

Parametric Study on Soil Nailed Deep Excavation Wall using FEM in Case of Addis Ababa City (Clay Major Soil)



Samrawit Leul Araya

A Thesis Submitted to the Department of Civil Engineering

School of Civil Engineering and Architecture

Presented in Partial Fulfillment of the Requirement for the Degree of Master's in
Geotechnical Engineering

Office of Graduate Studies

Adama Science and Technology University

June 2022

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Approval sheet

I/we, the advisors of the thesis “**Parametric study on soil nailed deep excavation wall using FEM in case of Addis Ababa city (clay major soil)**” and developed by **Samrawit Leul** hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

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We, the undersigned, members of the Board of Examiners of the thesis by **Samrawit Leul** have read and evaluated the thesis entitled “**Parametric study on soil nailed deep excavation wall using FEM in case of Addis Ababa city (clay major soil)**” and examined the candidate during open defense. This is, therefore, to certify that the thesis is accepted for partial fulfillment of the requirement of the degree of Master of Science in Geotechnical Engineering.

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Recommendation

I/we, the advisor(s) of this thesis, hereby certify that I/we have read the revised version of the thesis entitled “**Parametric study on soil nailed deep excavation wall using FEM in case of Addis Ababa city (clay major soil)**” prepared under my/our guidance by **Samrawit Leul** submitted in partial fulfillment of the requirements for the degree of Master’s of Science in Geotechnical engineering. Therefore, I/we recommend the submission of revised version of the thesis to the department following the applicable procedures.

Major Advisor

Signature

Date

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ACRONYMS

LEM	Limit Equilibrium Method
FEM	Finite Element Method
FEA	Finite Element Analysis
FS	Factor of Safety
FHWA	Federal High Way Administration
2D	Two Dimensional
MC	Mohr columb
FDM	Finite Difference Method
SSR	Shear Strength Reduction
SPT	Standard penetration test

ABBREVIATIONS

Ag	Cross Sectional Area of Grouted Soil Nail
B	Strain Interpolation Matrix
C	Cohesion
Cu	Undraind cohesion
d	Thickness of the Wall
DDH	Diameter of Drilled Hole
deq	Equiivalent Plate Thickness
E	Elastic Modules
EA	Axial stiffness
Eeq	Equiivalent Modules of Elasticity
Eg	Modules of lasticity of Grout
EI	Bending stiffness
En	Modules of Elasticity of Nail
K	Element Stiffness Matrix
Lp	Pullout Length
L _s	Space between nails
N	Number of Blows
q	Mobilized Shear Stress
Q	Load Transfer Rate
Qu	Pullout Capacity Per Unit length
Rp	Pullout Capacity
Sh	Horizontal Spacing of Nail
S _v	Nail vertical Spacing
To	Tensile force
UX	Nodal Movement to The x-axis
γ	Bulk unit wieght
γ_{sat}	Saturated unit wieght
Φ	Soil Angle of Internal Friction
ψ	Dilatancy Angle

D^e	Elastic Material Matrix
c'	Effective Cohesion of Soil
σ_n	Vertical Stress
τ_f	Shear Strength of Soil
α	Nail Inclination

ABSTRACT

Urbanization is one of the major driving forces behind the formation of today's land use systems. In order to decrease the land use for urban areas, basements are constructing widely. The excavation is deep for the basements and it induces lateral displacement in the soil mass. When the soil displaces more than allowable displacement failure will occur. Soil nailing stabilization method is practicing now a days in the developed world. The objective of this research was to conduct parametric study on soil nailed deep excavation wall. This parametric study was conducted to evaluate the effect of nail spacing, nail inclination, surcharge and depth on the lateral displacement and to compare the maximum lateral displacement of nailed wall and only diaphragm wall supported face for two (6m), three (9 m) and four (12 m) basements using FEM based software (plaxis 2D.v8.2) for static loading in case of Addis Ababa (clay major soil). The soil properties needed for the analysis were found from SPT correlations which were collected from organizations. The results show that there was influence of nail geometry (spacing and inclination), surcharge and depth on the maximum lateral displacement of excavation wall face. However, depth had significant effect when the surcharge was greater than 50 kPa. Excavation depth of 6 m was not capable of carrying load for more than 155.6 kPa which corresponds to maximum lateral displacement of 12mm. The 9-m depth of excavation can support a surcharge of up to 225 kPa which corresponds to the maximum lateral displacement of 18mm. The 12 m depth of excavation can carry a load of up to 286 kPa which corresponds to maximum lateral displacement of 24mm, but not more since they displace greater than the allowable displacement. The maximum lateral displacement increases with increase in the nail inclination and nail spacing. At 6 m the excavation depth can carry load up to 118 kPa without nailing (only diaphragm wall). At 9 m the excavation depth can carry load up to 130 kPa without nailing (only diaphragm wall). At 12 m the excavation depth can carry load up to 142.7 kPa without nailing (only diaphragm wall supported).

Keywords: *Plaxis 2D, Slope stability, FEM, Soil nailing.*

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CHAPTER ONE

INTRODUCTION

1.1 Background

One of the key driving forces behind the construction of today's land use systems is urbanization. It always involves changing non-urban to urban land use. Excavations for residential and commercial buildings have become increasingly significant as a result of urban development, city land size constraints, and population growth. Urban sprawl, a relatively widespread form of land take for urban uses with environmentally harmful impacts, has been labeled for much of the current temporary urbanization. However, urban land use change can take many different forms in terms of layout, building density, and change speed, to mention a few. Researchers have made significant progress in empirically addressing the many forms of urban land use and their change through time in recent decades. Now a day, in order to decrease the land use for urban areas basements are constructing (Rashidi, et al 2018). The basement excavation is deep as a result there will be slope instability as the excavation progresses.

Changes in stress conditions, an increase in the groundwater table, and rainfall can all contribute to slope instability. Similarly, many slopes that have been stable for years can suddenly fail due to changes in geometry, poor soil shear strength, or the action of external forces. Debris flows (also known as slope failures) occur without warning and can be devastating. As a result, it is essential to consider the mechanism of slope failure and apply protective measures (Digvijay et al., 2017).

For a deep excavation, the term "Retaining System" refers to the structural system that keeps earth and water from falling into the open cut. There are numerous alternatives for retaining systems. The best retaining system is determined by a variety of considerations including cost, soil conditions, neighboring structure protection, ease of construction, environmental concerns, and more. One of the most common soil slope stabilization techniques is soil nailing.

Soil nail walls are particularly well suited to excavation applications when vertical or near-vertical cuts are required. When the stability of natural or excavated soil slopes is considered inadequate, it is an effective technique for strengthening. Nails are used because they are semi-rigid passive elements that can withstand tension, shear, and bending pressures. Within the soil mass, the reinforcement elements are positioned nearly horizontal. The internal forces in the nails are induced by lateral displacements in the outer direction generated by progressive unloading of soil material caused by staged excavation. Their purpose is to reduce these displacements by introducing internal resistive forces (Lima et al., 2004).

Natural slopes and earth retaining structures have long been stabilized with soil nailing. Because of the interacting effects of the soil, nails, grout, and shotcrete on the mechanism of this stabilizing system, numerical modeling with high accuracy should be used to analyze the behavior of the soil-nailed walls (Singh, 2009).

1.2 Statement of the problem

One of the key driving forces behind the construction of today's land use systems is urbanization. Excavations for residential and commercial buildings have become increasingly significant as a result of urban development, city land size constraints, and population growth. Now a day, in order to decrease the land use for urban areas basements are constructing. The basement excavation is deep. As a result there will be slope instability as the excavation progresses. As a result, it is essential to consider the mechanism of slope failure and apply protective measures. In Ethiopia soil nailing are rarely used. But in the developed world, it is practicing very well due to many advantages.

There are different soil nailing standard codes. But there is no code for soil nailing in Ethiopia. This parametric study is proposed to analyze and specify nail inclination and nail spacing at different depths for different surcharges for basements in Addis Ababa clay major sites. This will be one step to introduce soil nailing to the construction industry since urbanization and land usage is main concern now a time.

1.3 Objective of the study

1.3.1 General Objective

The general objective of this study was to conduct parametric study on soil nailed deep excavation wall.

1.3.2 Specific objectives

The specific objectives of this study are to;

1. Evaluate the effect of variation of nail geometry (nail spacing and inclination), and depth on lateral displacement of nailed wall.
2. Evaluate the effect of variation of surcharge on lateral displacement of nailed wall.
3. Compare the change of maximum lateral displacement with nailing and without nailing (only diaphragm wall) for each depth.

1.4 Scope and Limitation of the work

The scope of this parametric study is limited to the provision of adequate support (specifying nail inclination and nail spacing) to deep excavation using nailing for basements in Addis Ababa with clay major heterogeneous soil considering only the static loading. The analysis was carried using FEM based software (plaxis 2D) on deep excavation for two (6 meter), three (9 meter) and four (12 meter) basements. Suitable data for numerical evaluation was obtained using SPT correlations which were gathered from organizations and some literatures. The performance of the nailing at selected depths was evaluated by comparing the lateral displacement with the allowable lateral displacement of nailed wall using plaxis 2D software. The study is worked only for clay type of soil. As a result, the work's limitation is that it is only applicable to clay soils.

1.5 Significance of the Study

Slope stability analysis is performed to assess the safe design of a human-made or natural slope such as embankments, road cuts, open-pit mining, excavations and landfills. This study will have a valuable significance to understand the effect of variation of nail inclination, nail spacing, surcharge and depth on the lateral displacement of the reinforced soil wall.

Furthermore, this study will provide adequate nail spacing and inclination for two (6 meter), three (9 meter) and four (12 meter) basements in Addis Ababa clay major soil. So engineers who constructs basement in Addis Ababa with clay major site can use the specified nail inclination and nail spacing as a starting point for the design, since there is no standard code for soil nailing in Ethiopia.

CHAPTER TWO

LITERATURE REVIEW

2.1 General

Terzaghi and Peck (1967) define deep excavation as any excavation that extends above 6-7 meters. A deep excavation, according to certain unpublished literature, is one that extends beyond 4.5 meters. For a variety of reasons, deep excavations are more difficult to design and execute than shallow excavations, which is why they are frequently undertaken by skilled specialists. Practical engineers are concerned about the stability of deep excavations. Excavation failure is characterized by the collapse of the support structure and the displacement of the surrounding soil. As a result, this section covered the reasons and mechanisms of failure, analysis methodologies, soil nailing, and plaxis software application.

2.2 Factors influencing stability of deep excavation wall

Slope stability is influenced by a number of factors. A change in any one of these characteristics, or a combination of them, could have an impact on the slope's steady state condition, reducing its stability and perhaps leading to slope failure. A relatively sudden triggering event of natural (earthquake, soil saturation) and anthropogenic activity may produce destabilization while the slope is at a critical stage of stability. Factors affecting the stability of deep excavation wall listed in the following section.

2.2.1 Internal factors

1. Increase in pore water pressure which reduces shear strength of the material.
2. Material properties such as, shearing strength of slope material.
3. Freezing and thawing soils.

2.2.2 External factors

1. Undercutting of slopes due to river erosion, quarrying, excavation for canals and roads.
2. Surcharge loads like traffic loads, buildings, reservoir, and alluvium accumulation on slopes.
3. Infiltration of rain fall which increases the unit weight of slope material & pore water Pressure.
4. Land use change like deforestation.

2.3 Finite Element Method

Finite Element Analysis (FEA) is a numerical method used to solve engineering and mathematical physics issues. It's useful for problems with complicated geometries, loadings, and material properties that can't be solved analytically. Interconnecting sub-regions or elements are used to approximate the FEM solution. Typically, these elements are simply geometrical shape. This flexibility in construction of elements allows FEM to accurately model complex geometries. (Safuan et al., 2013a). In finite element method the critical surface is automatically find out by various software's (Digvijay et al., 2017). The FE approach is used to model slopes with a high level of realism (complex geometry, loading sequence, presence of reinforcement material, water action, and complex soil behavior rules), as well as to better visualize the deformation of soil in place. The finite element method can be used to simulate progressive failure (Digvijay et al., 2017). The finite element method (FEM), also known as finite element analysis (FEA), is a computational methodology that is used in engineering to generate approximate solutions to boundary value issues (Bathe, 2016).

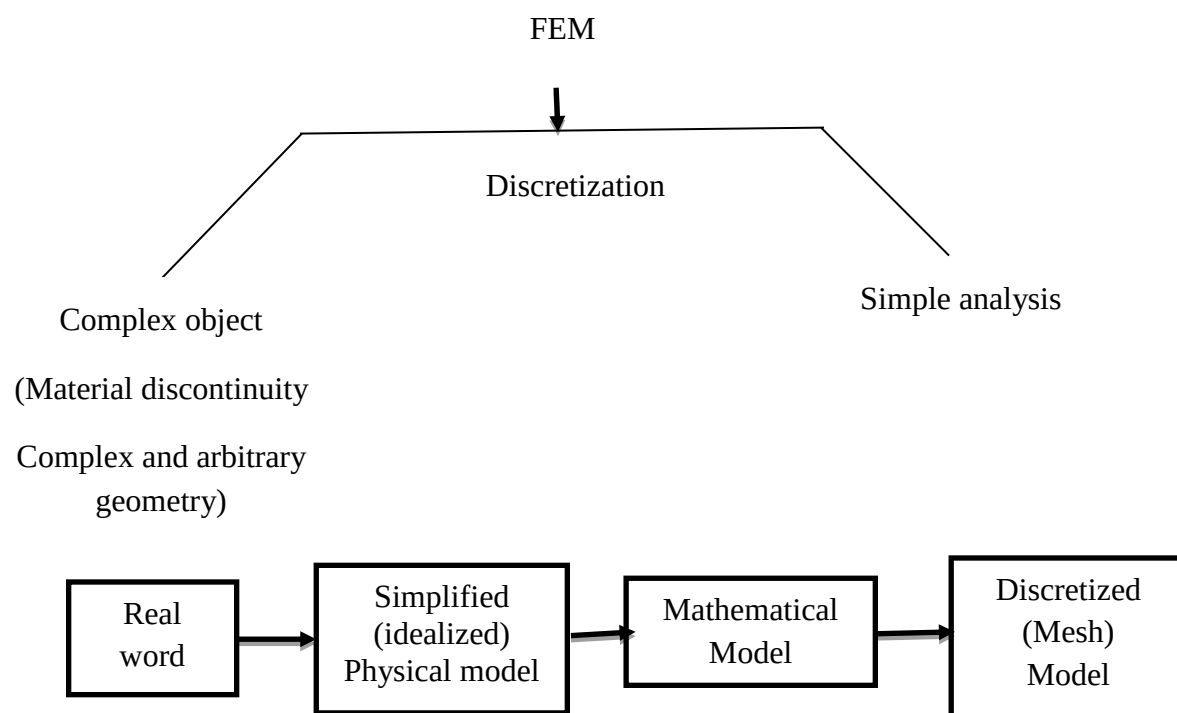


Figure 2. 1 Finite element analysis steps (Bathe, 2006)

FEM is a modern and powerful computational method in engineering that can replicate the physical behaviors of a problem without the requirement to simplify it using advanced computational tools. The ability to analyze any slope geometry in two and three dimensions, to analyze both linear and nonlinear problems, to provide FS without making assumptions about the shape and position of the sliding plane, to use various soil material models, and to consider the material's stress-strain relationship make FEM a preferred analysis (Kramer, 1996).

FEM also provides data on deformations at various operating stress levels and can track progressive failure, including total shear failure. For slope stability analysis using the finite element method, the stress rise and strength reduction approaches are often utilized. The gravity force is gradually increased until the slope fails in the Stress Increase method. The ratio of gravitational acceleration at failure to the actual gravitational acceleration is used to calculate FS. The soil strength parameters are reduced until the slope becomes unstable in the strength reduction method, and FS is calculated as a ratio of the initial strength parameter to the critical strength parameter. Strength decrease has no effect on the strength of structural objects like plates and anchors. The factor by which the initial shear strength parameters must be split to get the slope to the point of failure is defined as FS of a slope in this method (Burman et al., 2015).

2.4 soil Nailing

Soil nailing is a ground stabilization technique for improving slope and face stability by inserting soil nails into "in-situ" ground, which may be natural or deposited by man. The shear strength of soil, which is naturally weak in tension, can be improved by correctly aligned soil nails. When surrounding soil shears against relatively inextensible soil nails, frictional forces are mobilized. This results in:

1. An improvement of the slope's mass shear resistance; and
2. Development of tensile forces in the soil nails.

Depending on the soil nailing application, the soil nails can rely on other elements such as bearing pads, facing systems and drainage to be most effective. The effectiveness and efficiency of a soil nailing application is highly dependent upon the ground conditions. Soil nailing may be used to stabilize slope angles up to the vertical.

The success of a soil nailing scheme relies on the ability to form an adequate bond between the nail and the soil into which it is installed; for soil nailing to be economical, the soil should have sufficient shear strength to develop such a bond.

2.4.1 Soil Nail Advantages

Soil nail walls exhibit numerous advantages when compared to ground anchors and alternative top down construction techniques. Some of these advantages are described below (Piyush Sharma, 2015)

- Less disruptive to traffic and causes less environmental impact compared to other construction methods.
- Provide a less congested work place, particularly when compared to braced excavation
- Soil nail installation is relatively rapid and uses typically less construction materials
- Nail location, inclination, and length can be adjusted easily when obstructions (cobbles or boulders, piles or underground utilities) are encountered. On the Other hand, the horizontal position of ground anchor is more difficult to modify almost field adjustment.
- It is ideal for tight spaces.
- It can be used in irregular shapes.
- Minimum shoring is required.
- Lower load requirements than tieback anchors systems.
- Eliminates the time and expense of placing steel piles.
- Can be used to repair other existing wall systems.
- It can be used on new constructions, as temporary structures or on remodeling process.
- Wall height is not restricted.

2.4.3 Location of Critical Potential Failure Surface

Because the anchor bond zone must be situated sufficiently beyond the critical potential failure surface so that load is not transferred from the anchor bond zone into the "no-load" zone, the location of the critical potential failure surface must be evaluated. The "no-load" zone, also known as the unbonded length, is defined as the space between the critical potential failure surface and the wall. The unbonded length is usually extended a minimum distance of $H/5$, where H is the wall's height (Allerberger et al., 1996).

The distribution of tensile pressures in the soil nails varies over the wall system's cross-section. Because of the intricacies of load transfer inside individual nails, the position of maximum nail tensile pressures is often near to, but not always identical to, the critical failure surface discovered during global stability analysis. The maximum tensile force occurs behind the wall facing roughly $0.3 H$ to $0.4 H$. (Panigrahi, 2019). By comparing the given values for the unbonded length the maximum which is 4.5 m is used for all analysis.

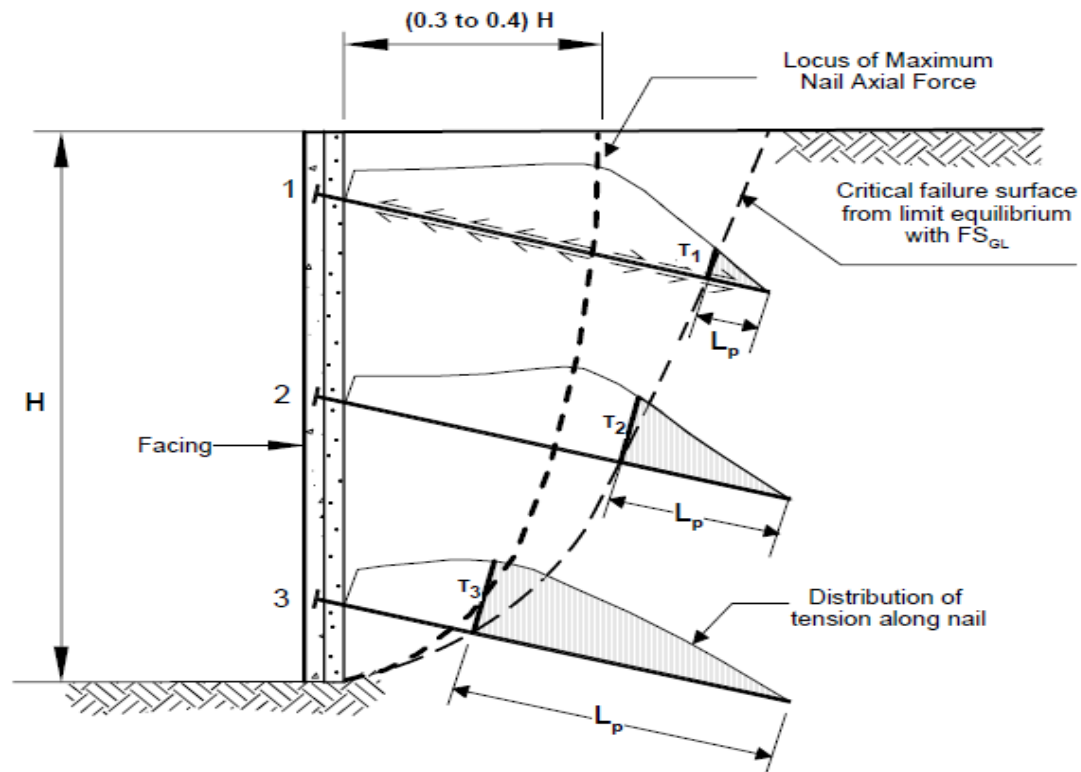


Figure 2. 2 Schematic Location of Soil Nail Maximum Tensile Force (FHWA, 2015)

The following recommendations are made by FHWA concerning soil nail length and distribution.

- Select uniform length pattern whenever possible.
- Select longer nails than required by the target factor of safety as a means to reduce wall Deformations in the upper portions of the wall.
- Avoid the use of “short” nails in top portion of wall
- Non-uniform nail length patterns may be used if soil layers with very dissimilar conditions are encountered

2.4.4 Steps to Design the Nail Bond Length

Steps for the design of nail bonded length is specified by FHWA. Pullout failure is the primary internal failure mode in a soil nail wall. This failure mode may occur when the pullout capacity per unit length is inadequate and/or the nail length is insufficient. In general, the mobilized pullout per unit length, Q , (also called the load transfer rate) can be expressed as:

$$Q = \pi q D_{DH} \quad (2.1)$$

Where:

q = mobilized shear stress acting around the perimeter of the nail-soil interface; and

D_{DH} = average or effective diameter of the drill hole.

Considering a single nail segment subjected to a tensile force, T_o at one end, and applying equilibrium of forces along the differential length of the nail. The tensile force can be related to the interface shear stress as:

$$dT = \pi D_{DH} q dx = Q dx \quad (2.2)$$

The above equation represents the transfer mechanism between the stresses on the nail-soil interface to tensile forces in the nail bar. In general, the tensile force (T) at a distance “ x ” along the bar is:

$$T(x) = \int_0^x \pi D_{DH} q dx = \int_0^x Q dx \quad (2.3)$$

Actual distributions of mobilized bond shear stress (and load transfer rates) are not uniform, as illustrated in Figure 3.9, and depend on various factors including nail length, magnitude of applied tensile force, grout characteristics, and soil conditions. As a simplification, the mobilized bond strength is often assumed to be constant along the nail, which results in a constant load transfer rate, Q . As a result, the nail force at the end of the pullout length, L_p , is:

$$T(L_p) = T_o = Q L_p \quad (2.4)$$

The pullout capacity, R_p , is mobilized when the ultimate bond strength is achieved and is expressed as:

$$R_{p1} = T_{\max} = Q_u L_p \quad (2.5)$$

With:

$$Q_u = \pi q_u D_{DH} \quad (2.6)$$

Where:

Q_u = pullout capacity per unit length (also referred to as load transfer rate capacity); and

q_u = ultimate bond strength

$$R_{p1} = \pi D_{DH} q_u L_p / S_v \quad (2.7)$$

In absence of insitu test. Pullout capacity is calculated from the following equation

$$R_{p1} = (c + \sigma_v \tan \psi) \pi D_{DH} \quad (2.8)$$

Ψ = mobilized soil nail interface friction angle $= \Psi = \frac{2}{3} \phi$

Φ = soil effective angle of internal friction

C = unit cohesion of soil

The undrained shear (total stress) is used for the calculation since it applies high pull out force on the nail.

For i th nail, $\sigma_v = \gamma' h_i + \text{surcharge}$

Since the nail is at an angle $R_p = R_{p1} / \cos i$

Where i is the nail inclination

$$\text{Design pull out capacity } (R_{PD}) = R_p / FOS_T \quad (2.9)$$

Where $FOS_T = 1.5$ in most cases

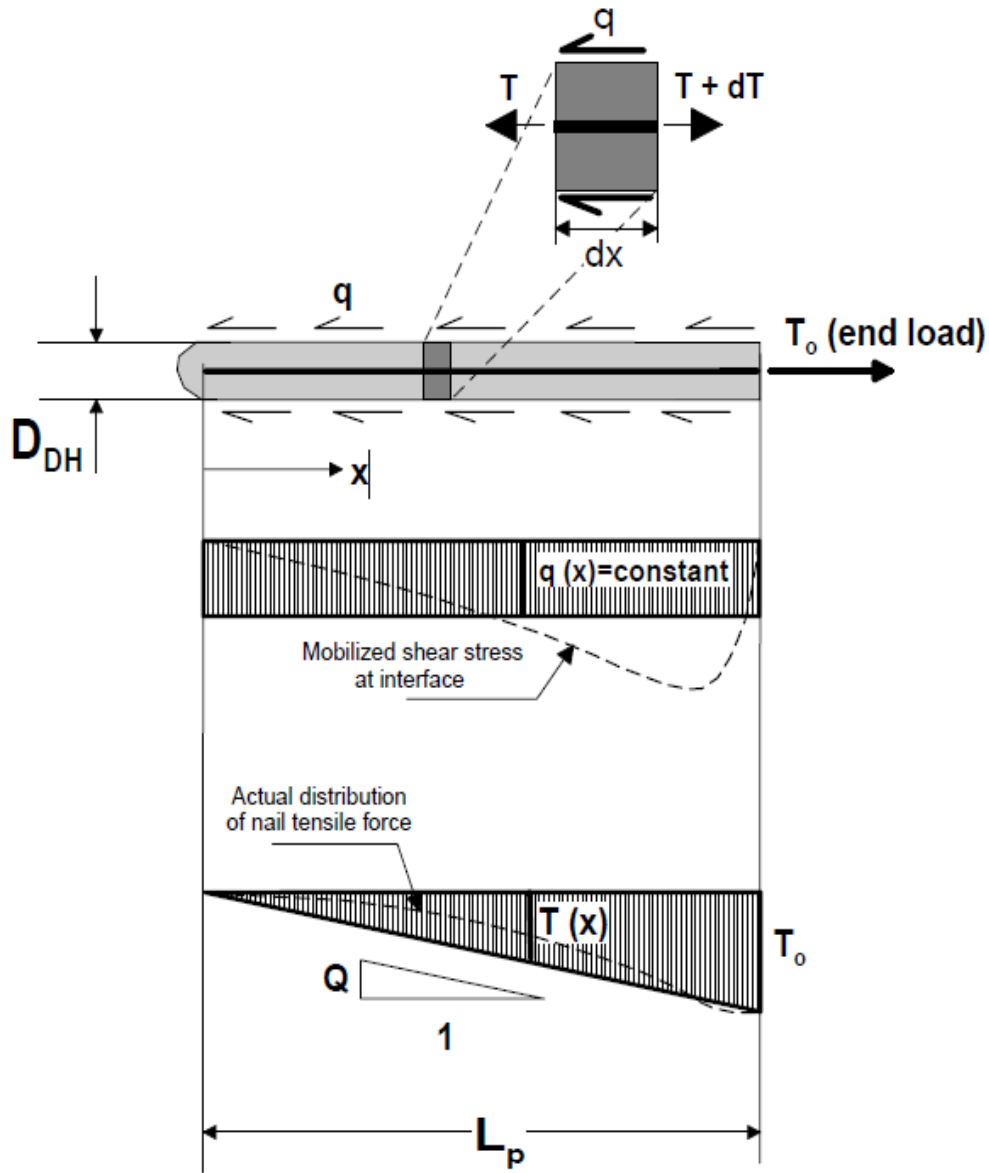


Figure 2. 3 Single Nail Stress-Transfer Mode (Carlos et al., 2003)

The anchor bond length load transfer capability is usually estimated based on previous field experience. Potential differences in capacity due to different installation and grouting methods must be addressed when calculating capacities based on past field data (FHWA, 2013). The first step in evaluating the minimum permitted capacity for a certain project is to assume a maximum anchor bond length. For this work, a bond strength of 30 KN/m was chosen for hard clay FHWA.

So,

$$L_p = R_{pd} * \frac{S_v}{\pi D_{DH} q_u} \quad (2.10)$$

2.5 Plaxis Software Analysis

Plaxis is a geotechnical finite element program in which soil models are used to simulate the behavior of the soil. The plaxis algorithm and associated soil models have been meticulously built. Despite the fact that extensive testing and validation has been carried out. It is impossible to generate a plaxis code that is error-free. Furthermore, using the finite element method to simulate geotechnical issues inevitably involves some numerical and modeling errors. The user's experience in issue modeling has a significant impact on the accuracy with which reality is approximated. The capacity to evaluate the validity of the computational results, as well as the understanding of soil models and their limitations, model parameter selection, and model parameter selection. As a result, plaxis can only be used by professionals with the aforementioned experience. When using computational results for geotechnical design, the user must be aware of his or her responsibilities(plaxis 2D, 2002).

Plaxis 2D has four features (i.e. input, calculation, output, and curves). Plaxis input is used for geometry, material, and boundary condition modeling, as well as mesh generation and initial condition generation (pore water pressure and initial stress condition). For the examination of various structures, loading circumstances, construction stages, and pore water conditions, plaxis calculations are used. It supports a wide range of computations, including plastic calculations, consolidation analysis, safety analysis, completely linked flow deformation analysis, and dynamic calculations with various load stepping processes. To analyze and present analysis findings, Plaxis output and curves are employed (Plaxis 2D v8, 2013).

Because of the user-friendly environment, simulation of soil nail structures with PLAXIS 2D is quick and easy. However, given the computational tool's capabilities, the accuracy of the analysis is heavily reliant on the user's knowledge of the tool and the problem. The following are some suggestions for using PLAXIS 2D for simulations and analyses of soil nail structures based on the literature and the authors' experiences.

In practice, a soil nail is rigidly connected to the wall facing (FHWA 2003; Joshi 2003) by means of bearing plate and hexagonal nut to the temporary facing which in turn is connected with permanent facing. In PLAXIS 2D, connection between two plate structural elements by default represents a rigid connection. Therefore, use of plate structural elements to simulate soil nails and wall facing is recommended to account for the rigid nail-facing connection.

2.5.1 Mohr-Coulomb Failure Criterion for Soil

Plaxis 2D describes the mechanical behavior of soils and rocks as a stress-strain relationship using constitutive models. Different soil models are incorporated in the software such as the HB model, the MC model, soft soil model, hardening soil model, soft soil creep model, and jointed rock model. The feature also enables the use of self-programmed or user-defined constitutive models.

Mohr-coulomb provides a failure criterion of soil material using a critical combination of normal and shear stress (Das, 2010). The Mohr-Coulomb criterion relates the shear strength of the material on the failure plane to the cohesion, normal stress, and angle of internal friction of the material (Eqn.2.11). As shown in Figure 2.5 any combination of normal and shear stress along with the MC failure envelop (B) leads slope failure, any combination of normal and shear stress below the MC envelope (A) is safe and no combination of normal and shear stress can be existing above the failure envelope (C) (Mekonnen, 2020).

$$\tau_f = c' + \sigma_n \tan \phi' \quad (2.11)$$

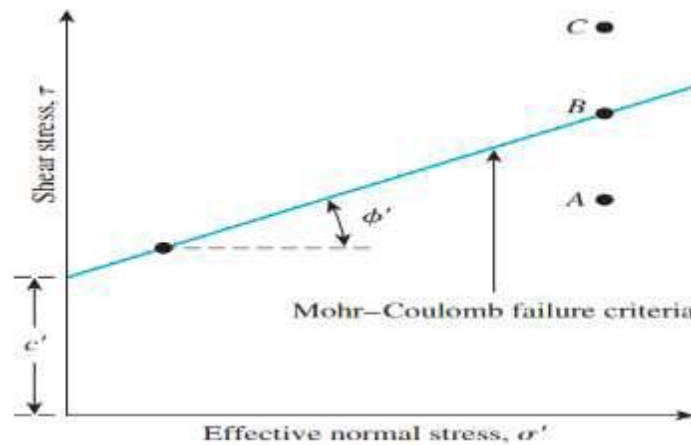


Figure 2. 4 Mohr-Coulomb failure criterion (Das et al., 2011)

In PLAXIS 2D, soil nail structures are modeled as a plane strain problem. The flexural rigidity (bending stiffness) EI and the axial stiffness EA are the most critical material input parameters for plate elements (for geogrid structural element only the axial stiffness EA is required). Plate and geogrid structural elements are both rectangular in shape, with an out-of-plane width of 1 m. Because the soil nails have a circular cross-section and are spaced at a specified horizontal spacing, corresponding axial and bending stiffnesses must be determined for the accurate modeling of circular soil nails as rectangular plate or geogrid elements. Following a full

discussion of the applicability of plate or geogrid structural elements to simulate soil nails, the general process for determining similar material properties is outlined (Singh, 2009).

For grouted nails, the equivalent modulus of elasticity E_{eq} must be calculated, taking into consideration the elastic stiffnesses of both the grout cover and the reinforcement bar. E_{eq} can be calculated using the foundations of material strength as follows:

$$E_{eq} = E_n \left(\frac{A_n}{A} \right) + E_g \left(\frac{A_g}{A} \right) \quad (2.12)$$

Where: E_g is the modulus of elasticity of grout material; E_n is the modulus of elasticity of nail;

E_{eq} is the equivalent modulus of elasticity of grouted soil nail A is the total cross-sectional area of grouted soil nail; $A_g = A - A_n$ is the cross-sectional area of grout cover; $A_n = 0.25 (\pi) d^2$ is the cross-sectional area of reinforcement bar and DDH is the diameter of drill hole. If, S_h is horizontal spacing of soil nails, knowing the equivalent modulus of elasticity (equation 2.10) for the grouted soil nail, the axial and bending stiffness can be determined using equations (2.3) and (2.4) respectively.

$$\text{Axial stiffness } EA = E_{eq} / S_h \left(\frac{\pi D^2 DDH}{4} \right) \quad (2.13)$$

$$\text{Bending stiffness } EI = E_{eq} / S_h \left(\frac{\pi D^4 DDH}{64} \right) \quad (2.14)$$

Substituting, EA and EI values in the material properties menu for Plate elements, PLAXIS automatically determines the equivalent plate thickness in meter deq using equation (2.5).

$$deq = \sqrt{12 \left(\frac{EI}{EA} \right)} \quad (2.15)$$

2.6 Related Works

Pham et al., (2020) investigated the effect of the inclination angle and length of soil nail Patterns, as well as surcharge loads, on the stability of soil nailed slopes and face deformation in a staged - excavation. The findings show that inclinations and length Patterns of soil nails might affect the efficiency of soil nail reinforcing. The basic result is that the more inclinations in the soil nails, the greater the reinforcing forces in the soil nails. Slope displacement properties have also been influenced by soil nail length patterns.

Ma et al., (2011) presented a case study of an excavation in Shanghai, China's soft deposit. Through a field case, this research investigated the interaction mechanism between the nail and the surrounding soil. A two-dimensional finite element approach (2D-FEM) was used to solve the problem. By comparing the calculated wall displacement to the field measured value, the effectiveness of 2D FEM is shown. The importance of four important design parameters, such as nail length, nail spacing, wall thickness, and wall stiffness, is discussed based on the FEM results. The findings showed that FEM analysis is a good approach to forecast nail displacement and internal force. Finally, a design chart for the soil-nail supported wall commonly used in excavation practice is proposed.

Lima et al., (2004) investigated at how the geometry of the nail and the depth of the excavation affected the stress-strain behavior of vertical nailed excavations. In gneissic residual soils, a parametric research was conducted for a typical slope. The result indicated that design geometry and excavation height have a significant impact on the horizontal displacements of nailed soils. When $L/H \leq 0.6$ and $S_v/L \leq 25\%$, the displacements are considered negligible. The highest limit of the S_v/L ratio can be adjusted to 50% for relatively small height excavations ($H \leq 5\text{m}$).

Anand et al (2019) studied that the effect of slope angle wall height on the FS. The findings showed that FS decreases when the slope angle and wall height increases. In this study the effect of ground water table and rainfall is not considered. Since ground water and rain fall decreases soil shear strength, both should have to be considered.

Alfat et al., (2019) investigated into how slope geometry affects slope stability. The study showed that in a single slope profile, FS increases when the slope angle decreases, and

additional benches increases FS far more. The software used in this investigation is based on the slide LE (fellenius method). This method examines slope stability by assuming an imaginary failure surface, which is a limitation.

M. Abbas, (2014) for both cohesive and cohesion less soil, an assessment was done for a slope system with different slopes. The entire geotechnical system was simulated and analyzed using a two-dimensional finite element technique. For both types of soils, the factor of safety in the structure of slope case reduced as the slope increased, according to this study. Furthermore, as the slope increases, the displacement in the points in which the section varies (in the critical zone) increases.

Hulagabali, (2019) conducted a parametric analysis of slope behavior for various cohesion, internal friction angle, slope angle, unit weight, and slope height. "GEO STUDIO – 2012" is used to model slope stability with various parameters. In this analysis, many parameters are changed. Slope was first modeled for several parametric studies. It was found that when the cohesiveness, angle of internal friction, and factor of safety increased, so increased the factor of safety. In addition, as the slope angle and height were increased, the factory of safety decreased. Because shear strength of soil is completely dependent on shear factors such as cohesiveness and angle of internal friction, a change in unit weight of soil had no effect on slope stability. The effect of the groundwater table and rainfall is not taken into account in this study. Because both ground water and rain fall reduce soil shear strength, both should be taken into account. Geo studio is an LEM-based application. This method examines slope stability by assuming an imaginary failure surface, which is a limitation.

Safuan et al., (2013) discussed the application of a soil nailing method based on a case study in Tehran, Iran. For a 15-story residual building wall, the overall excavation depth brought the cut face close to 12 meters. The diameter of the nails was 28 mm, and their lengths ranged from 8 to 10 m, with a 7-degree slope to the horizontal plane. The deformation during the excavation process was evaluated using the finite difference technique (FDM - FLAC 2D) and the finite element method in a series of numerical simulations (FEM - PLAXIS 2D). Shotcrete facing was used to simulate the pattern of wall deflection throughout the modeling phase. When the pattern of deformation in the initial and last steps of excavation was simulated, there was a good agreement between the results of both methods. The results obtained by FEM were

closer to the field measurement based on the maximum vertical displacement of shotcrete face, estimated using two approaches. However, for different stages of excavation, there was a good agreement between the values of factor of safety calculated by FLAC 2D and PLAXIS 2D.

Chow, *et al* (2012) presented two case studies in Malaysia, one of which involved the use of soil nailing technique in deep excavation. A 30-meter-deep excavation was the first case history. The geotechnical challenge is to design and construct a deep basement up to 30 meters deep while keeping low-rise structures close by. The Kenny Hill Formation, a metasedimentary rock formation, was found beneath the site. The bedrock is sandstone/siltstone with quartzite and phyllite. A characteristic feature of weathered soils is the presence of sandy silt/clay. The soil nails are 18 meters long, with four 15-degree inclination nails and 1.5-meter horizontal and vertical inclination, as well as nine 15-degree inclination nails with 1.5-meter horizontal and vertical spacing. The second case study included a 20-meter-deep excavation beneath a granite formation for the construction of a five-level basement. Granite comes in a variety of textures and compositions, ranging from coarse to very coarse grained. Upper nails have a 15° inclination, 0.75 vertical spacing, and 1m horizontal spacing, while middle nails have four 6m long nails with a 15° inclination, 0.75 vertical spacing, and 1m horizontal spacing, and last nails have three 12m long nails with a 15° inclination, 1m vertical and horizontal spacing.

Rashidi *et al*, (2018) conducted a parametric study to determine the deformations and settlements of nine stabilized walls with various features. The numerical simulations in this work were verified using data from Briaud and Lim (1999) for a stable wall. The main goal of this paper is to compare the performance of a soil nail wall to that of an anchorage method that was simulated in Plaxis 2D (finite element software), as well as the wall deformations achieved using these two approaches.

Singh, (2009) presented the capabilities of limit equilibrium methods and a finite element approach for study of slope stability problems in mining. The geometry of the model, material input parameters, Mohr-Coulomb failure criterion, loads, and boundary conditions required to predict slope failure accurately are all provided. The finite element approach is used to assess two models, and the findings are compared to those obtained using the limit equilibrium

method. The first model has a 'middle' mesh density, whereas the second has a 'fine' mesh density. It was also examined in a model with and without pore pressure.

Culligan et al., (2003) investigated how the stress-strain behavior of nailing excavations was affected by slope geometry and construction procedures. Various slope inclinations and modeling approaches were studied for replicating nails. Geotechnical data from a typical residual soil from southeast Brazil was used in a parametric study using a finite difference computer program. According to the data, a modest modification in slope inclination from 90 to 80 degrees reduces horizontal displacements at the top of the excavation by 70%. On gentler slopes, this effect is less noticeable. Horizontal displacement at the top of vertical excavations is reduced by 35% when stapled soil slopes with the nail end anchored to the concrete face are used.

Mahmud Sazzad et al., (2015) studied how to use a finite element approach to study the response of slopes of homogeneous and stratified soil using the shear strength reduction (SSR) technique. Slope failure can be caused by a loss in shear strength due to increased plastic strains caused by loading. The current method allows this failure mechanism, and the analysis is carried out using the Mohr-Coulomb and Drucker-Prager yield criteria, which minimize the shear strength parameters. Mesh's effect on the slope's factor of safety is investigated. The numerical result for slope stability analysis by the finite element SSR methodology is compared to the well-known classical methods of Bishop Method (1955), Fellenius Method (1936), and Spencer Method (1967) for homogeneous and layered soil. There is a good agreement between the traditional limit equilibrium techniques (LEM) and the finite element method (FEM). It's important to note that FEM has a higher safety factor than LEM. Furthermore, for both homogeneous and stratified soil slopes, the Drucker-Prager model provides a higher factor of safety than the Mohr-Coulomb model, regardless of the position of the surcharge and slope angles. The factor of safety increases for slopes of homogeneous and layered soil as the distance of surcharge grows from the summit of the hill up to a certain level, after which the surcharge's influence remains constant. The failure surfaces of the LEM and FEM have both been evaluated.

Mekonnen, (2020) studied slope profile stability and performance evaluation (single slope, multi-slope, and benched slope). The static and dynamic loading scenarios were analyzed

using limit equilibrium (slide program) and the finite element method (Plaxis software). The findings of the analysis in key areas demonstrate that the slope is marginally stable in the static state with FS ranging from 1.2 to 1.5 and unstable in the dynamic state with FS less than 1. According to a comparison of analysis approaches, the finite element method provides more useful information about a slope's failure surface than limit equilibrium analysis. According to a performance evaluation of geometric profiles, geometric modification results in improved and more cost-effective slope stability. Cut slope stability is greatly improved by benching (i.e. the use of 2m and 3m bench improves the factor of safety by 7.5 percent and 12 percent from single slope profile). The method is particularly successful on steep slopes. Similarly, using a multi slope profile in stratified soil with various strengths enhances slope stability. When used in conjunction with benches, the results are greatly improved. According to the study, drainage control and slope fortification are also indicated as restorative measures for cut slopes.

Kun-Ming, et al (2019) conducted research in order to analyze the stability and optimize the project of soil nailing engineering for expansive soil slope, the calculation formulas for unsaturated soils strength, soil nailing surface bond strength, and nail withdraw strength were deduced based on the Mohr-Coulomb strength formula and national regulations. The Bishop slice method was used to analyze the sliding resistance of the soil nailing; models for soil nailing expansive soil slope stability analysis were established; and the influence of fissure, rain infiltration, and soil nailing on expansive soil slope stability was also taken into account. Using an engineering example with a fixed slope soil parameter and matching soil nailing parameter, the influential elements were investigated by varying the inclination, installation position, and length of soil nails. As proven by engineering examples, the techniques and accompanying calculating procedure were suitable for stability analysis and design optimization for soil nails large soil slopes. It has a significant amount of engineering utility.

Youssef et al., (2017) conducted a parametric analysis using the PLAXIS v8.2 finite element computer to evaluate the global factor of safety, F.S, horizontal displacement, and maximum tension force of the reinforcement element as a function of aspect ratio (L/H) of a non-uniform soil nail wall. For non-uniform soil nailing constructions, the aspect ratio (L/H) was changed in four distinct ways. There are four non-uniform soil nail wall patterns in this study (A, B, C,

and D). Pattern (A): The length of nails in the lower two thirds of the wall remains constant, while the length of nails in the upper third varies from 0.25 to 1 from the wall's height. Pattern (B): The length of nails (in the lower two thirds of the wall) varied from 0.25 to 1, while the length of nails in the upper third remained constant. Pattern (C): Nail length reduces with depth, resulting in an average nail length of $0.7H$ or H , and Pattern (D): Nail length grows with depth, resulting in an average nail length of $0.7H$ or H . When the aspect ratio is more than 0.7, the results show that raising the aspect ratio increases the global factor of safety of soil-nail walls, has a considerable effect on the maximum tensile force, and has no effect on horizontal displacement. At two-thirds of the wall height from the wall base, the maximum horizontal displacement occurs. Furthermore, for all nail lengths utilized in this investigation, wall failure occurred when the aspect ratio in the upper half of the wall was less than 0.4 and the lower half was constant (Pattern A). When the aspect ratio was 0.25 in the lower half of the wall and the top half was constant, the factor of safety was less than the minimum value (1.5). (Pattern B). Patterns (C, D) had no influence on horizontal displacement, but the safety factor was raised due to the gradual nail length arrangement.

Table 2. 1 Summery of Related works

Researcher	Year	Content
Pham et al	2020	Studied on the effect of some parameters of soil nails on the stability of vertical slopes • Investigated the effects of the inclined angle and nail length • Subjected to surcharge loads • Soil nail reinforcement efficiency could be affected by inclinations and length Patterns of soil nails
L. Ma et al	2011	A case study of the behavior of soil-nail supported deep mixed Wall in the soft deposit of shanghai • The strength of this deposit is very low • Nail and surrounding soil through a field case

		• two-dimension finite element method (2D-FEM) was conducted • nail length, nail spacing, thickness of DM wall, and stiffness of DM wall, are discussed • FEM analysis is an efficient way to predict the displacement and internal force of nails
A. P. Lima et al	2004	Considerations on the soil nailing technique for stabilizing excavated slopes • Evaluated the influence of nail's geometry and excavation depth on the stress-strain behavior of vertical nailed excavations • Strong influence of design geometry and excavation height on the horizontal displacements of nailed soils
Anand et al	2019	Numerical Analysis of Slope Stability- A Parametric Study • Carried out the parametric study on the behavior of slope for different cohesion, angle of internal friction, slope angle, unit weight and height of the slope • "GEO STUDIO – 2012 is used. • As the cohesion, angle of internal friction increased, factor of safety increased.
S Alfat et al	2019	Slope stability analysis through variational slope geometry using Fellenius Method • Conducted a theoretical investigation of the variation of slope form to minimize the landslide of slope • Slopes with terraces will be easily stable
M. Abbas et al	2014	An assessment was done for a slope system with different slopes for both cohesive and cohesion less soil. • Two-dimensional finite element technique • As the slope increased, the displacement in the points in which the section varies (in the critical zone) increased.
Safuan et al	2013	Discussed the application of a soil nailing method based on a case study in Tehran, Iran. • Evaluated using the finite difference technique

		• Shotcrete facing was used to simulate the pattern of wall deflection throughout the modeling phase. • The results obtained by FEM were closer to the field measurement.
Chow et al	2012	Presented two case studies in Malaysia, one of which involved the use of soil nailing technique in deep excavation. • One a deep basement up to 30 meters • One 20-meter-deep excavation • Upper nails have a 15° inclination, 0.75 vertical spacing, and 1m horizontal spacing, • Middle nails have four 6m long nails with a 15° inclination, 0.75 vertical spacing, and 1m horizontal spacing, and • Last nails have three 12m long nails with a 15° inclination, 1m vertical and horizontal spacing.
Culligan et al	2003	Investigated how the stress-strain behavior of nailing excavations was affected by slope geometry and construction procedures • Finite difference computer program • Horizontal displacement at the top of vertical excavations is reduced by 35% when stapled soil slopes with the nail end anchored to the concrete face are used.
Mahmud Sazzad et al	2015	Studied how to use a finite element approach to study the response of slopes of homogeneous and stratified soil using the shear strength reduction (SSR) technique. • carried out using the Mohr-Coulomb and Drucker-Prager yield criteria • FEM has a higher safety factor than LEM • The factor of safety increases for slopes of homogeneous and layered soil as the distance of surcharge grows from the summit of the hill up to a certain level, after which the

		surcharge's influence remains constant.
Mekonnen	(2020)	Studied slope profile stability and performance evaluation • The static and dynamic loading scenarios were analyzed • Slope is marginally stable in the static state with FS ranging from 1.2 to 1.5 and unstable in the dynamic state with FS less than 1.
Kun-Ming et al	2019	Conducted research in order to analyze the stability and optimize the project of soil nailing engineering for expansive soil slope. • Mohr-Coulomb strength formula used • The Bishop slice method was used to analyze the sliding resistance of the soil nailing • The techniques and accompanying calculating procedure were suitable for stability analysis and design optimization for soil nails large soil slopes
Youssef et al	2017	conducted a parametric analysis using the PLAXIS v8.2 • finite element computer to evaluate the global factor of safety, F.S, horizontal displacement, and maximum tension force of the reinforcement element as a function of aspect ratio (L/H) of a non-uniform soil nail wall • At two-thirds of the wall height from the wall base, the maximum horizontal displacement occurs. • For all nail lengths utilized in this investigation, wall failure occurred when the aspect ratio in the upper half of the wall was less than 0.4 and the lower half was constant.

CHAPTER THREE

METHODOLOGY

3.1 Description of the study area

The research region is the city of Addis Ababa, notably the clayey areas. Addis Ababa, Ethiopia's political capital and most major commercial and cultural center, is situated in the country's heartland at 902°N latitude and 38045°E longitude. It has an average elevation of 2,400 meters above sea level, with the highest altitude reaching 3,200 meters at Entoto Hill to the north. Addis Ababa is thus one of the world's high-altitude capital cities. Addis Ababa covers a total area of 540 square kilometers and is bordered by rugged terrain (Coal & Portions, 2010). Standard penetration test samples are gathered from various sub-cities, with one typical soil profile being chosen.

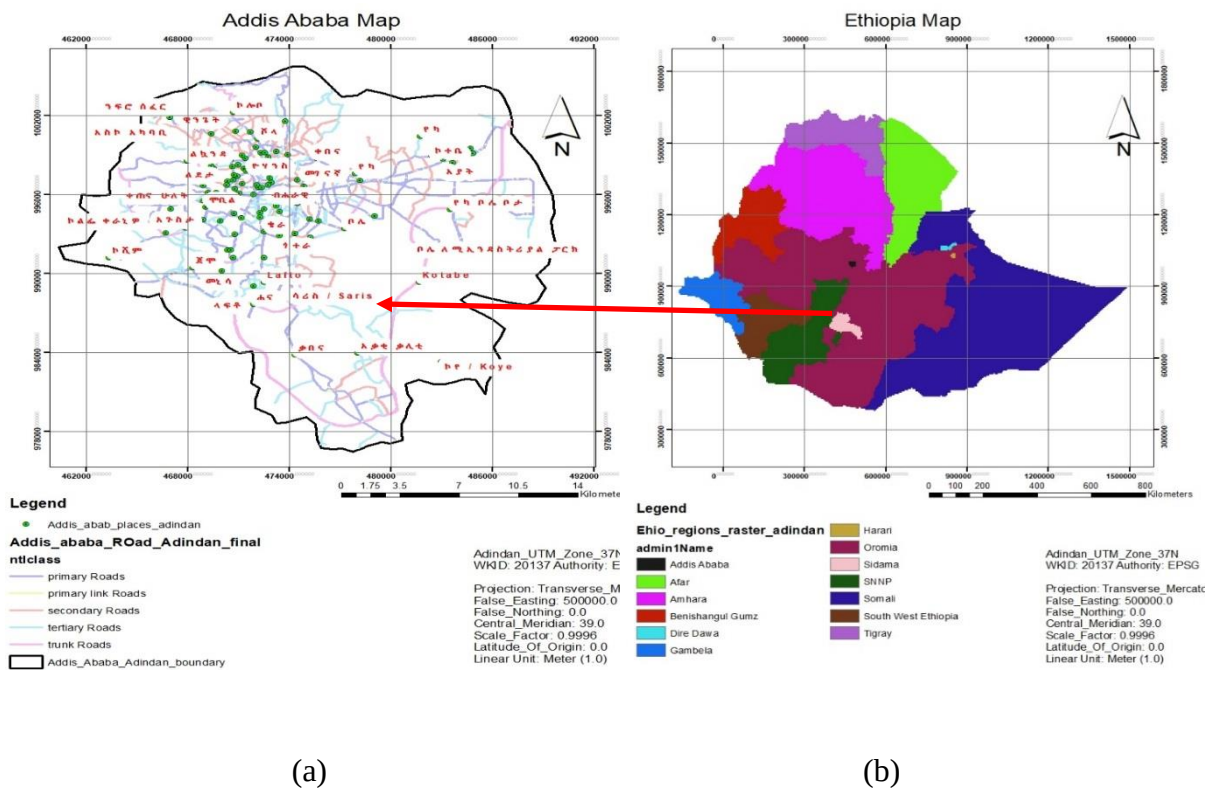


Figure 3. 1 Map of Addis Ababa (b) Map of Ethiopia

3.1.1 Soil type of the study area

Figure 3.2 shows the soil map of Addis Ababa.

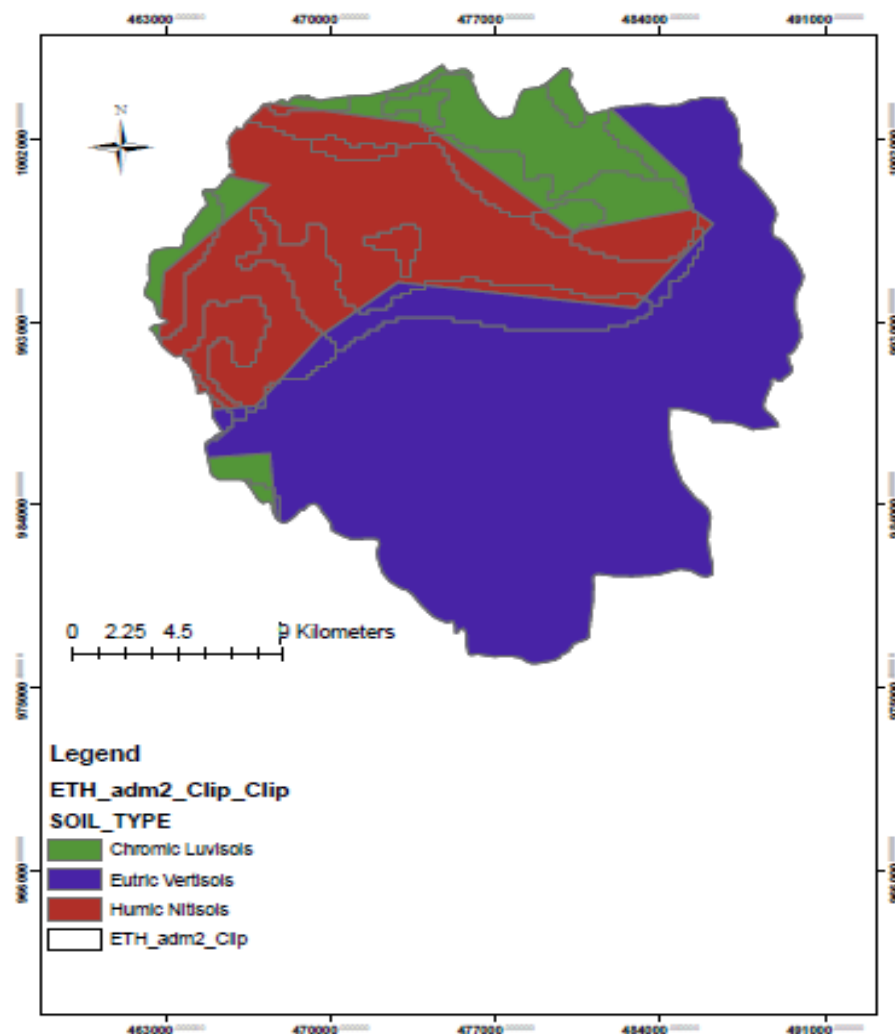


Figure 3. 2 Soil texture map of Addis Ababa

3.1.2 Rainfall, Temperature and Groundwater Table

Rainfall averages 1160 mm per year, with a peak from June to September, which is the only particularly rainy period. From November to February, there is minimal rain and only a few showers; from March to May, afternoon showers become more common, occurring 7/10 days per month, and practically every day in July and August. The average precipitation is shown below (World Climate Guide).

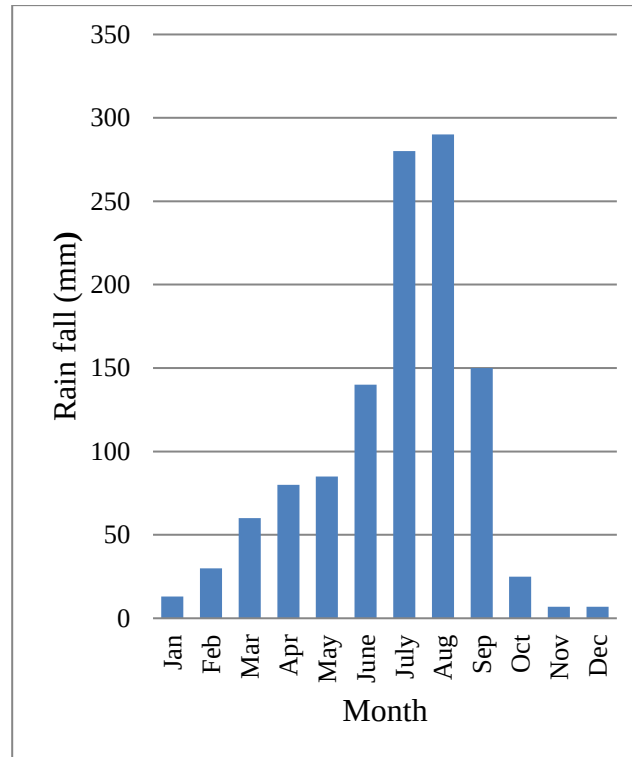


Figure 3. 3 Average Participation of Addis Ababa (World Climate Guide)

Nights are cool, even frigid, from November to February, when lows drop below 10 °C, while days are comfortable, around 23–25 °C, except in July, August, and September, when highs drop to around 20–21 °C due to the rainy season. The months of March to May are, as is common in Ethiopia, the hottest months of the year, albeit by a few degrees (world climate guide). Figure 3.4 shows the average temperature of Addis Ababa 1990-2020.

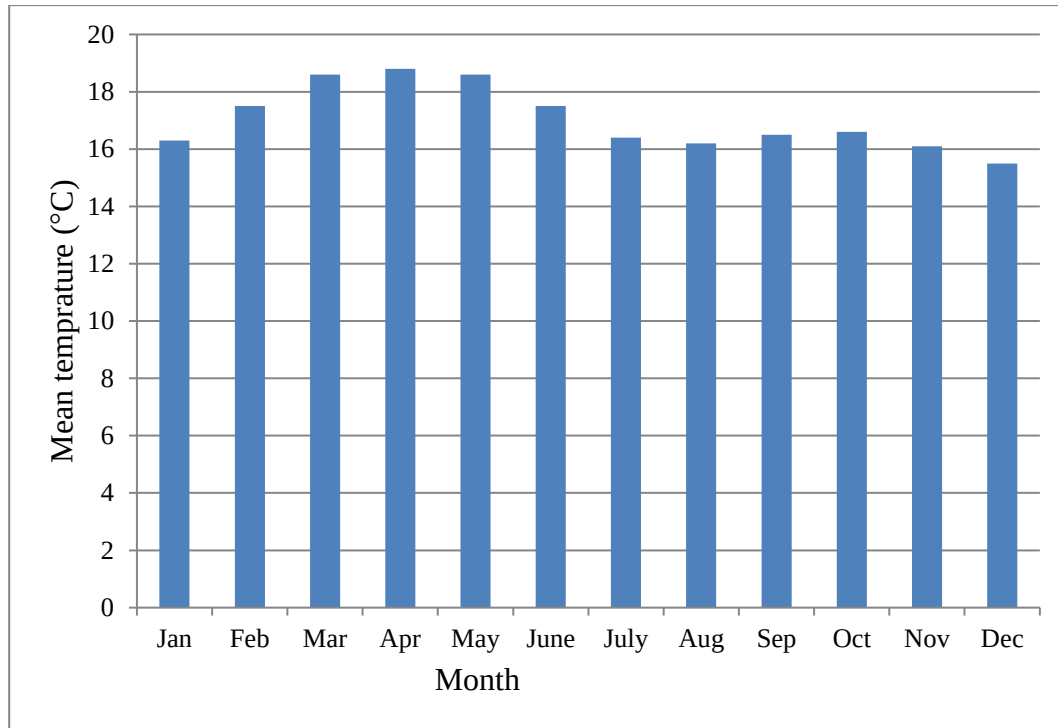


Figure 3. 4 Average Temperature of Addis Ababa (1990-2020), (World Climate Guide).

Figure 3.5 shows shallow ground water depths in the town's central and northern areas. The confinement in the town's center area ranges from 23 to more than 100 meters. The confinement is greatest along an elongated strip running north-south (Filwoha, Lideta, Mekanisa, Lafto, Kailty), and it diminishes in both the eastern and western directions from this elongated strip. The vast majority of the shallow groundwater area north of town is unconfined. The depth of the groundwater ranges between 40 and 70 meters, indicating that the un-saturated thickness is greater than 40 meters. The unconfined groundwater of Addis Ababa, with a depth of less than 80 meters, is considered very susceptible to numerous pollutants, with the exception of those that are easily absorbed and changed. The semi-confined aquifers of Addis Ababa area are considered (Coal, et al., 2010).

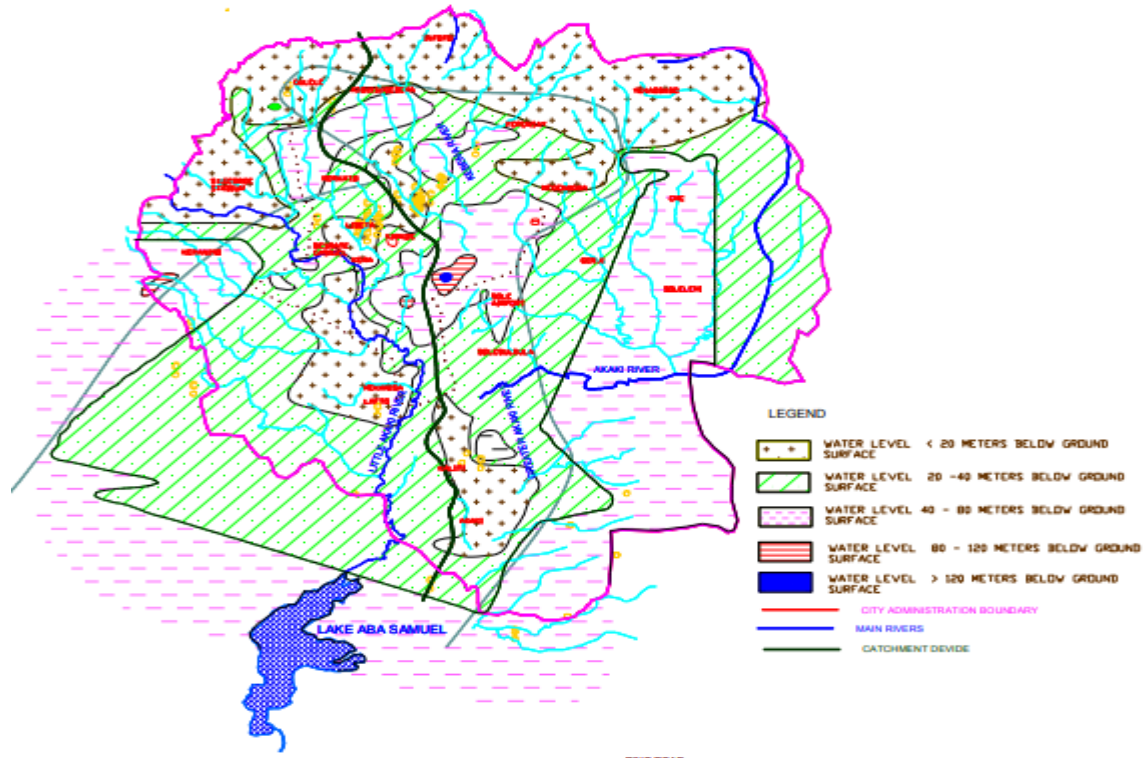


Figure 3. 5 Ground Water Depth of Addis Ababa Area (Coal et al, 2010)

3.2 Methodology

This section consists of the research methods used to achieve the study's general and specific aims. First, a thorough review of current literature and theories was conducted to gain a better knowledge of the cause and mechanism of slope failure, stability evaluation methodologies, and nailing design for the proposed deep excavation. The data for the deep excavation's stability study was then gathered from numerous organizations and through a literature review.

Secondary data, SPT data and groundwater table of the site, were collected from organizations. Literature and correlations were used to get the soil stiffness, nail, and grout characteristics needed for the numerical analysis.

FEM was used to undertake stability analysis by measuring horizontal displacement using data gathered from organizations and literature. Previous case studies were used to evaluate the modeling and analytic software. Finally, the variation of horizontal displacement in relation to wall height, surcharge, nail spacing and nail inclination was discussed. Figure 3.6 depicts the study's overall workflow for achieving the study's objectives.

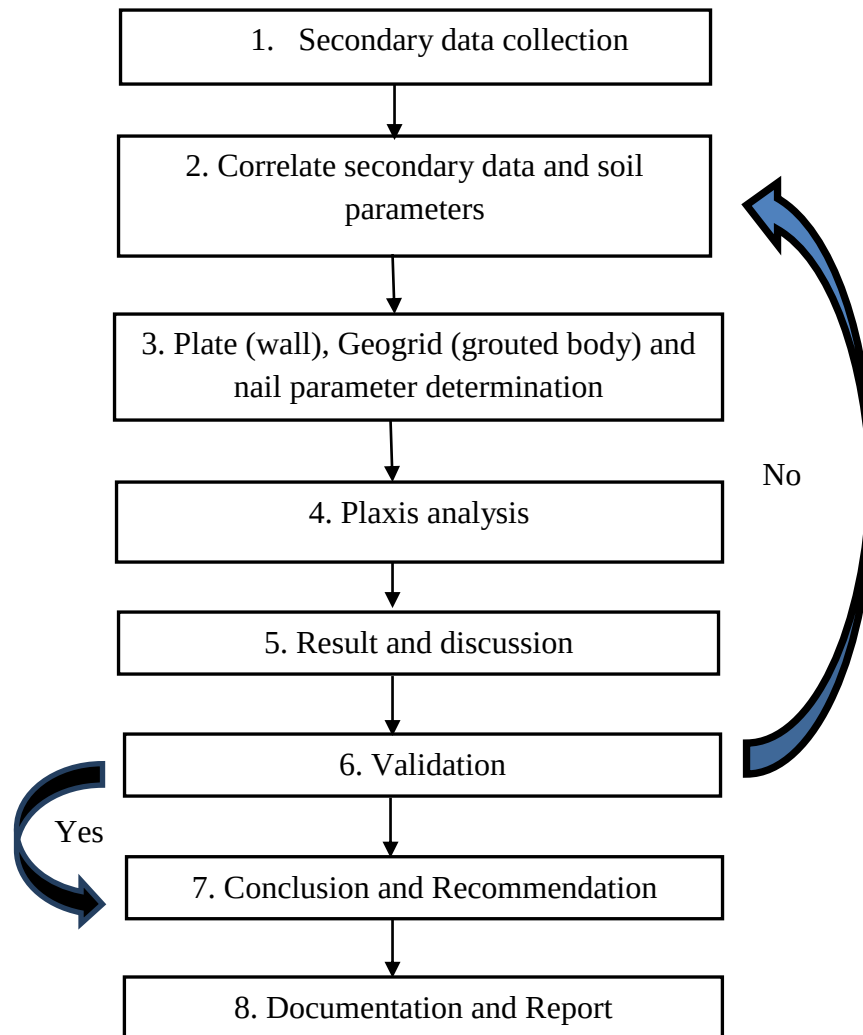


Figure 3. 6 General flow of work

3.2.1 Secondary Data Collection

Data was collected from a variety of sources, including research, correlations, and institutions. SPT data and ground water table information from yeka sub city were provided by institutions. The data was varied depending on the soil type and the depth of the groundwater. Only clay major soil was chosen from the data. After the clay major soils were selected, the dominating clay major soil type was selected. Up to 4 meter from the ground surface silty clay was the dominating one, from 4 meter up to 10 meter sandy silty clay was the dominating one, and from 10-14 is sandy clay dominating and 14-20 was tuff soil. The mean average of SPT value was calculated for each depth. Young's modules, poisons ratio, soil internal friction,

Cohesiveness, Dilatancy angle, and Unit weight were determined using correlations from the SPT data.

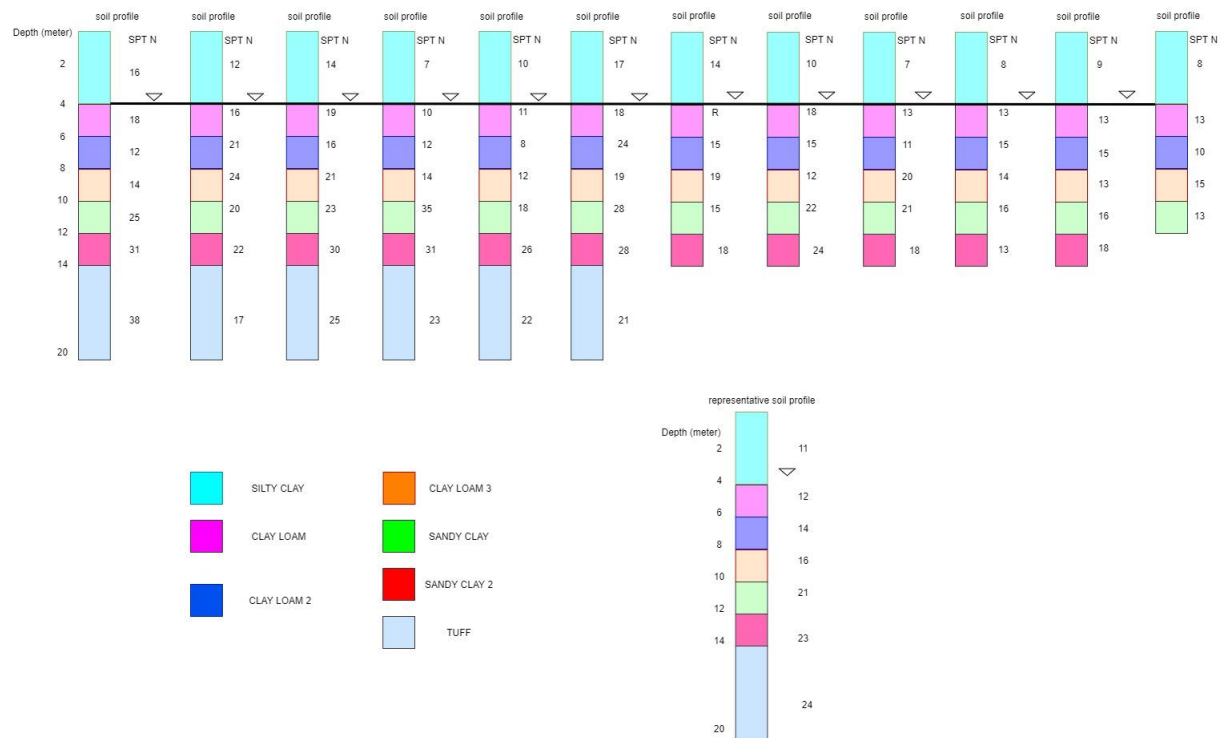


Figure 3. 7 Selected soil profiles and representative borehole

Rainfall intensity and temperature were obtained from the World Climate Guide's official website. The higher ground water table, which were 4 meters below the ground water table, was chosen because water reduces the shear strength characteristics of soil.

The values of soil parameters, Plate (wall), Geogrid (grouted body) and nail parameters used are given in the following section.

3.2.2 Correlation and Determination of Soil Parameters from SPT Value

Clay was supposed to be present in all strata, with a limited quantity of other soil types. The layers were called as clay soil assuming that they contains less than 40% of another type of soil. Minimum internal friction values were considered in order to obtain the maximum shear stress on the nails.

3.2.2.1 Cohesion Value Determination

$$C = -16.5 + 2.15N \text{ for fine grained soils (Kumar et al., 2016)} \quad (3.1)$$

Where N is uncorrected number of blows

Undrained cohesion for fine grained soils

$$C_u = 6.25N \text{ Terzaghi and Peck} \quad (3.2)$$

Where N (uncorrected number of blows) is between 10 and 30. Using the given formulas the cohesion of the each layer was determined.

Table 3. 1 Values of Cohesion for each layer

Soil profile cohesion values			
Depth	SPT N	Corrected SPT N value	Cohesion KPa
0-4	11	9	44.3 (bulk)
4-6	12	12	82.1 (saturated)
6-8	14	13	95.1 (saturated)
8-10	16	14	101.5 (saturated)
10-12	21	16	133.9 (saturated)
12-14	23	17	144.3 (saturated)
14-20	24	18	156.25 (saturated)

3.2.2.2 Internal Friction Determination

According to Minnesota Department of Transportation, Pavement Design, 2007 the internal friction of silty clay and clay loam ranges from 18°-32°. For safety minimum value 18° is used. For sandy clays the value of internal friction ranges 27°-35°. For safety the minimum 27° is used. For the last tuff soil the 28° by taking the average (Asniar et al., 2019). For the seventh soil which is tuff soil, the properties is given according to Asniar et al., (2019). The property is calculated from selected from Table A.1.

Table 3. 2 Values of Internal friction for each layer

Depth	Type of soil	Internal friction angle
0-4	Silty clay	18°
4-6	Sandy silty clay 1	18°
6-8	Sandy silty clay 2	18°
8-10	Sandy silty clay 3	18°
10-12	Sandy clay 1	27°
12-14	Sandy clay 2	27°
14-20	Tuff soil	28°

3.2.2.3 Young's Modules Determination

$$E_s = 4.32N \text{ (D' Appolonia et al)} \quad (3.3)$$

Table 3. 3 Values of Young's modules for each layer

Depth	SPT	Es (Mpa)
0-4m	11	47.52
4-6m	12	51.84
6-8m	14	60.48
8-10m	16	69.12
10-12m	21	90.72
12-14m	23	99.36
14-20m	24	108

3.2.2.4 Saturated and Unsaturated Unit Weight Determination

The saturated unit weight are determined from Table A.3 which are given by Terzaghi and Peck, (1943).

Table 3. 4 Values of Saturated unit weight for each layer

Depth of soil profile	Number of blows	Value of saturated unit weight (KN/M ³)
0-4	11	19.5
4-6	12	19.79
6-8	14	20.3
8-10	16	20.6
10-12	21	21
12-14	23	21.3
14-20	24	21.5

The bulk unit weight are determined from Figure A.4 which are given by (Bowles, 1977)

Table 3. 5 Values of Bulk unit weight for each depth

Depth of soil profile	Number of blows	Bulk unit weight (KN/m) ³
0-4	11	19
4-6	12	19.5
6-8	14	20
8-10	16	20.3
10-12	21	20.7
12-14	23	21
14-20	24	21.27

3.2.2.5 Poisons Ratio Determination

Table 3. 6 Value of Poisons ratio for each depth (Bowles, 1994)

Depth	Number of blows	Poisons ratio
0-4	11	0.35
4-6	12	0.37
6-8	14	0.38
8-10	16	0.39
10-12	21	0.4
12-14	23	0.419
14-20	24	0.425

3.2.3 Soil Nail Design

Soil nails are used to support excavation walls both temporarily and permanently. The FHWA manual is used to create the soil nail reinforcement. The link between the nail and the surrounding soil is crucial to the stability of soil nailing. In order to take the ease of construction and the site soil is not highly varied, uniform nail distribution is used for this study.

3.2.3.1 Nail Inclination and Nail Spacing

Vertical nail spacing, or S_v , and horizontal nail spacing, or S_h , are taken be the same for this study. For standard drilled and grouted soil nails, nail spacing ranges from 1.25 to 2 m (4 to 6.5 feet) (FHWA, 2015).

The soil nails are typically installed at a 5 to 15 degree slope from horizontal. For normal borehole and soil nail dimensions and conventional grout mixtures, this recommended range of soil nail inclination ensures that grout flows freely from the bottom of the hole toward the nail head (BS 8006, 2011). The nail spacing and inclination utilized in the investigation are shown in Table 3.7.

Table 3. 7 Maximum and minimum of nail spacing and inclination from (Carlos et al., 2003)

Parameters	Maximum	Average	Minimum
Nail spacing	2 m	1.625 m	1.25 m
Nail inclination	15 ⁰	10°	5 ⁰

3.2.3.2 Nail Bonded Length

The bonded lengths was calculated using steps specified in section 2.5.5.

Table 3. 8 Values of undrained shear at the middle of each layer and nail pullout capacity

Depth	γ_{sat}	C (kPa)	γ^*h+ surcharge	undrained shear	Rp	Rpd
2	19.5	44.43	264	99.87	47.04	31.36
5	19.79	82.1	323.95	150.13	70.71	47.14
7	20.3	95.1	367.1	172.19	81.1	54.07
9	20.6	101.5	410.4	187.68	88.4	58.93
11	21	133.5	456	279.42	131.61	87.74
13	21.3	144.3	501.9	304.91	143.61	95.74
17	21.5	156.25	590.5	357.02	168.16	112.10

The nail lengths are given in Table 3.9

Table 3. 9 Bonded and total Nail length for different depth of excavation and nail spacing

Depth (m)	Spacing (m)	Bonded nail length (m)	Total nail length (m)
6	1.25	5	10
6	1.625	7	12
6	2	8	13
9	1.25	7	12
9	1.625	8	13
9	2	10	15
12	1.25	10	15
12	1.625	11	16
12	2	14	19

3.2.4 Wall parameters Determination

As specified FHWA the thickness of temporary facing (h) is between the ranges of 100mm-200mm. for this study 200 mm was used. The Elastic modules of shotcrete was taken as 21 GPa, the density of concrete was taken to be 2400 Kg/m³. Using equation

$$\text{Flexural rigidity } EI = E \cdot \frac{h^3 \cdot b}{12} \quad (3.5)$$

$$\text{Normal stiffness } EA = E \cdot h \cdot b \quad (3.6)$$

$$\text{Unit weight of concrete} = \text{density of concrete} \cdot 9.81 \text{ m/s}^2 \quad (3.7)$$

$$\text{Weight of concrete per meter width} = \text{Unit weight of concrete} \cdot h \quad (3.8)$$

The static Poisson's ratio normally for hardened concrete varies between 0.15-0.25. for this study 0.2 was used.

3.2.5 Geogrid (grouted body) Parameters Determination

As FHWA specified the diameter of drilled borehole is between the ranges of 100mm-200mm. for this study 150 mm was used. Since the diameter of nail bar is small, it is neglected in the calculation. Elastic modules was taken as 21 MPa. The area of drilled hole was calculated using equation 3.9.

$$A = \pi r^2 \quad (3.9)$$

Where, r is the radius of drilled hole.

Table 3. 10 Summery Plate (diaphragm wall) and Geogrid (Grouted body) property

Material	Parameter	Value
Diaphragm wall	Flexural stiffness per meter (EI)	4.2 Mpa
	Axial stiffness per meter (EA)	14 KPa
	Thickness of wall (h)	0.2 m
	Weight of wall per meter	4.7 KN/m
	Poisson's ratio v	0.2
Geogrid (grouted soil nail)	Axial stiffness (EA)	3.78*10 ⁵ KPa

3.2.6 Horizontal displacement

The requirements for movement control will vary depending on project restrictions. Temporarily nailed walls built on granular soils with no neighboring structures, for example,

pose low risk of displacement. However, for a temporary excavation support system in a huge city, wall and ground movements may be the key design challenge. (FHWA, 2015).

For walls made of sands and strong clays, maximum lateral wall movements average around 0.2 percent H, with a maximum of about 0.5 percent H, where H is the wall's height. For this study, the allowable lateral displacement was calculated using 0.2%H. As a result, the horizontal displacement was compared to the allowable lateral displacement (0.2%H).

CHAPTER FOUR

NUMERICAL MODELING

4.1 Introduction

Finite Element Analysis (FEA) is a numerical method used to solve engineering and mathematical physics issues. The solution is piecewise represented in terms of defined basis functions in FEM discretization. In FEM, a solution is approximated using interconnecting sub-regions or elements rather than a grid of hexahedral cells or nodes. Typically, these elements are simply geometrical shapes (Safuan et al., 2013).

When using FEM to investigate slope stability, there are two methods. The first way is to increase the gravity load of the soil element, while the second is to reduce the strength characteristics of the soil mass, which is commonly referred to as the Shear Strength Reduction (SSR) technique. The SSR technique was used in this work with the help of plaxis 2D, a robust FEM-based software. The slope materials were believed to have elasto-plastic behavior in the SSR approach. The SSR is based on the gradual lowering of soil strength parameters until slope failure occurs (Mahmud Sazzad et al., 2015). The processes for modeling and analyzing slopes in FEM (plaxis 2D) are given in this section.

4.2 Formulation of Finite Element Method

The method of discretizing a continuum into small pieces or elements to describe the behavior or actions of individual parts and reconnecting them to represent the behavior of the continuum as a whole is known as finite element modeling. In deformation analysis, each element contains some nodes, and each node has some degrees of freedom corresponding to displacement components (Mekonnen, 2020).

Specifying the problem domain is the first phase in FEM, which includes defining the geometry model, material model, and boundary conditions. The domain can then be discretized into a sequence of continuum components to create a mesh that connects nodes to transfer forces and displacements. Plaxis 2D software was used to discretize soils using triangular areal elements. The element stiffness functions are created when the problem domain has been established and discretized into finite elements.

$$[K]\{d\} = \{f\} \quad (4.1)$$

$$K = \int B^T D^e B dV \quad (4.2)$$

Where K is element stiffness matrix, d is a vector of an unknown degree of freedom or nodal displacement and f is a resultant vector of element nodal forces. The elastic element stiffness matrix is also expressed as a function of strain interpolation matrix (B) and an elastic material matrix (D^e).

4.3 Modeling with plaxis 2D

4.3.1 Geometric modeling

Before generating a finite element mesh, the full geometry model was completed. This signifies that the geometry model's boundary conditions and model parameters were entered and applied first (plaxis 2D - Version 8, 2013).

Because there is a uniform cross section and corresponding stress state and loading scheme over a specified length perpendicular to the cross section (z-direction), a plain strain model was utilized for the analysis. In the z-direction, displacements and strain are assumed to be zero. Normal strains in the z-direction, on the other hand, are fully considered (plaxis 2D v8, 2013).

The research provided use of a 15-node triangle. It is a precise ingredient that has resulted in high-quality stress outcomes for complex challenges. The use of 15-node triangles consumes a lot of memory and makes calculations and operations take a long time (plaxis 2D, 2013).

The current paper conducted 81 analyses for various wall depths, surcharges, nail inclination, and nail spacing. The height of the soil below and the width of soil from the excavation depth must be adequate up to not influence the lateral displacement of the soil. The depth is specified using sensitivity analysis. As Figure 4.1 shows effect of depth below the tip of the wall becomes negligible. So 20m depth is used for the analysis. And the width is taken as 25 meter which is $2H$. The geometric model of a layered soil with a major clay type of soil and the location of the retaining wall is shown in Figure 4.2.

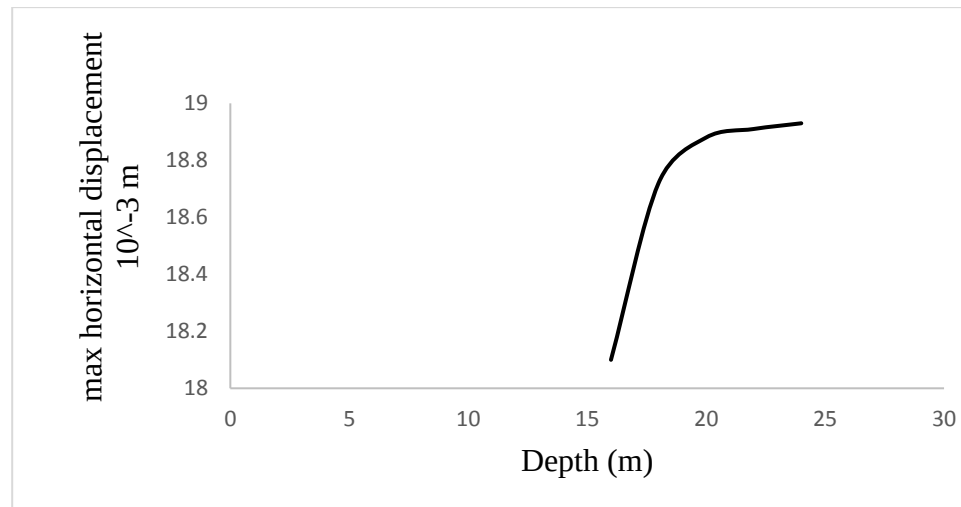


Figure 4. 1 Sensitivity analysis for determination of depth of soil below tip of wall

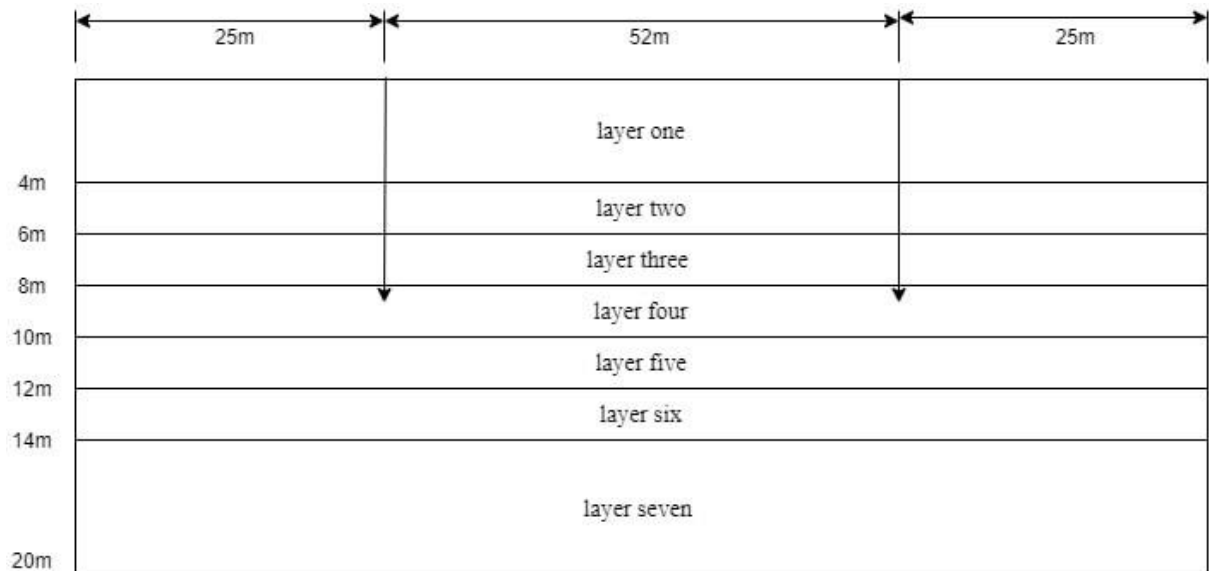


Figure 4. 2 Depth of layers and position of wall

4.3.2 Boundary Condition (fixities) and Loading

To depict the actual situation, appropriate boundary conditions (both essential and natural) must be applied during modeling. Fixities, surcharge loads, and regulated displacements were all part of this. The geometric boundaries were horizontally constrained on the left and right sides ($u_x = 0$ and $u_y = \text{free}$) and entirely fixed ($u_x = u_y = 0$) at the bottom of the geometry in the models. As a result, roller supports were represented by two vertical parallel lines, whereas full fixity was represented by crosshatched lines. The loads tool is used to draw a distributed load in both directions (walls).

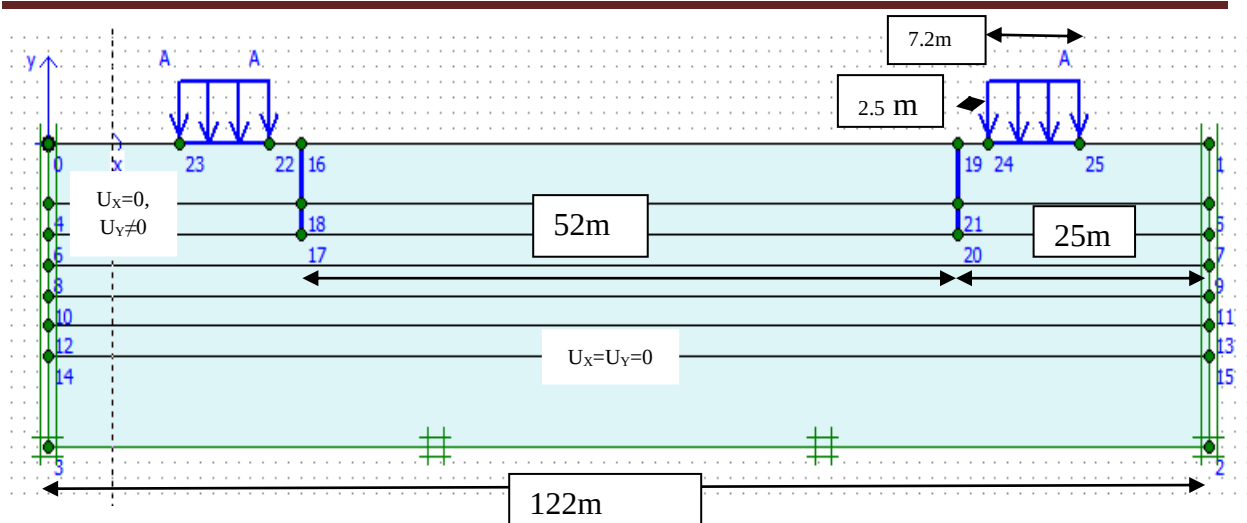


Figure 4. 3 Boundary condition and loading

4.3.3 Material Modeling

Plaxis distinguishes between soil data sets, plates, nails, and geogrid material data sets. Table 4.1, Table 4.2, Table 4.3, Table 4.4, Table 4.5, and Table 4.6 shows the soil properties used for layer 1, layer 2, layer 3, layer 4, layer 5, layer 6 and respectively. There were seven layers of soil to consider. Plate structural elements were used to simulate the wall, Geogrids for the grouted soil nails, and struts for the ungrouted soil nails. The following tables show the values of soil parameters used. The Mohr columb soil model was used for this study.

Table 4. 1 Soil parameters for layer 1

Layer	Parameters	Value
Layer 1	Young modules (E)	47.52 Mpa
	Poisson's ratio (ν)	0.35
	Unsaturated unit weight (γ)	19 kN/m ²
	Saturated unit weight (γ_s)	19.5 kN/m ²
	Cohesion (C)	44.43 kPa
	Frictional angle (ϕ)	18 ⁰
	Dilatancy angle (ψ)	0 ⁰

Table 4. 2 Soil parameters for layer 2

Layer	Parameters	Value
Layer 2	Young modules (E)	51.84 Mpa
	Poisson's ratio (ν)	0.37
	Unsaturated unit weight (γ)	19.5 kN/m ²
	Saturated unit weight (γ_s)	19.7 kN/m ³
	Cohesion (C_u)	82.1 kPa
	Frictional angle (ϕ)	18 ⁰
	Dilatancy angle (ψ)	0 ⁰

Table 4. 3 Soil parameters for layer 3

Layer	Parameters	Value
Layer 3	Young modules (E)	60.48 Mpa
	Poisson's ratio (ν)	0.383
	Unsaturated unit weight (γ)	20 kN/m ²
	Saturated unit weight (γ_s)	20.3 kN/m ³
	Cohesion (C_u)	95.1 kPa
	Frictional angle (ϕ)	18 ⁰
	Dilatancy angle (ψ)	0 ⁰

Table 4. 4 Soil parameters for layer 4

Layer	Parameters	Value
Layer 4	Young modules (E)	69.12 Mpa
	Poisson's ratio (ν)	0.39
	Unsaturated unit weight (γ)	20.3 kN/m ²
	Saturated unit weight (γ_s)	20.6 kN/m ³
	Cohesion (C_u)	101.5 kPa
	Frictional angle (ϕ)	18 ⁰
	Dilatancy angle (ψ)	0 ⁰

Table 4. 5 Soil parameters for layer 5

Layer	Parameters	Value
Layer 5	Young modules (E)	90.72 Mpa
	Poisson's ratio (ν)	0.4
	Unsaturated unit weight (γ)	20.7 kN/m ²
	Saturated unit weight (γ_s)	21 kN/m ³
	Cohesion (C_u)	133.9 kPa
	Frictional angle (ϕ)	27 ⁰
	Dilatancy angle (ψ)	0 ⁰

Table 4. 6 Soil parameters for layer 6

Layer	Parameters	Value
Layer 6	Young modules (E)	99.36 Mpa
	Poisson's ratio (ν)	0.419
	Unsaturated unit weight (γ)	21 kN/m ²
	Saturated unit weight (γ_s)	21.3 kN/m ³
	Cohesion (C_u)	144.3 kPa
	Frictional angle (ϕ)	27 ⁰
	Dilatancy angle (ψ)	0 ⁰

Table 4. 7 Soil parameters for layer 7

Layer	Parameters	Value
Layer 7	Young modules (E)	108 Mpa
	Poisson's ratio (ν)	0.425
	Unsaturated unit weight (γ)	21.27 kN/m ²
	Saturated unit weight (γ_s)	21.5 kN/m ³
	Cohesion (C_u)	156.25 kPa
	Frictional angle (ϕ)	28 ⁰
	Dilatancy angle (ψ)	0 ⁰

The analysis was conducted for 6m, 9m and 12m wall heights (plate heights) with 12,118,225 kPa. The properties of the plate (diaphragm wall), geogrid (grouted soil nails) and node to node (struts) is presented in Table 4.8.

Table 4. 8 Properties of wall and grouted soil nail

Material	Parameter	Value
Diaphragm wall	Flexural stiffness per meter (EI)	4.2 Mpa
	Axial stiffness per meter (EA)	14 kPa
	Thickness of wall (d)	0.2 m
	Weight of wall per meter	4.7 Kn
	Poisson's ratio ν	0.2
Geogrid (grouted soil nail)	Axial stiffness (EA)	3.78×10^5 kPa

Table 4.9 shows the nail properties for wall heights of 6m, 9m, and 12m.

Table 4. 9 Nail property

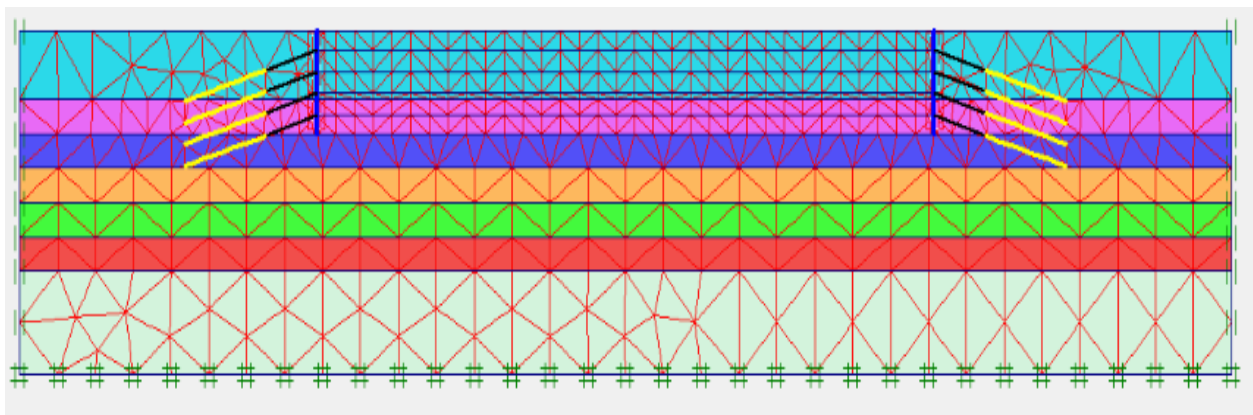
For 6m wall height					
property	Nail spacing (m)				
	1.25	1.625	2		
Strut length (m)	4.5	4.5	4.5		
Grogrid length (m)	5	7	8		
EA of geogrid (KPa)	3.78*10 ⁴	3.78*10 ⁴	3.78*10 ⁴		
EA of strut (KPa)	7.74*10 ⁴	7.74*10 ⁴	7.74*10 ⁴		
L _s (m)	1.25	1.625	2	2	2
No of nails	4	3	3		
For 9m wall height					
property	Nail spacing (m)				
	1.25	1.625	2		
Strut length (m)	4.5	4.5	4.5		
Grogrid length (m)	7	8	10		
EA of geogrid (KPa)	3.78*10 ⁴	3.78*10 ⁴	3.78*10 ⁴		
EA of strut (KPa)	1.02*10 ⁵	1.02*10 ⁵	1.02*10 ⁵		
L _s (m)	1.25	1.625	2	2	2
No of nails	6	6	4		
For 12m wall height					

property	Nail spacing (m)		
	1.25	1.625	2
Strut length (m)	4.5	4.5	4.5
Grogrid length (m)	10	11	14
EA of geogrid (KPa)	3.78×10^4	3.78×10^4	3.78×10^4
EA of strut (KPa)	1.64×10^5	1.64×10^5	1.64×10^5
L_s (m)	1.25	1.625	2
No of nails	9	7	6

4.3.4 Mesh Generation

The finite element numerical approach is based on the discretization of a continuous into finite elements, with each element's behavior or actions described and then connected to represent the continuum. Different sorts of elements, such as a beam, shell, or solid, as well as different types of forms, such as triangular, quadrilateral, and tetrahedral, can be employed in FE discretization, depending on the nature of the project. The 4th order 15-node triangular element was used in this study. In general, decreasing element size and increasing the order of interpolation improves the accuracy of the answer (i.e. quadratic is more accurate than linear). However, when the element size decreases and the sequence of interpolation increases, the computation time increases (plaxis 2D, 2013).

Finally, mesh generation for all 81 analyses was completed. A sample mash for a 6 meter wall height, 15 degree nail inclination, and 1.25 meter nail spacing is shown in Figure 4.4.



.Figure 4. 4 Mesh of 6m wall at 15° nail inclination and 1.25 nail spacing

4.3.5 Initial Conditions

The initial stress state and the initial configuration were specified after the geometry model and finite element mesh were created. The water table in the ground is 4 meters below ground level. Pore water pressure for a 6 meter wall height, 15° nail inclination, and 1.25 meter nail spacing is shown in Figure 4.5.

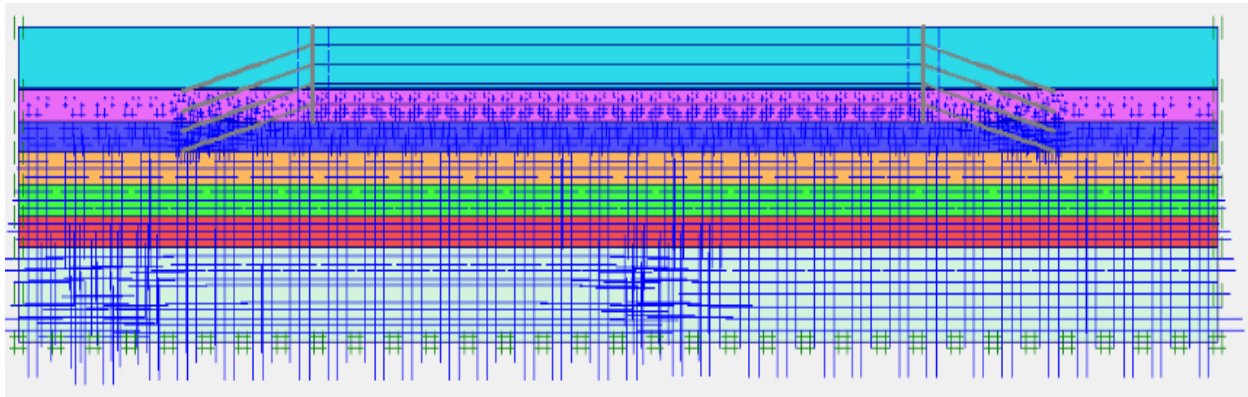


Figure 4. 5 Pore water pressure for 6 meter wall height, 5° nail inclination and 1.25 meter nail spacing.

4.3.6 Calculation

For the project, a plastic calculation was employed to do an elastic-plastic deformation study in which the degradation of surplus pore water pressure with time was not taken into account. Unless the soft soil creep mode is selected, a plastic calculation does not take time effects into account, despite the fact that a time interval can be given. For the limiting scenario of entirely undrained behavior, a plastic calculation must be utilized when considering a fast loading of water-saturated clay-type soils (plaxis 2D - Version 8, 2013). Under the ground water table, the soil was considered as an undrained computation.

When soil plasticity is factored into the equations, the equations become nonlinear. This means that each computation phase must be completed in a sequence of steps (load steps). The selection of step sizes and the solution algorithm to be employed is an important aspect of the non-linear solution technique (plaxis 2D - Version 8, 2013).

Plastic calculations with staged constructions loading input and standard values for all other parameters were used in all calculation phases. The common phases defined within the staged building mode were as follows.

Table 4. 10 Phases for the staged construction

Stags of construction	Activity
Phase 1	1. Activate the wall. 2. Activate the surface loads and assign load value
Phase 2	3. De-activate (excavate) the upper cluster of the Excavation
Phase 3	4. Activate the upper strut and Geogrid for both sides of excavation
Phase 4	5. De-activate (excavate) the second cluster of Excavation
Phase 5	6. Dc-activate the second strut and Geogrid in both sides of excavation.
Phase 6	7. De-activate (excavate) the third cluster of excavation.
The iteration continues until the last cluster is de-activated (excavated).	

4.3.7 Out Put

The next two Figures show one of deformed mesh and extreme horizontal displacement mesh at the end of calculation phases for 6m wall height and 12 kPa surcharge with 15° nail inclination and 1.25 nail spacing. The failure of the soil behind the wall face is occurred because, As the wall face moves horizontally the soil behind the face will settle. The soil surrounding the surcharge and beneath the surcharge moves downward. This type of failure is called punching shear failure.

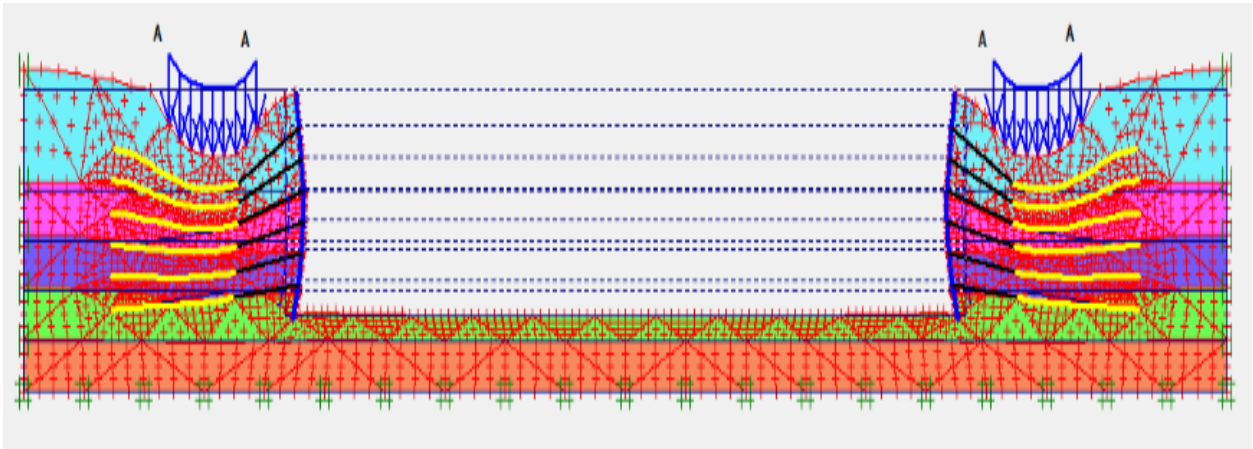


Figure 4. 6 Deformed mesh of

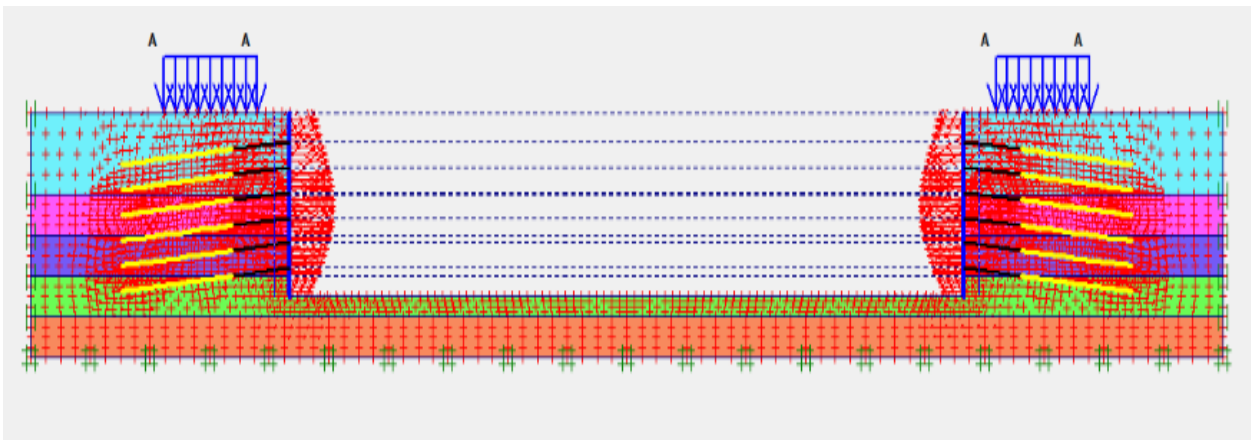


Figure 4. 7 Horizontal displacement mesh

CHAPTER FIVE

RESULT AND DISCUSSION

By using Plaxis 2D software, the results of stability analysis of soil - nailed slopes were modeled and presented with special attention paid to the effect of surcharge, depth of excavation, nail spacing and nail inclination on the lateral displacement of soil nailed slopes.

5.1 Discussion on the pattern of deformation on single analysis

The sample computed horizontal displacement profiles for 6m, 9m, and 12m deep excavation are shown in Figures 5.1, 5.2, and 5.3. In all analyses, the deformed wall had the same shape. It also depicts the magnitude of horizontal displacements at the excavation's top and bottom. Most of the analyses show that the maximum deflection occurred at 4 meters for all depths, with some of them occurred near the midpoint of the excavation depth. It usually increased up to 4 meters before starting to decrease. This is due to the fact that the sum of the overburden pressure and surcharge is greatest at 4 meters below ground level (high mobilized shear stress). This is due to high effective stress above 4m depth and high stiffness below 4m excavation depth. The horizontal displacement at the top of the wall is less than the lateral displacement at the bottom of the wall when the excavation depth was 6m. For 9 and 12 meter excavation depths, the opposite is true. This is due to the fact that the influence of surcharge diminishes as the depth increases, and the soil stiffness.

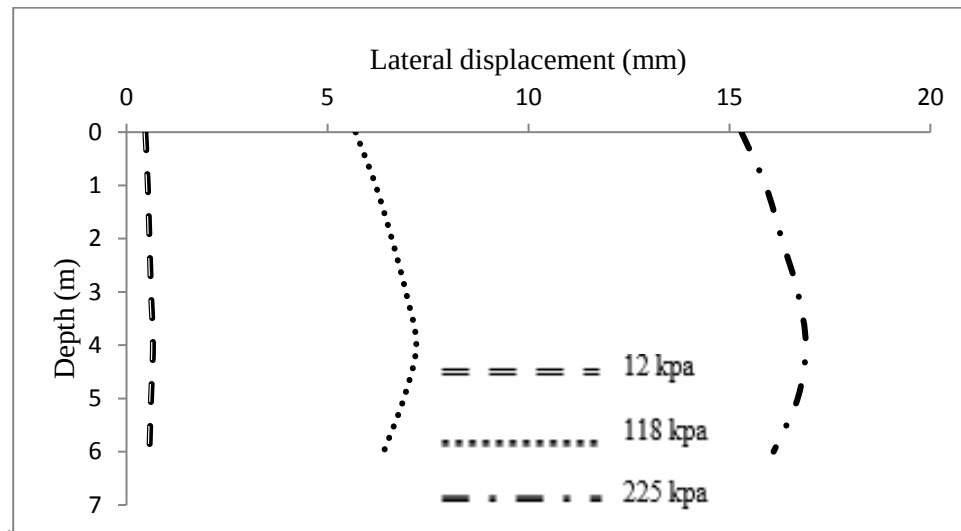


Figure 5. 1 Horizontal displacement on wall face for 6m single analysis

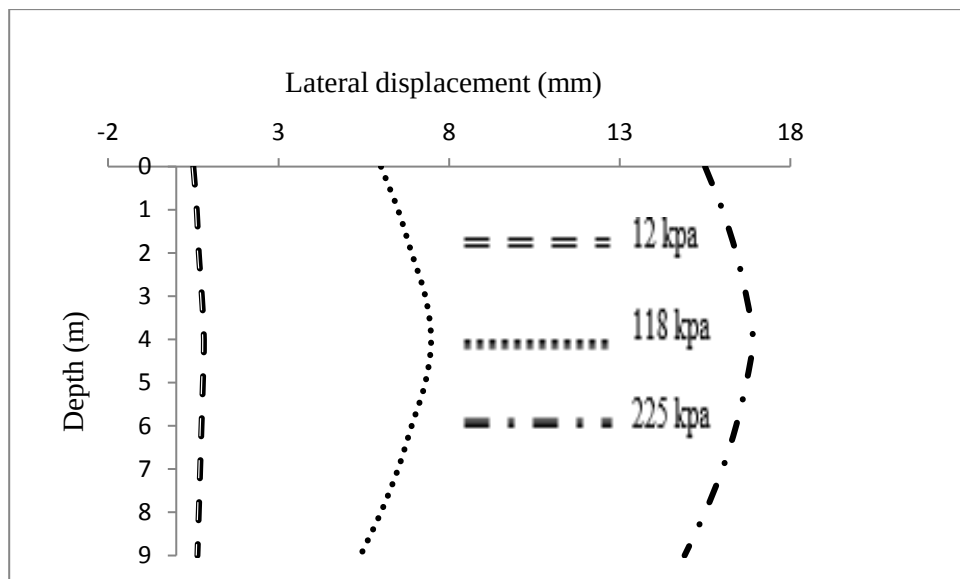


Figure 5. 2 Horizontal displacement on wall face for 9m single analysis

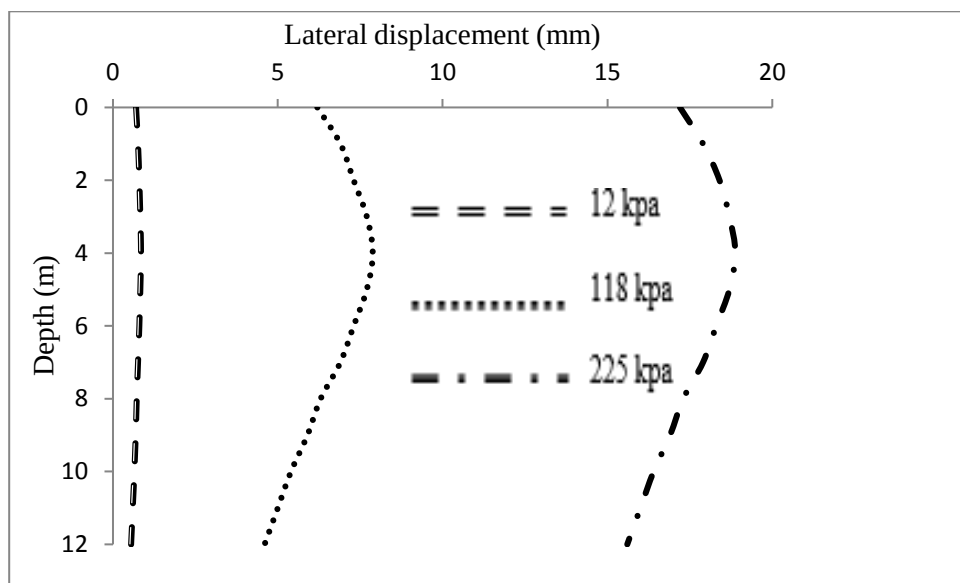


Figure 5. 3 Horizontal displacement on wall face for 12m single analysis

5.1.1 Validation for the pattern of deformed wall

The lateral displacement of a nailed wall was validated using a study by Ma et al., (2011). The maximum lateral displacement increased up to a certain depth and then decreased, as shown in figure 5.4. These excavation were modeled using Plaxis -2D software with the same depth of excavation, mesh size and boundary condition. Accordingly, the results obtained from the present evaluation show a good agreement with the result of Ma et al (2011).

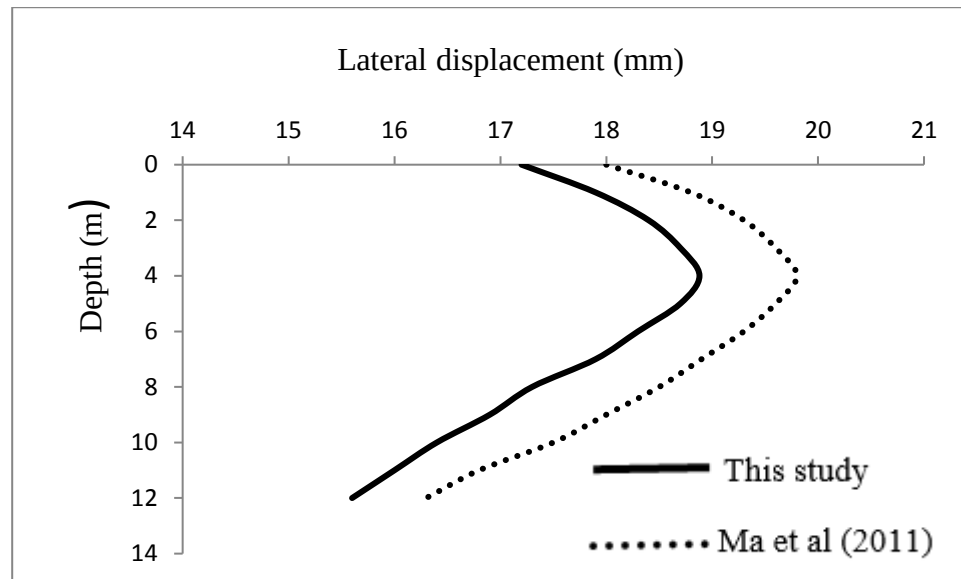
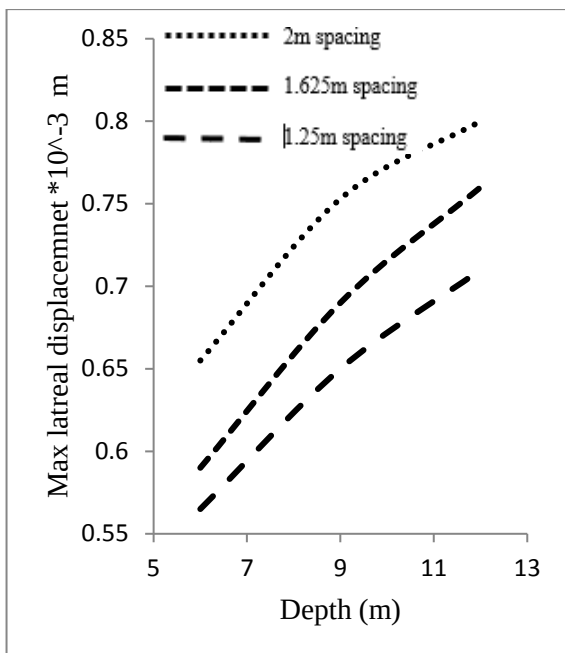


Figure 5. 4 Horizontal displacement from Ma et al., (2011) and this study

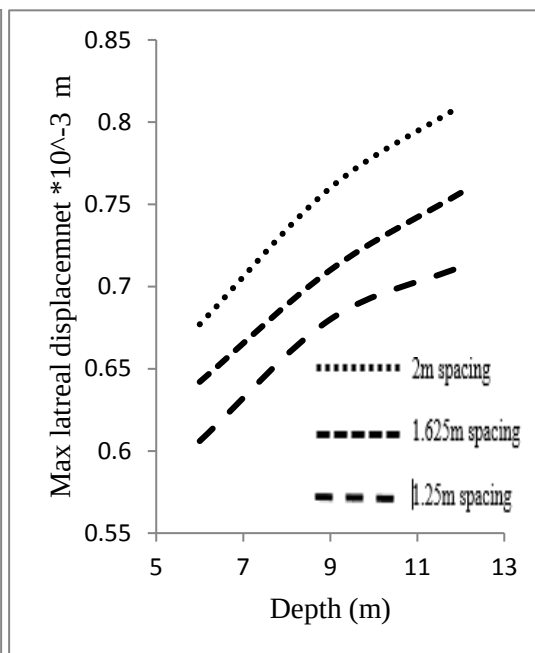
5.2 Effect of depth of excavation on the Lateral Displacement of Wall Face

Figure 5.5 shows the maximum lateral displacements for 12 KPa surcharge with a) 5°, b) 10° and c) 15° nail inclination, 118 surcharge with d) 5°, e) 10° and f) 15° nail inclination and 118 surcharge with g) 5°, h) 10° and h) 15° nail inclination at different excavation depth.

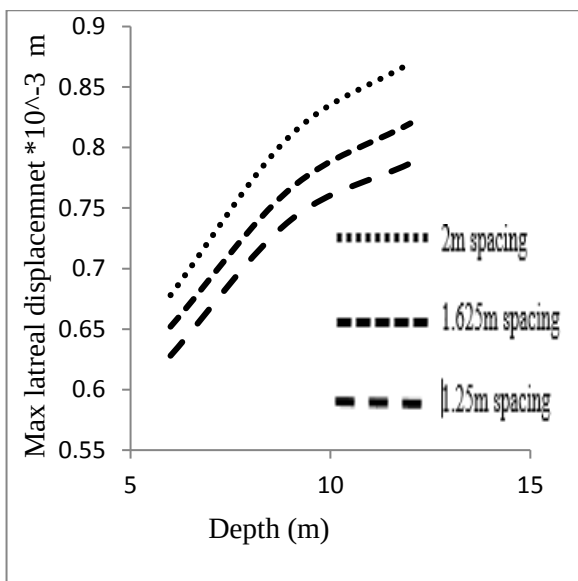
As the findings indicated, the depth of excavation had a significant impact on the size of the lateral displacement. At a given depth, the maximum lateral displacements increases. This is because as the depth grows, the soil confinement increases as well, and the influence of surcharge diminishes. When the depth was increased by 50%, the maximum lateral displacement was increased by 9-10%, and when the depth was increased by 33%, the maximum lateral displacement was increased by 5%.



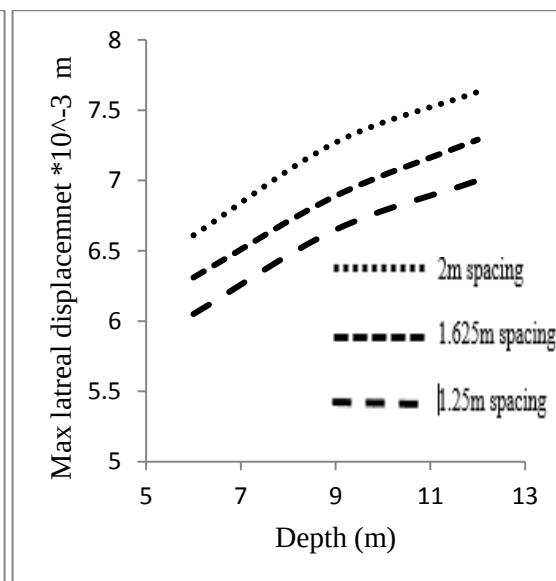
a) $q = 12$ KPa , $\alpha = 5^\circ$



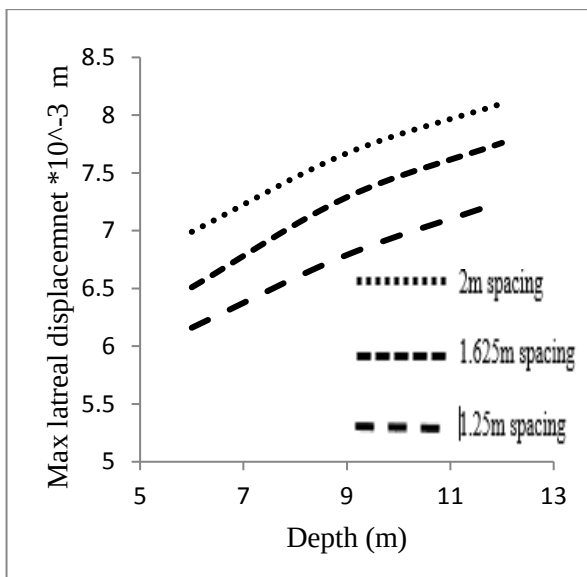
d) $q = 12$ KPa , $\alpha = 10^\circ$



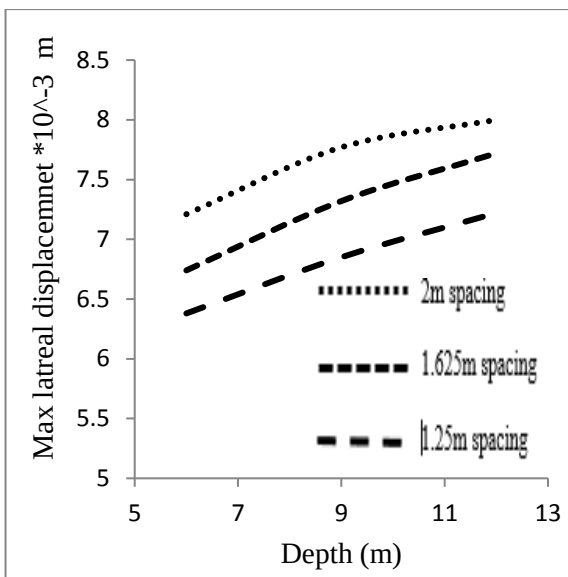
c) $q = 12$ KPa , $\alpha = 15^\circ$



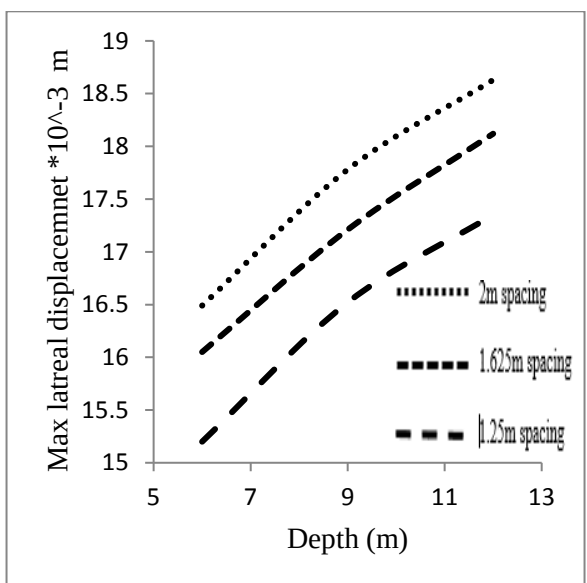
d) $q = 118$ KPa , $\alpha = 5^\circ$



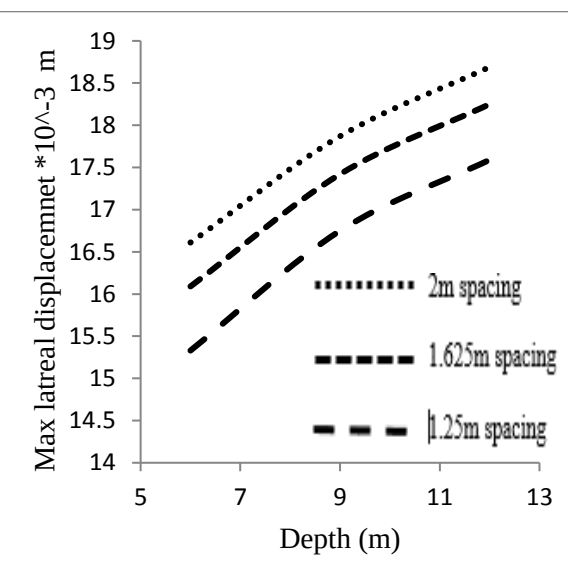
e) $q = 118 \text{ KPa}$, $\alpha = 10^\circ$



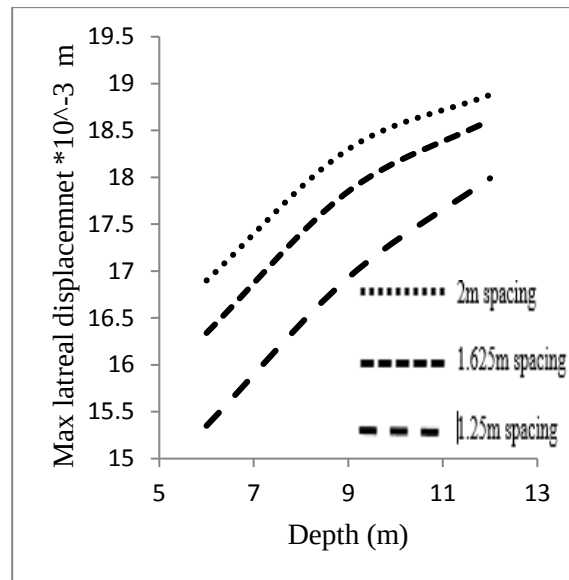
f) $q = 118 \text{ KPa}$, $\alpha = 15^\circ$



g) $q = 225 \text{ KPa}$, $\alpha = 5^\circ$



h) $q = 225 \text{ KPa}$, $\alpha = 10^\circ$



i) $q = 225 \text{ KPa}$, $\alpha = 15^\circ$

Figure 5. 5 Effect of depth of excavation on the maximum lateral displacement

5.3 Effect of Surcharge on the Lateral Displacement of Wall Face

Figure 5.6 illustrates the computed maximum lateral displacements at various excavation depths for 1.25 meter nail spacing with a) 5°, b) 10°, and c) 15° nail inclination, 1.625 meter nail spacing with d) 5°, e) 10°, and f) 15° nail inclination, and 2 meter nail spacing with g) 5°, h) 10°, and h) 15° nail inclination.

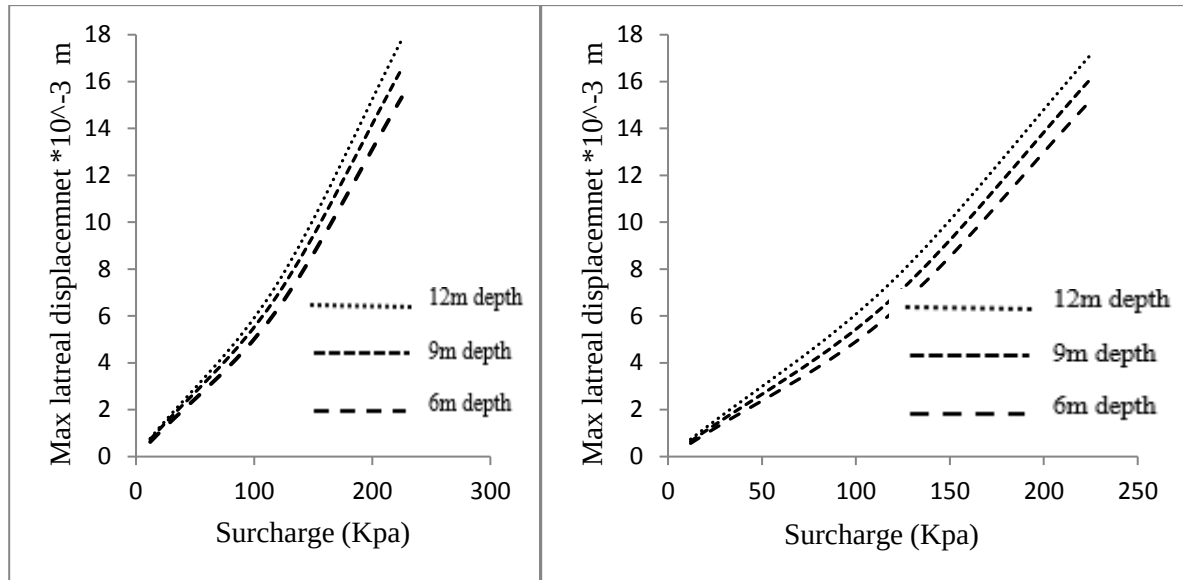
The results show that the surcharge increased mobilized shear stress, which increased the magnitude of the lateral displacements. The maximum lateral displacement increased by 10 times and 2.5 times as the surcharge increased by 9.8 times and 1.9 times, respectively.

According to FHWA the allowable lateral displacement is $0.002H$. Some of the results are above the allowable displacement.

Table 5. 1 Allowable lateral displacement

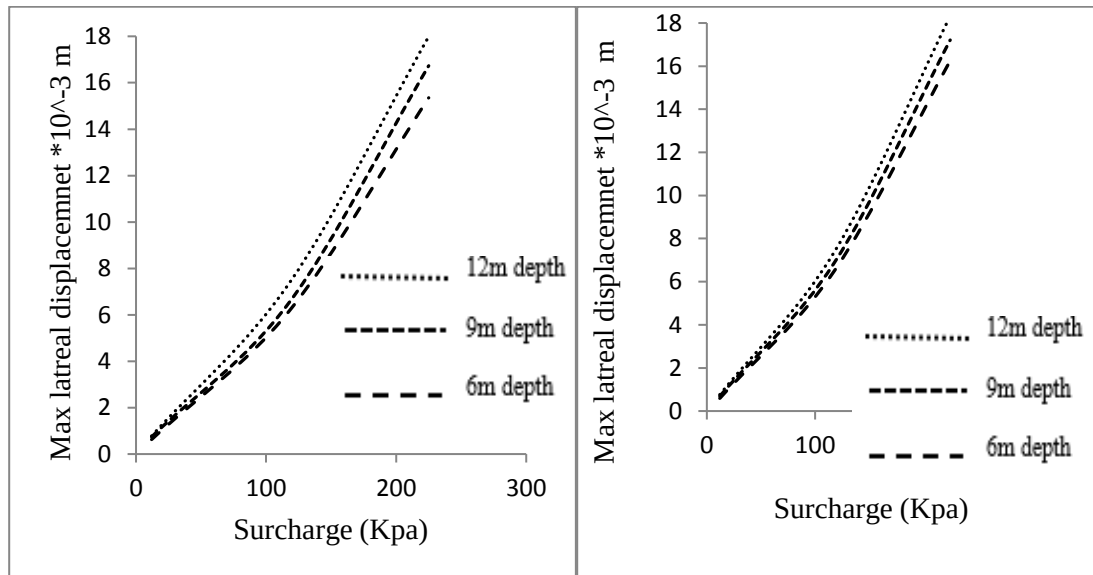
Depth (m)	Maximum allowable lateral displacement ($0.002H$) (mm)
6	12
9	18
12	24

The maximum lateral displacement of 6 meter depth of excavation with 225 surcharge, as shown in the graph, exceeds the allowable lateral displacement of 12mm. For a 12mm lateral displacement, the corresponding surcharge is 155.6 KPa. This means that if a surcharge of larger than 155.6 KPa is applied to a 6 meter depth of excavation with the given nail spacing and inclination, the allowable lateral displacement will be exceeded. For a 9-meter depth of excavation, the load surcharge for 18 mm is 225 KPa, while for a 12-meter depth of excavation, the load surcharge for 24 mm is 286 KPa. The influence of depth on maximum lateral displacement grows as surcharge increases, as the results indicate. Starting from 50 KPa, the depth has a significant impact on the maximum lateral displacement.



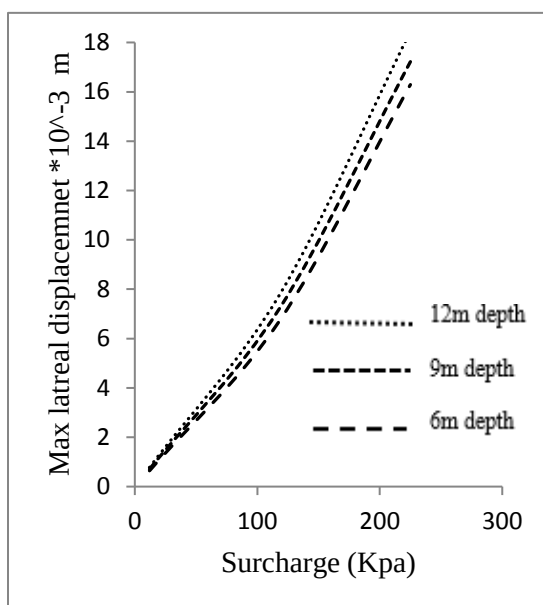
a) $S_v=1.25$ m, $\alpha=5^\circ$

b) $S_v=1.25$ m, $\alpha=10^\circ$

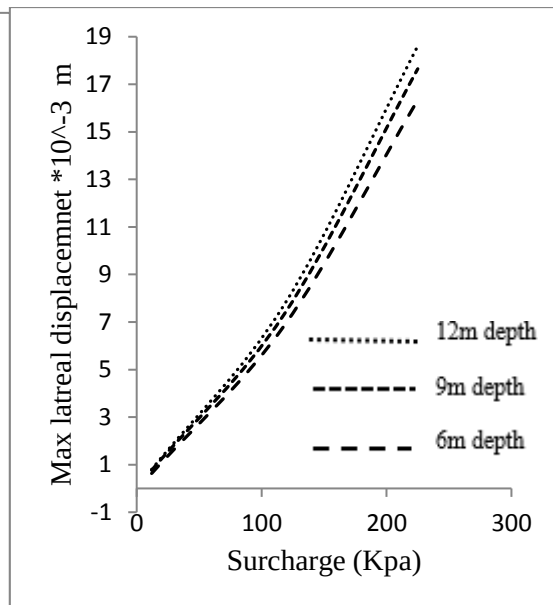


c) $S_v=1.25$ m, $\alpha=15^\circ$

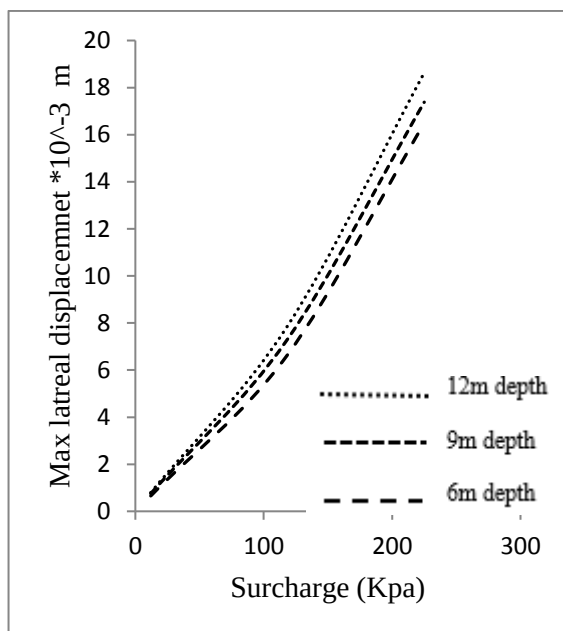
d) $S_v=1.625$ m, $\alpha=5^\circ$



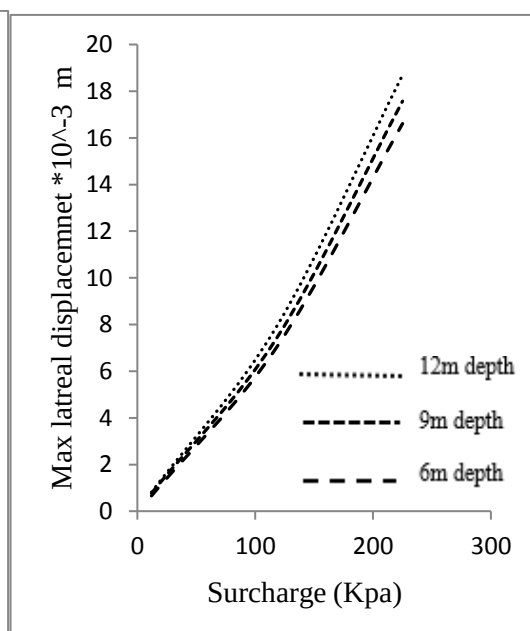
e) $S_v=1.625$ m, $\alpha=10^\circ$



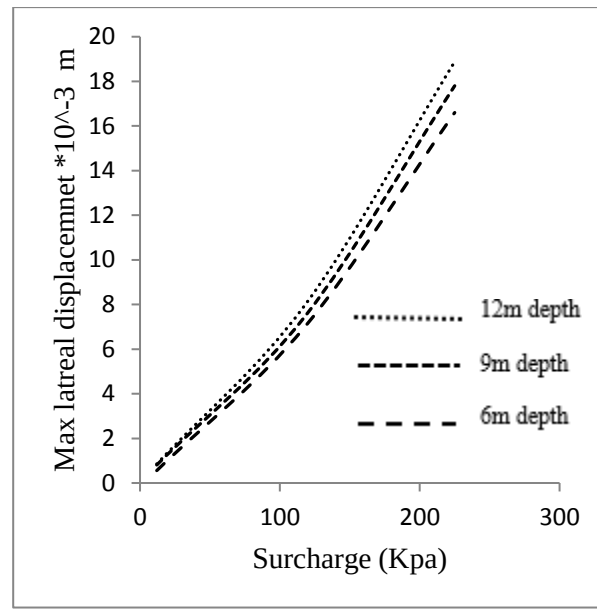
f) $S_v=1.625$ m, $\alpha=15^\circ$



g) $S_v=2$ m, $\alpha=5^\circ$



h) $S_v=2$ m, $\alpha=10^\circ$



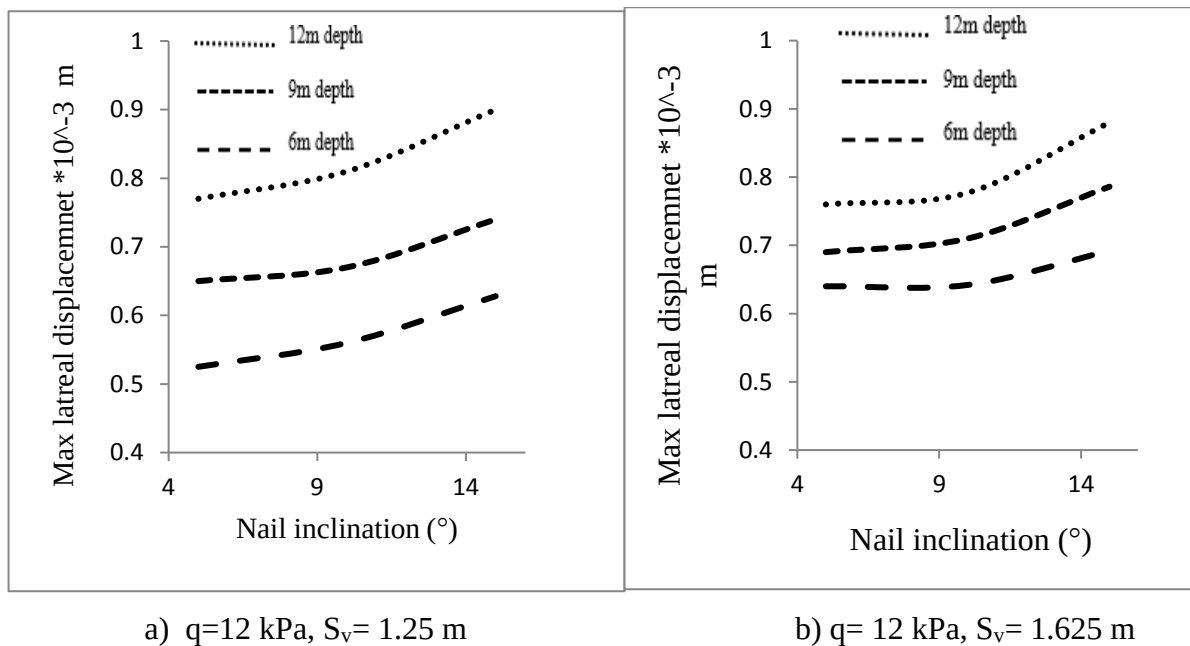
i) $S_v=2$ m, $\alpha=15^\circ$

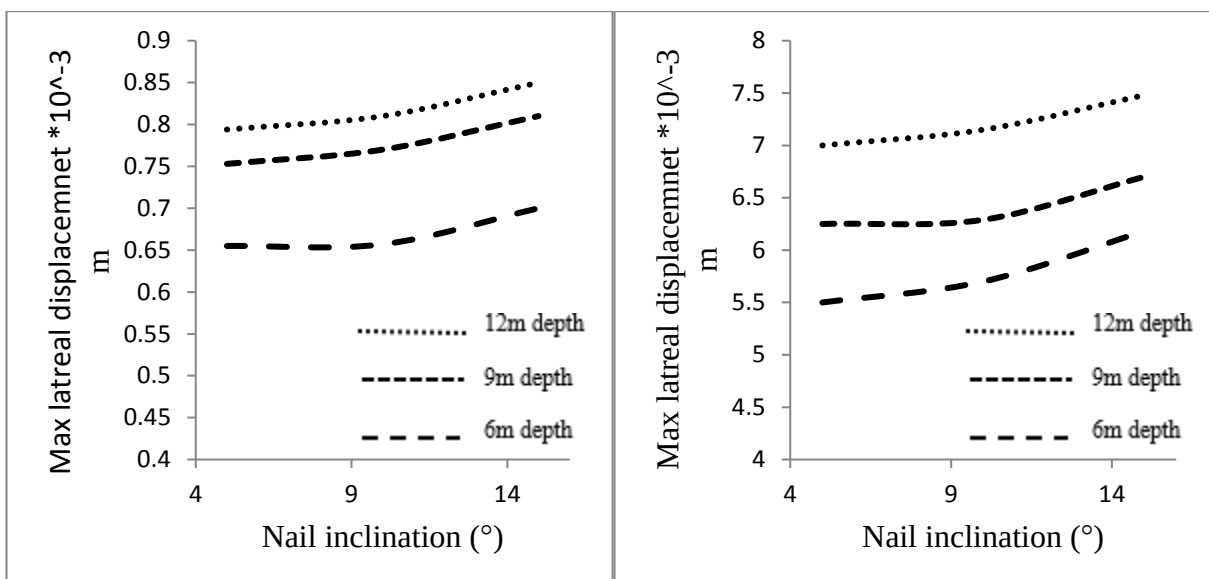
Figure 5. 6 Effect of surcharge on the maximum lateral displacement

5.4 Effect of Nail Inclination on the Lateral Displacement of Wall Face

Figure 5.7 illustrates the maximum lateral displacements for 12 KPa surcharge with a) 1.25 m, b) 1.625 m, and c) 2 m nail spacing, 118 KPa surcharge with d) 1.25 m, e) 1.625 m, and f) 2 m nail spacing, and 225 KPa surcharge with g) 1.25 m, h) 1.625 m, and i) 2 m nail spacing at different excavation depths.

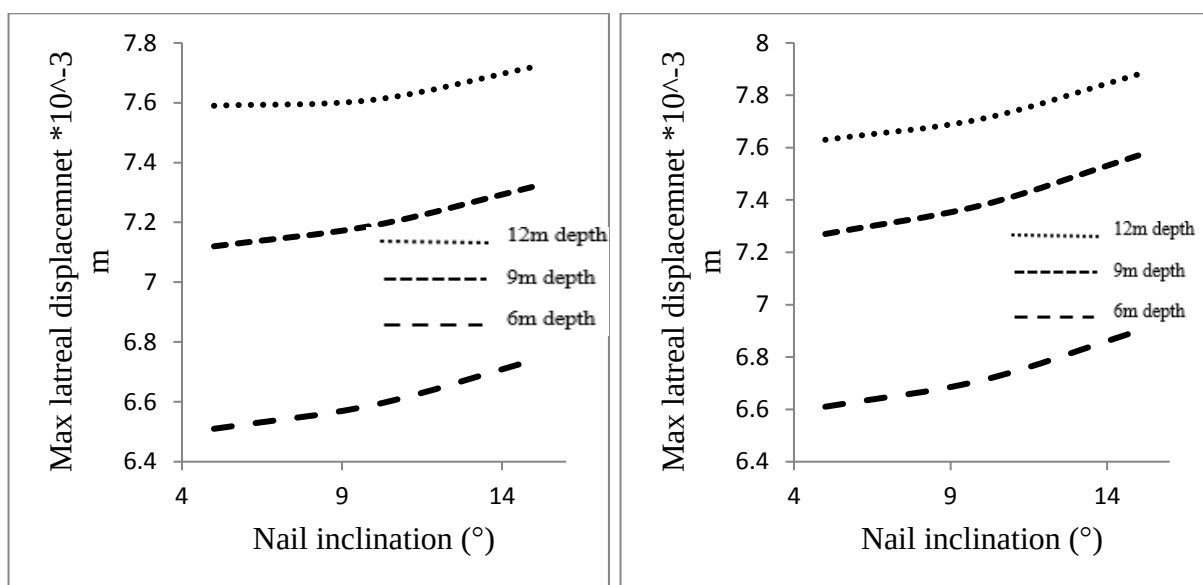
The pull out force toward the excavated face increases as the nail gets more inclined. The maximum lateral displacement increases by 3.6 percent and 8.6 percent correspondingly when the nail inclination is increased 2 times and 0.5 times.





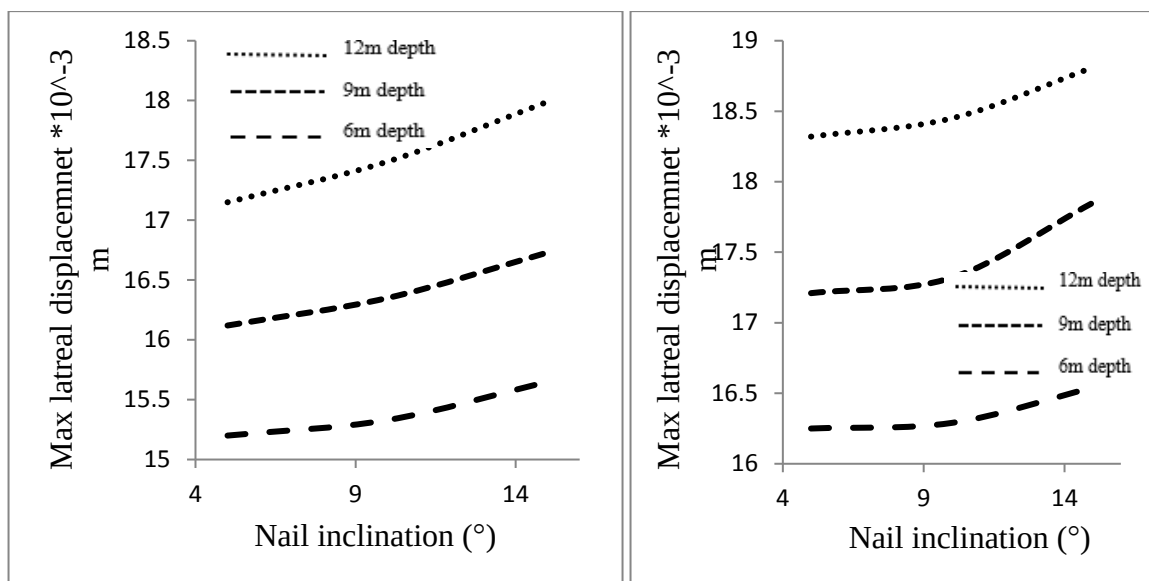
c) $q = 12 \text{ kPa}$, $S_v = 2 \text{ m}$

d) $q = 118 \text{ kPa}$, $S_v = 1.25 \text{ m}$



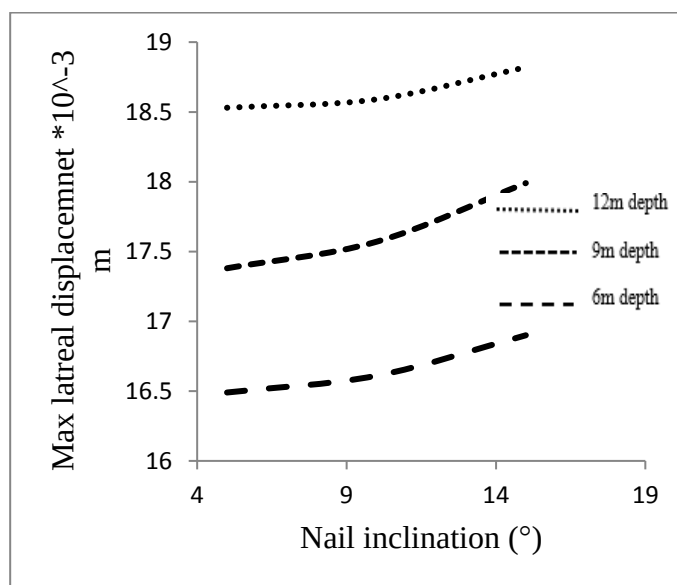
e) $q = 118 \text{ kPa}$, $S_v = 1.625 \text{ m}$

f) $q = 118 \text{ kPa}$, $S_v = 2 \text{ m}$



g)) $q = 225 \text{ kPa}$, $S_v = 1.25 \text{ m}$

h) $q = 225 \text{ kPa}$, $S_v = 1.625 \text{ m}$



i) $q = 225 \text{ kPa}$, $S_v = 2 \text{ m}$

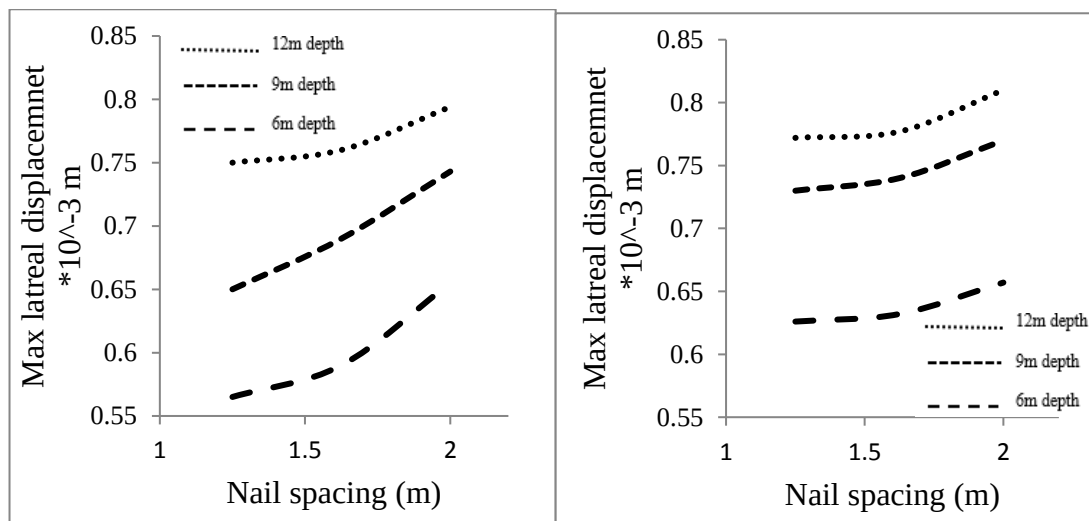
Figure 5. 7 Effect of nail inclination on the maximum lateral displacement

5.5 Effect of Nail Spacing on the Lateral Displacement of Wall Face

Figure 5.4 shows the computed maximum lateral displacements for 12 KPa surcharge with a) 5°, b) 10°, and c) 15° nail inclination, 118 KPa surcharge with d) 5°, e) 10°, and f) 15° nail inclination, and 225 KPa surcharge with g) 5°, h) 10°, and h) 15° nail inclination at various excavation depths.

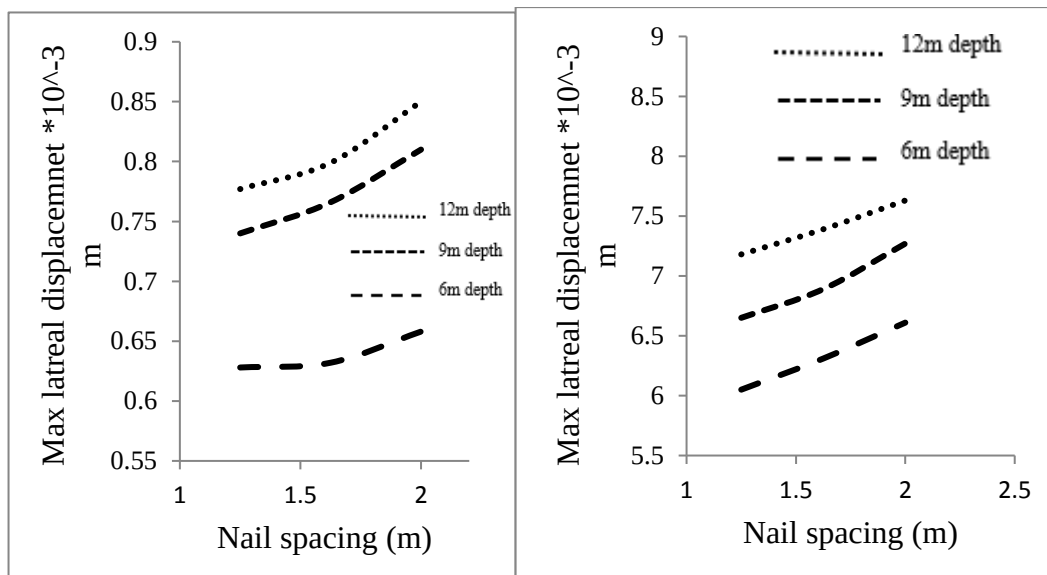
As the results indicate, nail spacing had an effect on lateral displacement. When the nail spacing is increased, the lateral displacements increase. This is because the lateral force applied on the nail increases as the nail spacing increases.

The maximum lateral displacement increases by 30 percent and 23 percent correspondingly when the nail inclination is increased 1.5 times and 0.75 times.



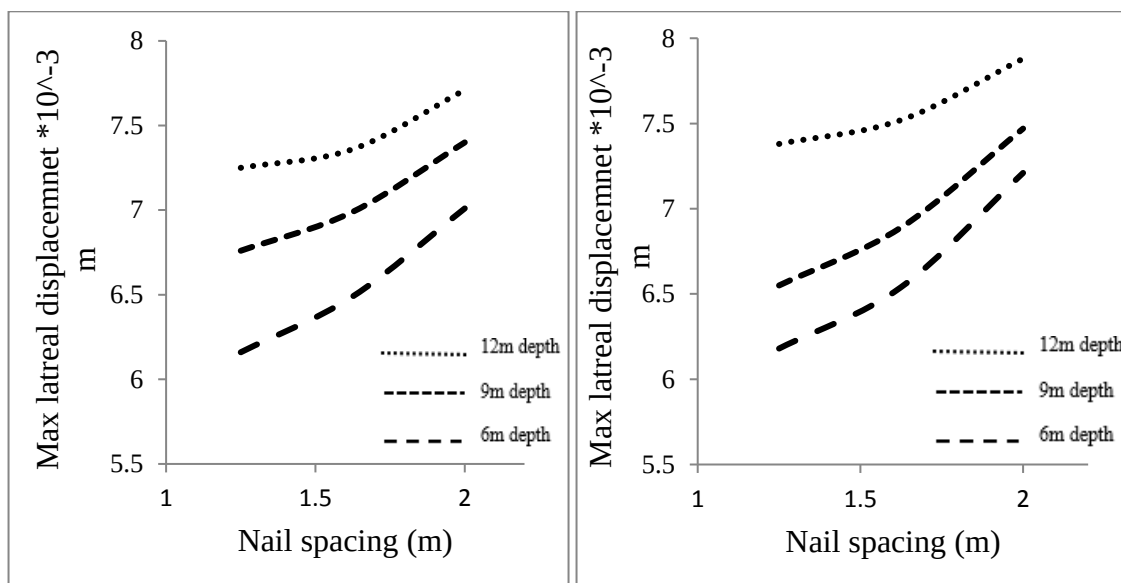
a) $q = 12$ kPa, $\alpha = 5^\circ$

b) $q = 12$ kPa, $\alpha = 10^\circ$



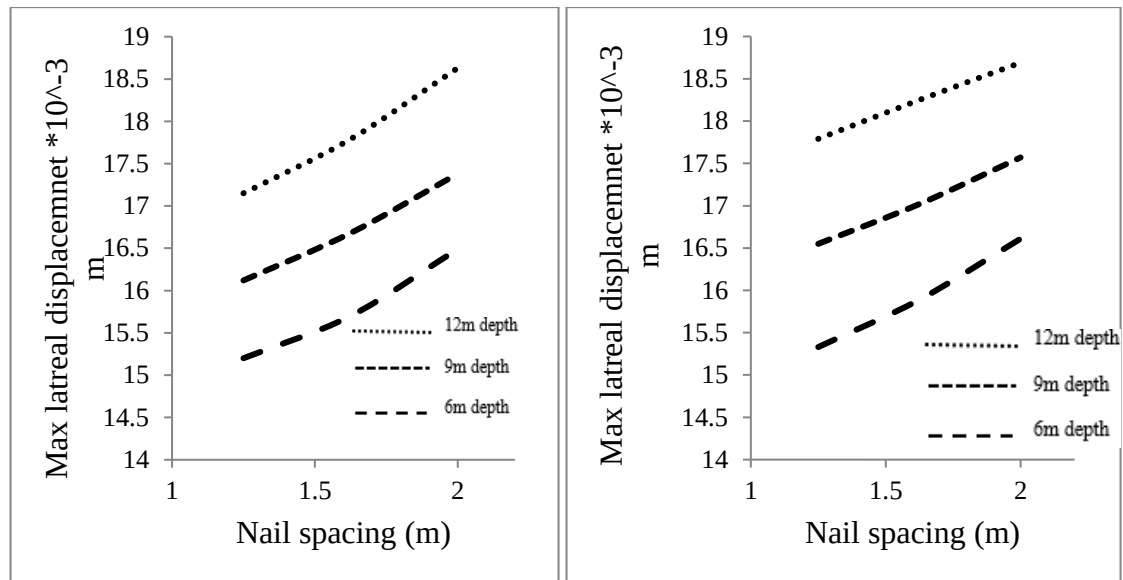
c) $q=12 \text{ kPa}$, $\alpha=15^\circ$

d) $q=118 \text{ kPa}$, $\alpha=5^\circ$



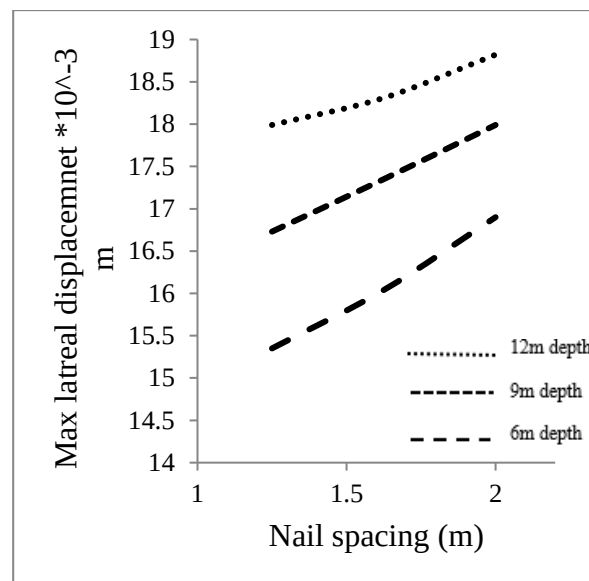
e) $q=118 \text{ kPa}$, $\alpha=10^\circ$

f) $q=118 \text{ kPa}$, $\alpha=15^\circ$



g) $q = 225 \text{ kPa}$, $\alpha = 5^\circ$

h) $q = 225 \text{ kPa}$, $\alpha = 10^\circ$



i) $q = 225 \text{ kPa}$, $\alpha = 15^\circ$

Figure 5. 8 Effect of nail spacing on the maximum lateral displacement

5.6 Comparison of Lateral Displacements With and Without Nails (only diaphragm wall)

Soil nailing is an in-situ soil reinforcement technique for improving slope, retaining wall, and excavation stability. To stabilize the soil mass, soil nails, which are closely spaced, relatively slender structural components, are installed into the ground. By introducing tensional capacity, soil nails reduce the lateral displacement of the wall. Figure 5.11 illustrates the difference between the maximum lateral displacement of a wall with and without nails at 6m, 9m, and 12m. The corresponding surcharge for 12mm displacement for only diaphragm wall supported face was 118 kPa for 6m depth of excavation. The corresponding surcharge for 18mm displacement for only diaphragm wall supported face was 130 kPa. And the corresponding surcharge for 24 mm displacement for only diaphragm wall supported face was 142.7 KPa. The shape of deformed wall face in the nailed wall is different from the only diaphragm wall supported face. In the nailed wall the horizontal displacement increased from the top and start to decrease at some point. But without nail (only diaphragm wall), the maximum lateral displacement is maximum at the top and decreases.

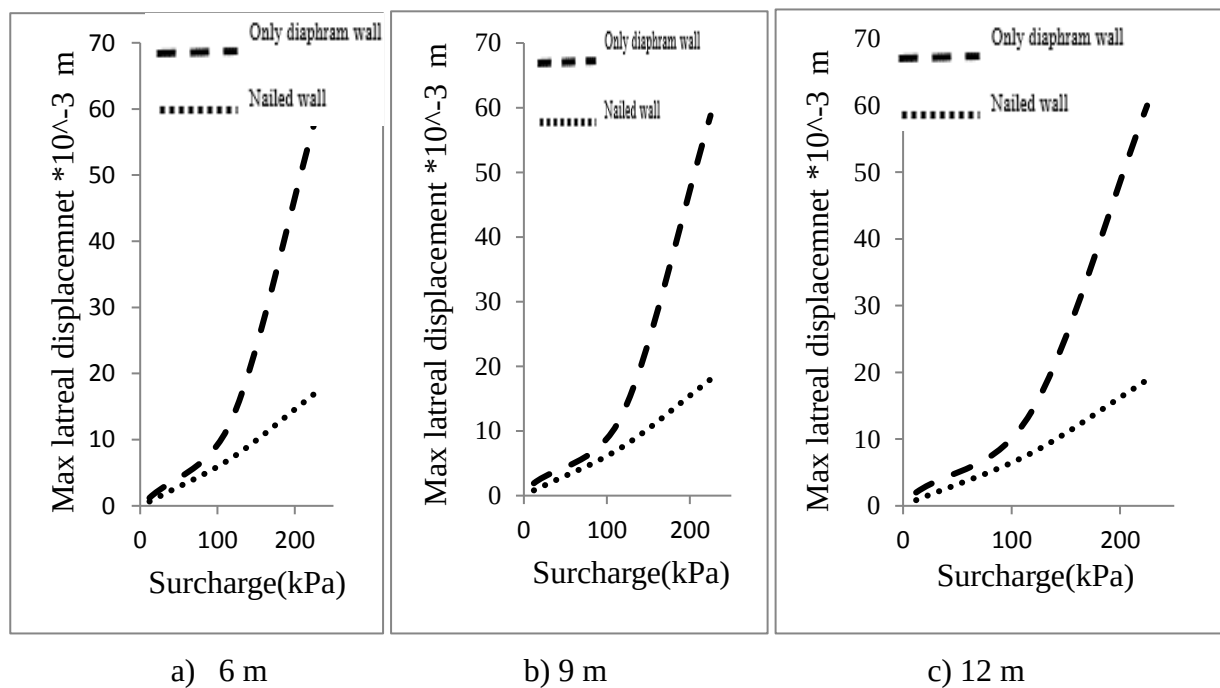


Figure 5. 9 The change maximum lateral displacement nails and using only diaphragm wall for a) 6m, b) 9m and c) 12m

6. CONCLUSION AND RECOMMENDATION

6.1 Conclusion

This study was carried out on Addis Ababa for clay major type of soils for two (6 meters deep), three (9 meters deep) and four (12 meters) basements. The soil is heterogeneous and the ground water table is on 4m below the ground surface. Hence, this study emphasizes on the stability analysis and performance evaluation of nailing on the stability of nail deep excavation.

The objective of the stability analysis was to determine the effect of surcharge, excavation depth, nail inclination, and nail spacing on the lateral displacement of the nail deep excavation wall and Compare the change of maximum lateral displacement with nailing and without nailing (only diaphragm wall) for each depth. The soil parameters for the analysis were derived from SPT data gathered from several organizations. Then, using the finite element approach, the stability was evaluated in an undrained state with static loads (plaxis 2D). All finite element analyses used the elastoplastic Mohr-Coulomb material model. The following conclusions are made:

1. As the surcharge increases, the maximum lateral displacement increases as well. Excavation depths of 6 meters are not capable of carrying loads more than 155.6 KPa. The 9-meter depth of excavation can support a weight of up to 225 KPa, but not more. The 12 meter depth of excavation can carry a load of up to 286 KPa, but not more.
2. The maximum lateral displacement increases at increasing rate when the nail inclination and nail spacing increases.
3. The maximum lateral displacement is certainly influenced by the depth of excavation. The greatest lateral displacement increases as the depth of excavation increases. The variation of maximum lateral displacement with respect to depth is small when the surcharge is small.

Some of the results are below the allowable lateral displacement, although some of them are above it. When the results of lateral displacements were compared to the allowable lateral displacement, the most cost-effective and easy-to-construct nail inclination and spacing were

chosen for the various surcharges and excavation depths. Higher vertical nail spacing is chosen since it reduces the number of nails used and higher nail inclination is chosen for ease of construction. The nailing method will become more cost effective as the number of nails decreases. Comparing the change in maximum lateral displacement of nailed wall and only diaphragm supported wall, the following conclusions is made:

4. The horizontal displacement of a nailed wall and a diaphragm wall supported wall differs significantly. Without nailing wall (Only a diaphragm wall) can support a surcharge of up to 118 KPa, which corresponds to 12mm maximum lateral displacement, for the depth of 6 meters of excavation. Without nail wall (only the diaphragm wall), a 9 meter depth of excavation can hold up to 130 KPa surcharge which corresponds to 18mm and 12 meter depth of excavation can handle up to 142.7 KPa of load which corresponds to 24mm.

Table 6. 1 Specified nail spacing and inclination for each depths

Depth (m)	surcharge	Nail spacing	Nail inclination
6	118	1 nail at 3 meter depth	15°
	155.6	2	15°
9	130	2	15°
	225	2	15°
12	142.7	2	15°
	286	2	15°

6.2 Recommendation

The work focused on deep excavation stability analysis and nailing performance evaluation using the finite element method. As a result, the following recommendation is made:

1. The research focuses on clay soil. As a result, it is strongly advised to evaluate it with different soil types.

SPECIAL ACKNOWLEDGEMENT

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ASTU/SM-R/629/22

Adama, Ethiopia

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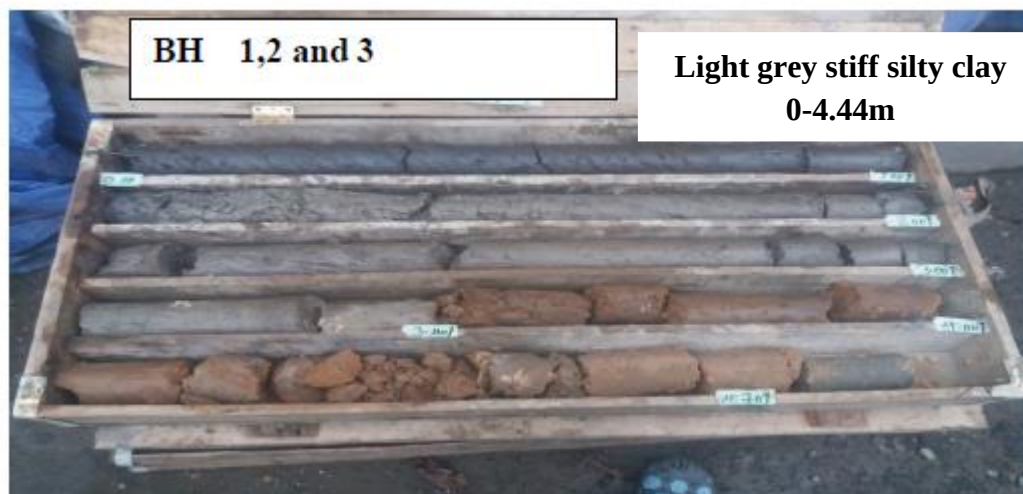
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Appendix

Appendix A Sample core logs from organization

Some of the core logs are presented in the following section. Since there are many data from different site these are attached for sample.









Light Gray Silty Clay 0-4m



Light gray sandy silty clay 4-16m



Tuff soil 13-20m



Figure A. 1 Sample Core Logs from Organizations

Appendix B Properties of tuff soil

Table A. 1 Tuff soil Properties from the world and Indonesia (Asniar et al., 2019)

Material of Tuff	Geotechnical Properties							
	γ (kN/m³)	Bulk Density (gr/cm³)	σ (Mpa)	C (Mpa)	φ (°)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
South Queensland		2,05	0,109	0,1	10,3			
Tenerife	22-30		0-0,2		23-40			
Tizi, Algeria	25,9					28	13	15
Antalya	10,37-13,92							
Qishan				0,040 (dry) 0,052 (natural) 0,032 (saturated)	33,12 (natural) 32,31 (saturated)			
Eskişehir	20-21		0,03-0,24	0,013-0,044	11-22			
Mt. Burangrang						45-49		11-28
Malino, Manipi	4,089							
Transek, Liwa				0,007	38,4	58,35	30,68	27,6

Appendix C Soil properties related to SPT N value

The average SPT values for each depth are calculated.

Table A. 2 Approximate ranges of C_u and corresponding SPT-N for cohesive soils (Terzaghi and peck, 1967)

SPT N values	consistency	Undrained shear strength C_u KPa
0-2	Very soft	0-12.5
2-5	Soft	12.5-25
5-10	Medium stiff	25-50
10-20	Stiff	50-100
20-30	Very stiff	100-200
>400	Hard	>200

Table A. 3 Correlation of saturated unit weight with SPT result (Terzaghi and Peck, 1943)

Standard penetration resistance, N-SPT	Consistency	Saturated unit weight
0-2	Very soft	16-19
2-5	Soft	16-19
5-10	Medium stiff	17-20
10-20	Stiff	19-22
20-30	Very stiff	19-22
>30	Hard	19-22

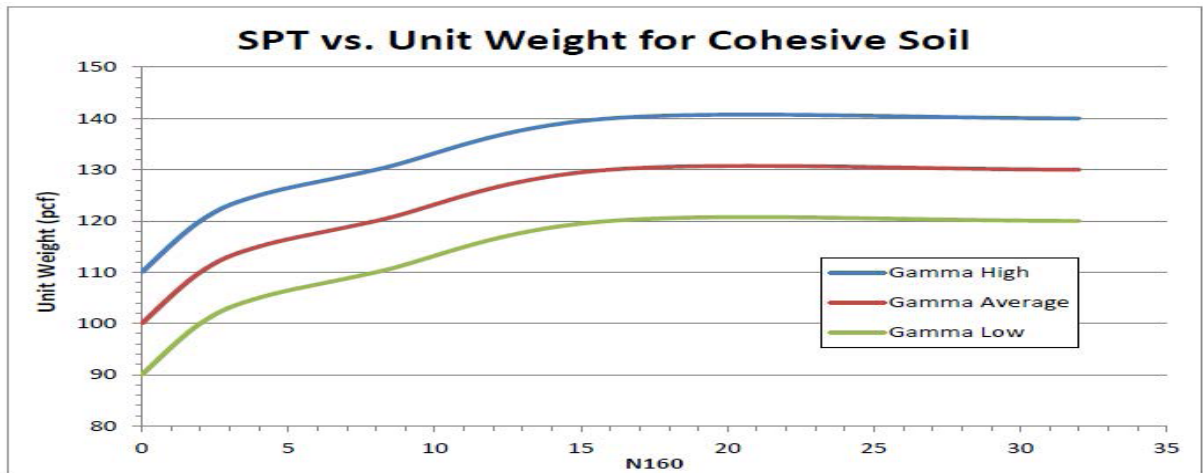


Figure A. 2 Correlation between SPT and bulk unit weight (Bowles, 1977)

Appendix D SPT value calculations for each depth

Table A. 4 SPT value for the upper 4 meter depth soil

Depth (m)	SPT	SPT*dep	W _{spt}
4	16	64	11
4	12	48	
4	14	56	
4	7	28	
4	10	40	
4	17	68	
4	14	56	
4	10	40	
4	7	28	
4	8	32	
4	9	36	
4	8	32	
48		528	

Table A. 5 Weighted SPT

Depth	SPT 4-6m	SPT 6-8m	SPT 8-10m	SPT 10-12m	SPT*de4	SPT*d6	SPT*d8	SPT*d10
2	18	12	14	25	36	24	28	50
2	16	21	24	20	32	42	48	40
2	19	16	21	23	38	32	42	46
2	10	12	14	35	20	24	28	70
2	11	8	12	18	22	16	24	36
2	18	24	19	28	36	48	38	56
2	0	15	19	15	0	30	38	30
2	18	15	12	22	36	30	24	44
2	13	11	20	21	26	22	40	42
2	13	15	14	16	26	30	28	32
2	15	15	13	16	30	30	26	32
2	13	16	15	13	26	32	30	26
24					328	360	394	504

Table A. 6 Weighted average SPT value

W.SPT 4-6 m	13.6666667	13
W.SPT 6-8 m	15	15
W.SPT 8-10 m	16.4166667	16
W.SPT 10-12 m	21	21

Table A. 7 Weighted average SPT value for 12-14 meter

Depth	SPT 12-14 m	SPT*depth	W.spt	
2	31	62	23.55	23
2	22	44		
2	30	60		
2	31	62		
2	26	52		
2	28	56		
2	18	36		
2	24	48		
2	18	36		
2	13	26		
2	18	36		
22		518		

Table A. 8 Weighted average SPT value for 12-14 meter

SPT 14-20	Depth	SPT*depth	W. average	
38	6	228	24.33333333	24
17	6	102		
25	6	150		
23	6	138		
22	6	132		
21	6	126		
	36	876		

Table A. 9 Corrected SPT N value for each depth

depth	N	N60	N160	N160
0-4m	11	9.16666667	9	9
4-6m	13	10.83333333	12.916667	12
6-8m	15	12.5	13.75	13
8-10m	16	13.33333333	14.166667	14
10-12m	21	17.5	16.25	16
12-14m	23	19.1666667	17.083333	17
14-20.	24	20.8333333	17.916667	18

Appendix E Undrained shear calculation

To calculate the nail bonded length and nail diameter for each depth, the following steps followed.

1. Calculate the Undrained shear strength of soil.
2. Calculate mobilized shear stress acting around the perimeter of the nail-soil interface.
3. Divide it to get the design pull out capacity.
4. Divide the design pull out capacity by cosine of the nail inclination in order to get the inclined design pull out capacity.
5. Calculate the bonded length for each nail spacing at each depth.
6. Make the pull out capacity concentrated and divide to the nail yield strength in order to get the nail area.
7. Compare the nail area to the nail area given by ASTM a615 and select the one

Appendix F Nail Pull out capacity at the middle point of each depth

The nail pull out capacity is calculated at the center of each depth 0° to the horizontal axis.

Table A. 10 Nail pull out capacity

Depth (m)	γ_{sat} (KN/m ³)	C (KPa)	$\gamma^*h +$ surcharge (KPa)	Undrained shear (KPa)	Rp (KPa)	Rpd (KPa)
2	19.5	44.43	264	99.87	47.04	31.36
5	19.79	82.1	323.95	150.13	70.71	47.14
7	20.3	95.1	367.1	172.19	81.1	54.07
9	20.6	101.5	410.4	187.68	88.4	58.93
11	21	133.5	456	279.42	131.61	87.74
13	21.3	144.3	501.9	304.91	143.61	95.74
17	21.5	156.25	590.5	357.02	168.16	112.1

Appendix G Bonded length for 1.25, 1.625 and 2 meter spacing

The nail pullout capacity and bonded nail length is calculated at 5°, 10° and 15° nail inclination with 1.25m spacing at the center of each depth.

Table A. 11 Nail pullout capacity and bonded nail length for 1.25 spacing

Depth (m)	Rpd /cos5°	Rpd /cos10°	Rpd /cos15°	Lp of 5° and 1.25m spacing	Lp of 10° and 1.25m spacing	Lp of 15° and 1.25m spacing
2	31.68	32	32.67	2.8	2.83	2.89
5	47.62	48.1	49.10	4.21	4.25	4.34
7	54.61	55.17	56.32	4.83	4.88	4.98
9	59.53	60.14	61.39	5.27	5.32	5.43
11	88.62	89.53	91.39	7.86	7.92	8.09
13	96.71	97.7	99.73	8.56	8.64	8.82
17	113.25	114.39	116.78	10.02	10.12	10.33

The bonded nail length is calculated at 5°, 10° and 15° nail inclination with 1.625m spacing at the middle point of each depth.

Table A. 12 Bonded nail length of 1.625 spacing

Depth (m)	Lp of 5° and 1.625m spacing	Lp of 10° and 1.625m spacing	Lp of 15° and 1.625m spacing
2	3.64	3.68	3.76
5	5.48	5.53	5.65
7	6.28	6.34	6.48
9	6.85	6.92	7.06
11	10.19	10.3	10.51
13	11.12	11.23	11.47
17	13.02	13.16	13.43

The bonded nail length is calculated at 5°, 10° and 15° nail inclination with 2m spacing at the center of each depth.

Table A. 13 Bonded nail length for 2m spacing

Depth (m)	Lp of 5° and 2m spacing	Lp of 10° and 2m spacing	Lp of 15° and 2m spacing
2	4.48	4.53	4.62
5	6.73	6.81	6.95
7	7.73	7.81	7.97
9	8.43	8.51	8.69
11	12.54	12.67	12.94
13	13.69	13.823	14.12
17	16.03	16.19	16.53

Since the bonded length of the angle has slight difference the bigger one is selected.

Table A. 14 Threaded bar properties [ASTM a615, grades 420 and 525 Mpa (60 and 75 Ksi)]

Nominal Bar Designation		Cross-Sectional Area		Nominal Unit Weight		Max. Diameter w/Threads		ASTM Grade	Yield Strength		Max. Axial Load	
English	mm	in. ²	mm ²	lbs/ft	kg/m	in.	mm	English	ksi	MPa	kips	kN
#6	19	0.44	284	1.50	2.24	0.86	21.8	60	60	414	26.4	118
								75	75	517	33.0	147
#7	22	0.60	387	2.04	3.04	0.99	25.1	60	60	414	36.0	160
								75	75	517	45.0	200
#8	25	0.79	510	2.67	3.98	1.12	28.4	60	60	414	47.4	211
								75	75	517	59.3	264
#9	29	1.00	645	3.40	5.06	1.26	32.0	60	60	414	60.0	267
								75	75	517	75.0	334
#10	32	1.27	819	4.30	6.41	1.43	36.3	60	60	414	76.2	339
								75	75	517	95.3	424
#11	36	1.56	1,006	5.31	7.91	1.61	40.9	60	60	414	93.6	417
								75	75	517	117.0	520
#14	43	2.25	1,452	7.65	11.39	1.86	47.2	60	60	414	135.0	601
								75	75	517	168.8	751

Source FHWA, 2013