

Su et al. (2010) carried out a study on a three dimensional finite element model to investigate the influence of overburden pressure and soil dilation on soil nail pullout resistance in a CDG soil by using Mohr-coulomb failure criterion. The variations of stress release surrounding the soil nail during the process of drilling, grouting, and pullout were simulated by the FE model and compared with the results available from the laboratory test. They concluded that simulated FE model results were in a good agreement with measured results from the laboratory pullout tests. In addition they pointed out that the 3D FE model can capture the main characteristics of the responses of soil pressure changes, pullout shear stresses, and displacements.

A similar study was conducted by Zhou et al. (2011) in soil nails installed in a compacted and saturated completely decomposed granite (CDG) pullout box under different overburden pressures and grouting pressures to simulate the pullout behaviour of the soil nail using modified DruckerePrager Cap model. In this study, Zhou et al. (2011) concluded that finite element method (FEM) can be used well to simulate the pullout behaviour. Zhao and You (2014) carried out a study on pullout and failure mechanisms of soil anchors in 3 D. Moreover, Rawat and Gupta (2016a, b) used two-dimensional (2D) FEM to study the failure mechanism and performance of soil nail in model slope.

Researchers have analysed using more rough surfaced soil nails or soil anchors to achieve a comprehensive soil- nail interaction (Frydman and Shaham, 1989; Lutenegger, 2009). Further detailed studies in this concern have been performed on the pullout capacity of screw piles by Kurian and Shah (2009) or helical screw piles by Rao et al. (1991). Many studies have been conducted to study the interface behaviour of soil nail by using helical plates, circular discs, helical piles (Rao et al., 1991, Lutenegger, 2009, Merifield, 2011, Gupta and Rawat., 2017).

It is a proof from Tan et al. (2008) that the pullout behaviours of soil nails can be well simulated by vertical pullout testing of soil nail in axisymmetric condition using the Plaxis 2D package. The inclination to horizontal is simulated by applying a horizontal load on the free boundary. This accounts for the overburden pressure when soil nail is pulled out horizontally.

CHAPTER 3 EQUIPMENT DESIGN

3.1 Test Box Used for Soil Nail Pullout Test

Ranjan Kumara and Kulathilaka (2016) reported that the variability of the natural soil profile imposed considerable difficulties in the assessment of pullout capacity in the field. Thus it was decided to develop a soil embankment in a test box. The box was made with 10 mm transparent perspex sheets on three sides and the front face with slotted opening to install four nails. The box is placed on a platform under a steel frame and a surcharge on the soil fill in the box can be imposed by jacking against the frame. In this study a uniform soil mass was obtained by compacting a soil at its optimum moisture content with a compaction effort equivalent to standard Procter. This eliminates the material variability and influence of matric suction could be isolated. Figure 3.1 shows the complete details of the test box used in the study of pull-out testing.

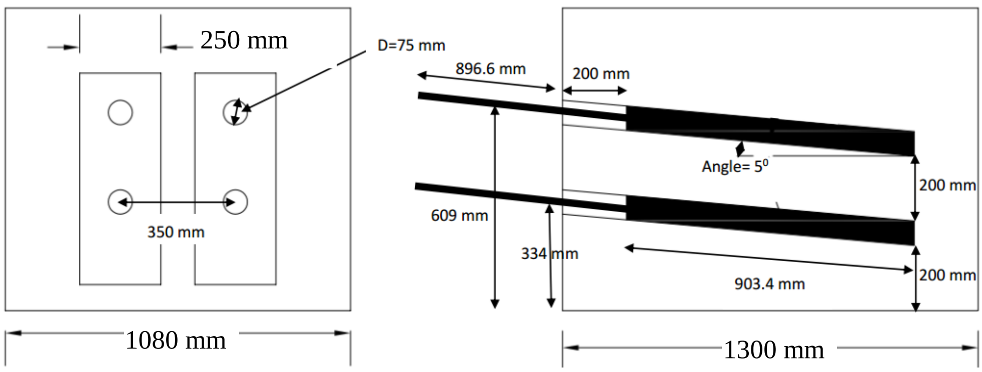


Figure 3.1 the complete test box arrangement with dimensions

Dimension of the Perspex box is 1.3 m x 1.08 m x 0.96 m. Figure 3.2 shows that a clear distance of 200 mm was kept in all sides to avoid the influence of boundary effects following the suggestion by Zhou et al. (2011). A flexible Polyethene sheet was placed on the vertical side of the Perspex box with lubricating oil between the sheet and perspex to minimize the friction between the perplex glass walls and the soil. The soil type was silt with high plasticity (MH). The soil was obtained from the residual soil formation within the university premises and was sieved through a 5 mm sieve. Soil was placed in the test box and compacted in layers of thickness 150 mm with manual compactor (with a 5.835kg modified hammer) providing an energy equivalent to that of standard Proctor test. The compaction process was consistent for all the layers in the test box. According to the in situ density measurements a relative compaction close to 100% had been achieved.

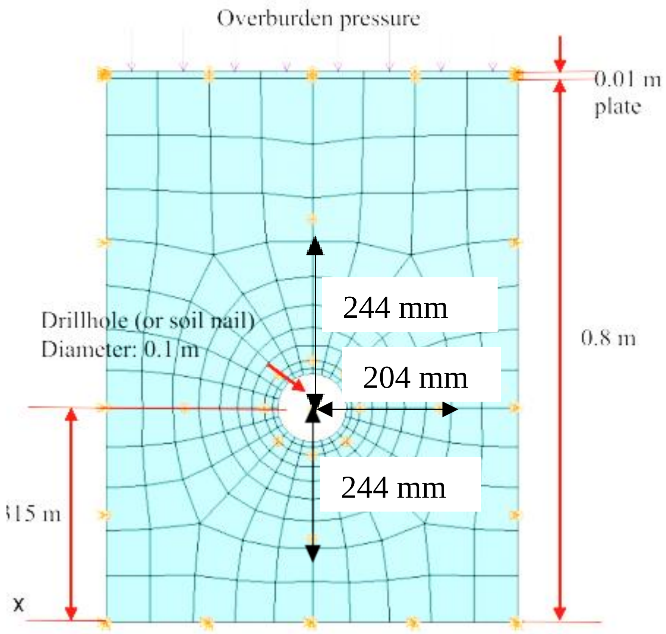


Figure 3.2 Influence zone on test box following the suggestion of Zhou et al. (2011)

A layer of geo textile was placed on the top of the compacted soil mass and a perforated network of pipes was installed and covered by a coarse aggregates layer of thickness 50 mm. The coarse aggregates were used to facilitate infiltration of water to the compacted soil mass and to prevent damage to the pipe network by surcharge to be applied. The geotextile was used as a separator to prevent the penetration of coarse aggregates into the soil mass. The as compacted soil was unsaturated and possessed a high matric suction. The matric suction within the fill was changed as desired by sending water through the system of pipes or by allowing it to dry, while being monitored through the tensiometers installed. Three tensiometers were placed at different depths in the box to measure the matric suction prevailing at respective location.

Two layers of timber planks and steel plates were placed above the coarse aggregate to distribute the point loads applied from hydraulic jack and to convert it to a uniform surcharge on the nails installed. This arrangement was necessary to obtain a significant surcharge (overburden pressure) on the soil nails in view of the relatively small depth of soil. Figure 3.3 illustrates the complete experimental arrangement of test box.

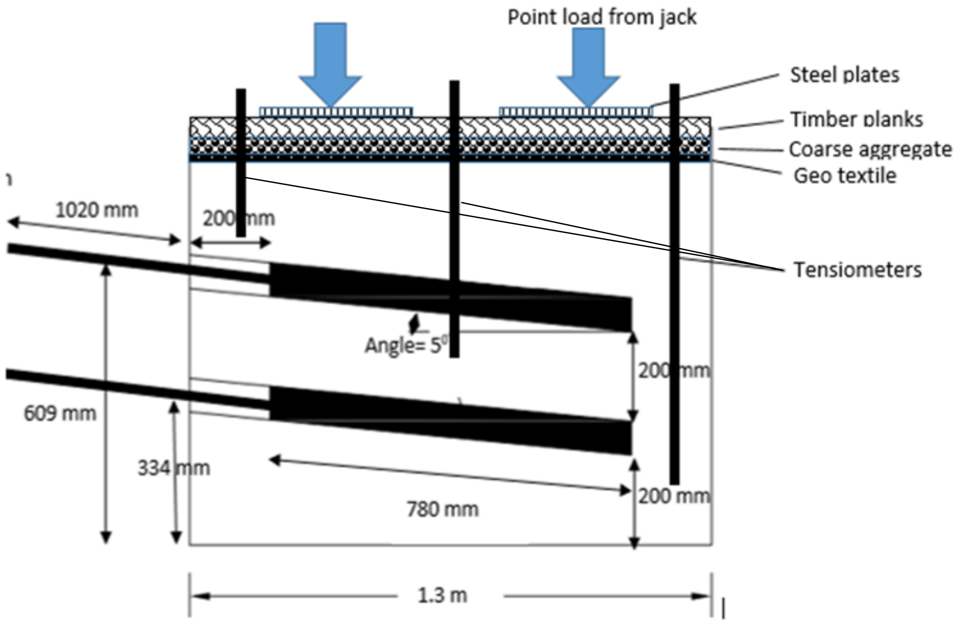


Figure 3.3 Complete experimental arrangement of the test box used in the study

3.2 Drilling and Installation of Soil Nails

Soil nails were installed at an inclination of 5° downward to the horizontal. The drill holes were done manually with an auger of diameter 75 mm. The length of the hole was 0.98 m from the face of the box but only the lower 0.78 m was grouted. Drilling of hole for soil nail installation, test box after drilling and drilled hole are shown in Figure 3.4 (a), 3.4 (b) and 3.4 (c) respectively.

The grouting was done at a pressure of 50 kN/m^2. The grouting device used was a sealed PVC cylinder with a 160 mm inner diameter. There were two valves at the inlet, which were used to inject additional cement grout and to apply air pressure.

During the pressure grouting process, air pressure valve and the grout outlet valve were opened. A pressure gauge was used to monitor the grouting pressure. Once the grout was used up the valve for injecting grout was opened and fresh quantity was taken in. Figure 3.5 shows (a) the apparatus used for pressure grouting (b) the process of grouting.

Figure 3.4 (a) Drilling of hole for soil nail installation

(b) Test box after drilling (c) A drilled hole

The ultimate bond strength at the grout-soil interface is strongly influenced by the characteristics of the grout. Tokyo super ordinary Portland cement and the admixture Flow cable 50 were used in the mix proportion of 200:1 for grouting with a water cement ratio is 0.45. The grout was mixed well with electric grout mixing equipment. This mix proportion is commonly used in most of the soil nailing applications in practice. The grout injection was done in one continuous operation to avoid formation of air voids. Flow cable 50 which is used as expanding plasticizing admixture, eliminates the settlement and shrinkage, and improves the flowing properties.

Figure 3.5 (a) The apparatus used for pressure grouting (b) The process of grouting.

A reinforcement bar of 25 mm diameter Qst Ribbed steel was used as the nail. The yield strength of the bar is 500 N/mm^2. Each soil nail was installed with two centralizers (Figure 3.6) and a tremie grout tube. Tremie grout tube was progressively removed out with the injection of pressurized grout. A 200 mm segment of the drill hole closer to the opening surface was left without grouting.


Figure 3.6 Soil nails with two centralizers

3.3 Instrumentation

The force applied and displacement of the nail during the pull-out test were recorded by the load cell and the dial gauge fixed at the end of the nail (Figure 3.7). The matric suction was monitored at the top and the bottom tensiometers located near the nail continuously through a data logger and a channel box. The Figure 3.8 shows the instruments used in the measurement of matric suction, vertical stress and pullout force.

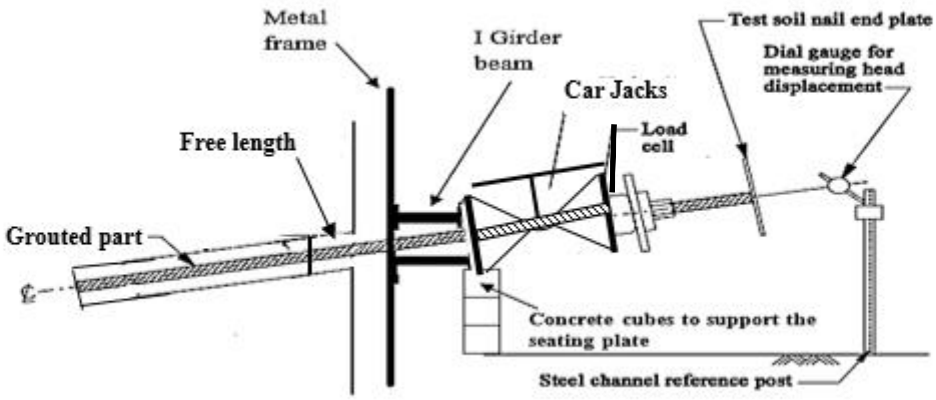


Figure 3.7 Pullout test arrangement

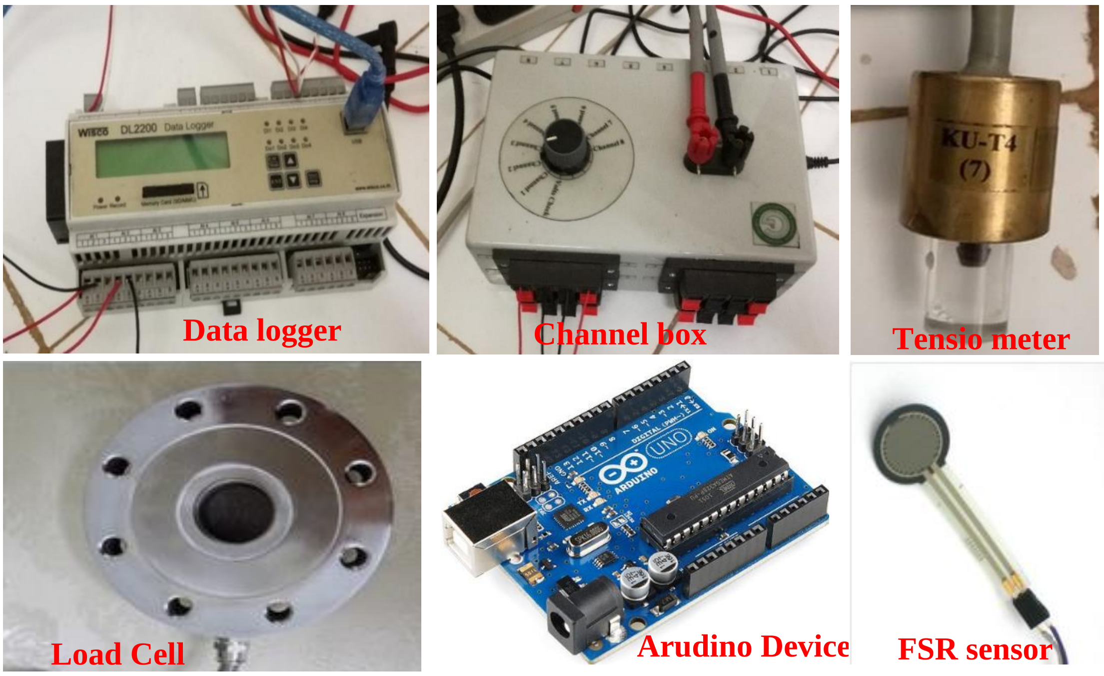


Figure 3.8 Instruments Used in the Study

The stresses in the soil in the vertical and horizontal directions were measured through a force sensitive resistor (FSR) and Arudino device. FSRs are sensors that is used to detect physical pressure, squeezing and weight. They are simple to use and are of low cost. The FSR on Figure 3.8 is specifically the Interlink 402 model. The 1/2" diameter round part is the sensitive bit. The FSR is made of 2 layers separated by a spacer. With the increasing pressure, the more of those Active Element dots touch the semiconductor and that makes the resistance go down. FSRs are basically a resistor that changes its resistive value (in ohms Ω) depending on how much it is pressed.

Surcharge of 60 kN/m^2 and 30 kN/m^2 is applied to the soil mass by placing two hydraulic jack. The pull-out force was applied by two car jacks assembled in parallel and kept between the load cells and loading frame (Figure 3.9). With this arrangement it was possible to apply the pull-out load in small increments in a well-controlled manner.

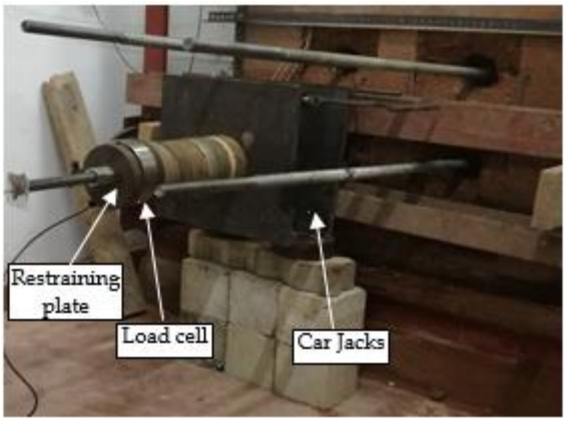


Figure 3.9 Arrangement before pullout testing

3.3.1 Tensiometers

There are many methods to measure matric suction. They are; axis translation, tensiometer, and filter paper. The tensiometer used in the test program was a Miniature tensiometer. It consists of Micro Electro Mechanical Systems (MEMs) pressure sensor, 1 bar (100 kN/m^2) high air entry porous ceramic and transparent acrylic tube. This miniature tensiometer was developed at Department of Civil Engineering, Kasetsart University, Thailand. Hence referred to as KU tensiometer. This KU tensiometer has an operating range of matric suction from 0 – 86 kPa.

There is a ceramic tip in the tensiometer, a small probe which can be filled with water. The purpose of the ceramic tip is to create a saturated connection between the unsaturated soil and the water in the tensiometer, through the use of a pressure sensor. A typical KU tensiometer is shown in Figure 3.10.

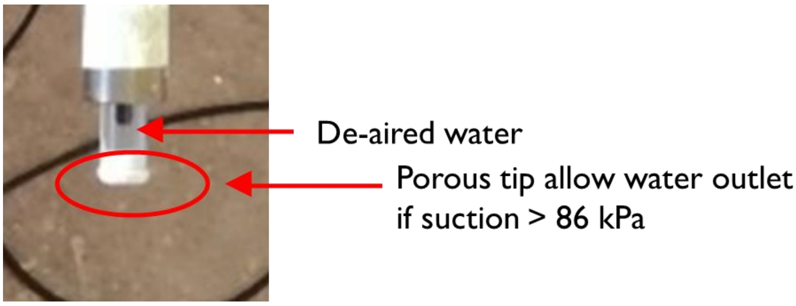


Figure 3.10 KU tensiometer

The ceramic material is made of kaolinite, which has high air entry value. This material is used for the measurement of matric suction. The surface tension on the ceramic material to maintain the gas-liquid interfaces. This tension separates the air and water phases acting as a membrane. Figure 3.11 shows the cross-section of a typical saturated ceramic disk.

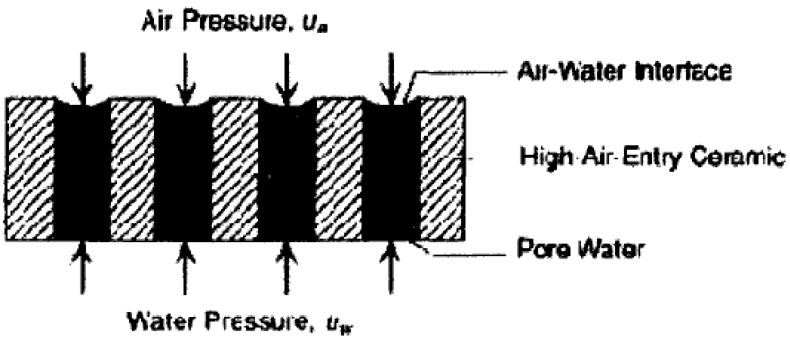


Figure 3.11 Operating principle of HAE ceramic cup (Lu and Likos, 2004)

3.3.1.1 Preparation of tensiometers

The tensiometer should be de-aired to get an accurate reading. The saturation of the tip and the tensiometer is an important part in this experiment. The tensiometer was kept in the distilled water, which was placed inside the desiccator and air was pumped out by a vacuum pump. After continuous de-airing of 2 or 3 days, the tensiometer were checked with the suction value which usually lie at 0.8, 0.8-0.3 in water and in air respectively. If any air bubbles are present the further de-airing should be done again. Figure 3.12 shows the equipment used for de-airing.

Figure 3.12 Equipment used for de-airing

3.3.2 Force sensitive resistor calibration (FSR)

The manual of force sensitive resistor (FSR) gave a brief description about coding in arudino software. In the code, all the values are variable with analog input except a dividing factor (Figure 3.13). Calibration of the FSR is finding the suitable value / formula for dividing factor. But the dividing factor varies non-uniformly with the increase in conductance. As such, best suits quadratic function was developed for the dividing factor.

```
if (fsrConductance <= 1000) {
  fsrForce = fsrConductance / 80;
  Serial.print("Force in Newtons: ");
  Serial.println(fsrForce);
```

Constant dividing factor

Figure 3.13 Extract from the coding of FSR manual

The procedure followed in developing the quadratic function

- First the FSR was placed on digital balance and the sensing area was touched completely by a probe tip connected to a PVC pipe as shown in Figure 3.14.
- For each load, analog inputs (conductance) from the FSR were sensed by the arudino and saved in text file until the FSR readings get stabilized (about 30 minutes).
- Then the load was incremented to about 150 grams and the recording was carried out for 30 minutes until it reach 2 kg.
- The recorded texts were imported into excel and the graphs were drawn for FSR sensed force and time as shown in Figure 3.15.
- The stabilized force at a constant applied load was estimated by trend lines.
- Those stabilized force and the applied loads as shown in Table 3.1 to find the dividing factor at that applied load.
- These sets of dividing factors are plotted in a dividing factor Vs conductance graph as shown in Figure 3.16 and the quadratic function was developed.

Figure 3.14 experimental arrangement for the Calibration of FSR

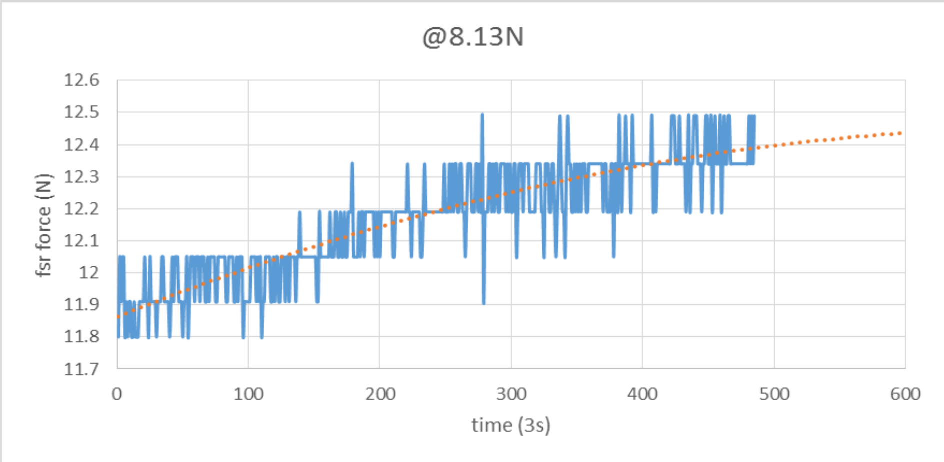


Figure 3.15 Fsr measured force versus time plot

3.1 Table depicting the calculation

Actual load (N)	Stabilized force (N)	Conductance (A/V)	Dividing factor
3.61	6.80	544	150.7
5.02	8.85	708	141.0
6.64	10.90	872	131.3
8.13	12.45	996	122.5
9.55	14.10	1128	118.1
11.00	16.00	1280	116.4
12.54	17.50	1400	111.6
14.03	19.00	1520	108.3
15.57	20.30	1624	104.3
17.04	21.80	1744	102.3
18.55	23.50	1880	101.3
20.06	25.30	2024	100.9

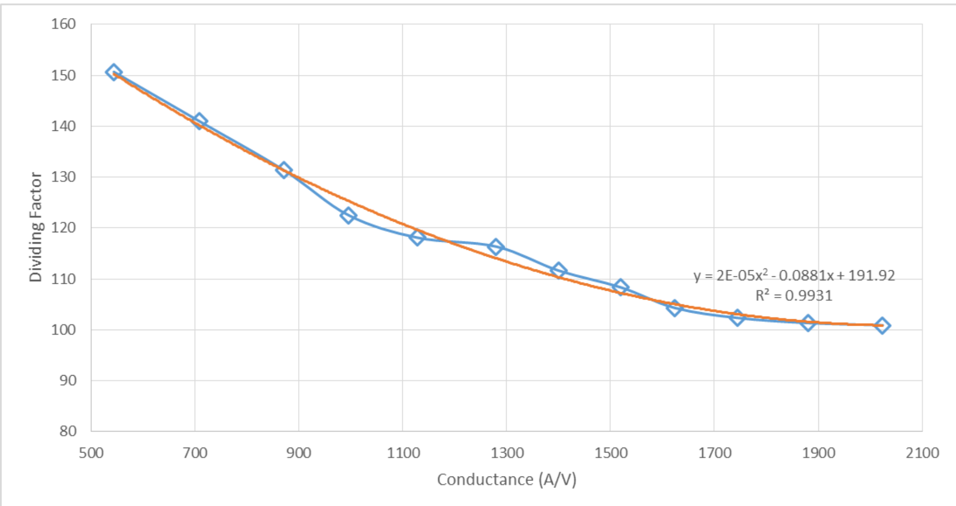


Figure 3.16 Plot to determine suitable dividing factor for FSR

CHAPTER 4 TESTING PROGRAM AND RESULTS

4.1 Basic Laboratory Tests

The soil used for this research was obtained from the University of Moratuwa Premises. The soil was first sieved using 5 mm sieve to remove the gravel and impurities present. The removal of large size particle was necessary as the direct shear test and consolidation tests were conducted with small specimen to determine the soil characteristics. A series of laboratory tests were conducted on the soil to determine its basic properties, shear strength parameters. The different test conducted were;

Particle size distribution (Wet sieve analysis, Hydrometer test)

Plastic characteristics of soil

Standard Proctor compaction test

Specific density test

4.1.1 Particle size analysis

For a wet sieving, Soil sample of 500 g was measured and washed within 0.075mm sieve and the remains were dried and sieving was done.

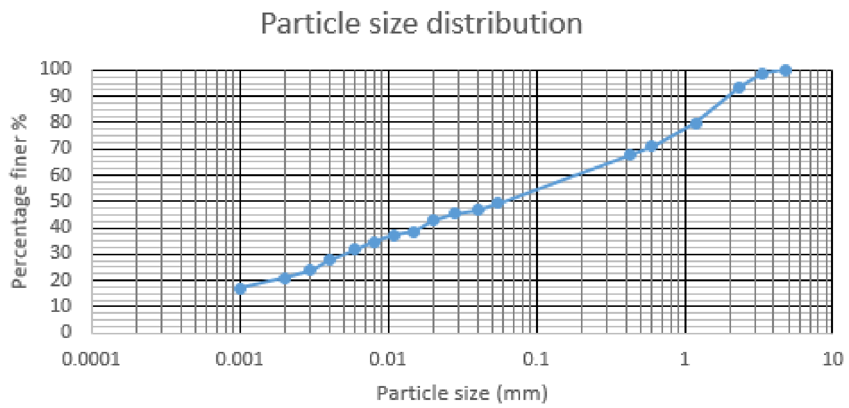


Figure 4.1 Combined wet sieve and hydrometer analysis

The hydrometer test was done with the soil sample of 50g passed through 0.425mm sieve. The combined results of wet sieve and the hydrometer test are shown in the Figure 4.1.

4.1.2 Plastic characteristics of soil

The liquid limit of soil was determined using casagrande apparatus accordance with the ASTM D 4318. Liquid limit and plastic limit obtained were 55.80% and 36.35 % respectively. The Figure 4.2 shows the variation of moisture content with no of blows.

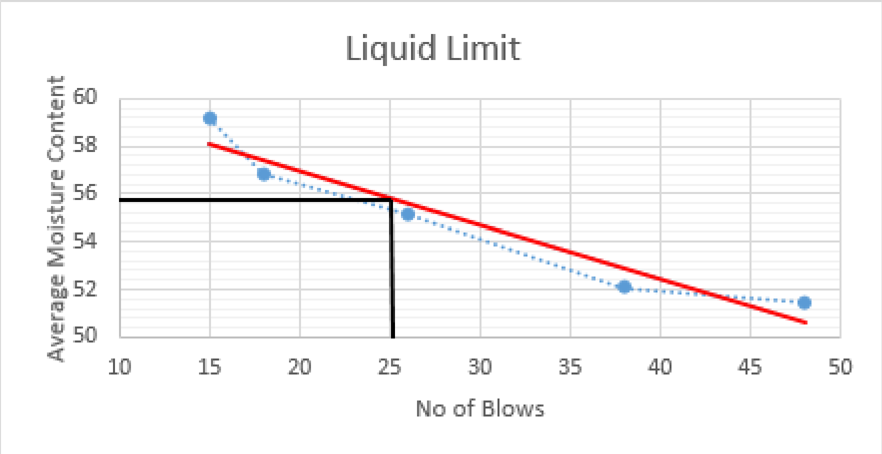


Figure 4.2 Variation of moisture content Vs. no of blows.

From the particle size distribution and plasticity test the soil was found to be silt with high plasticity (MH)

4.1.3 Compaction test (standard Proctor test)

The compaction test was conducted in accordance with ASTM D 638 on the soil used. The optimum moisture content and the maximum dry density of the soil were 23.1% and 1554 kg/m^3 respectively. The soil in the test box was compacted at optimum moisture content using manual modified proctor compactor. Figure 4.3 illustrates the relationship between the dry density and moisture content.

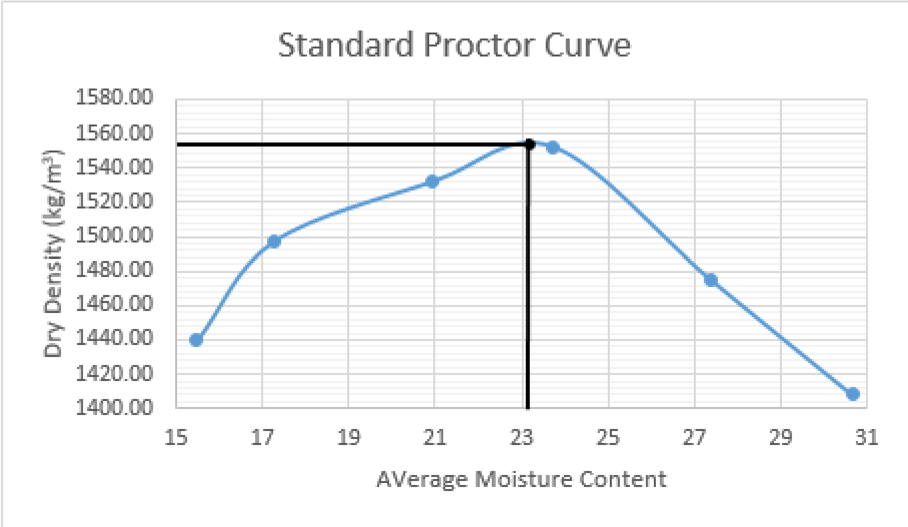


Figure 4.3 Standard proctor curve

4.1.4 Specific gravity test

The specific gravity tests were performed in the laboratory in accordance with ASTM D854 (2004). The average value of the specific gravity of the soil used is equal to 2.61 from three tests.

4.1.5 Summary of the properties of the tested soil

Table 4.1 shows the summary of the laboratory tests performed with the findings.

Table 4.1 Summary of the soil characteristics

Soil properties	Value/ description
Type of soil	silt of high plasticity (MH)
Optimum moisture content	23.10%
Maximum dry density	1554 kg/m^3
Specific gravity	2.61
Liquid limit	55.80 %
Plastic limit	36.35 %
Plastic index	19.45 %

4.2 Direct Shear Test

To estimate the pull out resistance of the soil nail it is necessary to determine the interface shear strength parameters between relevant soil and grout. The direct shear test is a simple laboratory test used to determine the shear strength properties of soil or interface. In this study a set of direct shear tests were performed for soil grout interface at saturated state to determine the interface parameters at saturated conditions and another set of tests were performed for soil, soil grout interface to determine the strength reduction factor.

4.2.1 Direct shear test at different matric suction

Dilanthi et al. (2018) performed a series of direct shear test under different matric suctions and at saturated state to determine corresponding soil-soil interface apparent cohesion component (C_a) and effective friction angle (ϕ) of the same soil (MH soil). Table 4.2 summarizes the results of the direct shear test. This values were used in the finite element analysis. Figure 4.4 shows the plot of measured apparent cohesion Vs matric suction.

Table 4.2 Unsaturated parameters obtained from the direct shear tests for high plastic silt

Test No.	Saturation (%)	Measured apparent cohesion, c_a/(kN/m^2)	Measured matric suction,(u_a -u_w) (kN/m^2)
1	81	58.6	60
2	85	50.9	40
3	92	42.1	30
4	100	23.9	0

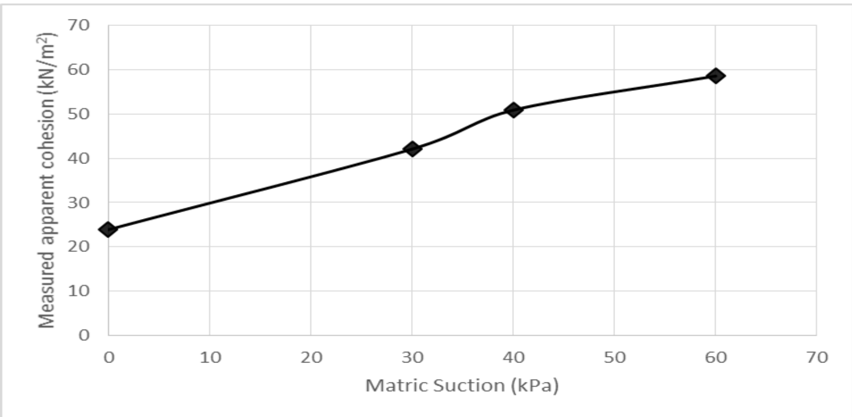


Figure 4.4 Variation of measured apparent cohesion with matric suction (Dilanthi et al., 2018)

4.2.2 Interface direct shear test at saturated state

The test is performed on four identical specimens from disturbed soil sample. The soil samples were obtained from proctor mould after attaining the maximum dry density by compacting at optimum moisture content. Each specimen is placed in the shear box; the contact between the two rings is at the mid-height of the sample. A confining stress is applied vertically to the specimen, and the upper ring is forced to move apart laterally until the sample fails. The load applied, the horizontal displacement and vertical displacement are recorded at intervals specified in ASTM 3080. After failure, next sample was prepared and tested at varying normal load to determine the shear strength parameters, the adhesion (c_w) and the apparent of internal friction (**δ**).

The compacted sample was brought to a saturated state by adding water (submerge in water body) and keeping for 24 hrs to get the saturation condition before testing. A set of consolidation tests were performed by Gangani (2017) under saturated condition and values of coefficient of consolidation were in the range of 9-18 mm^2/min. Direct shear tests are to be conducted under drained conditions and the rate of shearing has to be decided ensuring that 90% consolidation achieves much earlier than the failure. Accordingly a rate of shearing of 0.125 mm/min was used (Dilanthi et al., 2018). Using the shear stress Vs shear displacement graph of each test the shear strength Vs normal stress graphs were plotted to obtain the interface shear strength parameters.

4.2.2.1 Preparation of test specimen for interface direct shear

150 mm × 150 mm × 150 mm of grout cubes with a minimum strength of 30 MPa were casted. After 7 days of curing, grout cubes were cut into small cuboid so that it can be accommodated inside the lower part of the direct shear box. Considering the findings of previous researches (Hossain and Yin, 2013; Gurpersaud, 2010), the researchers have created an interface condition where the shearing plane is forced to develop approximately 2 mm from the counterface (grout penetration zone). Therefore 2 mm gap was placed in the present study between the grout cube and failure plane. Then upper part of shear box was fixed and the fresh grout was applied on the top of the

grout cube filling the 2 mm gap and the extruded compacted soil sample was inserted into the shear box so that the soil can be penetrated into the fresh grout. Before testing the specimen was cured for 3 days. Figure 4.5 (a) shows the grout cube used in the interface test (b) shows the applied grout to the cube (c) shows the grout penetration zone.

Figure 4.5 (a) The grout cube used in the interface test (b) The applied grout to the cube (c) The grout penetration zone

4.2.2.2 Evaluation of Effective Interface Shear Strength Parameters c_w and δ

The shear stress Vs horizontal displacement curves obtained from the five specimens are presented in Figure 4.6. The plots of shear stress Vs normal stress at failure for the five specimens are presented in Figure 4.7. The interface strength parameters at saturated state are summarized in Table 4.3.

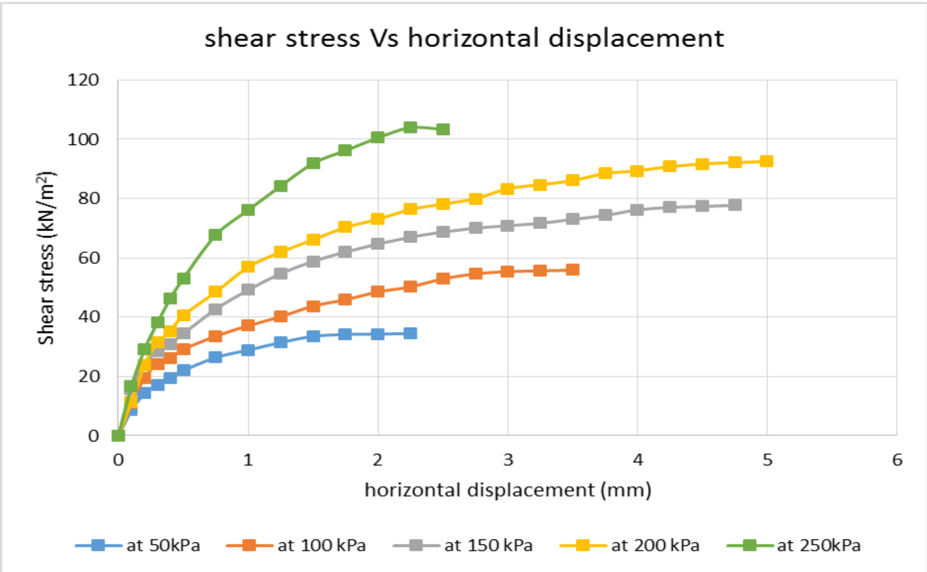


Figure 4.6 Shear stress Vs horizontal displacement curves

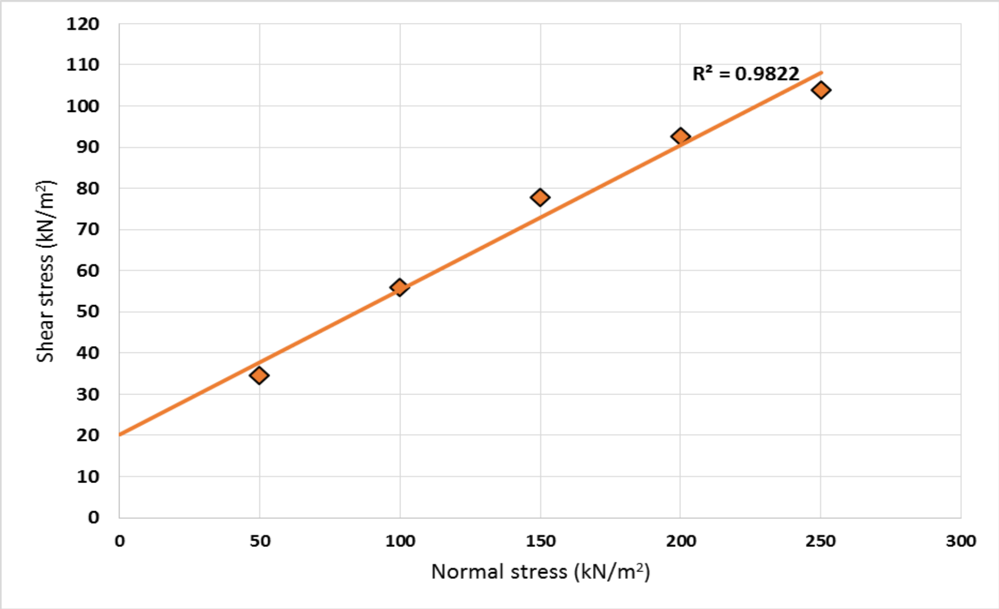


Figure 4.7 Shear stress Vs normal stress at failure

Table 4.3 Interface strength parameters at saturated state

Test No	1	2	3	4	5
Normal Stress kN/m^2	50	100	150	200	250
Peak Shear Stress kN/m^2	34.484	55.891	77.752	92.589	103.944
Rate of Strain mm/min	0.125	0.125	0.125	0.125	0.125
Strain of Peak Shear Stress %	3.75	5.83	7.92	8.33	4.17
Sample Preparation	Disturbed				
Interface Shear Strength Parameters					
Interface Friction angle (in degree) δ	**19.35**				
Adhesion (C_w) kN/m^2	**20.25**				

4.2.3 Interface shear test at as compacted condition of matric suction 40 kPa

The matric suction of the as compacted soil was around 40 kPa. Hence a set of interface shear tests was carried out at a matric suction of 40 kPa to evaluate the shear strength parameters at that suction to check whether that can be substitutable in a simplified design formula. The preparation of test specimens and testings were similar to the section 4.2.2.1. But to maintain the matric suction of 40 kPa it was placed inside the pressure plate apparatus after saturation of test specimens and allowed to reach the equilibrium. Thereafter, it was placed on direct shear test apparatus and normal load was applied. Shearing of the specimen was carried out after it attained the lost matric suction of 40 kPa due to applied normal load. The matric suction variation during shearing is shown in the Figure 4.8.The shear stress Vs horizontal displacement curves obtained from the four specimens are presented in Figure 4.9 and Figure 4.10 presents the plot of shear stress Vs Normal stress at failure. Table 4.4 shows the shear strength parameters at 40 kPa of matric suction. The failure shear stress obtained at normal load of 200 kPa gave a lesser value compared to other normal load as such it was omitted from further calculations.

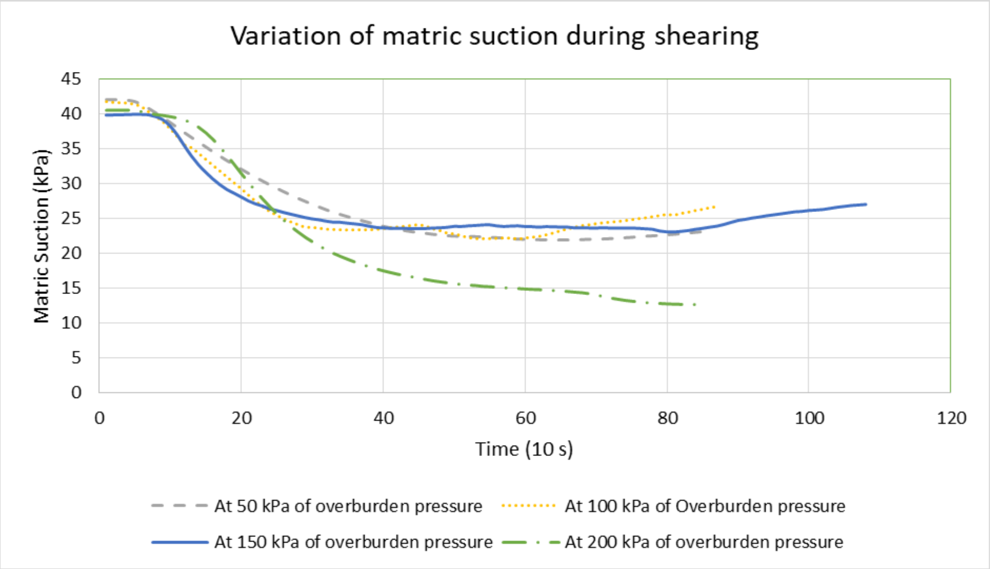


Figure 4.8 Matric suction variation during shearing

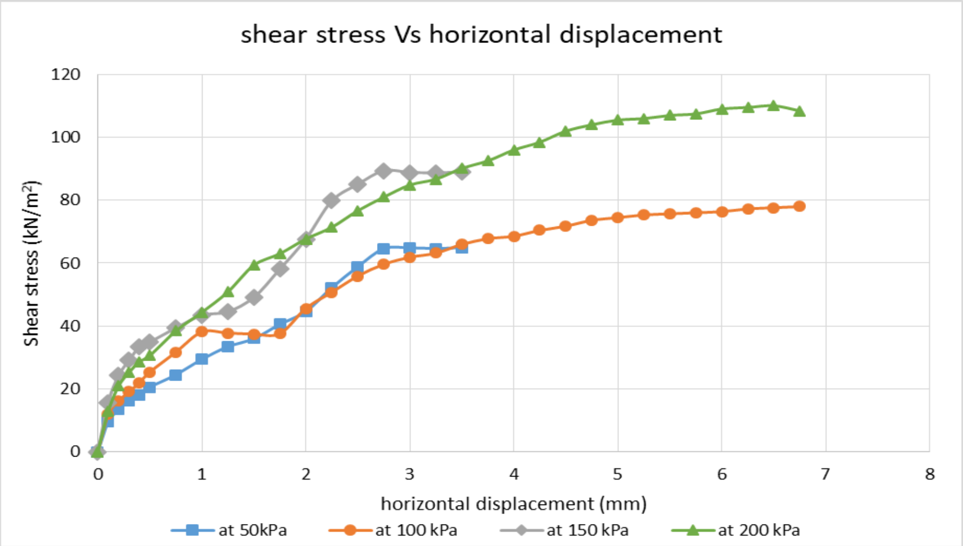


Figure 4.9 Shear stress Vs horizontal displacement curves

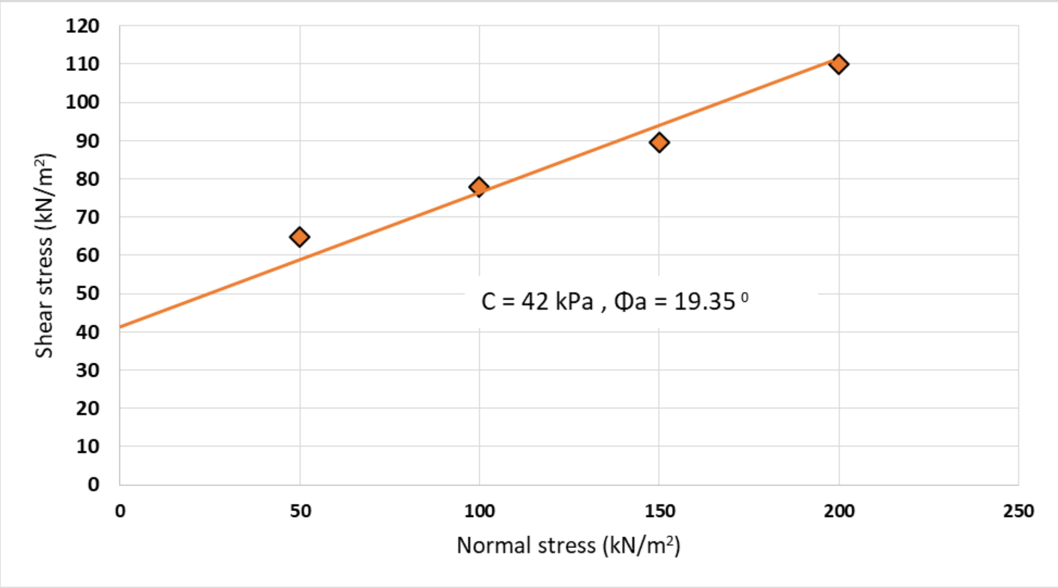


Figure 4.10 Shear stress Vs normal stress at failure

Table 4.4 Interface shear strength parameters at matric suction of 40 kPa

Test No	1	2	3	4
Normal Stress kN/m^2	50	100	150	200
Peak Shear Stress kN/m^2	64.9	78.0	89.4	110.01
Rate of Strain mm/min	0.125	0.125	0.125	0.125
Sample Preparation	Disturbed			
Interface Shear Strength Parameters				
Interface Friction angle (in degree)	**19.01**			
Adhesion (C_w) kN/m^2	**42**			

4.2.4 Direct shear tests to estimate the strength reduction factor

Another set of direct shear tests were performed to estimate the strength reduction factor at saturated state under a vertical stress of 30 kPa and 60 kPa. This test was carried out for soil-soil interface and soil-grout interface. Testing was done at two overburden pressure to justify the value obtained from the first test. Figure 4.11 shows the shear stress Vs shear displacement graph for different interface. Table 4.5 shows the summary of the failure shear strength.

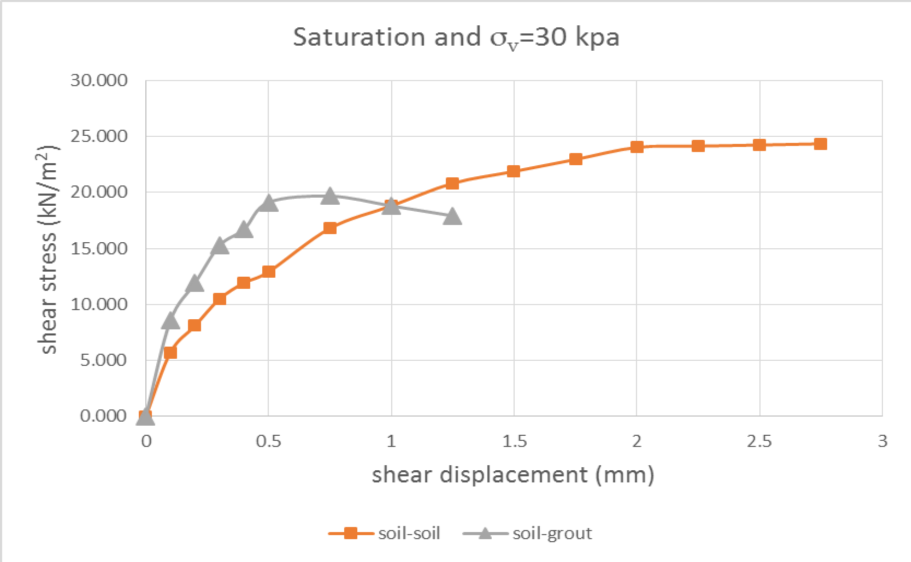
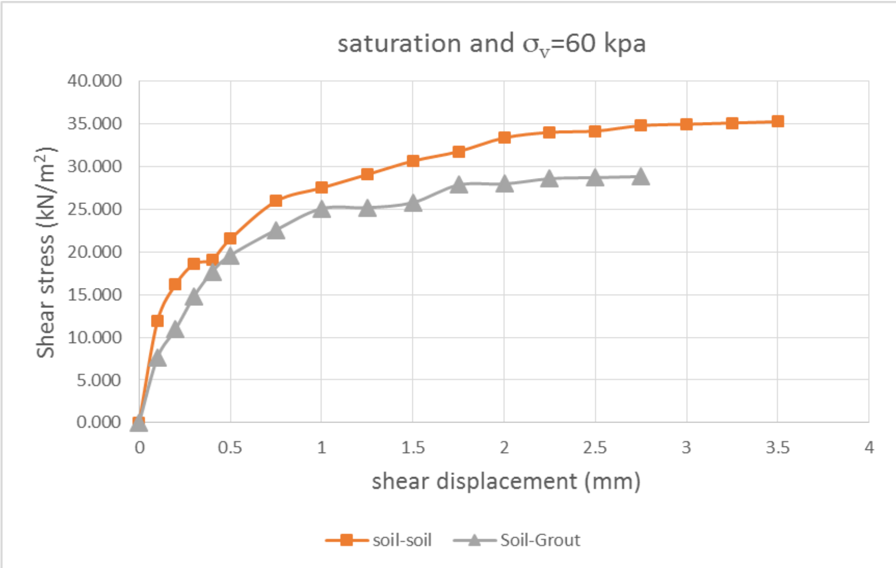


Figure 4.11 (a) Direct shear at saturated state and normal pressure of 30 kPa



(b) Direct shear at saturated state and normal pressure of 60 kPa

Table 4.5 Summary of the failure shear strength

Normal stress (kN/m^2)	Condition of soil	Soil-Soil peak shear stress (kN/m^2)	Soil-Grout peak shear stress (kN/m^2)
30	saturated	22.86	19.69
60	saturated	35.25	28.82

4.3 Compaction of Soil Mass in Test Box

The soil moisture content was measured initially and for a fixed quantity of soil, the water is added to bring it to the optimum moisture content and mixed well and left over night to bring it to moisture equilibrium. Thereafter it was placed in a layer of 125 mm and the compaction was provided manually with modified hammer which has a weight of 5.935 kg. After completion of compaction, soil samples were extruded with a modified core cutter which has a diameter of 44 mm and height of 109 mm at each layers to assess the compaction achieved. Figure 4.12 (a) shows the modified hammer and (b) shows the modified core cutter. Level of compaction was measured in each layer and the results are summarized in Table 4.6. The insitu density tests show that level of compaction in each layer of 125 mm is around 100%. This showed that the soil in the box was brought to a uniform level of compaction.


Figure 4.12 (a) Modified hammer (b) Modified core cutter

Table 4.6 Compaction results

layer	moisture content	dry density	percentage of compaction
first	22.10	1590.0	102.3
second	21.62	1636.7	105.3
third	20.83	1598.0	102.8
fourth	22.96	1587.1	102.1
fifth	25.36	1418.9	91.3
sixth	23.16	1609.0	103.5
seventh	23.36	1612.6	103.8
eighth	25.47	1563.0	100.6

4.4 Grout Cube Test

Grout cubes (100 x 100 x 100 mm) were prepared and compressive strength was determined as per ASTM CI09-08 for hydraulic cement mortars. The average compressive strength of six grout cubes was determined after 7, 14 and 28 days and produced results of 38.4, 51.15 and 56.7 MPa respectively. The nominal strength required for soil nails in the practice is 30 MPa after 28 days.

4.5 Performance of Pullout Test

Two methods are adapted to perform pull-out tests. Such as displacement controlled test and force controlled test. Displacement controlled tests are used to establish the actual ultimate pull-out capacity of the nail. Force controlled pull-out tests provides a rough estimation of the peak capacity of the nails and commonly used in the field because it is easier to conduct (Barley et al, 1997; Junaideen et al., 2004). Force controlled test procedure was followed here as it is easy to conduct and equipment necessary to a displacement control test were not available.

The sequence adapted after compaction of the soil are;

- The soil nails were installed with manual drilling and grouting
- The suction was measured in the compacted soil at three places and the sample was wetted until a measurable matric suction is achieved.
- Conducted the pullout test using force controlled method in a controlled manner.

Pullout testing procedure

- The pullout testing setup was done and the pullout force was applied in increments of 100kg.
- For every 100kg increment, displacement readings at 0, 1, 2, 5 minutes in the dial gauge was recorded until 0.5 DL (loading).
- After reaching 0.5 DL unloading was initiated.
- This was done in 2 cycles (0.5 DL, 0.75 DL) and the third cycle until the failure was achieved.

4.5.1 Pullout testing near saturated state

Two tests were performed near saturated state at two different overburden pressure of 30 kPa and 60 kPa. The objective of these tests was to evaluate the pullout capacity of soil nails near saturated state to compare with the pullout capacity obtained at unsaturated state. The soil was gradually saturated from the top using the installed pipe system while measuring the matric suction. Initially an overburden pressure of 30 kPa was applied to the soil mass. Near saturated state the pullout testing was performed. The Figure 4.13 and Figure 4.14 shows the pullout testing results near saturated state.

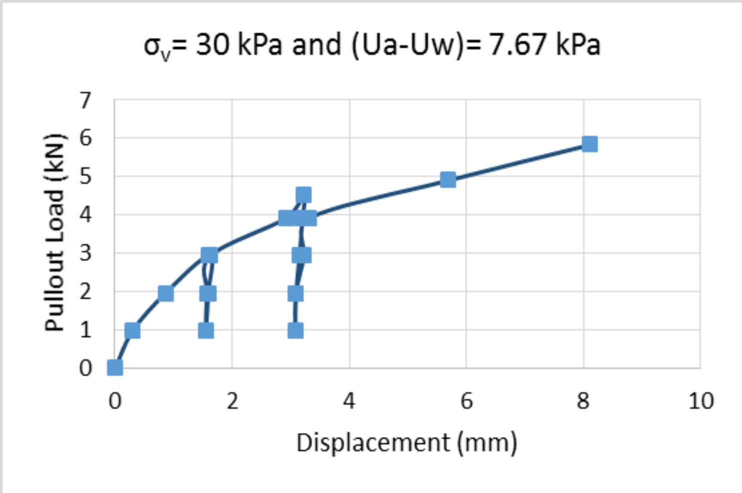


Figure 4.13 Pull-out at an overburden pressure of 30 kPa with a matric suction of 7.67 kPa

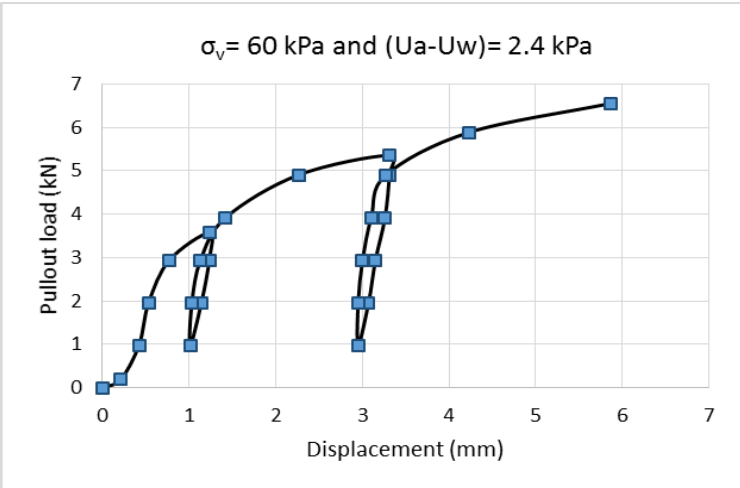


Figure 4.14 Pull-out at an overburden pressure of 60 kPa with a matric suction of 2.4 kPa

4.5.2 Pullout testing at higher suction (Unsaturated state)

Similar to the saturated state, other two soil nails were tested at a higher matric suction under overburden pressures of 30 kPa and 60 kPa. Matric suction was increased by allowing moisture to evaporate from the soil mass. The pullout testing was performed at a matric suction of 40 kPa. The Figure 4.15 and Figure 4.16 shows the pullout testing results near 40 kPa

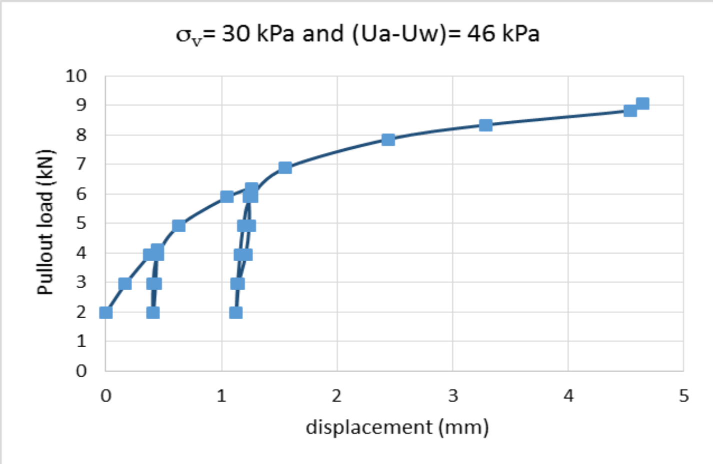


Figure 4.15 Pull-out at overburden pressure of 30 kPa with a matric suction of 46kPa

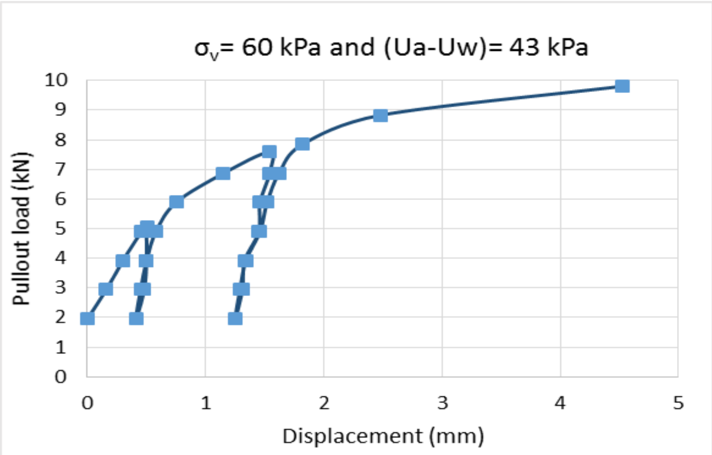


Figure 4.16 Pull-out at overburden pressure of 60 kPa with a matric suction of 43 kPa

4.5.3 Estimation of average matric suction value

There is a non-linear relationship between matric suction with respect to depth as indicated from the results obtained from tensiometers in the test box. However, matric suction variation was considered to be linear with respect to the depth for this research.

The suction was taken as the average value of two readings obtained by tensiometers placed in the top and bottom of the soil nail.

4.5.4 Measurement of vertical overburden pressure with FSR during pullout

There were 6 force sensitive resistors (FSR) were installed in the test box to measure the lateral and vertical pressure acting around the soil nail. The applied overburden pressure was controlled by proving ring and the fsr sensors reading were recorded. But an abnormal readings were obtained in 5 sensors except one. The damage cause to the sensor during the compaction process may be the reason for this observed behaviour. The FSR recording of vertical pressure during the pullout is shown in Figure 4.17.

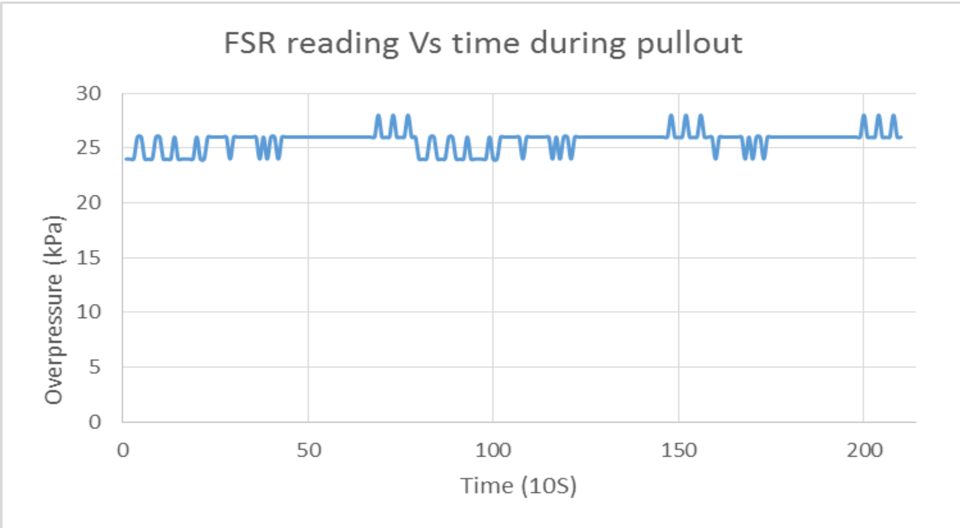
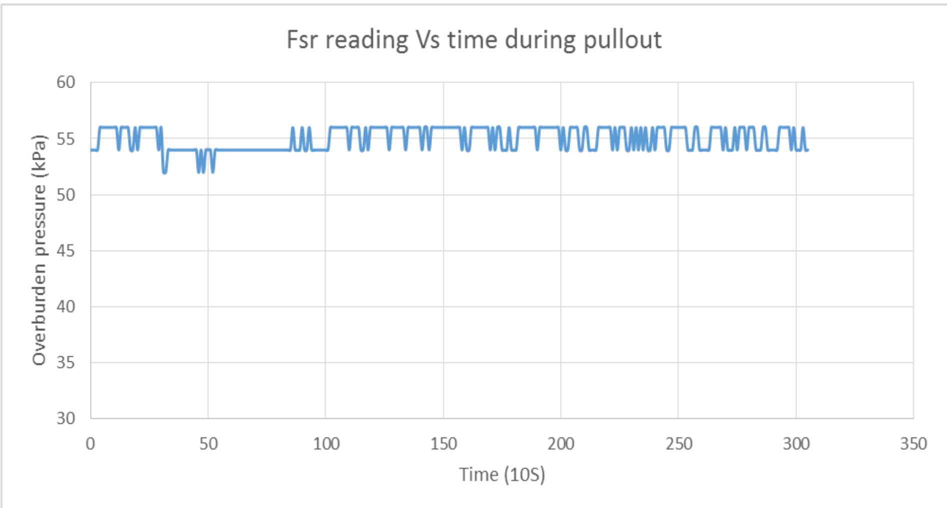


Figure 4.17 (a) FSR reading at an applied overburden pressure of 30 kPa



(b) FSR reading at an applied overburden pressure of 60 kPa

4.6 Complete Pullout of Soil Nails

After completion of pull-out testing, the test nails were removed and the perimeters of the grouted nails were measured as shown in Table 4.7 to determine the actual nail diameter. It was found that the average diameter of the grouted body is 10.16% greater than drilled diameter of 75 mm. Figure 4.18 shows the completely pulled out soil nails.

Figure 4.18 Completely pulled out soil nails.

Table 4.7 Percentage increase in diameter of the grouted nails

nail no	Circumference (cm)				length (cm)	Diameter (cm)	percentage increase of diameter
	C1	C2	C3	C_{ave}			
1	25.50	25.70	25.60	25.60	75.00	8.15	8.64
2	28.20	26.00	26.00	26.73	77.00	8.51	13.45
3	26.50	26.00	25.00	25.83	75.00	8.22	9.63
4	26.50	25.00	25.50	25.67	79.00	8.17	8.92

CHAPTER 5 STIMULATION OF SOIL NAIL PULLOUT RESISTANCE USING FINITE ELEMENT MODEL

5.1 Introduction

Simulation of the interaction between the grouted body of soil nail and soil is an important aspect in the soil nail design. To study the complex behaviour of the soil nail interaction, often rigorous computational techniques based on finite element or finite difference methods are used often. Plaxis 2D software was used by many researchers to study the complex soil structure interaction, to estimate the performance and stability of the soil nail structures (Shiu et al., 2005; Fan and Luo, 2008).

Soil nails with overburden pressure has bending and shear resistances in addition to the pullout resistance (axial force). Literature states that the soil nails with the inclination (10^0 -15^0) would tend to undergo a local rotation to approach horizontal direction of maximum soil extension, developing bending stiffness over soil nail force (Juran et al. 1990). Jewell and pedley (1992) concluded that the bending and shear resistance effects can be ignored in the design thus makes the design simple and commonly acceptable.

The following points were considered in the modelling using Plaxis 2D.

- The stress change around the soil nail pullout is mainly due to the grouting pressure used in the process of installation of soil nails. So it is assumed that installation produces minimal disturbance to the soil.
- The boundary stresses in the vertical and horizontal directions are approximately equal, and the stresses in the soil immediately surrounding the soil nail may be nearly axisymmetric (Morris, 1999).
- The normal stress generated in the axisymmetric method is uniformly distributed on the perimeter of the soil nails. But in actual scenario the distribution of circumferential normal stress is non-uniform. (Ann et al. 2004).

5.2 Finite Element Mesh and Boundary Conditions

In this study, Finite element software Plaxis 2D is used for the numerical modelling of soil nail pullout behaviour. The horizontal soil nail is simulated by a vertical inclusion in a circular soil tank using axisymmetric condition. In the axisymmetric conditions the x axis is used as the radius and Y axis as the symmetrical axis of the soil model. Brinkgreve et al. (2012) stressed the theory of using axisymmetric condition for the simulation of cylindrical elements such as soil nail, piles and soil anchors. Figure 5.1 show the axisymmetric model of the soil nail. 15 node mesh boundary is used for finite element generation and the mesh density is fine (Babu and Singh, 2009). In order to compare the results obtained from laboratory experiment, the exact model used for pullout was developed. The left and right boundaries are fixed in horizontal ways and the bottom boundary is fixed in vertical direction and the top boundary is simulated as a free boundary.

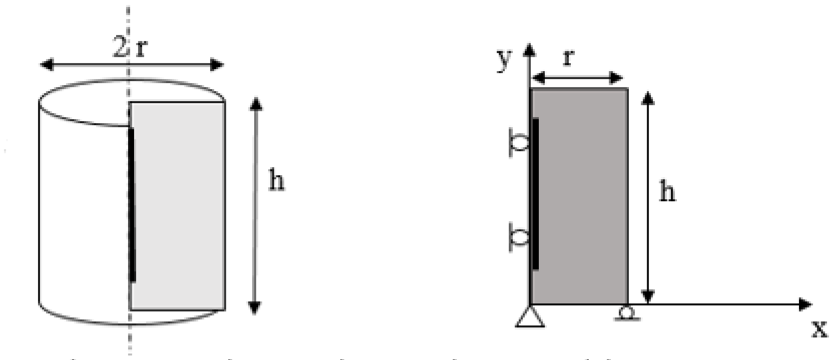


Figure 5.1 Axisymmetric finite element model

5.3 Constitutive Models, Model Parameters and Modelling Procedures

In Plaxis 2D, the soil nail can be designed as plate element as well as a geogrid element. Plate element accounts for the bending stiffness in analysis but geogrid element ignores it. Jewell and Pedley (1992) concluded that in the design and analysis of soil nailing the effects of bending and shear resistances can be ignored with marginal conservatism. Similarly, Schlossor (1991) stated that the influence of bending stiffness and shear on the global factor is small (less than 15%). Hence soil nail is considered as a plate element. HS model (Hardening soil model) is commonly used to stimulate in-situ soil for excavation and retaining structure as it can yield plastic straining due to soil expansion. But if all input parameters are unavailable, Babu and Singh (2009) suggested to choose Mohr-Coulomb material model. The suction in the soil mass was incorporated in the cohesion component of soil model to acquire the effect of suction on soil. Table 5.1 gives the summary of the shear strength values at different suction used for the simulation (Dilanthi et al., 2018) while soil parameters used for the simulation are given in Table 5.2.

Table 5.1 Summary of the shear strength values at different suction (after Dilanthi et al., 2018)

Matric suction (kPa)	2.4	7.67	43	46
Apparent Cohesion , Ca (kPa)	24	29	52.06	53.21
Friction angle	32.62	32.62	32.62	32.62

Table 5.2 Soil parameters used for the simulation

Type of Soil model	Condition of Soil Model	γ (kN/m^2)	γ_{sat} (kN/m^2)	Poisson's ratio	Elastic modulus (kN/m^2)
Mohr Coulomb	Drained	15	17	0.3	11500

The interaction between the soil and nail is simulated by creating the interface. Plaxis 2D uses a strength reduction factor (R_{inter}) to manage small displacements and permanent slip (elastic and plastic behaviour) within the interface. The interface shear strength parameters are controlled by R_{inter}. Brinkgreve et al. (2012) suggested the following equations to use in the modelling:

$$C_{interface} = R_{inter}\, C_{soil} \quad (1)$$

$$\tan \bullet_{interface} = R_{inter} \tan \bullet_{soil} \quad (2)$$

The strength reduction factor was set to 0.8 as laboratory direct shear tests on interface parameter gave the same value. Drained analysis was carried out in the simulation. The soil suction did not vary during the pullout test which resulted in the above assumption. A dilation angle of 10% of friction angle was taken for the analysis (DS 415). Soil nail was modelled as an elastoplastic plate element as suggested by Babu and Singh (2009). The bending stiffness (EI) and the axial stiffness (EA) of the soil nail are 7.909×10^{11} Nmm^2/mm and 1.56×10^8 N/mm, respectively. The Poisson's ratio and unit weight used for the simulation of the nail is 0.3 and 0.024 N/mm/mm respectively. Figure 5.2 shows the complete nail pullout model.

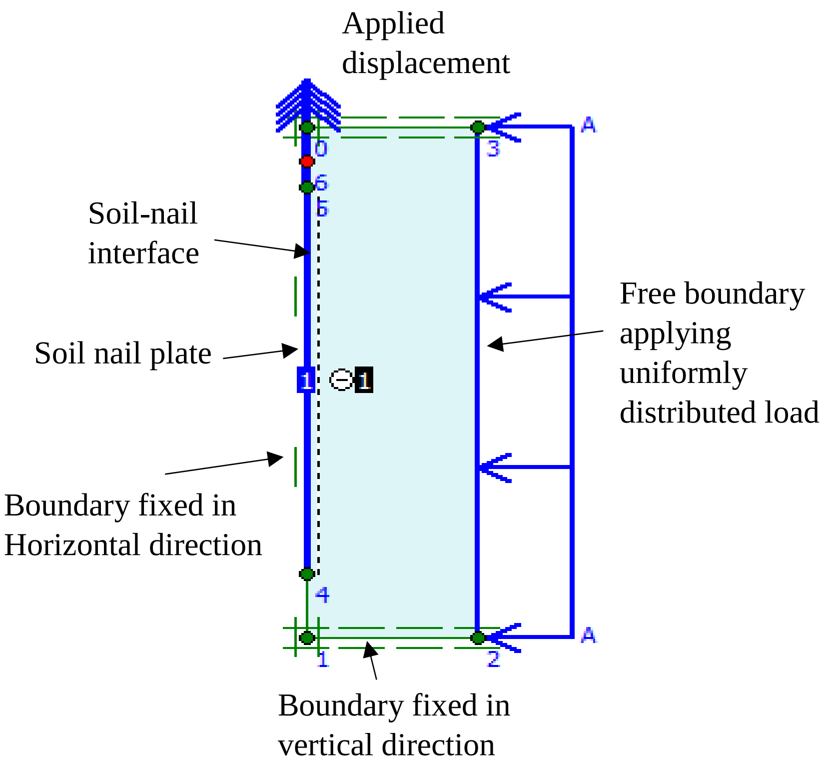


Figure 5.2 Complete model of soil nail Pullout

5.4 Simulation of Soil Nail Model

The phreatic line is positioned in the bottom of the numerical model to create drained conditions. Using Plaxis 2D, the soil stress variations surrounding the soil nail during drilling, grouting and saturation cannot be simulated. For the simulation, the known displacement from the laboratory pullout tests were the input while the corresponding failure loads were obtained as the output. Plastic calculation type was used and staged construction process was employed.

The stage construction used three steps in simulation as follows.

1. Activate the plate element (Soil nail)
2. Activate the distributed load (surcharge load)
3. Activate the prescribed displacement

For each pullout simulation, the displacements corresponding cohesion parameter and overburden pressure was changed. Table 5.3 shows the results of the pullout analysis from the Plaxis 2D and Figure 5.3 shows the axial force developed in the nail after the simulation at different matric suctions and overburden pressures.

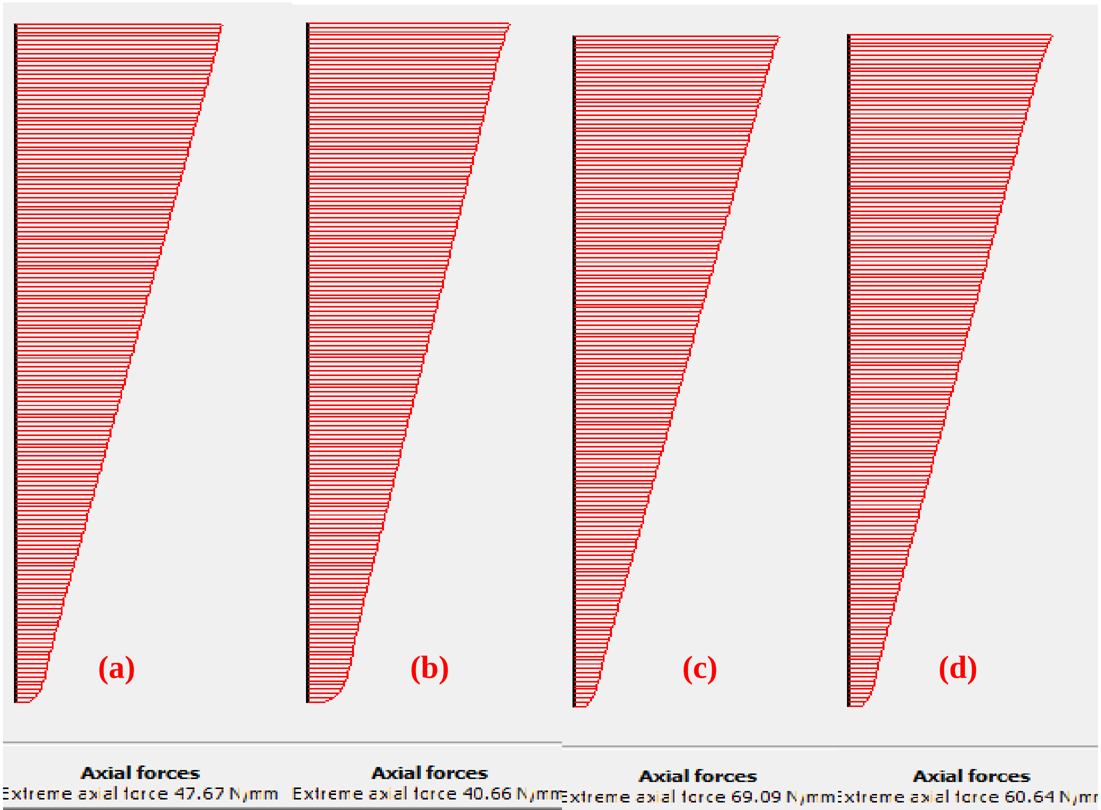


Figure 5.3 The developed axial force in the nail after the simulation at (a) $(U_a\text{-}U_w)$ =2.4 kPa and σ_v = 60 kPa (b) $(U_a\text{-}U_w)$ =7.67 kPa and σ_v = 30 kPa (c) $(U_a\text{-}U_w)$ =43 kPa and σ_v = 60 kPa (d) $(U_a\text{-}U_w)$ =46 kPa and σ_v = 30 kPa

Table 5.3 Finite element Pullout analysis results

Average Matric suction (kPa)	Applied Overburden pressure (kN/m^2)	Displacement (mm)	Measured Pullout Capacity (kN)	Output from FEM (kN)
2.40	60	5.86	6.55	6.22
43.00	60	8.12	9.81	9.00
7.67	30	4.53	5.84	5.30
46.00	30	4.65	9.05	7.91

The variation of axial load with displacement using plaxis output, experimental and proposed equation by Gurpersaud (2011) is shown in the Figure 5.4. The displacement simulation of pullout in plaxis 2D resulted in higher displacement than experimental. Pullout capacity obtained in the both methods are approximately equal.

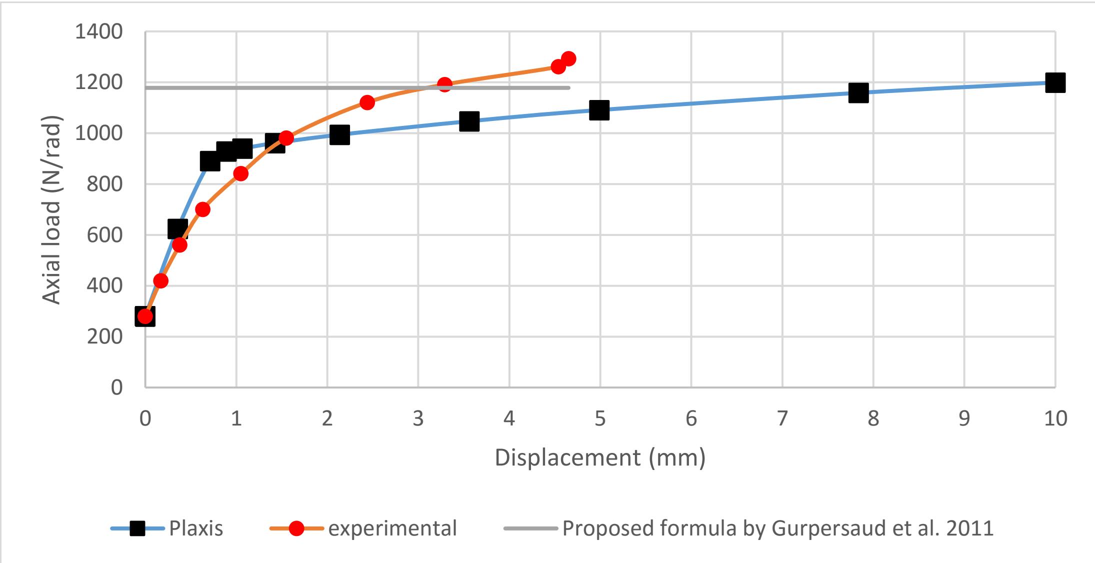


Figure 5.4 Variation of axial load with displacement from the results of the current study

CHAPTER 6 DISCUSSION OF THE TEST RESULTS

6.1 Introduction

The present study was carried out to investigate the influence of matric suction on pullout resistance of soil nails installed in a silty soil and to study the applicability of formula proposed by different researchers for the estimation of pullout capacity. A comprehensive experimental program was undertaken using a compacted residual soil. The results obtained from the test program and the interpretation are discussed in this chapter and the test results from the present study are also analyzed using other equations available in the literature to estimate the pull-out capacity.

6.2 Interpretation of the Test Results

6.2.1 Interpretation of the measured pullout resistance

To investigate the influence of matric suction on pullout capacity two tests were performed near saturated state and another two at a higher matric suction (near 40 kPa). The pullout results obtained were presented in the Table 6.1 under the different conditions. Under overburden pressure of 60 kPa, the measured pullout resistance at unsaturated state was observed to be 1.5 times greater than the nail pullout near saturated state. Similarly 1.55 times enhanced pullout capacity was observed at unsaturated state under 30 kPa of overburden pressure. This clearly states that the matric suction influence on pullout resistance of the soil nails. The Figure 6.1 shows the measured pullout capacity in a graphical plot.

Table 6.1 Measured pullout resistance under different conditions

Average Matric suction (kPa)	Applied Overburden pressure (kN/m^2)	Measured Pullout Capacity (kN)	displacement (mm)
2.40	60	6.55	5.86
43.00	60	9.81	4.53
7.67	30	5.84	8.12
46.00	30	9.05	4.65

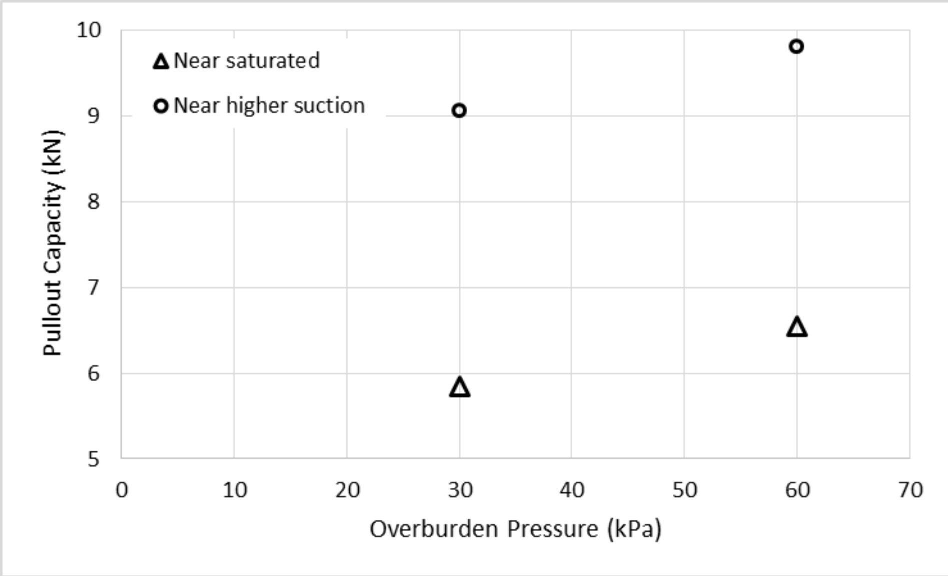


Figure 6.1 Measured pullout capacity under different overburden pressures

Apart from matric suction influence, the influence of overburden pressure was clearly observed near the saturated and unsaturated conditions. The peak pullout resistance of soil nails near saturated state under surcharge loading of 60 kPa was higher than surcharge loading of 30 kPa. Similar behaviour was observed at higher matric suction of 40 kPa as presented in Figure 6.2.

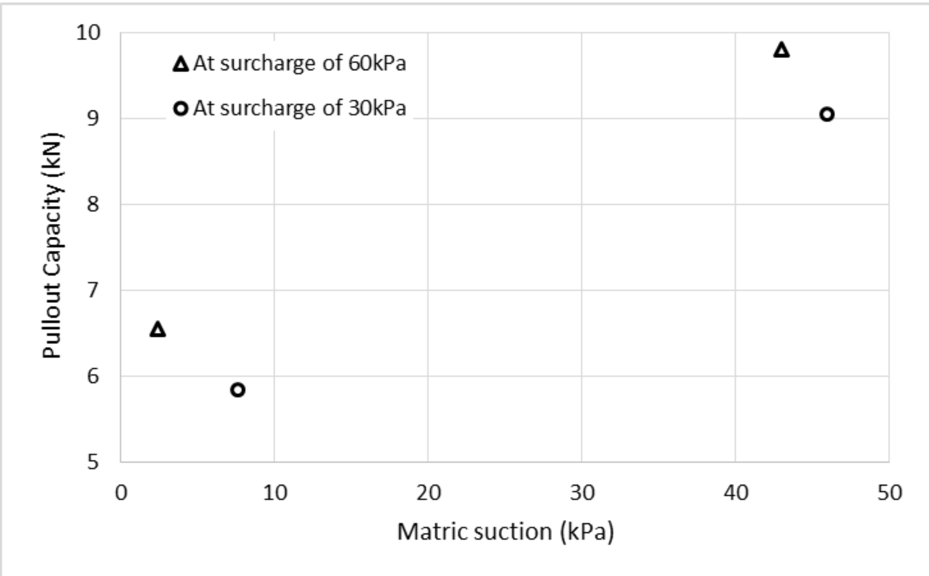


Figure 6.2 Measured pullout capacity at different matric suctions

6.2.2 Creep test results

Creep test consists of measuring the movement of the soil nail at a constant load over a specified period of time (holding period of 10-60 minutes). This test is performed to ensure that the nail can safely carry the designed load throughout the service life with low displacement under constant load. This test was performed at 0.75 Design Load for each soil nail. Table 6.2 shows the results of the nail displacement at 6 and 60 minutes. The soil nail can prevail safely if the creep displacement is less than 2 mm or 0.1% of the bond length (0.001 × 780 mm). The creep displacement obtained for each nail is less than 0.78 mm.

Table 6.2 Nail displacement at 6 and 60 minutes at 0.75 Design load

Average Matric suction (kPa)	Applied Overburden pressure (kN/m^2)	Displacement at 6 minutes (mm)	Displacement at 60 minutes (mm)	Creep displacement (mm)
2.40	60	2.87	3.31	0.44
43.00	60	1.20	1.54	0.34
7.67	30	2.96	3.23	0.27
46.00	30	1.05	1.26	0.21

6.2.3 Effective diameter of the soil nails

Auger of diameter 75 mm was used for the drilling process but there was an increase of 12.16 % in the diameter of the completely pullout nail. Similar behaviour was observed by Ranjan Kumara and Kulathilaka (2016), Gurpersaud et al. (2011). The penetration of grout into the soil and the expansion of grout during hardening are the main reasons for the observation.

6.2.4 Strength reduction factor determination

To estimate the strength reduction factor, the interface shear stress at failure and direct shear stress at failure were used at saturated condition and under two overburden pressure. The Table 6.3 shows the determined values for strength reduction factor. In both cases strength reduction factor was found to be 0.8. Thus this value was used in the FEM using Plaxis 2D

Table 6.3 Strength reduction factor at saturated state

Overburden pressure (kPa)	Shear stress soil-soil (kPa)	Shear stress soil-grout (kPa)	Strength reduction factor
30	24.359	19.686	0.808
60	35.247	28.822	0.818

6.2.5 Comparison of pullout with direct shear tests at saturated state

Both direct shear test and pullout test mechanisms had similar behaviour where shear resistance was mobilized gradually with the shear displacement. Thus the comparison of direct shear test and interface shear test with pullout test provide some insight of pullout mechanisms. So it was decided to plot all three test results together. A similar comparison was done by Chu and Yin (2005) and Ranjan Kumara and Kulathilaka (2016). The loads at different stages was converted in the pullouts to shear stresses by dividing it by the effective nail surface area. The Figure 6.3 and Figure 6.4 represent graphically the pullout test results compared with direct shear test at 30 kPa and 60 kPa overburden pressure. The value obtained for pullout failure is much greater than direct shear value as the matric suction at this particular graph is 7 kPa. As such, gave an enhanced shear stress.

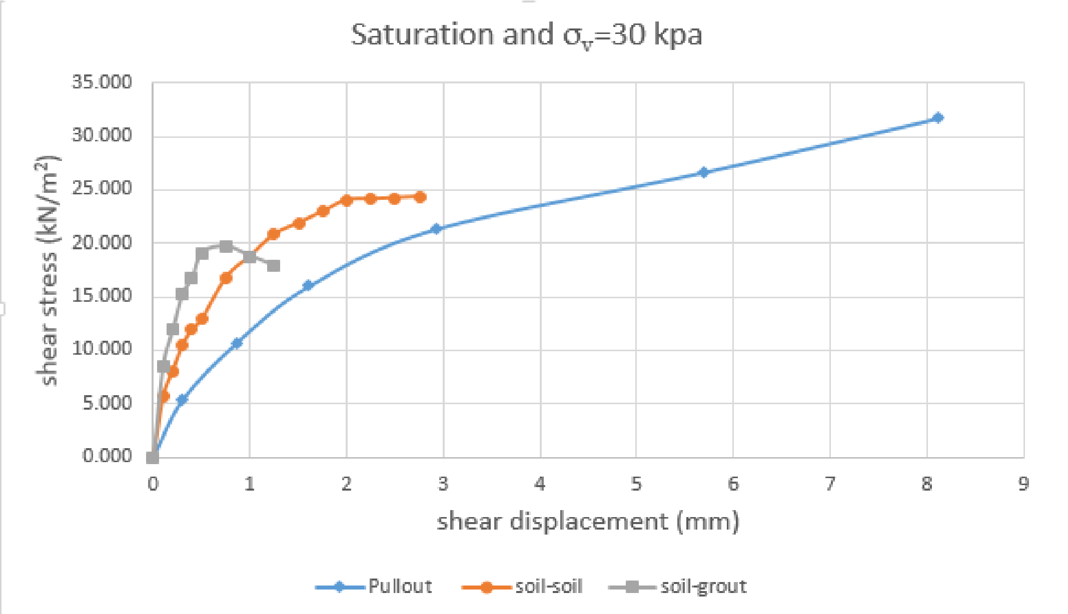


Figure 6.3 Comparison pullout test with direct shear test at overburden pressure of 30 kPa

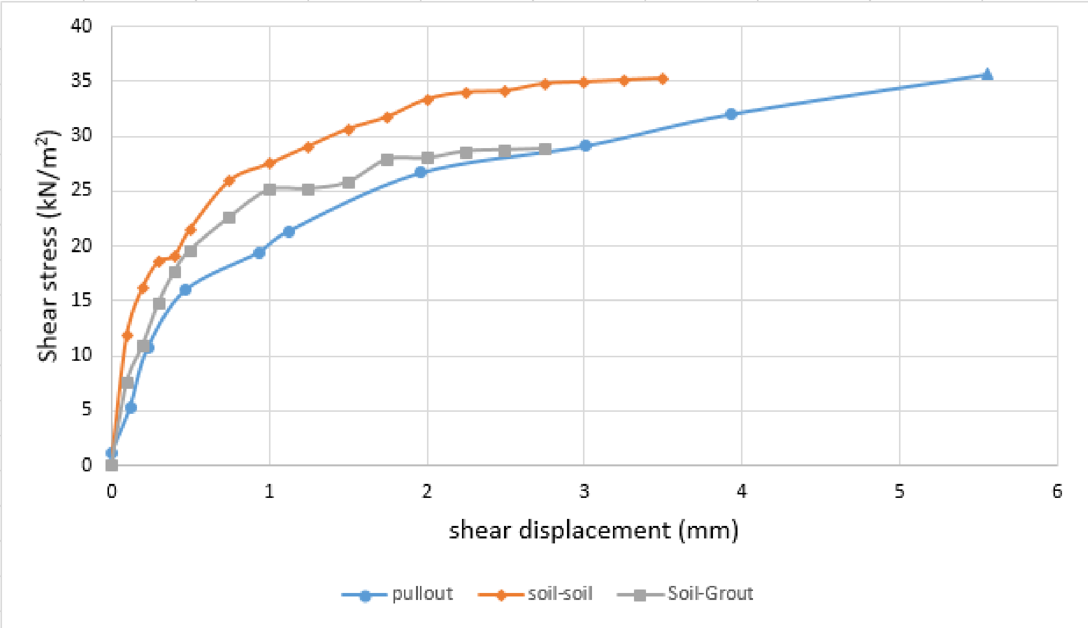


Figure 6.4 Comparison pullout test with direct shear test at 60 kPa

The shear strength failure envelopes for pullout tests and interface shear tests show similar trends as soil –soil direct shear test. The failure displacement for pullout is greater than direct shear displacement as direct shear uses a small specimen. But in pullout to mobilize the shear stress nail will displace more. Further the shear stress at failure for soil-soil interface is observed to be greater than interface shear stress at failure in both tests.

6.3 Estimation of pullout resistance

Through the complete study of literature it is well understood that a number of different formulae are used at present by design engineers to estimate the pullout resistance in soil nailing. Some of those methods are used in this study to estimate the pullout resistance. This comparison between the estimated pullout capacities by different formulae with measured pullout resistance provides a clear idea about the most appropriate formula that can be used in the estimation of the pullout capacity. Some of the commonly used equations in the literature are discussed next;

6.3.1 Equation proposed by Schlosser and Guilloux (1981)

The equation proposed by Schlosser and Guilloux (1981) has been adopted in Hong Kong to estimate the ultimate pull-out resistance of grouted soil nails
(Watkins and Powell, 1992).

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

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 • '.)

This equation does not account for the effects of matric suction and dilation.

6.3.2 Equation proposed by Chu and Yin (2005)

The following equation was proposed by Chu and Yin (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

This equation does not account for the effects of matric suction and dilation on the pull-out capacity of soil nails.

6.3.3 Equation proposed by Zhang et al. (2009)

Zhang et al. (2009) extended the formulae 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.

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

Where:

D = diameter of grouted nail

$(u_a - u_w)$ = matric suction

- ^{b} = internal friction angle with respect to soil suction

υ= Poisson's ratio

ψ = dilation angle

K_o = coefficient of earth pressure at rest

6.3.4 Equation proposed by Gurpersaud et al. (2011)

Gurpersaud et al. (2011) extended the formula proposed by Vanapalli et al. (2010) to estimate the carrying capacity of pile shaft. The equation proposed by Gurpersaud et al. (2011) accounts for both matric suction and dilation. They have shown that this equation can be used to estimate the pullout capacity of soil nails installed in the sandy soils.

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

Where,

$\beta = k_\theta \tan(\delta+\psi)$, $k_\theta/k_0 = 1+(1-k_0)/2k_0 \times (1/\cos 2\theta)$

$Q_{F(us)}$ = ultimate pull-out resistance (kN)

β = Bjerrum-Burland coefficient

S=degree of saturation

c_a = apparent cohesion of the soil

K=fitting parameter

ψ = dilation angle

θ= angle of cut slope

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

δ= Interface friction angle

k_{θ}= Coefficient of lateral earth pressure,

k_0= coefficient of lateral earth pressure at rest

L=effective length of the soil nail

d=diameter of the nail

The fitting parameter (K) for compacted soils can be expressed as follows (Garven and Vanapalli, 2006)

$K = -0.0016 (I_p^2) + 0.0975 (I_p) + 1$

Where, I_p – plasticity index

6.3.5 Conventional formula

This formula is widely used in Sri Lanka to estimate the pullout capacity of soil nails. This was an extended formula of HA 68/94 (1994).

$$Q_{f(us)} = \{C_a + \sigma_{ave} \tan\delta\}\pi dL$$

Where,

$Q_{F(us)}$ = ultimate pull-out resistance (kN)

c_a = apparent cohesion of the soil

σ_{ave} = average of effective vertical and horizontal stress

δ= Interface friction angle

L= effective length of the soil nail

d= diameter of the nail

6.3.6 Estimation of pull-out capacity using equations available in literature

Table 6.4 presents a summary of the results obtained by using the different equations described under sections 6.3.1 - 6.3.5. A comparison of the estimated and measured results for the soil nails are presented graphically in Figures 6.5. Table 6.5 shows the estimated value as a percentage of measured value.

Table 6.4 Summary of the results obtained by using the different equations

Average Matric suction (kPa)	Applied Overburden pressure (kN/m^2)	Measured Pullout Capacity (kN)	Proposed formula by Gurpersaud et al. (2011) (kN)	Zhang et al. (2009) (kN)	schlosser and Guilloux (1981) (kN)	Chu and Yin (2005) (kN)	conventional formula (kN)
2.40	60	6.55	6.69	7.50	9.85	7.14	7.07
43.00	60	9.81	9.33	12.87	9.85	7.14	7.07
7.67	30	5.84	5.89	6.69	7.36	5.63	5.52
46.00	30	9.05	8.25	11.68	7.36	5.63	5.52

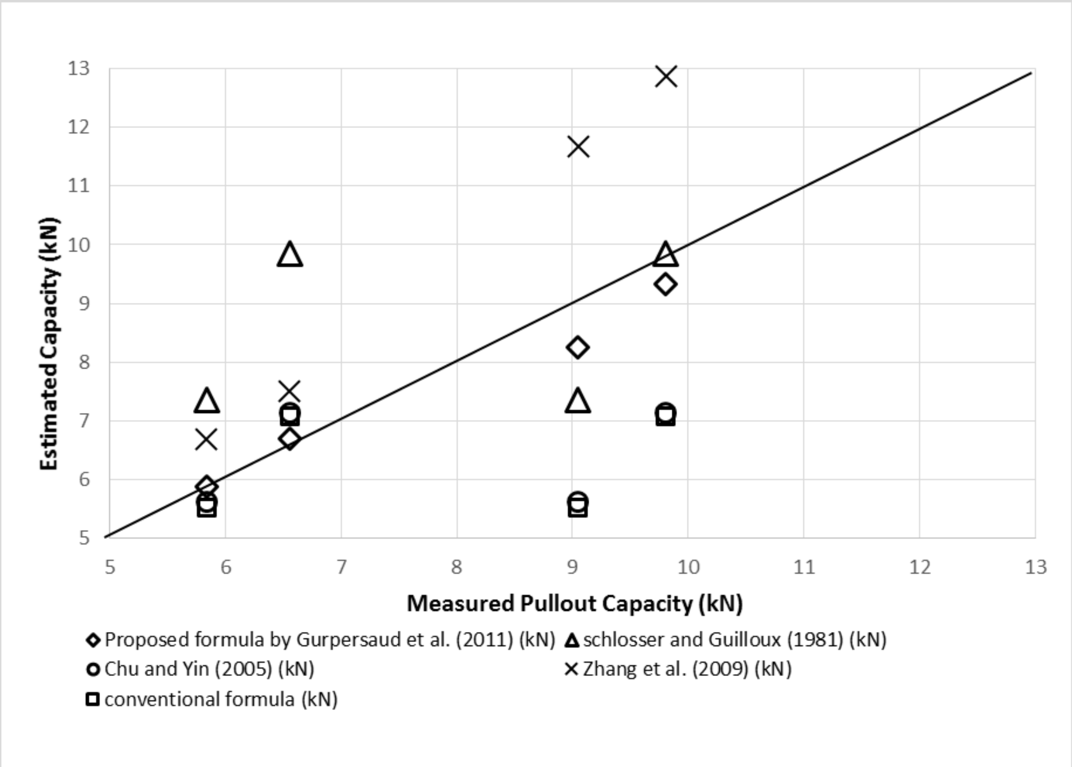


Figure 6.5 Comparison of the estimated and measured results for the soil nails

Table 6.5 Estimated value compared to measured pullout in percentage

Average Matric suction (kPa)	Applied Overburden pressure (kN/m^2)	Proposed formula by Gurpersaud et al. (2011) (kN)	Zhang et al. (2009) (kN)	schlosser and Guilloux (1981) (kN)	Chu and Yin (2005) (kN)	conventional formula (kN)
2.40	60	102	114	150	109	108
43.00	60	95	131	100	73	72
7.67	30	101	115	126	96	95
46.00	30	91	129	81	62	61

All estimates under unsaturated conditions are greater than the soil nails pulled out near saturated condition. The comparison of test results presented in Figure 6.5, indicates a good agreement in the estimation of pull-out capacities using Gurpersaud et al. (2011) and measured values. The formula proposed by Zhang et al. (2009) accounts for the matric suction and dilation but deviates significantly from the measured values as it include both soil strength parameters and interface strength parameters.

The proposed formula by Schlosser and Guilloux (1981) over estimates the pullout capacity even at saturated states as they used soil strength parameters instead of interface parameters to estimate the pullout capacity. But the pullout capacities obtained from the formula proposed by Chu and Yin (2005) and conventional formula are quite close to the measured values under saturated condition and did not change under 40 kPa matric suction as these formulae do not accounts for the influence of matric suction and dilation.

Table 6.6 clearly depicts that the estimated values obtained from Gupersaud et al. (2011) are very close to the measured value. From this it can be concluded that the formula proposed by Gurpersaud et al. (2011) can be used to estimate the pullout capacity in silt with high plasticity soil. The Figure 6.6 compares the results of Gurpersaud et al. (2011) and finite element analysis output with measured capacity. However, the proposed formula by Gurpersaud et al. (2011) needs to be simplified as such the commonly practicing engineers can use. In order to develop simplified formula further pullout testing and interface direct shear testing with different soil type needs to be done.

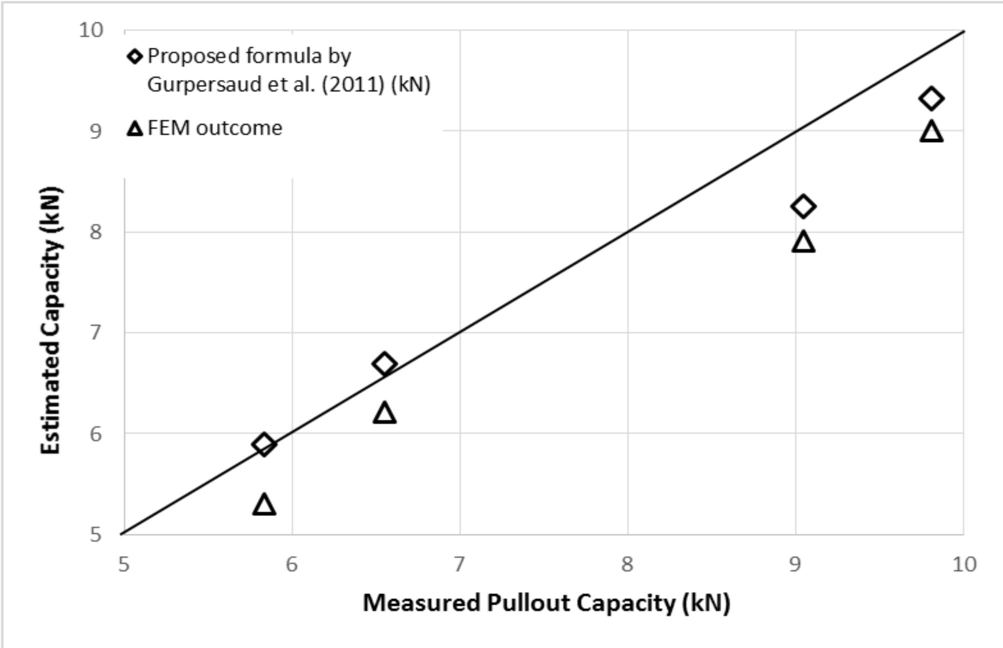


Figure 6.6 Comparison of results of Gurpersaud et al. (2011) and finite element analysis output with measured capacity.

6.3.7 Estimation of pullout resistance with interface shear strength parameters at matric suction of 40 kPa

Conventional formula is often used in the estimation of pullout resistance. As described earlier it doesn't account for the influence of matric suction. The evaluated shear strength parameters from the interface shear test performed at a matric suction of 40 kPa was used in the convectional formula to estimate the pullout capacity. The conventional formula was modified as follows;

$$Q_{f(us)} = \{C_a + \sigma_{ave} \tan\delta\}\pi dL$$

Where,

$Q_{F\,(us)}$ = ultimate pull-out resistance (kN)

c_a = adhesion at the matric suction of 40 kPa

σ_{ave} = average of effective vertical and horizontal stress

δ= Interface friction angle at the matric suction of 40 kPa

L= effective length of the soil nail

d= diameter of the nail

The evaluated values were used in the formula to compare with the pullout results and to investigate the adaptability of modified conventional formula which is much simpler than Gurpersaud (2011) to estimate the pullout capacity of soil nails installed in unsaturated embankments. Table 6.6 shows the estimated results from modified conventional formula with pullout results.

Table 6.6 Estimated results obtained from modified conventional formula

Overburden Pressure (kPa)	Modified Conventional formula (kN)	Laboratory Pullout results (kN)
30	10.18	9.05
60	11.81	9.81

The larger estimated value is as a result of the larger apparent adhesion recorded in the tests. This adhesion is smaller than the apparent cohesion of the compacted soil at a matric suction of 40 kPa. The modified results obtained at a matric suction of 40 kPa is greater than the pullout capacity obtained at the same. The comparison of the above table clearly depicts that modified formula gave an overestimated values. More tests need to perform at different matric suction with known pullout capacities to check the applicability of the modified conventional formula into practice so that the complex design formulae can be omitted.

CHAPTER 7 CONCLUSION AND RECOMMENDATION

When soil nailing is used as a stabilization technique attempts are made to lower the ground water table (GWT). Improvements are done in surface and sub-surface drainage leading to lowering of GWT. With these measures nails would prevail under significant matric suction conditions and the study of the influence of matric suction on pullout resistance is particularly important. Significant advancement were made during the last decades to incorporate the matric suction in the pull-out formula which was neglected in the past. The enhanced pullout resistance due to matric suction was shown by several researches. Gurpersaud et al. (2011) modified beta formula developed by Vanapalli et al. (2010), to find a reasonable approximation to the pull-out capacity.

In this study an experimental program was undertaken to show the influence of matric suction in pull-out resistance and to check the applicability of formula proposed by different researchers in residual soil (soil with high plasticity classified by USCS). The measured pull-out capacity and estimated capacity by the formula proposed by Gurpersaud et al. (2011) were very close and the measured pull-out capacity used in the study in compacted unsaturated state was found to be more than 1.5 times higher than the pull-out capacity near saturated condition. Further, numerical analysis by Plaxis 2D showed an increase in pullout capacity with the increase in matric suction.

With the limitation of matric suction measurement using the tensiometer (up to 86 kPa) and difficulties in maintaining the matric suction profile, the testing of pullout was limited to two; near saturated state and near 40 kPa suction. Further studies should be conducted by developing tensiometer with higher capacity of measuring matric suction.

The study should be extended to different soil types compacted under laboratory conditions. Thereafter it should be extended to an actual field situation measuring the matric suction and getting the soil parameters corresponding to the soil in the region where the pullout resistance is developed.

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