ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF GRADUATE STUDIES
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

A STUDY ON SOIL NAILING AS A DEEP EXCAVATION SUPPORT IN RAILWAY TRACK

BY: GIRMA MOGES

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ADVISOR: DR. TENSAY GEBREMEDHIN

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ABSTRACT

Soil nailing is one of the in-situ soil reinforcement and deep excavation support that has been used worldwide, with the main components as nail and facing. This study covers both conventional and state-of-the-art numerical analysis. It mainly focuses on the numerical analysis using PLAXIS 2D due to the drawback of deformation prediction in the conventional limit equilibrium method. The soil properties and initial wall parameters are taken from measurements on sites, were appropriately modeled and used in the finite element based software, PLAXIS 2D for the simulation. The global factor of safety as computed using limit equilibrium was greater than that of the finite element method. The effect of mesh density on the factor of safety and wall deflection was explained. Modeling of soil nail using a plate and geogrid elements were illustrated and from the analysis result, bending stiffness showed no effect on the global factor of safety at the end of the construction stage. However, it showed a significant effect on the other construction stages. A parametric study was conducted to investigate how the lateral displacement and the global factor of safety are influenced by varying the length, inclination, and spacing of the nails. When the nail length increases the lateral displacement decreases and the factor of safety increases. It was observed that when the nail inclination increases the global factor of safety increases however, for higher length to wall height ratio the global factor of safety decreases after 15^0 inclination. Based on the analysis result, the nail inclination angle between 10^0 and 15^0 is recommended as the optimal use of the nail.

Keywords: Soil Nail Wall, Lateral Deflection, Global Factor of Safety, PLAXIS 2D

TABLE OF CONTENT

TABLE OF FIGURE

LIST OF TABLES

LIST OF SYMBOLS AND ABBREVIATIONS

D_{DH}	Drill hole diameter
Q	Surcharge load
W	Weight of the sliding wedge
θ	Failure plane inclination from horizontal
q_u	bond strength of soil
λ	Nail inclination
L	Nail Length
H	Height of soil nail wall
EA	Axial stiffness
EI	Bending stiffness
S	Nail spacing
S_V	Vertical nail spacing
S_H	Horizontal nail spacing
L/H	Nail length to wall height ratio
γ	Unit weight of soil
c	Cohesion of soil
φ	Friction angle of soil
ψ	Dilatancey angle of soil
E	Elasticity modulus
ν	Poison’s ratio
h	Thickness of facing ;
a_{vn}	Reinforcement cross-sectional area in the vertical direction at the nail head
a_{vm}	Reinforcement cross-sectional area in the vertical direction at mid-span
a_{hn}	Reinforcement cross-sectional area in the horizontal direction at the nail head
a_{hm}	Reinforcement cross-sectional area in the horizontal direction at mid-span;
fy	Reinforcement tensile yield strength
ASD	Allowable stress design method
FHWA	Federal Highway Administration
WWM	Welded wire mesh

1. INTRODUCTION

1.1. General

Deep excavation is any man-made cut, cavity, trench, or depression in an earth surface, formed by earth removal to some depth below the ground level. There are several types of in-situ walls that are used to support deep excavations. The criteria for the selection of such walls depend on the depth and width of the excavation, the type of soil in the site, the location of groundwater level, the adjacent structures, the easiness of construction, cost, the speed of work and others. Some of this wall is Braced walls, soldier pile, and lagging walls, Sheet-piling or sheet pile walls, Pile walls (contiguous, secant and tangent), Diaphragm walls or slurry trench walls, Prefabricated diaphragm walls, Reinforced concrete (cast-in-situ or prefabricated) retaining walls, soil nail walls, Cofferdams, Caissons, Jet-grout and deep mixed walls, Top-down construction, Partial excavation or island method which will be discussed in literature review in detail. (Ergun, 2008)

From this deep excavation support, soil nail walls are one of the supporting systems for deep excavation. A soil nail is a structural element, which provides load-transfer to the ground in excavation reinforcement applications. The "nail" may simply consist of a steel tendon, but most commonly, the tendon is encapsulated in a cement-grouted body to provide corrosion protection and improved load-transfer to the ground (Byrne R.J., 1998). The main components in the soil nailing are the facing materials (shotcrete) and nail parameters i.e. length, nail inclination, and diameter.

1.2. Statement of the Problem

Nowadays, deep excavation is increasing every year due to the need for construction of multi-story buildings, highways, railways, and other underground-buried structures. This is because of high vertical or lateral loads and searching for the bearing stratum at a greater depth, space constraints and so on. Cities, which are very congested due to buildings and streets, it is difficult to construct any construction through open excavation and/or slope construction technique.

It is also mandatory to protect the excavation from collapsing and keeping the adjacent structures without any disturbance. The cause of collapsing of a soil may be from the lateral earth pressure, surcharge load, water pressure, etc. As a result, the need for deep excavation with excavation support arises.

Therefore choosing the appropriate excavation support is the main concern due to economy and performance. From the deep excavation supports, soil nailing with shotcrete is the most common method and easy for construction as well as economical. However, for its design, the facing materials and the nail parameters play a vital role and have a great influence. As a result, an investigation into the behavior of soil nails and identifying influential design parameters will be very important.

1.3. Objectives:

1.3.1. General Objective

- To study the behavior of soil nailing as deep excavation support in railway track using both analytical and numerical methods.

1.3.2. Specific Objectives

- To show the staged construction in soil nailing analysis
- To evaluate the effect of nail length to wall height ratio, nail inclination, and spacing on lateral deformation and global factor of safety of the soil nail wall.
- To compare the analysis using the conventional and the current state-of-the-art analysis approaches.
- To suggest the optimum soil nail inclination for deep excavation support.

1.4. Research Methodology

The methodology engaged in this thesis is mainly based on a detail literature review, analytic and numerical investigations. To accomplish the objective of this research,

- The parametric soil properties have been taken from measured sites, which is suitable and appropriate for soil nailing - here soil nailing is used for ground conditions that have cohesion and no groundwater table. It should be stable until the excavation of the consecutive excavation stages finished and nails are inserted i.e the soil should stand a minimum of two days without collapsing.
- The initial wall parameters such as wall layout (wall height) and the patterns of the nail on the wall face were taken from the real measurement.
- The study more focused on the current state-of-the-art analysis approaches comparative to the conventional method.
- For the conventional analysis excel and slope/W were used, and the numerical analysis was evaluated through a finite element model (FEM) using PLAXIS 2D programs to study the performance of soil nailing.
- Through varying the length to wall height ratio, nail inclination, and nail spacing, their effect on the lateral deformation and global factor of safety of the wall studied.
- Conclusions, discussions, and recommendations have been provided based on the results obtained from the analytic and numeric methods.

1.5. Scope of the Study

This thesis includes both the conventional and the current state-of-the-art analysis of soil nailing as deep excavation support in railway track. For the conventional analysis, the main resistances considered are the skin friction between soil and grouting, the tensile yield load of the inserted reinforcing material and the failure plane in the design.

For the numerical analysis, the geometry of the excavation model used is 2D with plane strain condition. It studies only static analysis.

As the title shows that, it focuses on a railway track. It does not include excavation for multistory buildings. If it is proposed to use for multistory building excavation, the length to width ratio of the excavation should be greater than ten.

1.6. The significance of the Study

This thesis fills the gap for those who use the soil nailing as deep excavation support with appropriate nail inclination, spacing, and length to choose the influential parameters for the design. In addition, it helps to choose and compare the conservative method from the conventional and the current state-of-the-art analysis.

1.7. Organization of the Thesis

Chapter 1: presents an introduction about soil nailing, objective, scope, significance, and organization of the thesis.

Chapter 2: it is all about literature review related to soil nail wall i.e. history, advantage and disadvantage, application, construction sequence, components, design consideration.

Chapter 3: discusses limit equilibrium analysis of soil nailing as deep excavation support.

Chapter 4: presents the finite element method for the analysis of the soil nail wall as deep excavation support. A comparison of the limit equilibrium and the finite element is presented in this chapter. It includes different parametric studies and their effects.

Chapter 5: conclusion and recommendation are presented.

2. LITERATURE REVIEW

2.1. Deep Excavation Supports

Here, the choice of deep excavation supports depends on soil type, the existence or level of the water table, the adjacent structures, the depth and size of the excavations, the performance and cost of the supports, the availability of construction and speed of work. Some of the deep excavation supports are: (Ergun, 2008)

2.1.1. Braced Wall

The braced wall is the placement or driven of soldier piles (steel H, I or WF sections or timber) around the excavation at about 2 to 3-meter intervals and at each level, horizontal waling beams and supporting elements (struts) are constructed.

For braced wall construction, soils with some cohesive properties in the absence of water table are suitable or dewatering can be used if required and allowed. Strut support is commonly preferred in narrow excavations for pipe laying or similar works but also used in deep and large excavations.

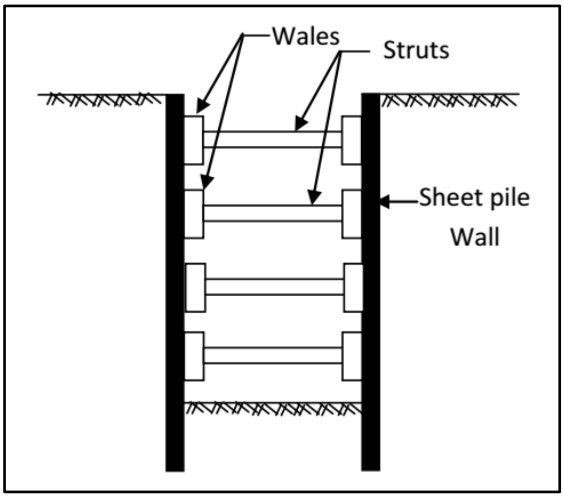
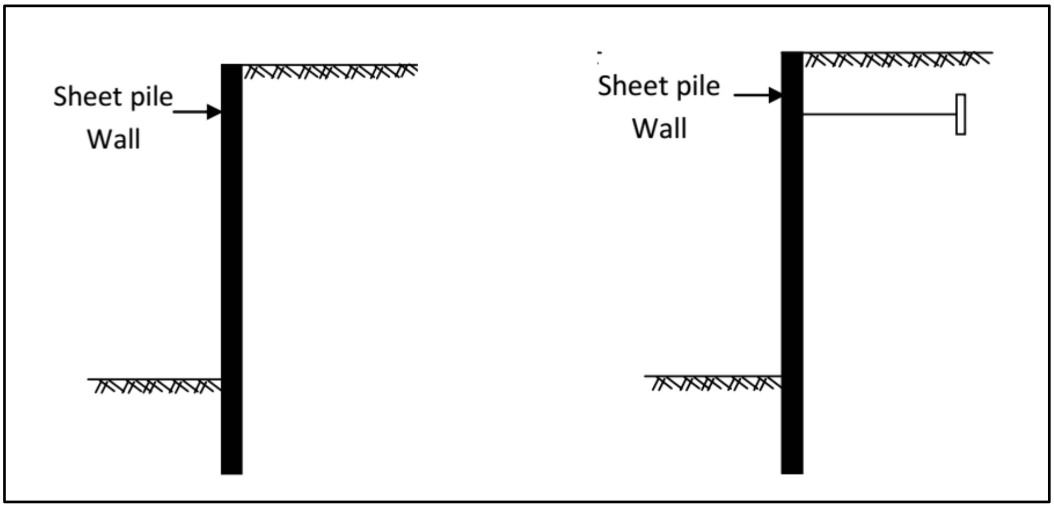


Figure 2.1: Braced Walls

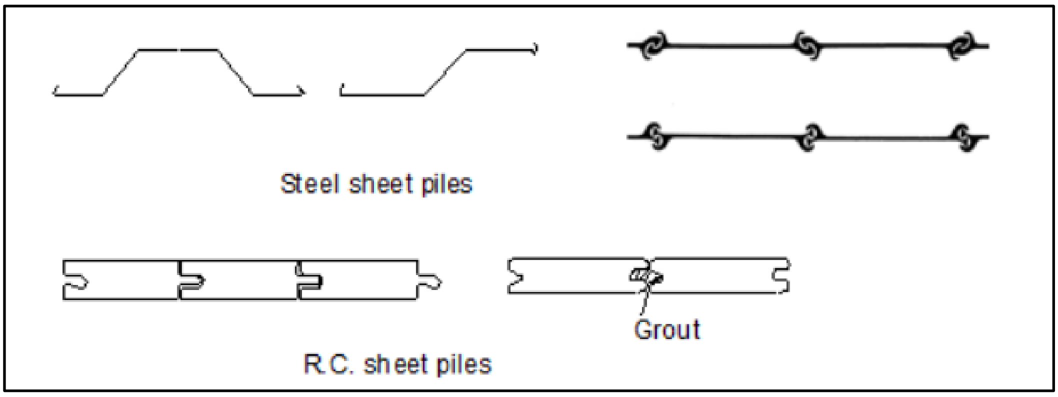
2.1.2. Sheet Pile Walls

Sheet pile wall is a watertight steel wall constructed from thin steels sections (7-30 mm thick) 400-500 mm wide, which is manufactured in different shapes like U, Z, and straight sections. There are interlocking watertight grooves at the sides, and they are driven into the soil by hammering or vibrating. Due to their environmental problems like noise and vibrations, their use is restricted in urbanized areas.

In soft soils sometimes reinforced concrete precast sheet pile sections are used if the drive is not difficult. Even though driving steel sheet piles in very dense, stiff soils or in soils with boulders is difficult, jetting is used for ease penetration.



a. Without and With Anchored Sheet Pile (side view)



b. Steel and Reinforced Sheet Pile Walls (top view)

Figure 2.2: Sheet Pile Walls

2.1.3. Pile Walls

There are different types of pile walls as shown in Figure 2.3. Contiguous (intermittent) bored piles are piles whose spacing between the piles is greater than the diameter of piles. The spacing depends on the type of soil and level of design moments but it should not be too large to avoid the drop off pieces of lumps. Cohesive soils or soils with some cohesion without water table are suitable. Here some acceptable amount of water can be collected at the base and pumped out. Common diameters of contiguous pile are 0.60, 0.80, 1.00 m.

Tangent piles are similar to that of contiguous pile except spacing i.e. the piles are tangent to each other. Tangent piles with grouting in between can be used when secant piling or diaphragm walling equipment is not available (i.e. in cases where groundwater exists).
Secant bored pile walls are formed by keeping the spacing of piles less than the diameter (S<D). It is a watertight wall and maybe more economical compared to a diaphragm wall in small to medium scale excavations due to the cost of site operations and bentonite plant. Here primary unreinforced piles are constructed first and then reinforced secondary piles are formed by cutting the primary piles. Pile construction methods may vary in different countries for all types of pile walls like full casing support, bentonite support, continuous flight auger (CFA), etc.

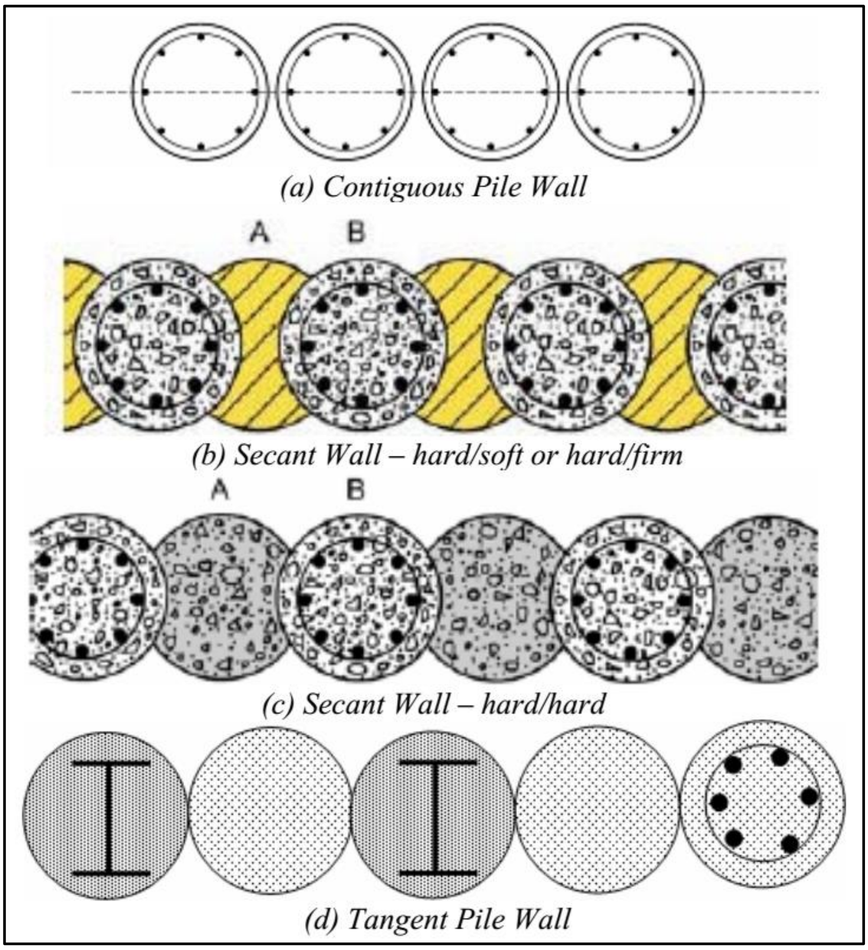


Figure 2.3: Types of Pile Wall (Where A Primary Pile- Unreinforced and B Secondary Pile-Reinforced Piles) (Venkata Ramasubbarao Godavarthi, 2011)

2.1.4. Diaphragm Wall

A diaphragm wall is a continuously reinforced concrete, which provides structural support and water tightness. These also called slurry trench walls due to the reference given to the construction technique where excavation of the wall is made possible by filling and keeping the wall cavity full with bentonite-water mixture during excavation to prevent the collapse of the excavated vertical surfaces. Wall thickness varies between 0.50 m and 1.50 m. The wall is constructed panel by panel in full depth. Panel lengths are 2 m to 10 m. Short lengths (2-2.5 m) are selected in unstable soils or under very high surcharges.

Panel excavation is made by cable or kelly supported buckets and then tremie concrete is placed in the slurry starting from the bottom after lowering reinforcement cages. The joint between the panels is a significant detail in water-bearing soils and steel pipe, H-beam or water stops are used.

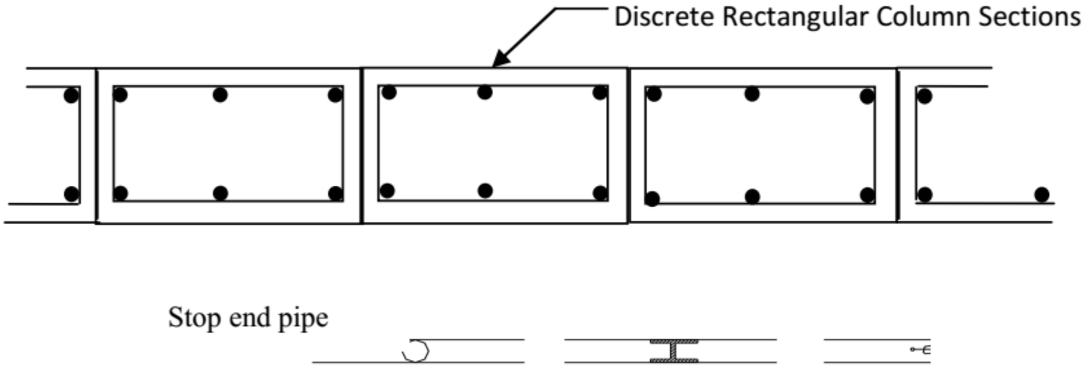


Figure 2.4: Diaphragm Wall and Joints between the Panels in Diaphragm Walls

2.1.5. Soil Nail Wall

In soil nail wall support excavation is made step by step (1.5 to 2 m high). Shotcrete is common for facing and wire-mesh is used. Soft facing is also possible making use of geotextiles. The hole is drilled, ordinary steel bars are lowered, and grout is placed without any pressure. Soil should be somewhat cohesive and no water table or significant water flow should be present. (Ergun, 2008)

2.2. Soil Nail Walls

2.2.1. Development of Soil Nailing

Soil nailing is a process in which excavation walls and slopes are stabilized in situ by installing relatively short, fully bonded steel bars or other reinforcing materials according to a regular and relatively closely spaced pattern. At every stage during the top-down construction, excavation beneath the toe of the stabilized soil is required. This is only possible if the soil is able to provide self-supporting stability during the excavation. (Rowe, 2001)

Soil nailing has been used in a variety of civil engineering projects as support of deep excavation as well as slope stabilization. It appears that the technique has emerged as an extension of rock bolting and of the "New Austrian Tunnelling Method" (NATM) developed by Rabcewicz (1964) which combines reinforced shotcrete and rock bolting which is usually 3 to 6 m long, to provide a flexible support system for the construction of underground excavations by minimizing the amount the final lining required as shown in Figure 2.5. (Clouterre, 1991). In North America, it was started in the early 1970s for temporary excavation support. In Europe, it was started as a retaining wall construction in France (1972), and Germany (1976), in connection with highway cut slope construction or temporary building excavation support.

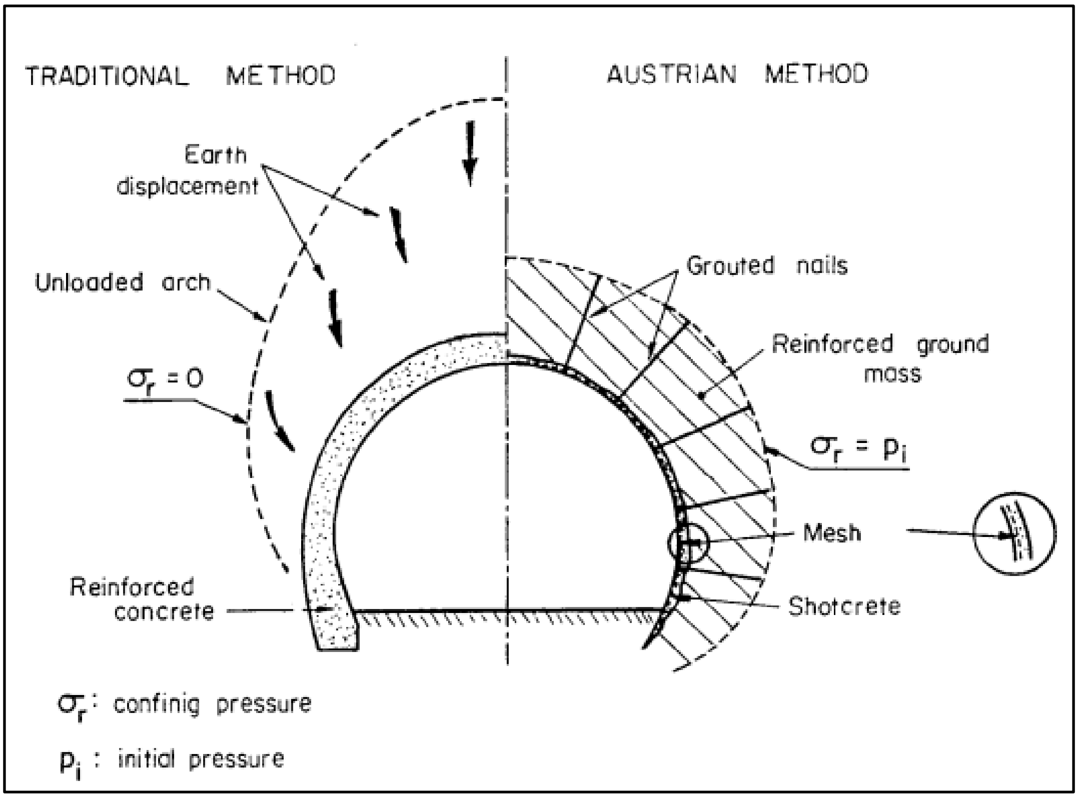


Figure 2.5: Traditional and Austrian Tunneling Method for Lining A Gallery (Clouterre, 1991)

2.2.2. Types of Soil Nail Walls

There are various types of soil nail walls, based on the nail installation techniques:

- **Drilled and grouted soil nails:** Holes of 100–200mm (4–8 inch) diameters are first drilled, and steel bars are placed in the holes and then grouted. The holes are typically 1.5m apart. Grouted soil nails are used for temporary and permanent applications.
- **Driven soil nails:** The nails are relatively small in diameter (19–25mm or 0.75–1.0 inch) and are mechanically driven into the soil. Grout is not used. This type of installation is fast, but it cannot provide good corrosion protection.

- **Hollow bar soil nails:** The nails are hollow and grout is simultaneously injected through the hollow bar with the drilling. This soil nail type allows for faster installation than drilled and grouted soil nails, and it can provide some level of corrosion protection. This method is commonly used as a temporary retaining structure.
- **Jet-grouted soil nails:** Jet grouting is used to cut the soil; after the holes are drilled, steel bars are installed using the Vibro-percussion through the grouted holes. The grout provides corrosion protection.
- **Launched soil nails:** In this new technique, soil nails are launched into the soil at a high speed using a firing mechanism involving compressed air. The installation is fast, but it is difficult to control the soil penetration depth of nails, particularly when the subsoil contains cobbles. (Xiao, 2015)

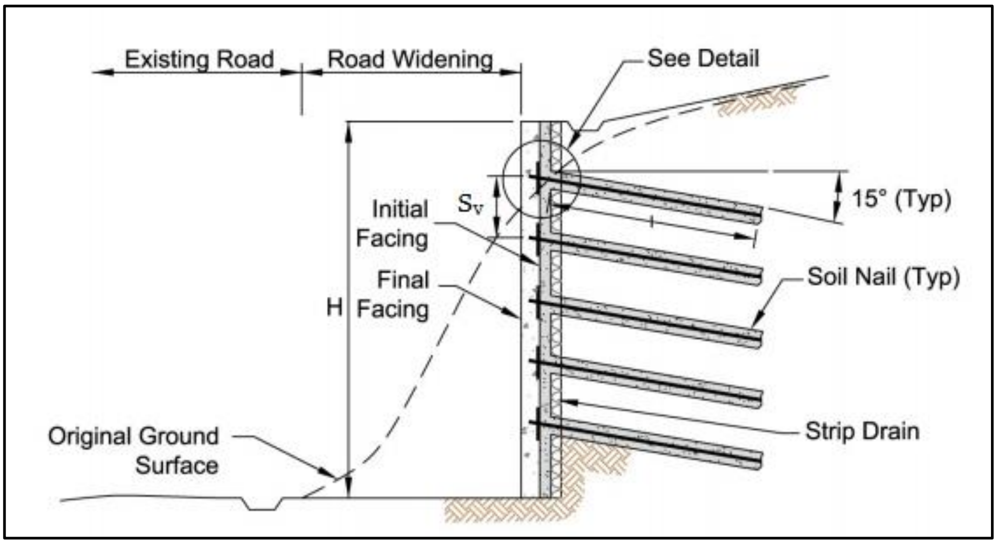
2.2.3. Component of Soil Nailing

The common type of soil nail wall is drilled and grouted soil nail wall, in which a steel bar is inserted in a pre-drilled hole and then grouted. Figure 2.6 shows a cross-section of a typical soil nail wall and each component is described as below: (Lazarte, 2003)

1. **Steel reinforcing bars** – The main component of soil nail wall systems are solid steel reinforcing bars. After the hole is drilled, these elements are inserted and grouted in place. The main purpose of these components is to provide tensile resistance to the deformation of the retained material during subsequent excavation.
2. **Spacers or centralizers** – it is polyvinyl chloride (PVC) material used to center the steel reinforcing bar and it is paced at regular intervals, typically not exceeding 2.5m along the length of the nail and at a distance of about 0.5m from each end of the nail.
3. **Grout** – A cement and water mix is injected to the predrilled borehole after the steel reinforcement bar is inserted. Grout is placed in the drill holes under gravity using the tremie pipe. The primary function of the grout is to transfer the stress from the ground to the steel bar. It also provides protection for the corrosion of the nail.
4. **Nailhead** – The nail head is the threaded end of the soil nail that projects from the wall facing.
5. **Hex nut, washer, and bearing plate** – These components attach to the nail head and are used to connect the soil nail to the facing. Bearing plate distributes the force at nail end to temporary shotcrete facing.
6. **Temporary and permanent facing** – The facing provides structural connectivity i.e. the nails are connected to the excavation face through these elements. The temporary

facing serves as the bearing surface for the bearing plate and supports the exposed soil. This facing is placed on the unsupported excavation prior to the advancement of the excavation grades. The permanent facing is placed over the temporary facing after the soil nails are installed and the hex nut has been tightened.

7. **Drainage System** – A drainage system is installed behind the soil nail wall before the facing is installed and used to collect perched groundwater or infiltrated surface water that is present behind the facing and used to discharge the collected groundwater away from the facing wall. Geosynthetic drainage strips, also referred to as geocomposite strip drains are commonly used for this purpose. The drainage system does not provide full coverage of the wall area, but rather covers commonly 10-20%, or more, of the excavation face, depending on the selected strip drain spacing and commercial widths that are available. (Lazarte, et al., 2015)
8. **Corrosion Protection** – Soil nails used in permanent applications require chemical and/or physical protection against corrosion. The required level of corrosion protection is greater for soils with higher corrosion potential and for projects with lower risk tolerance.



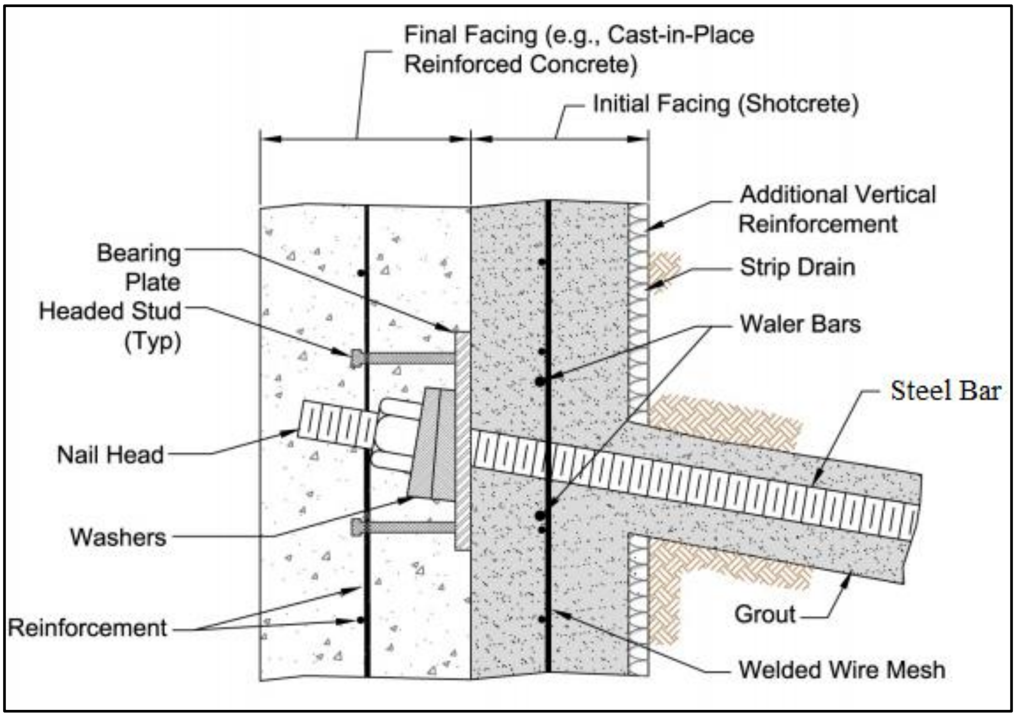


Figure 2.6: Typical Cross-Section of Soil Nail Wall (Lazarte, et al., 2015)

2.2.4. Construction Sequence of Soil Nail Wall

The construction sequence of a typical simple solid bar is described and illustrated schematically in Figure 2.7.

Step 1. Excavation. Initial excavation is done with excavation lift between 1 to 2 m (3 and 6 ft) depth for which the excavation face remains unsupported for at least one to two days. For this purpose, the soil should be suitable soil for soil nailing applications (see section 2.2.6.). The excavation lift reaches slightly below the elevation where nails will be installed and there must be sufficient platform or bench width to provide access to the installation equipment.

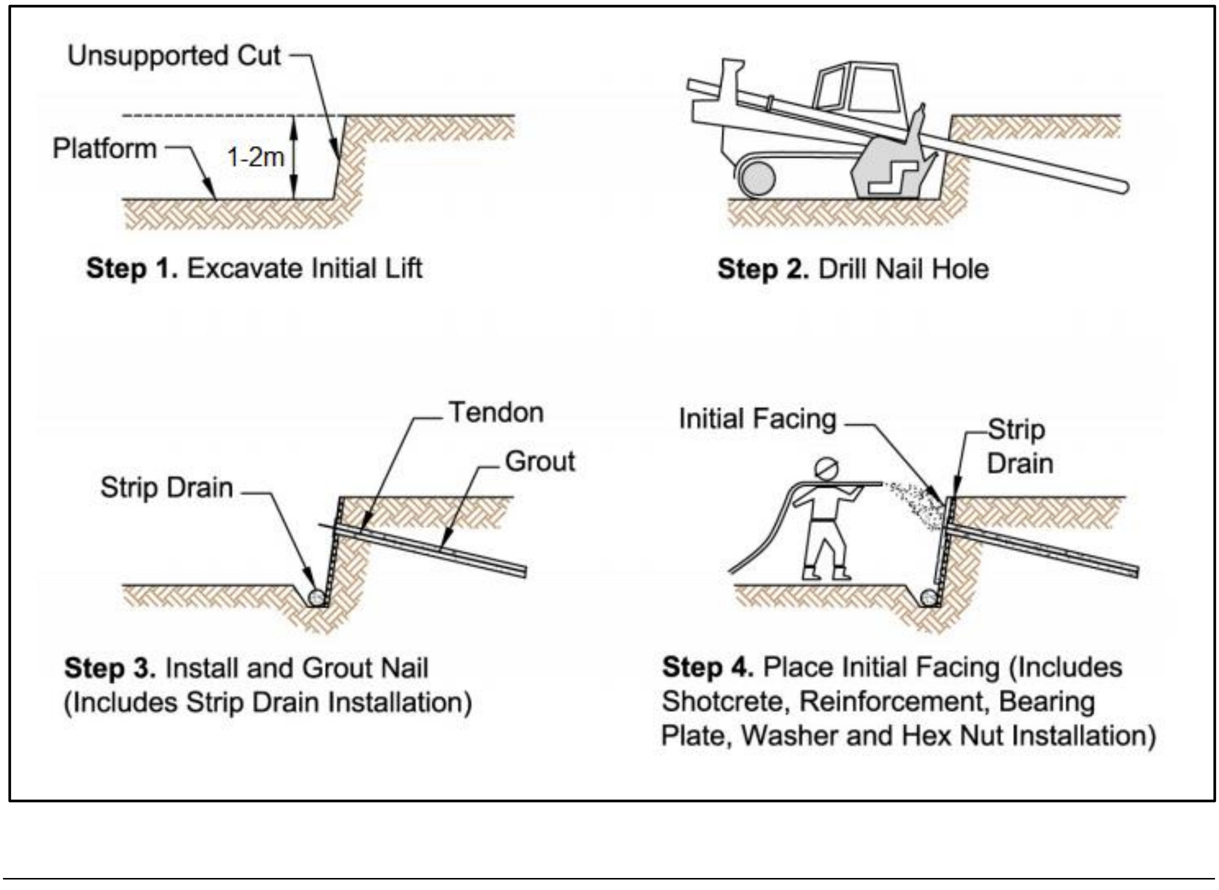
Step 2. Drilling Nail Holes. After the unsupported initial excavation, drill-holes are drilled according to a specified length, diameter, inclination, and horizontal spacing.

Step 3. Nail Installation and Grouting. Solid or hollow nail bars (most commonly solid bars) are placed in the pre-drilled hole. Here centralizers are attached to the nails before insertion to maintain the alignment within the center of the hole and to provide sufficient grout coverage over the nail. During the insertion of the nail bar, a tremie (grout pipe) is also inserted. Depending on the life of the structure as well as the high need for corrosion, protection plastic sheathing can be used to avoid corrosion. Then through the tremie pipe, the drill-hole is filled with cement grout. Prior to step 4 (construction of facing), geo-composite drainage strips are installed.

Step 4. Construction of Temporary Shotcrete Facing. After the installation of nail bars and before the next lift of soil is excavated, a temporary facing system is constructed to support the open-cut soil section. The most typical temporary facing thickness is 100mm thick. The reinforcements typically consist of welded wire mesh (WWM), which is placed in the middle of the facing thickness. Following appropriate curing time for the temporary facing, a steel bearing plate is placed over the nail head protruding from the drill-hole. A hex nut and washers are subsequently installed to secure the nail head against the bearing plate. The hex nut is tightened to a required minimum torque after the temporary facing has sufficiently cured. This usually requires a minimum of 24 hours. Before proceeding with subsequent excavation lifts, the shotcrete must have cured for at least 72 hours or have attained at least the specified 3-day compressive strength (typically 10.5 MPa).

Step 5. Construction of Subsequent Levels. Steps 1 through 4 are repeated for the remaining excavation lifts. At each excavation lift, the vertical drainage strip is unrolled downward to the subsequent lift. A new panel of WWM is then placed overlapping at least one full mesh cell. At the bottom of the excavation, the drainage strip is tied to a collecting toe drain.

Step 6. Construction of a Final, Permanent Facing. After step 1 through 5 is finished and the final excavation depth is reached, the final facing is constructed. The final facing may consist of cast-in-place (CIP) reinforced concrete, reinforced shotcrete, or prefabricated panels. (Lazarte, 2003)



Step 1. Excavate Initial Lift

Step 2. Drill Nail Hole

Step 3. Install and Grout Nail (Includes Strip Drain Installation)

Step 4. Place Initial Facing (Includes Shotcrete, Reinforcement, Bearing Plate, Washer and Hex Nut Installation)

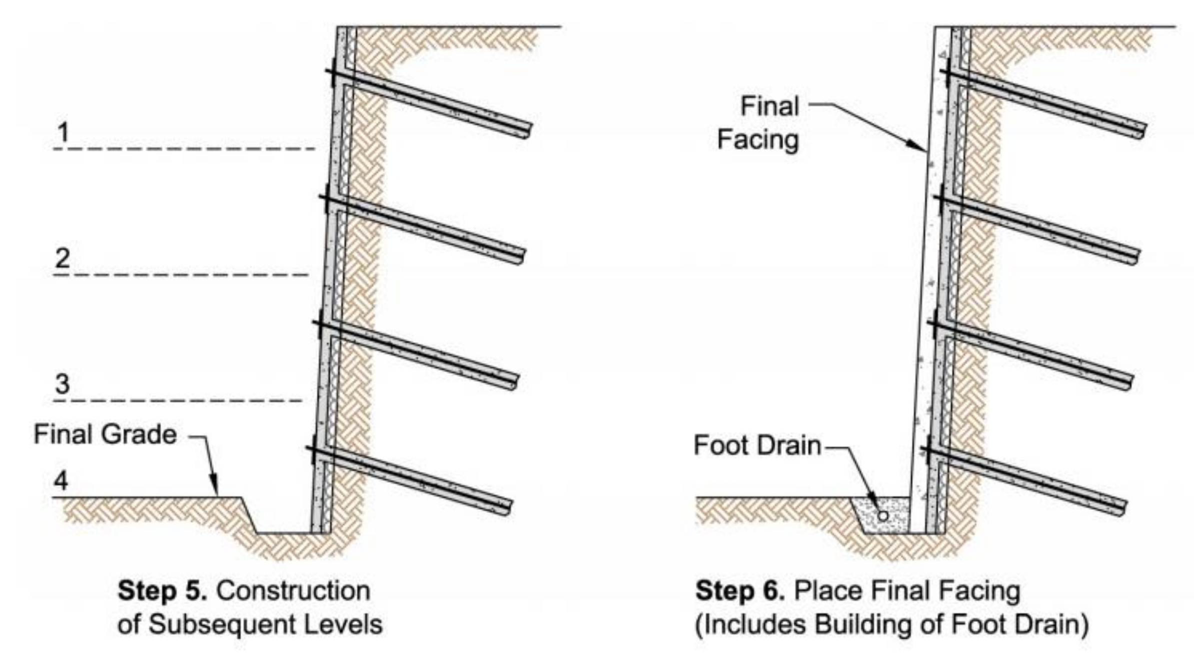


Figure 2.7: Typical Soil Nail Wall Construction Sequence; Adopted from (Lazarte, et al., 2015)

2.2.5. Advantage and Disadvantage of Soil Nailing

Soil nailing has the following advantages and disadvantages comparing to other computing structures. (Lazarte, 2003), (Clouterre, 1991)

Advantages:

- less disruptive to traffic and causes less environmental impact compared to other construction techniques such as drilled shafts or soldier pile walls, which require relatively large equipment.
- requires smaller right of way than ground anchors as soil nails are typically shorter;
- there is no need to embed any structural element below the bottom of excavation as with soldier beams used in ground anchor walls;
- installation of soil nail walls is relatively rapid and uses typically fewer construction materials than ground anchor walls;
- easy adjustments of nail inclination and location can be made when obstructions (e.g., cobbles or boulders, piles or underground utilities) are encountered; on the other hand, the horizontal position of ground anchors is more difficult to modify almost making adjustments in the field costly;
- provides a less congested bottom of the excavation, particularly when compared to braced excavations;

- because significantly more soil nails are used than ground anchors, adjustments to the design layout of the soil nails are more easily accomplished in the field without compromising the level of safety;
- overhead construction requirements are smaller than those for ground anchor walls because soil nail walls do not require the installation of soldier beams; this is particularly important when construction occurs under a bridge;
- soil nailing is advantageous at sites with remote access because smaller equipment is generally needed;
- In heterogeneous soils with cobbles, boulders, and weathered zones or hard rock zones, it offers the advantage of a small diameter shorter drill holes for nail installation and eliminates the need for soldier pile installation which is disproportionately costly to install under these conditions;
- Soil nail walls are relatively flexible and can accommodate relatively large total and differential settlements. Measured total deflections of soil nail walls are usually within tolerable limits. Soil nail walls have performed well during seismic events owing to overall system flexibility;
- Surface deflections can be controlled by the installation of additional nails or stressing in the upper level of nails to a small percentage of their working loads.
- Soil nail walls are more economical than conventional concrete gravity walls when conventional soil nailing construction procedures are used. It is typically equivalent in cost or more cost-effective than ground anchor walls. According to (Cornforth, 2005), comparing the cost of soil nail wall with tieback, soil nail wall save 10 to 30 percent to that of the tieback. These may be due to the structural facing (shotcrete) used in the soil nail wall is less costly.

Disadvantages:

- In the case of soil nailing, the system requires some soil deformation to mobilize resistance. Hence, soil nailing is not recommended for applications where very strict deformation control is required. Post-tensioning of soil nails can overcome this shortcoming, but this step, in turn, increases the project cost.
- Nail capacity may not be economically developed in cohesive soils subject to creep, even at relatively low load levels.
- Soil nail walls are not well suited for grounds with a high groundwater table for the difficulty in drilling and excavation due to seepage of groundwater into the excavation, corrosion of steel bars and change in grout water ratio.

- The existence of utilities such as buried water pipes, underground cables, and drainage systems behind the wall will likely create restrictions on the location, inclination, and length of soil nails, particularly in the upper rows. In addition, horizontal displacements may be somewhat greater than with pre-stressed tiebacks, which may cause distortions to immediately adjoining structures.
- Construction of soil nail walls requires a specialized and experienced contractor.

2.2.6. Suitable and Unsuitable Ground Conditions for Soil Nailing

Soil nail walls can be used for a different types of soils and conditions. From project histories, soil nailing is cost-effective over other deep excavation supports indefinite favorable ground conditions. On the other hand, there are soil conditions that have minimal applications for soil nailing and make them risky and/or too costly compared to other support systems. Generally, if the individual layers of the soil profile consist of suitable and stable materials, soil nail walls can be constructed in a mixed stratigraphy without complications. The following sections discuss the suitable and unsuitable ground conditions for soil nailing. (Lazarte, et al., 2015)

2.2.6.1. Suitable Ground Conditions for Soil Nailing

Soil nail walls can be constructed effectively in various types of soils. In suitable soil conditions, construction problems and long-term difficulties can be avoided. It has been confirmed that soil nailing is economical and feasible when (Lazarte, et al., 2015):

- The excavated soil is able to stand unsupported for 1to 2 m high vertical or nearly vertical cut for a minimum of one to two days;
- The soil nails are located above the groundwater table;
- If the soil nails are below the groundwater table, the groundwater table does not adversely affect the excavation face, the interface bond strength between the nail grout and the surrounding ground or the long-term functionality of the soil nails.
- The drill holes remain stable without casing until the nail bars are installed and grouted.

Soil conditions are assumed suitable for the construction of soil nail walls if the field test results indicate good soils. The standard penetration test (SPT) can be used as preliminary to identify suitable soil conditions.

Based on the general criteria for suitable conditions explained above, the following soil types are considered well suited for soil nailing applications (Lazarte, et al., 2015).

i. *Stiff to hard fine-grained soils*: these soils include stiff to hard clays, clayey silts, silty clays, sandy clays, and sandy silts. If the SPT N values of fine-grained soils are 9 blows/300mm or greater, they are classified as stiff. However, the consistency of fine-grained soils should be conducted with great care when SPT N values are obtained with non-automatic hammers and other outdated methods. Therefore, the consistency property should be supported by other field and/or laboratory tests. The potential for excessive long-term, creep-like, lateral displacements of soil nail walls is low in fine-grained soils with a plasticity index (PI) of less than 15.

ii. *Dense to very dense granular soils with some apparent cohesion.* These soils include sand and gravel with SPT N-values greater than 30 and percent of fines less than 10 to 15 or with weak natural cementation that provides cohesion. Due to capillary forces, apparent cohesion may also occur in moist fine sands. Generally, to get reasonable stand-up times the apparent cohesion of these soils should be larger than five kPa. To avoid the reduction of apparent cohesion (the breakage of capillary forces), some measures should be taken. These measures may include: (i) to avoid soil desiccation limiting the exposure of the cut excavation in dry weather is important; (ii) redirecting the surface water away from the excavation face; and (iii) preventing seepage toward the excavation face.

iii. *Weathered rock with no weakness planes.* Weathered rock may provide suitable supporting material for soil nails as long as weakness planes occurring in unfavorable orientations are not prevalent (e.g., weakness planes dipping into the excavation).

iv. *Engineered fill.* Soil nails can be installed in existing engineered, structural fill if this material is a mixture of well-graded granular material (approximately 90 percent of the mix or more) and fine-grained soil with Liquid Limit (LL) and PI values of less than 40 and 20, respectively. and if they were placed with acceptable compaction methods and acceptable levels of compaction energy (at least 90 percent of Standard Proctor). In addition, the age of engineered fills is critical for suitability and stability. Young embankment fills may be problematic.

v. *Glacial soils.* Glacial outwash and glacial till materials are suitable for soil nailing applications because they are naturally dense, well-graded granular materials with some amount of fines.

vi. *Residual Soils.* Some residual soils (i.e., those soils created from the in-place weathering of the parent rock material) and lateritic soil (a highly weathered tropical soil) may be acceptable for soil nailing. For lateritic types of soil, specific consideration should be given to the soil spatial variability and its ability to drain.

2.2.6.2. Difficult Soil Conditions for Soil Nailing

Soil conditions that are found between suitable and unsuitable soil conditions are considered as difficult or marginal. Soil nail walls can be installed in such soils, but it may not be cost-effective options. Examples of difficult or marginal soil conditions include (Lazarte, et al., 2015):

- *Non-engineered fill.* Soil nails may be installed successfully in existing non-engineered fill if this material has characteristics that are similar to those of engineered fills. But the embankment of the non-engineered fill may not have inadequate compaction, as a result, it may be problematic due to the irregular nail capacity and excessive deformations
- *Residual soils with unsuitable conditions.* Some residual soils may contain materials (e.g., mica, shale) that may impart low strength or stiffness to these materials.

2.2.6.3. Unsuitable Soil Conditions for Soil Nailing

Designing and constructing soil nailing in unsuitable conditions makes it more expensive and difficult. The following are some of the unsuitable conditions for soil nailing (Lazarte, et al., 2015):

i. *Dry, poorly graded cohesionless soils.* These types of soils will have no natural cementation and apparent cohesion if it is dry and contain no fines. As a result, it is difficult to achieve vertical or nearly vertical cuts without support for a few hours.
ii. *Soils with high groundwater*. Groundwater behind the soil nail wall causes stability problem, as a result, it needs proper drainage. In addition to these drill holes (particularly in loose granular soils) collapse easily due to a large amount of groundwater, this process requires temporary casing and increases the cost of the soil nail wall. It also makes the construction of facing (shotcrete application) difficult.
iii. *Soils with cobbles and boulders*. The presence of large cobbles and boulders in the soil cause drilling difficulties and leads to construction costs and time delays.
iv. *Soft to very soft fine-grained soils.* These types of soils have SPT N-value less than 4 and are unsuitable for soil nailing due to their low bong strength at the grout-soil interface, these require very long nail length to get the required bond resistance. Hence, construction cost increases. Long-term deformations (creep) possibly will a major concern for highly plastic clays. Generally, creep deformations are less critical for temporary soil nailing applications.

v. *Collapsible soils*. These soils are competent but after saturation, they can experience large and sudden volume changes. These changes may occur in the absence of added loads. The collapse of the internal structure of these soils can cause problems during excavation or can deteriorate the long-term bond resistance at the grout-soil interface.

vi. *Organic soils*. Some organic soils such as organic silts, organic clays, and peat typically exhibit very low shear strengths and thereby low bond strengths, which causes uneconomical nail lengths. In addition, organic soils tend to be more corrosive than inorganic soils.

vii. *Highly corrosive soil or highly corrosive groundwater.* Some non-natural soil-like materials with high corrosion potential include cinder and slag (i.e., both residues derived from the smelting or refining of metals) are unfavorable for permanent soil nail wall.

viii. *Weathered rock with unfavorable weakness planes*. Weathered rock with joints, fractures, shears, faults, bedding, schistosity, or cleavage may affect the drill hole stability and make grouting difficult. Grouting in rock with very large open joints or voids will be very difficult and/or expensive due to excessive grout loss.

ix. *Karst formations*. Grouting in karstic formations is not appropriate due to the potential for excessive grout loss.

x. *Loess.* Loess may tend to collapse when it gets water after the installation of the soil nails. As a result, the collapse potential of these soils must be evaluated. Additionally, low soil shear resistance may arise for the wetted condition. In these cases, it requires long soil nail lengths.

xi. *Expansive soils.* These soils may induce localized pressures on the facing and may decline the bond resistance.

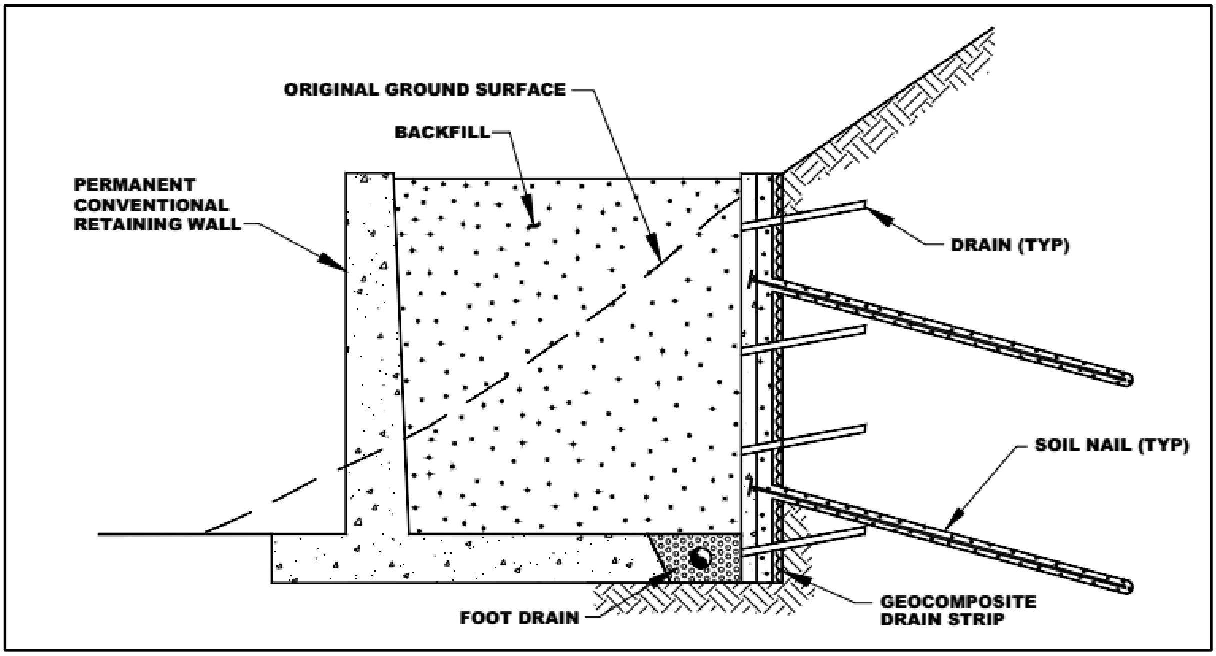
In addition to the above unsuitability and difficulties, other aspects such as repeated freeze-and-thaw cycles due to increased localized pressures; granular soils of very loose and loose soil due to excessive settlement; and loose and very loose saturated granular soil due to liquefaction should be considered.

2.2.7. Application of Soil Nailing

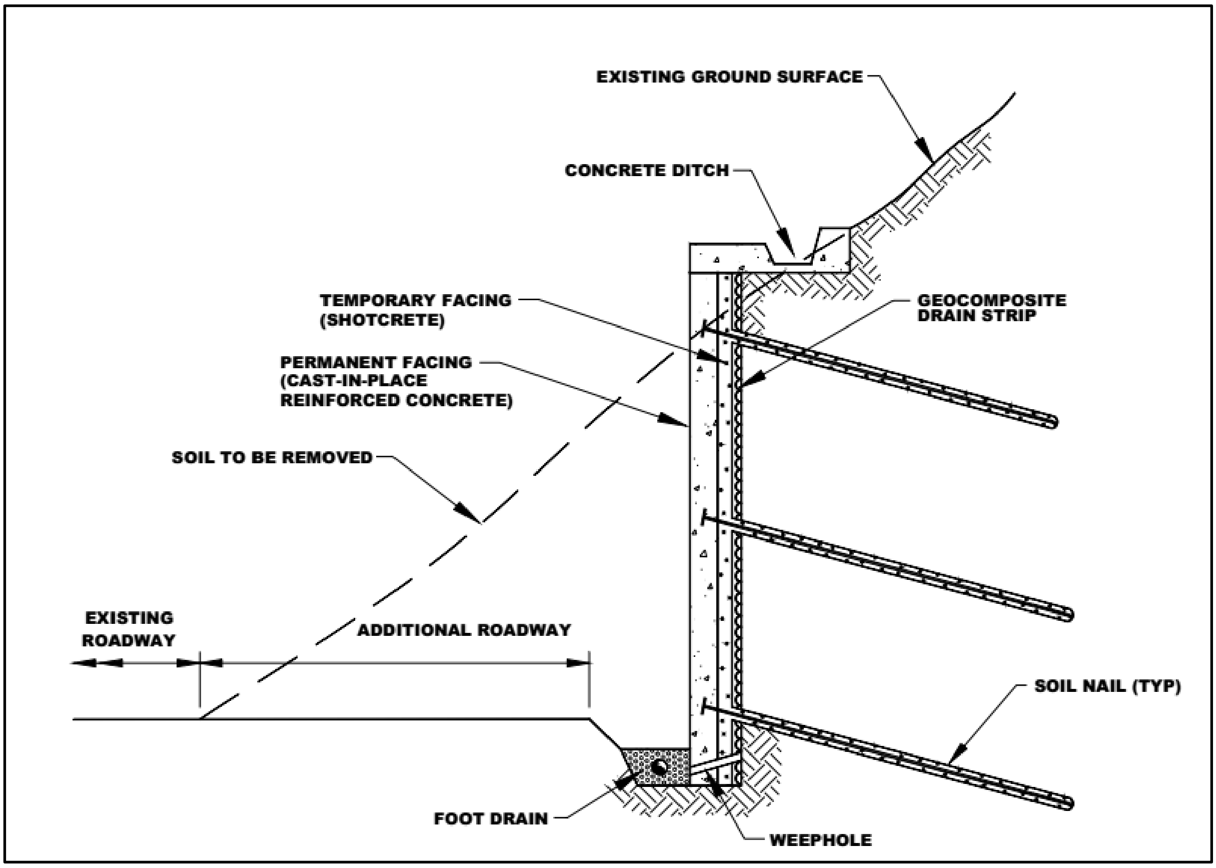
Soil nail walls have been successfully used in the temporary or permanent soil retaining (excavation) applications for ground conditions that require vertical or near-vertical cuts such as excavations in roadway cuts, excavation in the urban environment, etc. some of the application are discussed below (Lazarte, 2003; Lazarte, et al., 2015).

2.2.7.1. Retaining Structure in Cuts

In favorable ground conditions, soil nail walls can be used as retaining structures for any permanent and temporary vertical or near-vertical cut constructions, because of their stabilizing resistance in situations where other retaining structures such as anchor walls are used. The facing system in soil nailing is suitable for several aesthetic requirements. In this application, soil nailing is good looking because it has a tendency to minimize excavation, requires realistic right of way and clearance. Hence, it minimizes environmental impacts in the corridor of transportation systems. Soil nail wall is applicable for uphill widening projects that must be constructed within an existing right of way or on a steep slope. Figure 2.8 shows the application of soil nail walls in permanent and temporary cut excavations.



a) Application of soil nail wall for temporary shoring



b) Application of permanent soil nail wall in highway widening and traffic lane additions

Figure 2.8: Application of Soil Nail Wall for Temporary and Permanent Cut Slopes (Lazarte, 2003)

2.2.7.2. Retaining Structure Under Existing Bridge Abutments

As compared to conventional ground anchor support, soil nail walls are advantageous for underpass widening by removal of an existing bridge abutment end slope (see Figure 2.9). Soil nail walls can be installed at comparable costs with a ground anchor but the installation of soil nail walls does not interrupt the bridge traffic. If a ground anchor support wall were used, soldier beams or piles would have to be installed through the bridge deck because of limited overhead space under the bridge prior to excavating the end slope abutment. This process results in overpass traffic disruption and requires additional costs associated with lane closures and the procurement of large steel beams. Conversely, steel reinforcing bars used as soil nails are readily available. One disadvantage of the use of soil nail walls for end slope removal projects is that because the first level of soil nails is typically placed within 1 to 2 m from the top of the slope and because the nails are sloped downward, it is possible that the bridge girders will interfere with soil nail installation. This problem can usually be avoided by positioning the soil nails horizontally to be within the clear space between bridge girders.

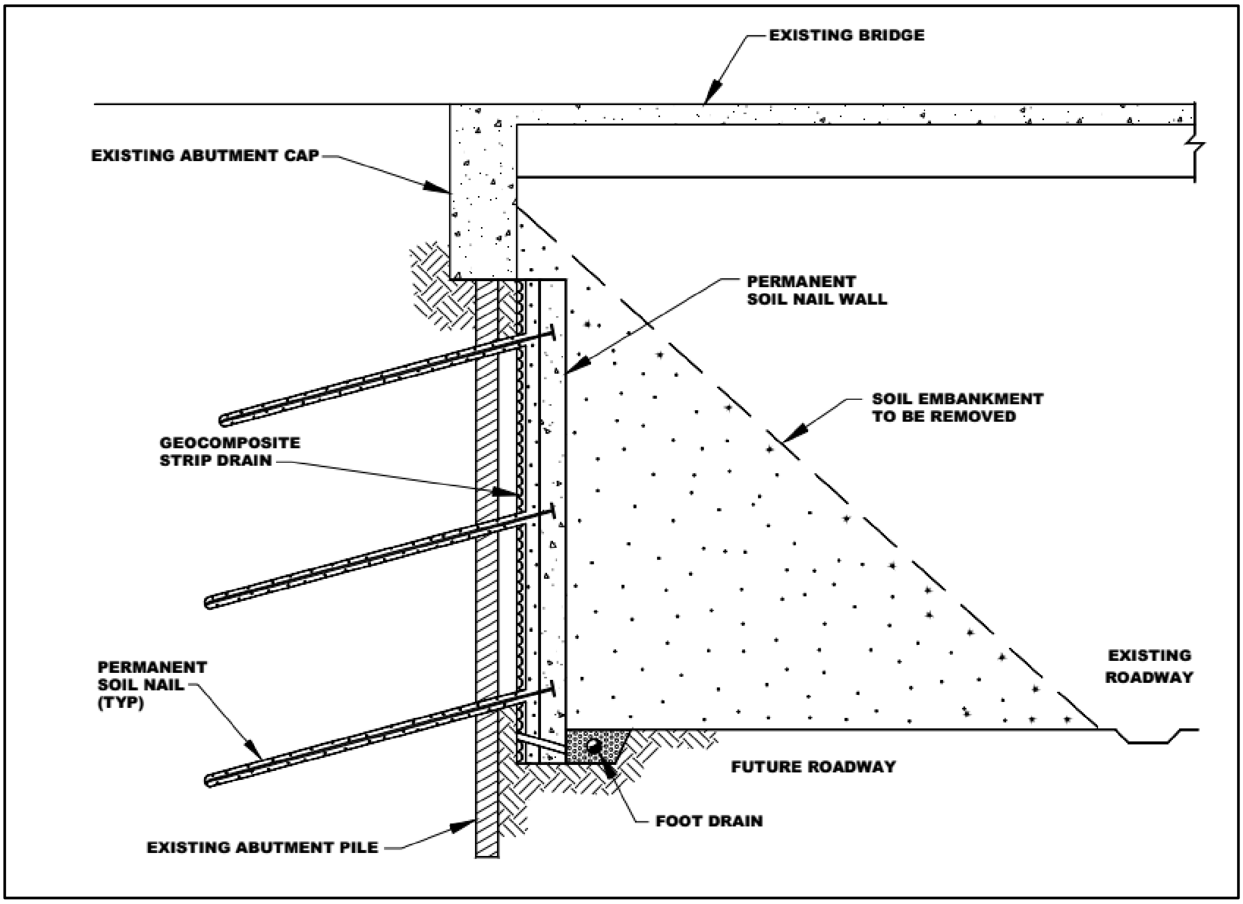


Figure 2.9: Road Widening Under Existing Bridge. (Lazarte, 2003)

2.2.7.3. Repair and Reconstruction of Existing Retaining Structures

Soil nails can be used to stabilize and/or strengthen failing or distressed retaining structures. For example, when some mechanically stabilized earth (MSE) walls may exhibit excessive deformation due to poor design and/or poor construction, soil nails can be installed directly through the face of an MSE wall if the existing face is sufficiently stable to resist drilling. As the MSE wall continues to deform, the backfill of the MSE wall and its facing would transfer loads to the installed soil nails, and these would transfer loads to stable soils lying behind the MSE-reinforced block of soil.

2.2.7.4. Tunnel Portals

The origin of soil nailing comes from supporting the tunnel lining. The application is similar to that of road cut stabilization but care should be taken during design and construction. The vertical stability of the shotcrete above the tunnel must be considered. The soil nail load transfer to the tunnel, the interaction between the soil nails and the initial shotcrete support and lining of the tunnel near the portal must be taken in to account. The layout of soil nails may be different than the conventional roadway soil nailing. Soil nails must be installed without affecting the tunnel support components.

2.2.7.5. Hybrid Soil Nail Walls

Soil nail wall can be used with other wall systems such as the ground anchor and MSE walls. This happens when there is a complex layout or the costs with other retaining structures are too high. Figure 2.10 shows an example of a hybrid MSE/soil nail wall. When there are utilities or underground obstructions exist and difficult to install soil nail walls in the top part, soil nails with ground anchors can be used. In addition, where there is a potential of deep-seated failure, ground anchors with soil nails can be used.

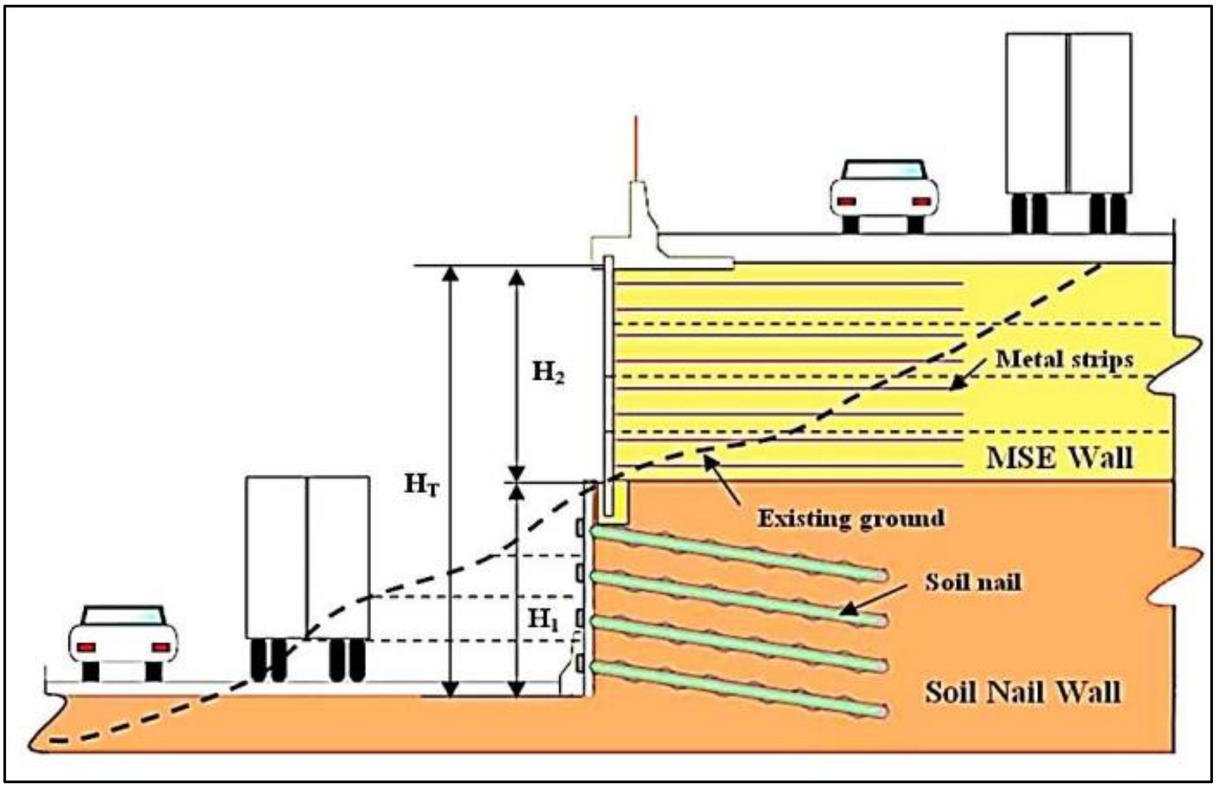


Figure 2.10: Hybrid Soil Nail/MSE Wall (Lazarte, et al., 2015)

2.2.7.6. Shored Mechanically Stabilized Earth (SMSE) Walls

Soil nail with MSE wall can be used for widening low volume roads by fill placement in steep terrain. It can also be used as shoring to stabilize the back slope (or back-cut) first, and then allow the construction of a conventional MSE wall in front of the soil nail wall. Figure 2.11 shows a generic cross-section of this combination. If the soil nail wall is designed as a permanent wall, nails can significantly reduce the long-term lateral pressures on the MSE wall. This configuration is known as an SMSE wall.

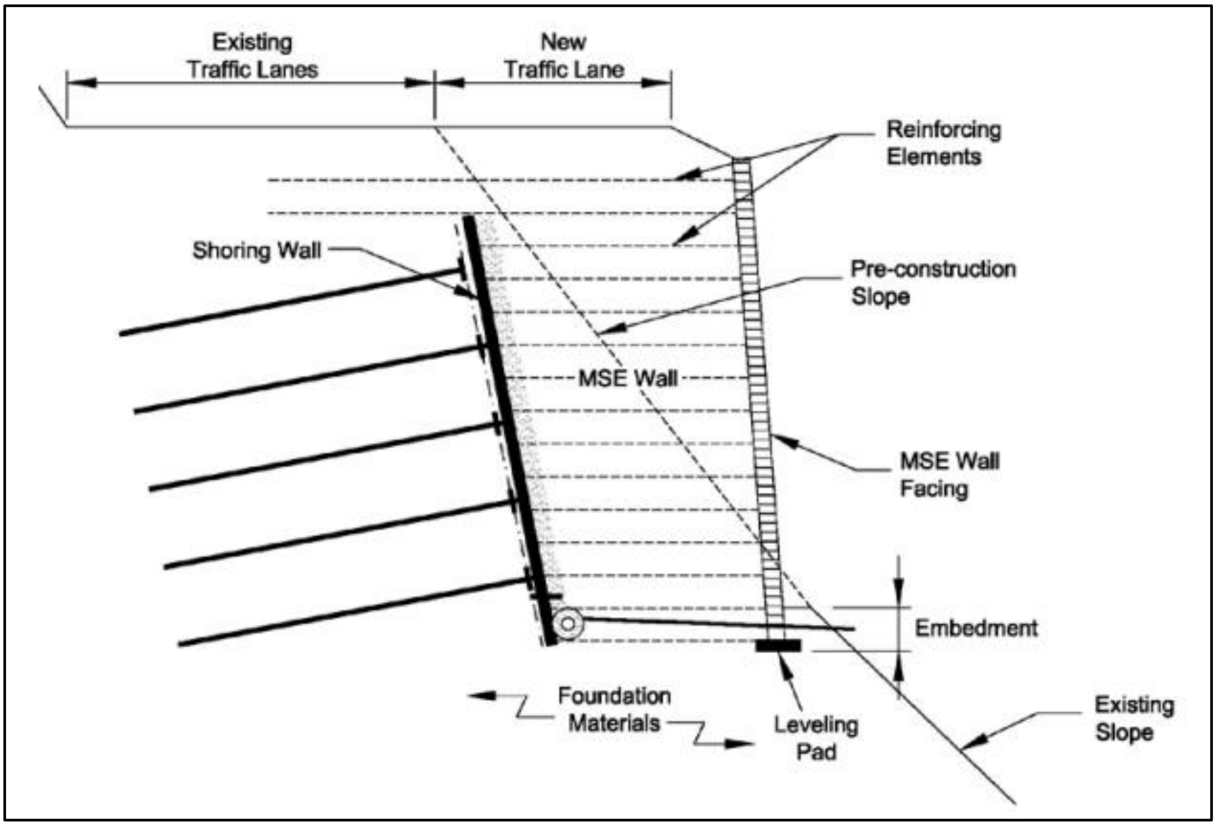


Figure 2.11: SMSE Wall for Steep Terrain (Lazarte, et al., 2015)

2.3. Load Transfer Mechanism in Soil Nail Wall

Soil nail wall construction is not undertaken the whole excavation at once rather through staged excavation i.e. excavating the first lift and installing the nail and facing. Then excavating the second lift, installing the nail and facing element. These procedures continued up to the bottom of the soil nail wall. When the first row of nails and facing are installed, the load from deformation of the upper soil is transferred to the nails through shear stresses and axial forces. The top portion of Figure 2.12 shows the axial force distribution of each nail at each phase.

After installation of the first row of the nail and facing element, the second phase continues by excavating the second lift and inserting the second rows. Here until the second row is installed, all the forces are resisted by the first row of the nail. Until the third phase is excavated and the third nail rows and facing are installed, the force derived from deformation is resisted by first and second nail row. The load is transferred to other rows of the nail in a similar manner (Lazarte, 2003).

To attain the factor of safety for global stability, the nails must extend beyond the potential failure surface. As lateral deformation increases due to subsequent excavation, additional shear stresses along with the soil nail/soil interface and axial forces of the previously installed nails are mobilized. As the depth of excavation increases, the size of the retained soil mass increases.

Generally, the analysis should consider both ‘during construction’ and ‘post-construction’ loading conditions to establish the most critical case at each soil nail level. Mostly in the absence of potential seepage, the most critical failure arises after the wall is completed.

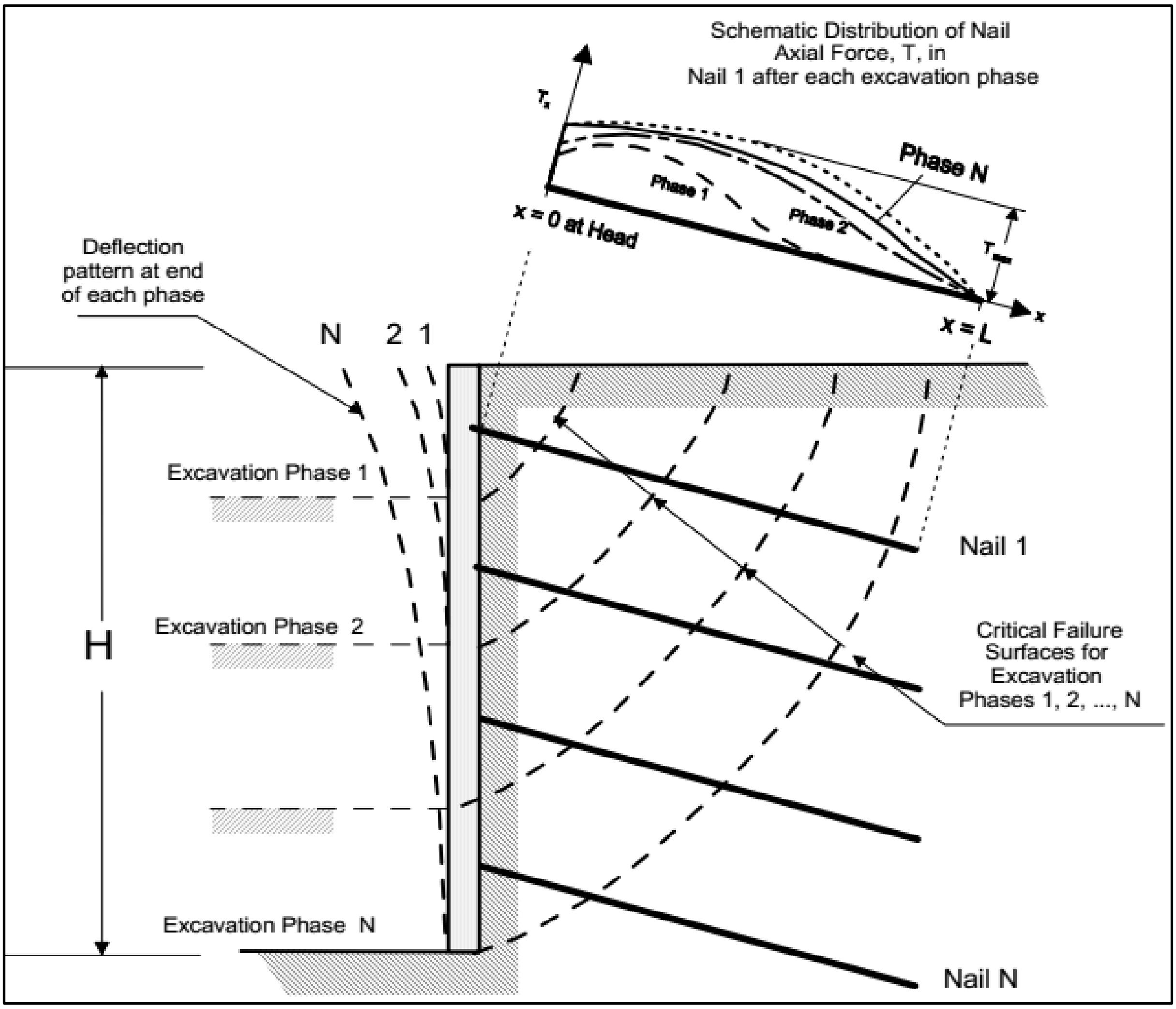


Figure 2.12: Potential Failure Surface and Load Transfer Concepts in Soil Nail Wall (Lazarte, 2003)

2.4. Bond Strength and Pull-Out Behavior of Soil Nail Wall

2.4.1. Bond Strength

The pullout capacity of a soil nail wall depends on the size of nail i.e. the length and diameter of the nail and the ultimate bond strength q_u i.e. the characteristics and density of the soil. The bond strength is the mobilized shear resistance along with the soil-grout interface.

The bond strength of drilled and grouted type of nail mainly depends on (Lazarte, 2003):

- ground conditions around the nail (soil type and conditions);

- soil nail installation (drilling method, grouting procedure, grout nature, grout injection (e.g.; gravity or under pressure); and
- The size of the grouted zone.

In addition to the above, for drilled and grouted nails in cohesionless soil, the magnitude of the overburden pressure and the nature of the granular soil affect the soil friction angle, which in turn affects the bond strength.

Typical values of the ultimate bond for drilled and grouted nails installed in various soils and using different drilling methods are presented in Table 2.1. The values in this table correspond to gravity grouting only.

Table 2.1: Estimated Ultimate Bond Strength of Soil Nails for Soil and Rock (Juran, 1991)

Material	**Construction Method**	**Soil/Rock Type**	**Ultimate Bond Strength, qu (kPa)**
Rock	Rotary Drilled	Marl/limestone Phyllite Chalk Soft dolomite Fissured dolomite Weathered sandstone Weathered shale Weathered schist Basalt Slate/hard shale	300-400 100-300 500-600 400-600 600-1000 200-300 100-150 100-175 500-600 300-400
Cohesionless Soils	Rotary Drilled	Sand/gravel Silty sand Silt Piedmont residual Fine colluvium	100-180 100-150 60-75 40-120 75-150
	Driven Casing	Sand/gravel low overburden high overburden Dense Moraine Colluvium	 190-240 280-430 380-480 100-180
	Augered	Silty sand fill Silty fine sand Silty clayed sand	20-40 55-90 60-140
	Jet Grouted	Sand Sand/gravel	380 700
Fine-Grained Soils	Rotary Drilled	Silty clay	35-50

	Driven Casing	Clayey silt	90-140
	Augered	Loess	25-75
		Soft clay	20-30
		Stiff clay	40-60
		Stiff clayed silt	40-100
		Calcareous sandy clay	90-140

2.4.2. Pull-Out Behavior of Soil Nail

In general, the pullout behavior of a soil nail is affected by bond strength. The mobilized bond shear stress is affected by nail length, the magnitude of the applied tensile force, grout characteristics and soil conditions. The mobilized pullout per unit length, Q, (also called the load transfer rate) can be expressed as:

$$Q = \pi\, q\, D_{DH} \tag{2.1}$$

Where

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

D_{DH} is the 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 shown in Figure 2.13, the tensile force can be related to the interface shear stress as:

$$dT = \pi D_{DH} q dx = Q dx \qquad (2.\ 2)$$

Then the tensile force (T) at a distance “x” along the bar is:

$$T(x) = \int_0^x \pi D_{DH} q dx = \int_0^x Q dx \qquad (2.\ 3)$$

At the end of the pullout length (l_p) the nailing force will be;

$$T(L_p) = T_0 = QL_p \qquad (2.\ 4)$$

The pull-out capacity of the nail is expressed as

$$R_p = T_{max} = Q_u L_p = \pi q_u D_{DH} L_p \qquad (2.\ 5)$$

Q_u is pull-out capacity per unit length, and q_u is the ultimate bond strength

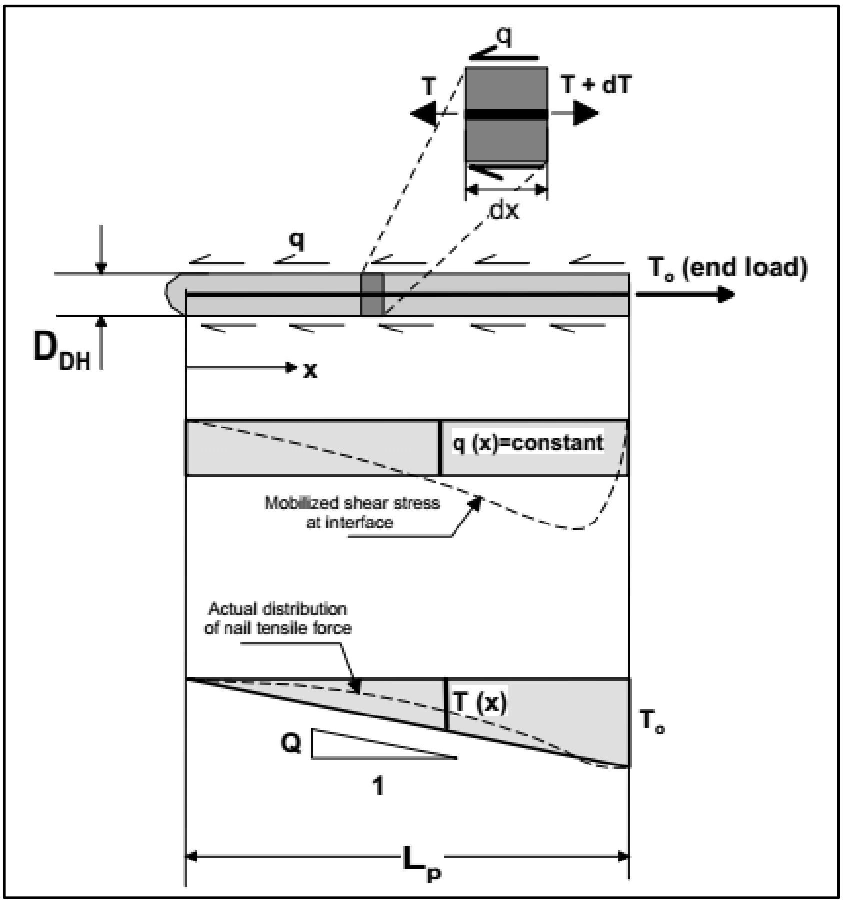


Figure 2.13: Nail Tensile Stress Transfer Mode (Lazarte, 2003)

2.5. Factors Affecting Soil Nail Pullout Resistance

The determination of soil nail pullout resistance is the key parameter in the design and analysis of the soil nail wall. There are both laboratory and field pullout tests for the determination of pullout resistance. The pullout resistance is influenced by the installation method, overburden pressure, grouting pressure, and roughness of nail surface, water content of the soil (degree of saturation of soil), soil dilation and soil nail bending. The pullout shear resistance of the nail can be calculated from the measured peak pullout force at the nail head divided by the active surface area of the nail.

$$\tau = \frac{P}{\pi DL} \tag{2. 6}$$

Where here P is the pullout force applied on the soil nail head; D is grouted nail diameter and L is grouted nail length.

2.5.1. Effect of Overburden Pressure on Pullout Resistance

The normal stress is the stress condition that will influence the pullout resistance of the nail.

According to (Su, et al., 2008), laboratory soil nail pullout test is performed on completely decomposed granite (CDG) fill at two degrees of saturation (38 and 75%) with overburden pressure of 40, 80, 120, 200, and 300 kPa before the soil nails are installed to simulate the actual construction procedure in the field. It is observed from the test that (1) during pullout; the vertical stress in the soil surrounding the soil nail is increased. (2) The development of pullout shear resistance was mainly derived from the constrained dilatancy of the soil. No apparent relationship between the soil nail pullout resistance and applied overburden pressure was observed which is consistent with (Clouterre, 1991) by field test.

2.5.2. Effect of Grouting Pressure on Pullout Resistance

Grouting pressure has an effect on nail pullout resistance. The test result shows that the amount of soil adhered to the nail surface after pullout increased with the increase in grouting pressure. With a constant degree of saturation, when the grouting pressure increases, the pullout resistance of the nail increases almost linearly with the increase in grouting pressure (Yin J.-H., 2009). When the grouting pressure is low, the soil around the drill hole is loose and the soil arching effect will exist to some time. However, when the grouting pressure is high, the loose soil is well compacted due to the pressure that the grout enters to surrounding soil and densify it. So that the strength of the pullout may even higher than before.

According to (Seo, et al., 2012), in the field test results, the pullout load resistance of the pressurized grouting is approximately 36% greater than that of gravitational grouting because of the pressurized grouting results in an enlarged diameter, higher roughness, and compaction of soil near the grouted hole.

2.5.3. Effect of Saturation on Pullout Resistance

The degree of saturation is an important factor, especially in permanent soil nail walls. The laboratory test result shows that the nail-soil interface adhesion is reduced to a high degree of saturation than at the natural moisture content (Pradhan, 2003). This indicates that the soil dilatancy is reduced when the soil is saturated and therefore the pullout resistance decreased. However, if the soil is completely dry, during grouting the water from the cement grouted is absorbed by the surrounding soil and may result in contraction of the grout. Therefore, it may reduce the bond strength between the nail surface and surrounding soil.

2.6. Soil Nail Interaction and Tensile Forces Distribution

The soil nail interaction behind the soil nail wall is complex. The load applied to the soil nails causes the outward movement during excavation. The portion of the nail behind the failure surface (i.e., the anchoring zone) is pulled out of the soil slope. The tensile forces in the soil nail, T, vary from the anchoring zone to the facing; they start as zero at the end of the nail, increase to a maximum, T_{max}, the value in the intermediate length, and decrease to a value T_o at the facing as shown in Figure 2.14.

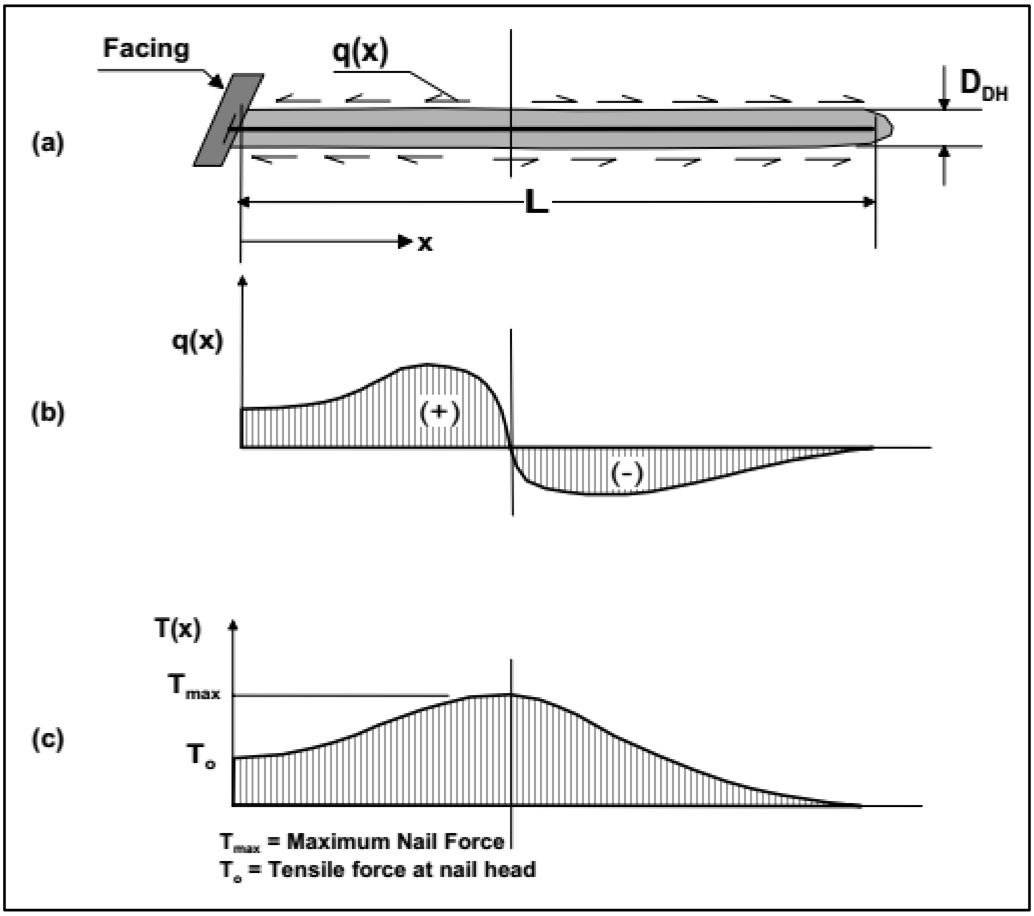


Figure 2.14: Soil Nail Stress-Transfer Mechanism.

To design the tensile force distribution the following three conditions related to the maximum tensile force are noted. The value T_{max} is bounded by three limiting conditions: the pullout capacity, R_P, the tensile capacity, R_T, and the facing capacity, R_F. The facing capacity, which is discussed in the next chapter.

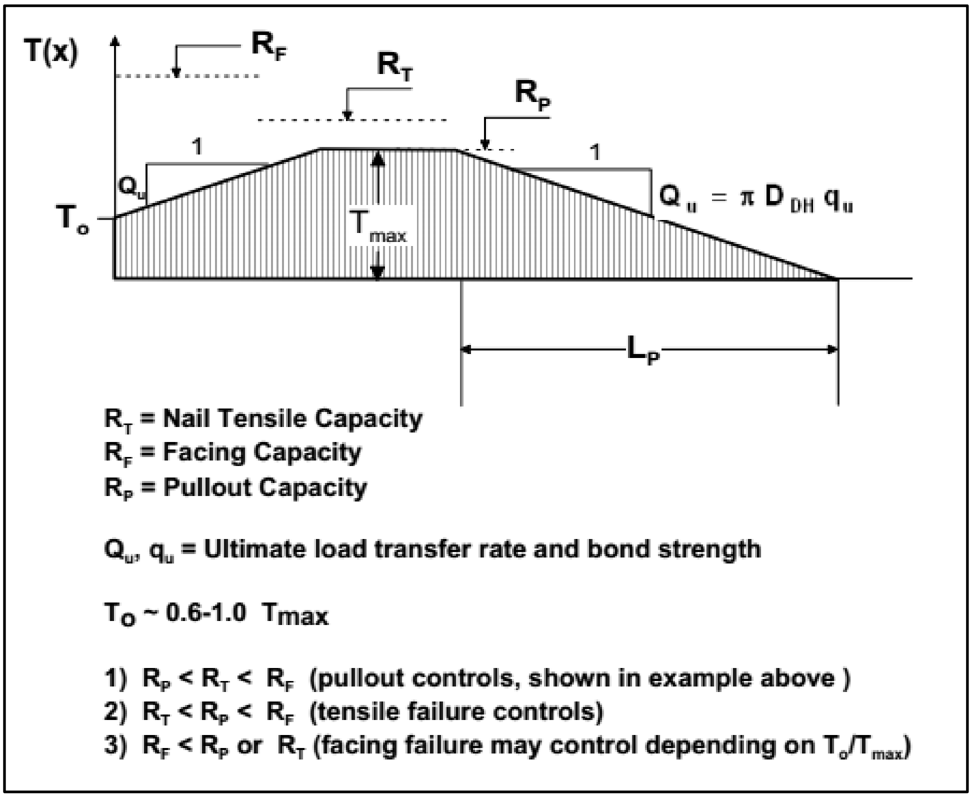


Figure 2.15: Simplified Distribution of Nail Tensile Fore (Lazarte, 2003)

Due to the complexities of load transfer within individual nails, the location of maximum nail tensile forces is close to, but generally does not coincide with, the location of the critical failure surface found during global stability analysis. Strain measurements in instrumented soil nail walls have indicated that in the upper portion of the wall, the maximum tensile force occurs approximately between 0.3 H to 0.4 H behind the wall facing (Byrne R.J., 1998). In the lower portion of the wall, the maximum tensile force occurs approximately between 0.15 H to 0.2 H behind the wall facing.

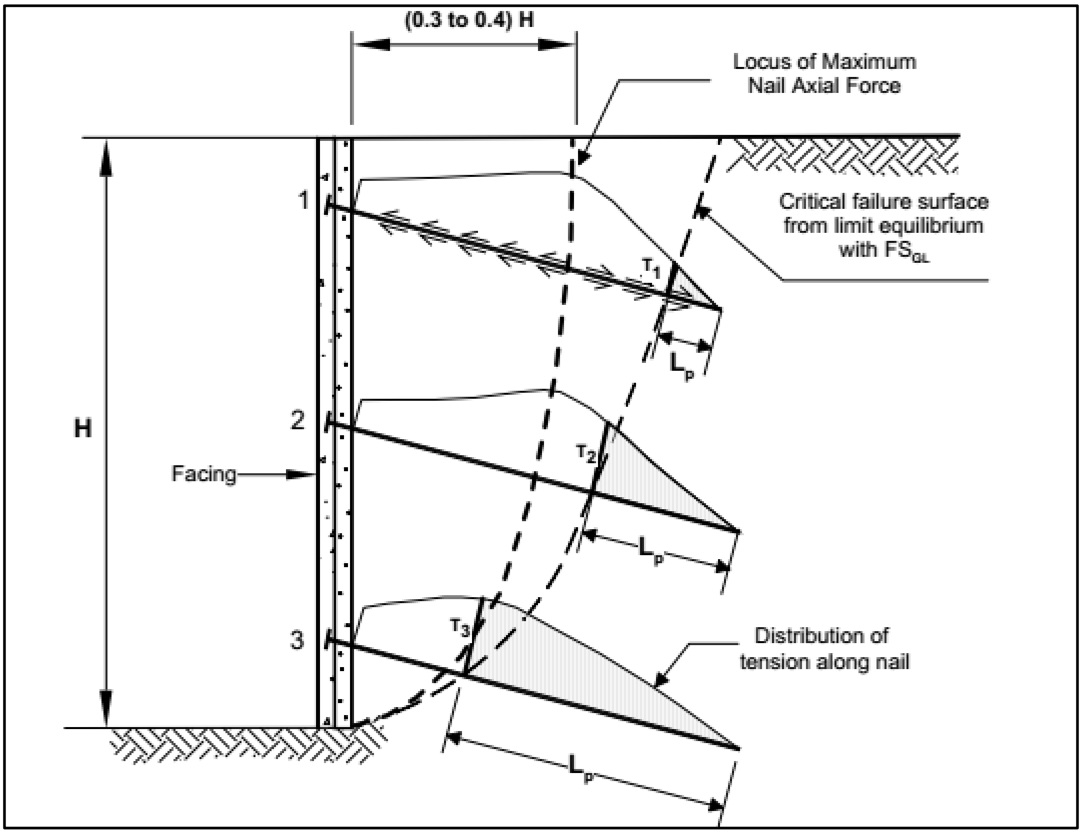


Figure 2.16: Location of Maximum Tensile Force in the Soil Nail Wall (Lazarte, 2003)

2.7. The Potential Failure Modes in Soil Nail Wall

According to (Lazarte, 2003), during the design and analysis of soil nail wall, there are two main critical conditions called strength limit state and service limit state to be considered. The limit state is based on the potential failure mechanisms and collapses state of the soil nail wall system whereas service limit state is based on the loss of service function due to excessive wall deformation.

Soil nail wall system may fail through different reasons. The potential failure modes in soil nail wall are classified as external, internal and facing failure modes as shown in Figure 2.17 The detailed analysis is presented in the next chapter.

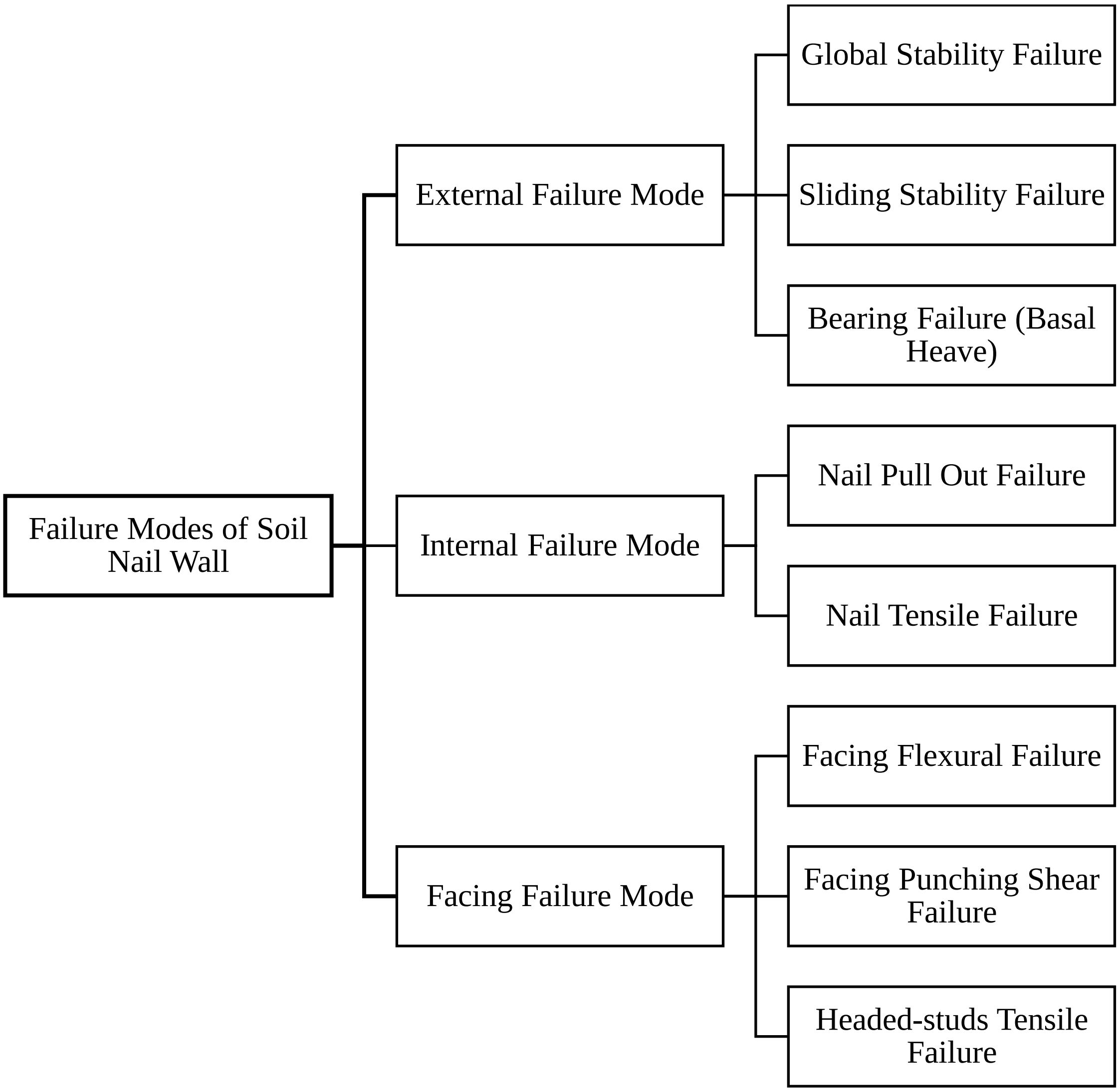


Figure 2.17: Different Failure Mode of Soil Nail Wall

2.7.1. External Failure Mode

External failure modes refer to the development of failure surface passing through or behind the soil nail mass. For external failure modes, the soil nail wall mass is treated as a block. Stability calculations take into account the resisting soil forces acting along the failure surfaces to establish the equilibrium of this block. If the failure surface intersects one or more soil nails, the intersected nails contribute to the stability of the block by providing an external stabilizing force that must be added to the soil resisting forces along the failure surface. External failure modes are classified as global stability, sliding and bearing failure as shown in Figure 2.18

2.7.2. Internal Failure Mode

Internal failure modes refer to failure in the load transfer mechanisms between the soil, the nail, and the grout. Soil nails mobilize bond strength between the grout and the surrounding soil as the soil nail wall system deforms during excavation. The bond strength is mobilized progressively along the entire soil nail with a certain distribution that is affected by numerous factors. As the bond strength is mobilized, tensile forces in the nail are developed. The two most common internal failure types are;

- Nail Pullout Failure: Nail pullout failure is a failure along with the soil-grout interface due to insufficient intrinsic bond strength and/or insufficient nail length, Figure 2.18d.
- Tensile Failure of the Nail: The nail can fail in tension if there is inadequate tensile strength, Figure 2.18f.

2.7.3. Facing Failure Mode

The most common potential failure modes at the facing-nail head connection are shown in Figure 2.18h-j.

- Flexure Failure: This is a failure mode due to excessive bending beyond the capacity of facing flexural. This failure mode should be considered separately for both temporary and permanent facings.
- Punching Shear Failure: This failure mode occurs in the facing around the nails and should be evaluated for both temporary and permanent facings.
- Headed-Stud Tensile Failure: This is a failure of the connection bolts at a nail head in tension. This failure mode is only a concern for permanent facings.

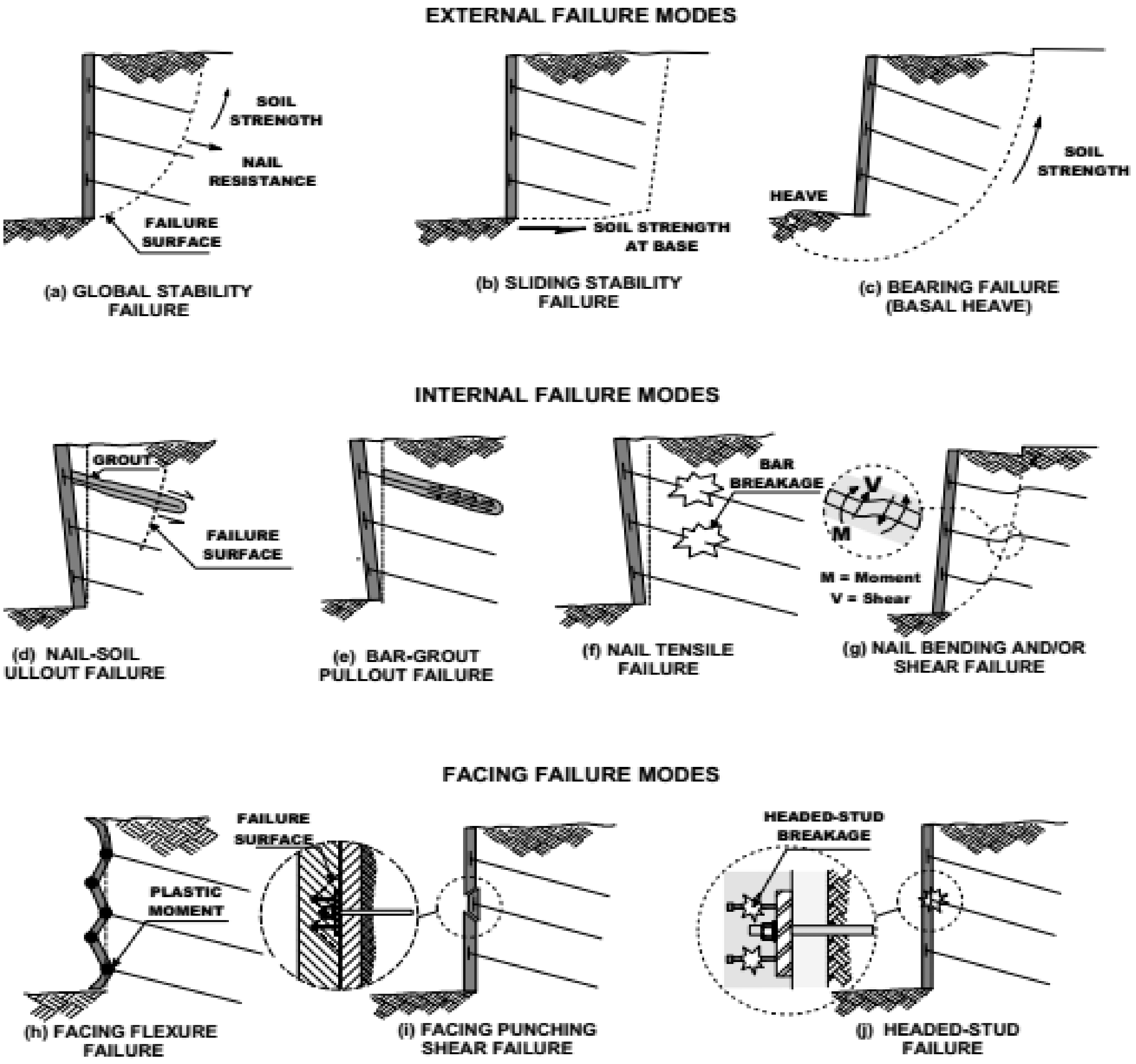


Figure 2.18: Failure Modes in Soil Nail Wall

2.8. Soil Nail Wall Design Approaches

There are different design approaches for soil nail wall design. Most methods are derived from the classical slope stability method with additional resistance force due to soil nailing. Some of the methods based on the limit equilibrium are as follows.

2.8.1. HA 68 Method

HA 68 method is based on the ultimate design state and serviceability limit state. The ultimate design state is considered when the structure is in the collapsed state while the serviceability limit state occurs during the life span (working and service condition) of the structure.

HA 68 method is developed in the UK. It gives a single unified effective stress design approach for all types of reinforced highway earthworks with slope angles to the horizontal in the range 10^0 to 70^0, and soil types in the strength range $\varphi=15^0$ to 50^0.

A limit equilibrium approach is adopted based on a two-part wedge mechanism with the inclusion of partial safety factors. The figure shows the geometry of HA 68’s two-part wedge mechanism. Equilibrium is reached when the driving forces, which consist of the self-weight of the structure and surcharge loads multiplied with the load partial factor (of predetermined value of unity) are in equilibrium with, the resisting forces which are the shear strengths of soil and the reinforcement forces divided by the material partial safety factors of predetermined values suggested by the Department of Transport. The assumption is made that the nails’ contribution is purely axial. Shear stress and bending stiffness are ignored in this design.

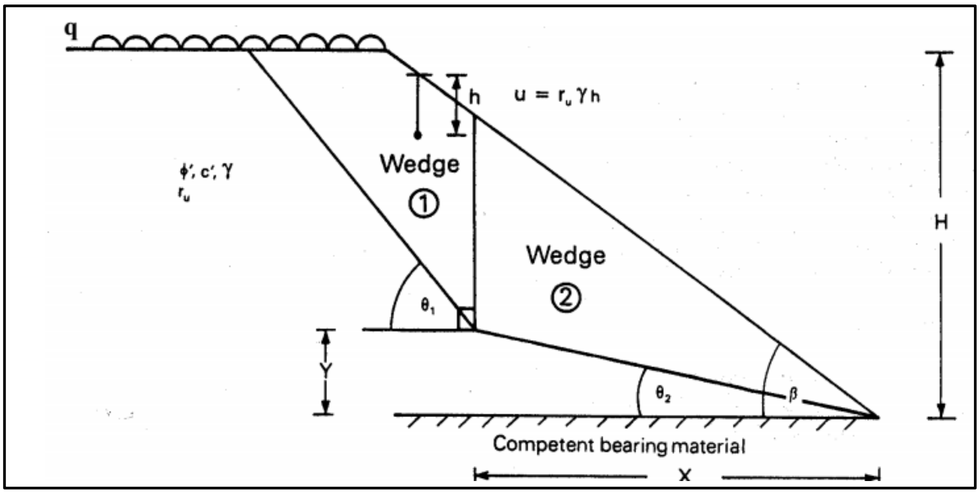


Figure 2. 19: Geometry of Two Wedge Methods of HA 68 Method.

2.8.2. BS 8006 (Code of Practice for Strengthened/Reinforced Soils Part 2: Soil Nail Design)

BS 8006 design approach is also based on the limit stated design (ultimate limit states (ULS) and serviceability limit states (SLS)). BS 8006 considers only the tensile (axial) capacity of the soil nails. The contribution of nail bending and shear resistance are second-order effects, only activated at high deformation levels, and conservatively ignored. The steps in BS 8006 for the design of soil nail structures are

- Determination of critical slip surface and resisting force to maintain equilibrium inactive zone.
- The determination of the tensile and shear loads for an initial constant spacing and inclination of nails of constant stiffness and length.
- A check for each level, allowing for stages of construction, against failure due to
  - tension in the nail at the slip surface,
  - pull-out of the length of the nail in the resistant zone,

- bending and shear in the nail near the slip surface, and
- Bearing failure of soil against the nail. (8006-2:2011, 2011)

2.8.3. The French Method (Program Clouture)

It is based on limit equilibrium approach and failure of nail reinforcements including tensile failure, pullout failure, grout reinforcement failures, bending/shear failure are considered. The failure surface is circular passing the toe of slopes and two-wedge method.

Based on this method at the intersection of failure surface and nail; normal force, shear force and bending moment are developed and taken into account. The determination of these forces and moment at a nail fail to form the consideration of soil nail friction interaction, soil nail lateral pressure interaction and consistent material (shear stress of material from which the reinforcement is made). (Clouterre, 1991)

2.8.4. FHWA Soil Nail Walls Reference Manual

Similar to other design methods the FHWA also uses limit equilibrium method for soil nail wall design. It considers two limit states; the strength limit state (ultimate limit state) and the service limit state. In these manual maximum lateral displacements, external and internal failure modes are explained briefly. It ignores the shear and bending effect of soil nail so that it is conservative relative to others. Therefore, from the above list of soil nailing design codes, FHWA is adopted in this thesis.

The manual consists of two design approaches; allowable stress design (ASD) and the Load and Resistance Factor Design (LRFD). Both of the approaches consider limit states in their calculations.

In ASD, allowable nail loads (tendon strength and pullout resistance of nail) are suggested for the reinforcement strength, recommended factors of safety are applied to the soil strength at both limit states in which the allowable nail loads, and factored soil strengths exceed the applied loads.

In contrast, in LRFD, at strength limit state, the soil and nail design strengths, which are obtained by applying resistance factors to their ultimate strengths, exceed the applied loads, which are multiplied by load factors.

From the comparative designs of ASD and LRFD methods (Lazarte, 2011) the required soil nail length calculated in both of the methods quite close to each other, approximately only 4% longer in the LRFD method. Therefore there is no significant difference between them, so allowable stress design (ASD) method is adopted in this thesis.

3. LIMIT EQUILIBRIUM ANALYSIS OF SOIL NAIL WALL SYSTEM

3.1. Analysis and Design of Soil Nail Wall Using FHWA Reference Manual

3.1.1. Global Stability of Soil Nail Wall

The global stability of the soil nail wall means the overall stability of the reinforced soil nail wall system. The slip surface may pass behind the wall or underneath the soil nail mass. The global failure occurs when the retained mass behind the soil nail wall is greater than the resistance of the soil along the slip surface and the intersected nails (Briaud, 2013).

According to (Lazarte, 2003), the global stability of the soil nail wall is calculated using two-dimensional limit equilibrium concept, which is similar to the conventional slope stability analysis. So that the potential sliding mass is modeled as a rigid block to establish the global force equilibrium. Then the factor of safety evaluated from the resisting and destabilizing forces. The trial slip surface is adopted until the critical failure surface is found. There is various failure surface such as planar, bi-linear with two wedges slipping mass, parabolic, log spiral and circular. However, research shows that the choice of the failure surface does not affect the final calculated factor of safety significantly (Long J.H., 1990).

The simplified single wedge failure mechanisms are used to illustrate the basic force that resists and destabilizing the global stability problem as shown in Figure 3.1.

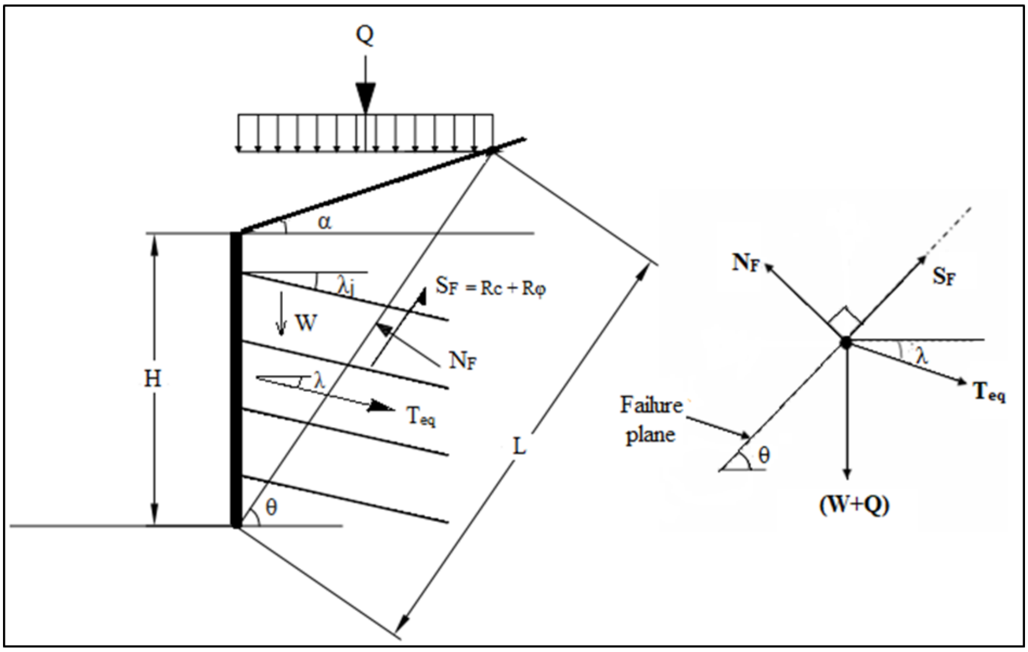


Figure 3.1: Single Wedge Failure Mechanism in Global Stability Adopted from (Lazarte, 2003)

Therefore, the factor of safety against global stability failure, which acts tangent to the potential failure plane, can be expressed as (Briaud, 2013):

$$FS_G = \frac{\sum Resisting\ Forces}{\sum Driving\ Forces} \tag{3. 1}$$

$$\sum \text{Normal force; } (Q+W)\cos(\theta) + T_{eq}\sin(\theta+\lambda) - N_F = 0 \tag{3. 2}$$

$$\sum \text{Tangential force; } (Q+W)\sin(\theta) - T_{eq}\cos(\theta+\lambda) - S_F = 0 \tag{3. 3}$$

Where $S_F = R_c + R_f = c_m L + N_F \tan\varphi_m$

$$tan\varphi_m = \frac{tan\varphi}{FSG} \tag{3. 4}$$

$$c_m = \frac{c\prime}{FSG} \tag{3. 5}$$

Therefore the global factor of safety will be

$$FS_G = \frac{\sum R}{\sum D} = \frac{cL + [(W+Q)cos\theta + T_{eq}sin(\theta+\lambda)]tan\varphi + T_{eq}\,cos(\theta+\lambda)}{(W+Q)sin\theta} \tag{3. 6}$$

Where

Q - Surcharge load;

W - Weight of the sliding wedge;

θ - Failure plane inclination;

λ - Inclination of resultant nail force;

λ_j - the inclination of an individual nail;

L - Length of failure plane;

α - slope angle;

H - Height of soil nail wall;

N_F - normal force on failure surface;

S_F - shear force on failure surface;

R_c - cohesive component of SF;

R_φ - frictional component of SF;

φ_m is the mobilized friction angle, and c_m is the mobilized cohesion. A single global factor of safety is used for the cohesive and frictional strength components of the soil (c’ and tanφ’, respectively). However, it is possible to select different safety factors for each strength component.

The length of the failure surface and the weight of sliding mass can be calculated as

$$L = H.\frac{cos\alpha}{sin(\theta-\alpha)} \tag{3. 7}$$

$$W = \frac{1}{2}\gamma H^2 \frac{cos\theta . cos\alpha}{sin(\theta-\alpha)} \tag{3. 8}$$

The equivalent resultant nail force (T_{eq}) is the sum of the force on each individual nail force and the force polygon is as shown in figure 3.2.

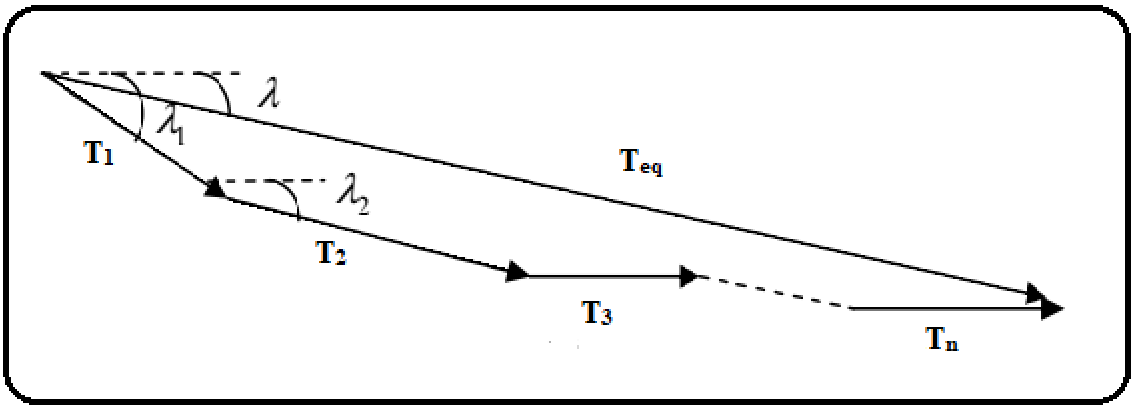


Figure 3.2: Force Polygon to Know the Equivalent Nail Force (T_{eq})

If the inclination angle (λ) is the same for all the nails, the equivalent nail force is the sum of each individual nail force.

$$T_{eq} = \sum_{j=1}^{n} Tj \tag{3. 9}$$

Where n is the number of nails and Tj is the nail force at each nail.

3.1.2. Sliding Stability of Soil Nail Wall

Sliding stability is the ability of the soil nail wall to resist sliding along the base of the retained system in response to lateral earth pressures behind the wall. Sliding occurs when the lateral earth pressure developed behind the wall is greater than sliding resistance along the base as shown.

The concept used here is similar to the gravity retaining structures using Rankine or Coulomb theories of lateral earth pressure. The soil nail wall system is considered as a rigid block where lateral earth pressure is applied behind the wall. Therefore, the factor of safety against sliding (FS_{SL}) is calculated as the ratio of the horizontal resisting forces to driving horizontal forces.

$$FS_{SL} = \frac{\sum R}{\sum D} \tag{3. 10}$$

Where;

$$\sum R = c_b\, B_L \; + \; [W + Q_D + P_A\, sin\alpha]\, tan\, \varphi_b \tag{3. 11}$$

$$\sum D = P_A cos\alpha \tag{3. 12}$$

$$P_A = 0.5\gamma H^2 K_a \qquad (3.13)$$

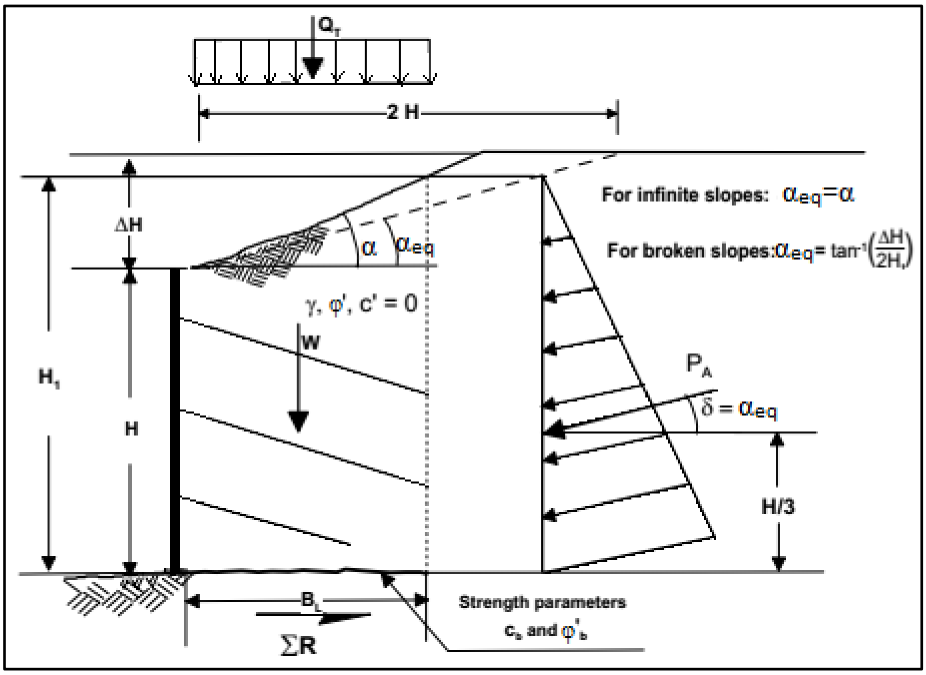


Figure 3.3: Sliding Stability of Soil Nail Wall Adopted from (Lazarte, 2003)

The terms in Figure 3.3 are:

H - Height of wall;

ΔH - slope rises up to bench (if present);

α - back slope angle;

α_{eq} - equivalent back slope angle;

c_b - soil cohesion strength along the base;

B_L - length of the horizontal failure surface where c_b is effectively acting;

W - Weight of soil nail block;

Q_D - a permanent portion of total surcharge load Q_T;

γ - Total unit weight of soil mass;

φ'_b - the effective angle of internal friction of the base (remolded or residual values may be needed if the significant movement takes place);

φ' - effective friction angle of soil behind soil nail block;

δ - wall-soil interface friction angle (for a broken slope, $\delta = \alpha_{eq}$, for infinite slope, $\delta = \alpha$);

H_1 - effective height over which the earth pressure acts [$H_1 = H + B \tan \alpha_{eq}$]; and

K_A - active earth pressure coefficient for soil behind the soil nail wall system.

The active earth pressure coefficient (K_A) can be obtained using the general Coulomb theory or the Rankine theory for cohesionless soil (assuming that the soil behind the soil nail wall behaves in accordance with c'= 0 in the long-term loading condition).

According to Coulombs theory

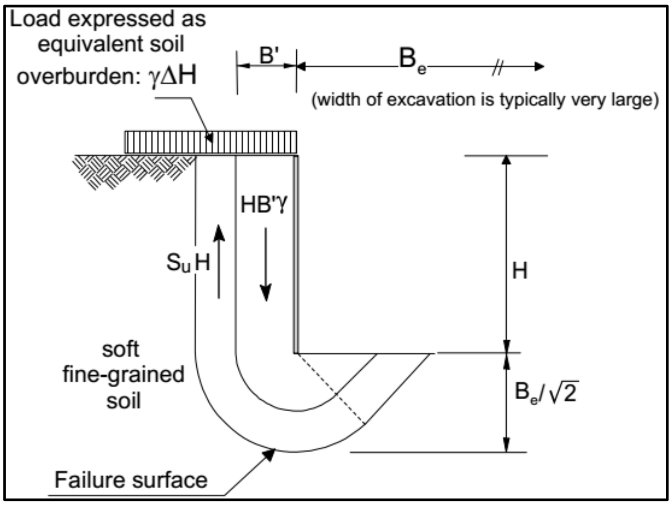
$$K_A = \frac{sin^2(90+\varphi')}{sin(90-\delta)\left[1+\sqrt{\frac{sin(\varphi'+\delta)\,sin(\varphi'-\alpha)}{sin(90-\delta)\,sin(90+\alpha)}}\right]^2} \qquad (3.\ 14)$$

According to Rankine for walls with face batter angle less than 8 degree and dry, sloping ground, the coefficient of active earth pressure is (Lazarte, 2003):

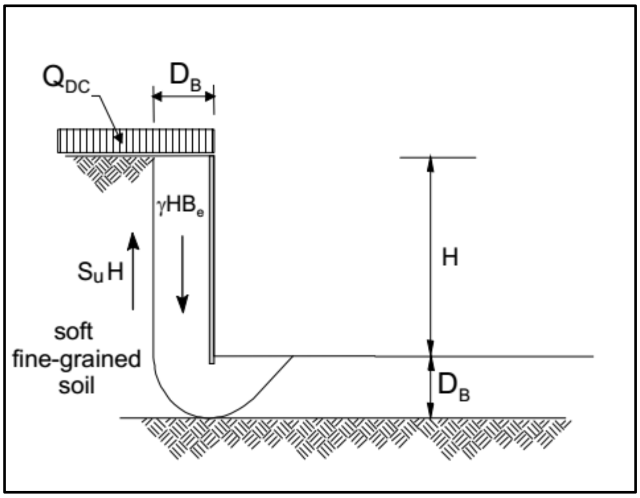
$$K_A = cos\alpha_{eq}\left[\frac{cos\alpha_{eq}-\sqrt{cos^2\alpha_{eq}-cos^2\varphi'}}{cos\alpha_{eq}+\sqrt{cos^2\alpha_{eq}-cos^2\varphi'}}\right] \qquad (3.\ 15)$$

3.1.3. Bearing Capacity of Soil Wall

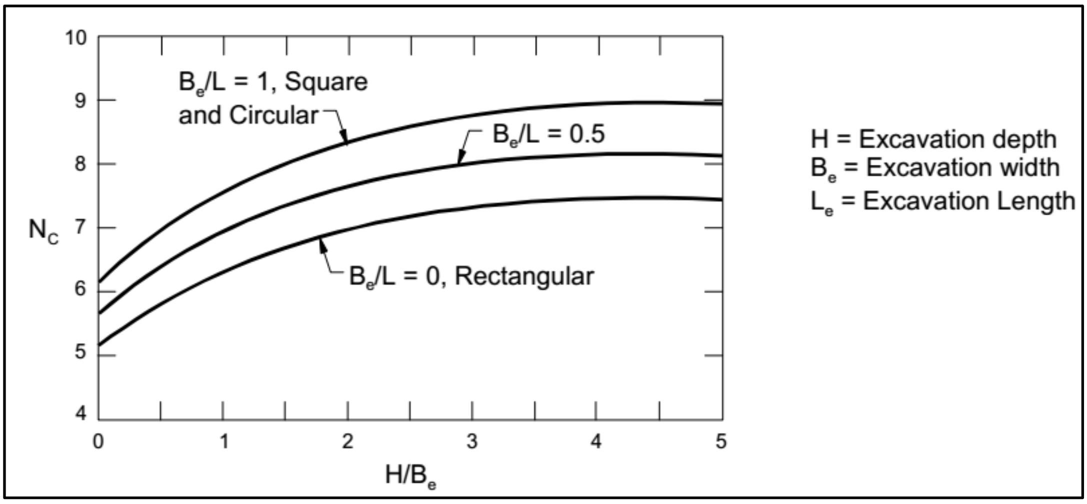
Bearing capacity is a major concern for fine-grained soils, soft soils. Since the facing does not penetrate below the bottom of the excavation, the unbalanced load caused by the excavation may cause heave and bearing capacity failure.



a. Deep deposit of soft fine-grained soil



b. Shallow deposit of soft fine-grained soil underlain by a stiff layer



c. Bearing capacity factor, Nc

Figure 3.4: Bearing Capacity Analysis for Heave from (Terzaghi K., 1996)

According to Terzaghi the factor of safety against heave is as follows (φ=0 and c=S_u):

$$FS_H = \frac{SuNc}{H_{eq}(\gamma - \frac{Su}{B'})} \tag{3. 16}$$

In addition, for c −φ soil the factor of safety will be:

$$FS_H = \frac{cN_c + 0.5\gamma B_e N_\gamma}{H_{eq}(\gamma - \frac{Su}{B'})} \tag{3. 17}$$

Where:

S_u - undrained shear strength of the soil;

N_c - bearing capacity factor;

γ - the Unit weight of the soil behind the wall;

H - Height of the wall;

Q_{DC} - the dead load.

H_{eq} - equivalent wall height = H+ΔH, with ΔH is an equivalent overburden;

and B' - width of influence, B' = B_e/ $\sqrt{2}$, where B_e - width of excavation.

For the wide excavation of soil nail wall H/ Be can be considered conservatively equal to 0 and for very long walls, it is conservative to adopt Be / Le = 0, and Nc =5.14.

3.1.4. Nail Pull-out Failure

As discussed in section 2.4.2 the pullout capacity of a soil nail wall is

$$R_p = T_{max} = \pi q_u D_{DH} L_p \tag{3. 18}$$

q_u is the bond strength of the soil nail interface.

L_p is the pull-out length for the case of pull-out test or length of the nail behind the failure surface in case of nailed structures.

For failure surface inclined 45+φ/2 with horizontal and horizontal back slope the pullout length can be calculated as:

$$L_p = L - \left[\frac{(H-z)cos\theta}{sin(\lambda+\theta)}\right] \tag{3. 19}$$

Where: θ - failure plane inclination; λ - the inclination of resultant nail force; L- length of each nail; z- depth form the top of the soil nail wall.

Pullout capacity is generally expressed as per the unit length of the horizontal spacing

$$R_p = T_{max} = \frac{\pi q_u D_{DH} L_p}{Sh} \tag{3. 20}$$

Where s_h is the horizontal spacing of the nail.

Generally, qu is obtained from the pull-out test. According to (Juran, 1991) prismatic value can be taken from Table 2.1. In the absences of pullout test data, some researchers calculate the bond strength as (Su L., 2008):

$q_u = c + \sigma_v tan\delta$ Where δ= the mobilized soil nail interface angle δ=2/3φ

σ_v= γh_j; hj - Depth of the midpoint of j th nail from the ground surface

c - cohesion of soil; γ - the unit weight of soil; φ - the effective angle of internal friction

During design the following allowable values of the bond strength or pull-out capacity is used;

$$q_{ALL} = \frac{q_u}{FS_p} \tag{3. 21}$$

$$R_{p\ ALL} = \frac{R_p}{FS_p} \tag{3. 22}$$

Where FS_P is the factor of safety against pull-out failure. In general, a minimum factor of safety of 2 is recommended against pull-out failure.

The factor of safety for each particular nail inserted form at a depth z can be expressed as:

$$FS_p = \left(\frac{R_p}{T}\right)_z \tag{3. 23}$$

Maximum axial force T at depth z can be obtained:

$$T_z[kN] = K(Q + \gamma z)s_h s_v \tag{3. 24}$$

Where k is Coulomb or Rankine earth pressure coefficient.

3.1.5. Nail Tensile Failure

Tensile failure of a soil nail takes place when the longitudinal force along the soil nail, T_{max-s}, is greater than the nail bar tensile capacity (R_T), which is defined as:

$$R_T = A_t f_y \tag{3. 25}$$

Where A_t is the cross-sectional area of nail and f_y is yield strength of the nail. To take into account uncertainties related to material strength and applied loads, allowable values of the nail tensile capacity are used in design as follows:

$$R_{T\ ALL} = \frac{R_T}{FS_T} \tag{3. 26}$$

Where FS_T is the factor of safety against soil nail tensile failure. In general, a minimum factor of safety of 1.8 is adapted for static loads.

3.1.6. Facing Flexural Failure

Different scholar's estimate the design tensile forces at the wall facing but the (Lazarte, 2003) recommends the tensile force developed on the facing as:

$$T_o = T_{max-s}\ [0.6 + 0.2(S_{max}[m] - 1)] \tag{3. 27}$$

Where: T_o - Design nail head tensile force;

T_{max-s} - Maximum design nail tensile force (when global factor of safety, $FS_G = 1.0$ (full soil mobilization), the safety factor for the tensile strength, $FS_T = 1.0$ (full nail tensile mobilization)); and

S_{max} - Maximum soil nail spacing. Use a maximum of S_V and S_H, the vertical and horizontal nail spacing, respectively.

The nail force is obtained when the soil pressure that causes facing failure can be applied to an influence area around the nail head. This force is designated as the facing flexure capacity, R_{FF}, and is related to the flexural capacity per unit length of the facing. The flexural capacity per unit length of the facing is the maximum resisting moment per unit length that can be mobilized in the facing section. Based on yield-line theory concepts, R_{FF} is estimated as the minimum of:

$$R_{FF}[kN] = \frac{C_F}{265} x(a_{vn} + a_{vm}) \left[\frac{mm^2}{m}\right] x \left(\frac{S_h h[m]}{S_v}\right) x f_y[MPa] \tag{3. 28}$$

$$R_{FF}[kN] = \frac{C_F}{265} x(a_{hn} + a_{nm}) \left[\frac{mm^2}{m}\right] x \left(\frac{S_v h[m]}{S_H}\right) x f_y[MPa] \tag{3. 29}$$

Where:

C_F - factor that considers the non-uniform soil pressures behind the facing (Byrne R.J., 1998);

h - Thickness of facing ;

d - half-thickness of facing;

a_{vn} - reinforcement cross-sectional area per unit width in the vertical direction at the nail head;

a_{vm} - reinforcement cross-sectional area per unit width in the vertical direction at mid-span;

a_{hn} - reinforcement cross-sectional area per unit width in the horizontal direction at the nail head;

a_{hm} - reinforcement cross-sectional area per unit width in the horizontal direction at mid-span;

S_H - nail horizontal spacing;

S_V - nail vertical spacing; and

fy - reinforcement tensile yield strength

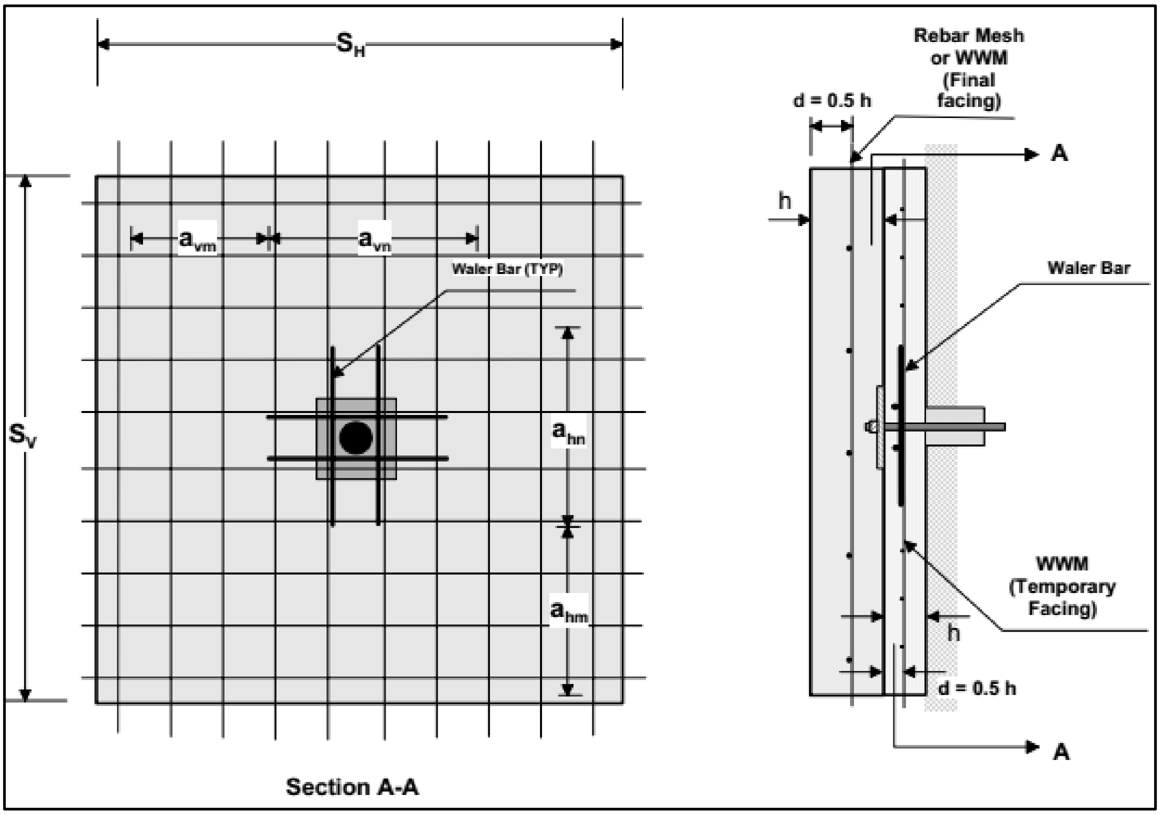


Figure 3.5: Geometry used in Flexural Failure Mode (Lazarte, 2003).

Table 3.1: Factors C_F

Type of Structure	Nominal Facing Thickness (mm)	Factor CF
Temporary	100	2.0
	150	1.5
	200	1.0
Permanent	ALL	1.0

The maximum moments in the facing are around a horizontal axis and the design of reinforcement in the vertical direction is more critical than the design of the horizontal reinforcement. However in practice, the cross-section area of reinforcement in the horizontal direction is the same as for the vertical direction (i.e., $a_{hm} = a_{vn}$ and $a_{hm} = a_{vm}$); therefore, the most critical case is the one that gives the minimum of SH/SV and SV/SH. When the same nail spacing and reinforcement are used in the horizontal and vertical directions, and 420 MPa steel (Grade 60) is used, the above equations simplify as:

$$R_{FF}[kN] = 1.6xC_F x(a_{vn} + a_{vm}) \left[\frac{mm^2}{m}\right] x\, h[m] \tag{3. 30}$$

Given the tensile force at the soil nailhead, T_o, and the facing flexure capacity, the safety factor against facing flexural failure can be defined (Lazarte, 2003).

$$FS_{FF} = \frac{R_{FF}}{T_o} \tag{3. 31}$$

The minimum and maximum reinforcement ratio are defined as (Lazarte, 2003):

$$\rho_{min}[\%] = 20\,\frac{\sqrt{f'_c[MPa]}}{f_y[MPa]} \tag{3. 32}$$

$$\rho_{max}[\%] = 50\,\frac{f'_c[MPa]}{f_y[MPa]}\left(\frac{600}{600+f_y[MPa]}\right) \tag{3. 33}$$

The placed reinforcement should be within $\rho_{min} \leq \rho \leq \rho_{max}$. And the ratio of the reinforcement in the nail head and mid-span zones should be less than 2.5 to ensure the comparable ratio of flexural capacities in these areas.

3.1.7. Punching Shear Capacity

Like for concrete structural slabs subjected to concentrated loads, the nail-head capacity (figure 3.6) must be assessed in consideration of the punching shear capacity, R_{FP}, and can be expressed as (Lazarte, 2003):

$$R_{FP} = C_P V_F \tag{3. 34}$$

Where V_F is the punching shear force acting through the facing section and C_P is a correction factor that accounts for the contribution of the support capacity of the soil.

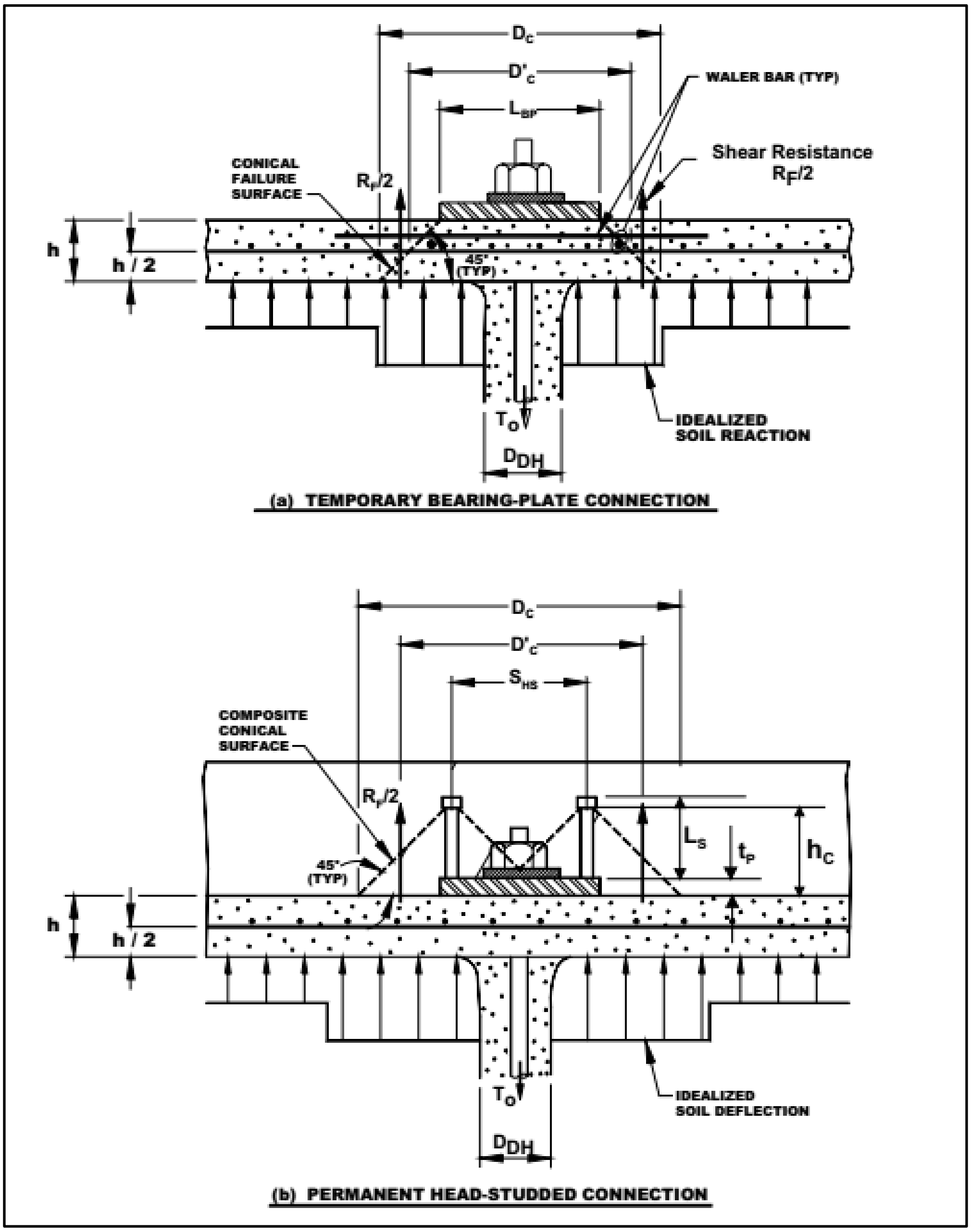


Figure 3.6: Punching Shear Failure Modes (Lazarte, 2003)

The punching shear force can be calculated using standard punching shear equations. These equations consider the size of a conical failure surface (with diameter D'$_C$ at the center of the facing and height h$_C$, as shown in figure 3.6) at the level of the concrete slab as:

$$V_F[kN] = 330\sqrt{f'_c[MPa]}\,\pi D'_c[m]h_c[m] \tag{3. 35}$$

Where: D'$_C$ - effective diameter of conical failure surface at the center of section (i.e., an average cylindrical failure surface is considered); and h$_C$ - an effective depth of conical surface.

Given the tensile force at the soil nailhead, T_o, and the punching shear capacity of the facing, R_{FP}, the safety factor against facing punching shear (FS_{FP}) can be defined as:

$$FS_{FP} = \frac{R_{FP}}{T_o} \tag{3. 36}$$

In general, a minimum factor of safety of 1.35 is adopted for static loads in temporary walls and 1.5 for static loads in permanent walls.

3.2. Deformation Characteristics of Soil Nail Wall

From the two limit state design consideration serviceability is one of the criteria to be applied during the design of soil nail wall. Serviceability depends on the amount of deformation the soil nail wall undergoes. During construction and after construction at any phase, the soil behind the wall has a tendency to deform outwards. Maximum horizontal displacements occur at the top of the wall and decrease progressively toward the toe of the wall. Vertical displacements (i.e., settlements) of the wall at the facing are generally small and are on the same order of magnitude as the horizontal movements at the top of the wall. In general, horizontal and vertical displacements of the facing depend on the following factors: (Lazarte, 2003)

- wall height, H, (deformation increases approximately linearly with height);
- wall geometry (a vertical wall produces more deformation than a battered wall);
- the soil type surrounding the nails (softer soil will allow more deformation);
- nail spacing and excavation lift heights (larger nail spacing and thicker incremental excavation lifts generate more deformation);
- a global factor of safety (smaller FS_G's are associated with larger deformation);
- nail-length-to wall-height ratio (shorter nail lengths in relation to the wall height generates larger horizontal deformation);
- nail inclination (steeper soil nails tend to produce larger horizontal deformation because of less efficient mobilization of tensile loads in the nails); and
- the magnitude of surcharge (permanent surcharge loading on the wall increases deformation)

Empirical data show that for soil nail walls with typical L/H between 0.7 and 1.0, negligible surcharge loading, and typical global factors of safety (FS_G) values of 1.5, the maximum long-term horizontal and vertical wall displacements at the top of the wall, δ_h and δ_v, respectively, can be estimated as follows (Lazarte, 2003):

$$\delta_v = \left(\frac{\delta_h}{H}\right)_i x\ H \tag{3. 37}$$

Where: $(\delta_h/H)_i$ is a ratio dependent on the soil conditions “i” indicated in Table 3.2; and H - wall height.

Table 3.2: Values of $(\Delta_h/H)_I$ and C As Functions of Soil Conditions (Lazarte, 2003)

Variable	Weathered Rock and Stiff Soil	Sandy Soil	Fine-Grained Soil
δ_h/H and δ_v/H	1/1,000	1/500	1/333
C	1.25	0.8	0.7

The size of the zone of influence as shown in Figure 3.7, where noticeable ground deformation may take place, is defined by a horizontal distance behind the soil nail wall (D_{DEF}) and can be estimated with the following expression:

$$\frac{D_{DEF}}{H} = C(1 - tan\,\alpha) \tag{3. 38}$$

Where α: is the wall batter angle; and C coefficient indicated in Figure 3.7.

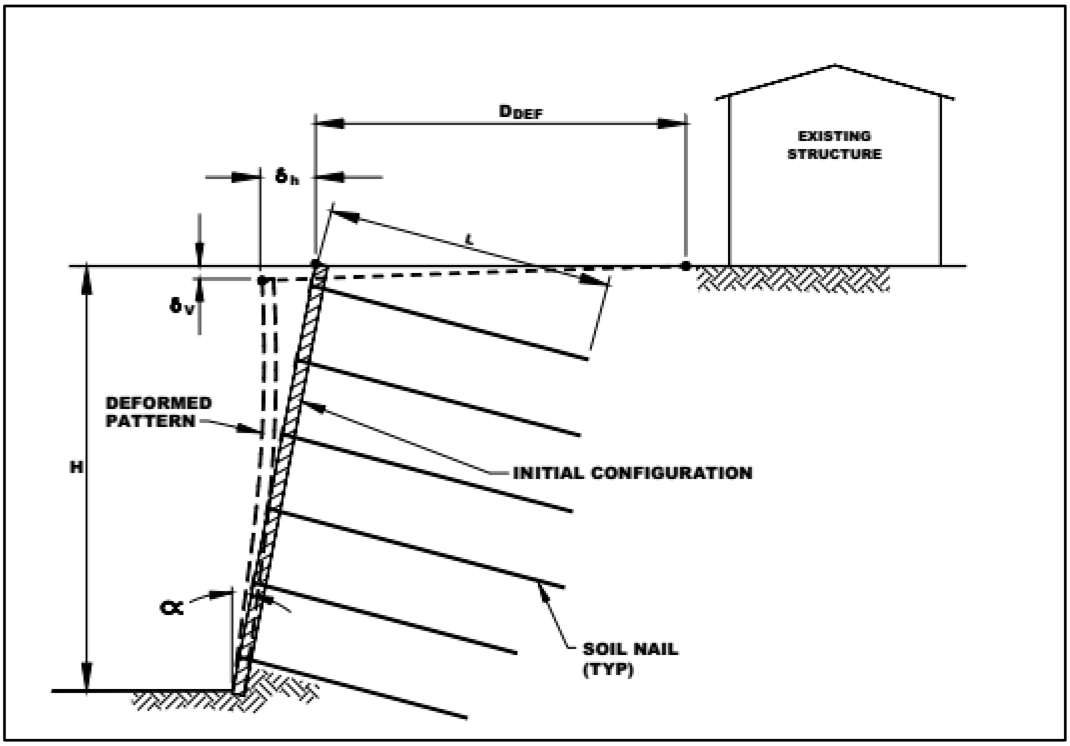


Figure 3.7: Deformation Behavior of Soil Nail Wall (Lazarte, 2003)

3.3. Initial Design Parameters and Conditions of Soil Nailing

The following initial parameters should be first determined and used in the stability design of soil nail walls. (Xiao, 2015), (Lazarte, 2003)

- Wall layout: The layout refers to the wall height (H), the length of the wall, and the inclination of the wall face (the typical range is from 0° to 10°).
- Soil nail vertical (SV) and horizontal (SH) spacing: SV is typically the same as SH. The nail spacing ranges from 1.25 to 2m (4–6.5 ft) for conventional drilled and grouted soil nails with a preferred routine value of 1.5m (5 ft). A reduced spacing for driven nails (as low as 0.5m or 1.5 ft) is required because driven nails develop bond strengths that are lower than those for drilled and grouted nails.
- Soil nail patterns on face: The patterns are square, staggered in a triangular pattern, and irregular. A square pattern results in a column of aligned soil nails and enables continuous and easy installation of geo-composite drain strips behind the facing. In practice, a square pattern is commonly adopted. A staggered pattern results in a uniform distribution of earth pressure in the soil nails, but its main disadvantage is the complicated installation of geo-composite drain strips behind the facing. Irregular nail spacing is project-specific where reduced spacing is needed.
- Soil nail inclination: The inclination ranges from 10 to 20 degrees with a typical inclination of 15 degrees to ensure easy flow of grout from the bottom of the hole to the nail head.
- Soil nail length distributions: They are (1) uniform length when the potential for excessive wall deformation is not a concern. it is beneficial to select uniform length distribution because it simplifies the construction and quality control; (2) variable length, when the wall deformation needs to be controlled; field data indicate that wall displacements can be significantly reduced if the nail lengths in the upper two-thirds to three quarters of the wall height are greater than those in the lower portion. In general practice, nail length in the lower rows should never be shorter than 0.5H, where H is the wall height.
- Soil nail materials: Appropriate grade of steel for the soil nail bars should be selected; for most applications, Grade 420 MPa (Grade 60) steel is used.
- Soil properties: The soil properties and strata are determined based on subsoil exploration.
- Drilling methods, Factor of safety & Loads should be considered

3.4. Recommended Factors of Safety for the Allowable Stress Design (ASD) Method

According to (Byrne R.J., 1998; Lazarte, 2003) the recommended minimum safety factor to be used in the design of soil nail walls, using the ASD method is listed in Table 3.3.

Table 3.3: Minimum Recommended Factors of Safety for the Design of Soil Nail Walls using the ASD method.

Failure Mode	Resisting Component	Symbol	Minimum Recommended Factors of Safety		
			Static Loads(1)		Seismic Loads(2) (Temporary and Permanent Structures)
			Temporary Structure	Permanent Structure	
External Stability	Global Stability (long-term)	FS_G	1.35	1.5$^{(1)}$	1.1
	Global Stability (excavation)	FS_G	1.2-1.3$^{(2)}$		NA
	Sliding	FS_{SL}	1.3	1.5	1.1
	Bearing Capacity	FS_H	2.5$^{(3)}$	3.0$^{(3)}$	2.3$^{(3)}$
Internal Stability	Pullout Resistance	FS_P	2.0		1.5
	Nail Bar Tensile Strength	FS_T	1.8		1.35
Facing Strength	Facing Flexure	FS_{FF}	1.35		1.1
	Facing Punching Shear	FS_{FP}	1.35		1.1
	H.-Stud Tensile (A307 Bolt)	FS_{HT}	1.8		1.5
	H.-Stud Tensile (A325 Bolt)	FS_{HT}	1.5		1.3

Notes: (1) For non-critical, permanent structures, some agencies may accept a design for static loads and long-term conditions with $FS_G = 1.35$ when less uncertainty exists due to sufficient geotechnical information and successful local experience on soil nailing.

(2) The second set of safety factors for global stability corresponds to the case of temporary excavation lifts that are unsupported for up to 48 hours before nails are installed. The larger value may be applied to structures that are more critical or when more uncertainty exists regarding soil conditions.

(3) The safety factors for bearing capacity are applicable when using standard bearing-capacity equations. When using stability analysis programs to evaluate these failures modes, the factors of safety for global stability apply.

3.5. Soil Nail Wall Analysis Example Using Limit Equilibrium Method

3.5.1. Hand Calculation Using Simple Wedge Method

For the purpose of illustration and parametric study of soil nail wall, the typical values of sand soil properties are taken from Ph.D. thesis (Alhabshi, 2006) with an average cohesion of 7.2 kN/m², friction Angle (φ') 38^0 and unit weight (γ) of 20.8 kN/m^3.

Wall geometries:

The vertical height of the wall: H = 8 m

Backslope angle: β = 0.0

Nailing type: grouted type

Soil nail spacing: Sh = Sv = 1 m

Soil nail inclination: $\lambda = 15^0$

Surcharge: Q = 0

Soil nail material: Grade Fe 415; fy = 415 MPa

The failure surface is assumed to be inclined with an angle equal θ=45+φ/2 from the horizontal.

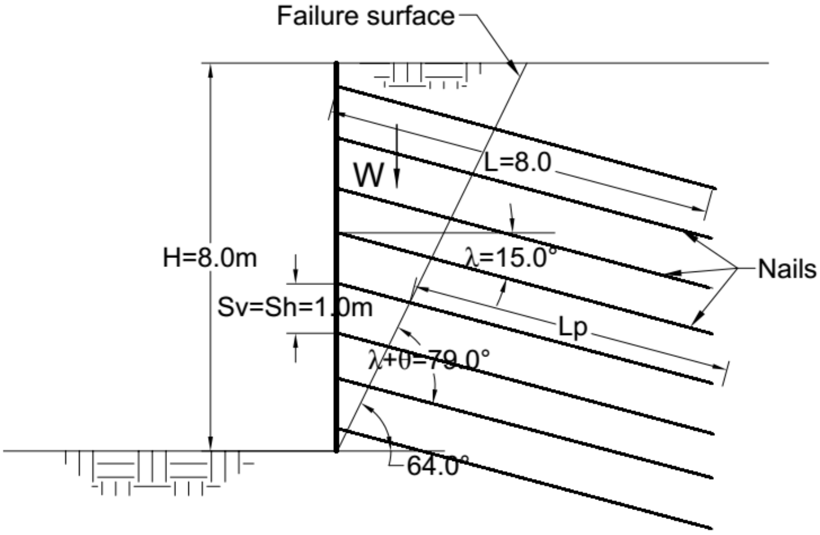


Figure 3.8: The Geometry of Soil Nail Walls for Limit Equilibrium Analysis

i. The global stability of soil nail wall

$$FS_G = \frac{\sum \mathrm{R}}{\sum \mathrm{D}} = \frac{cL + [(W+Q)cos\theta + T_{eq}\sin(\theta+\lambda)]tan\varphi + T_{eq}\cos(\theta+\lambda)}{(W+Q)sin\theta},$$

Where θ=45+φ/2= 64^0; $\theta + \lambda = 79^0$; slope angel (α) =0^0

Length of failure plane, $L_f = H.\frac{cos\alpha}{\sin(\theta-\alpha)} = 8\,x\,\frac{\cos 0}{\sin(64-0)}$ = 8.9m

Weight of active wedge, $W = \frac{1}{2}\gamma H^2 \frac{cos\theta.cos\alpha}{\sin(\theta-\alpha)} = \frac{1}{2}x20.8x8^2 \frac{cos64.cos0}{\sin(64-0)}$ = 324.63 kN/m

Nail pullout length, $L_p = L - \left[\frac{(H-z)cos\theta}{\sin(\lambda+\theta)}\right] = 8 - \left[\frac{(8-z)cos64}{\sin(79)}\right]$

The tensile capacity of the nail bar is

$R_T = A_t f_y = A = \frac{\pi d^2}{4} fy = \frac{\pi 0.0254^2}{4} 415x1000 = 210.28$ kN

$T_{eq} = \sum_{j=1}^{n} Tj$, the equivalent axial force is expressed as:

$T_{eq} = \frac{1}{S_h}\sum_{j=1}^{n} Tj,$

The bond strength is taken from the prismatic value of Table 2.1.

qu= 100kN/m^2

$R_p = \pi q_u D_{DH} L_p$= πx100x0.15xL_p=47.12L_p (kN)

Table 3.4: The Allowable Axial Force Carrying Capacity of Nail

Nail No. (from top)	Depth of nail z [m]	Nail inclination from failure surface (θ+λ)	Effective pullout length Lp [m]	Nail pullout capacity RP[kN]	Nail tensile capacity (kN/m2)	Allowable axial force carrying capacity of nail Tall [kN]
1	0.5	79	4.65	218.58	210.28	210.28
2	1.5	79	5.10	239.57	210.28	210.28
3	2.5	79	5.54	260.56	210.28	210.28
4	3.5	79	5.99	281.55	210.28	210.28
5	4.5	79	6.44	302.54	210.28	210.28
6	5.5	79	6.88	323.53	210.28	210.28
7	6.5	79	7.33	344.52	210.28	210.28
8	7.5	79	7.78	365.51	210.28	210.28
						1682.27

Therefore; $T_{eq} = \frac{1}{1} x1682.27$= 1682.27kN/m,
Substituting the above values on the global stability equation;

$$FS_G = \frac{\sum \mathrm{R}}{\sum \mathrm{D}} = \frac{cL+[(W+Q)cos\theta+T_{eq}\sin(\theta+\lambda)]tan\varphi+T_{eq}\cos(\theta+\lambda)}{(W+Q)sin\theta},$$

$$FS_G = \frac{\sum \mathrm{R}}{\sum \mathrm{D}} = \frac{7.2x8.9+[324.63\ +0)cos64+1682.27x\sin(79)]tan38+1682.27\cos(79)}{(324.63+0)sin64},$$

FS_G = 6.12

Based on the above equations the global factor of safety for different failure inclination is listed in Table 3.5.

Table 3.5: Global Factor of Safety for Different Failure Inclination

Failure inclination from bottom	10	15	20	30	40	50	60	**64**	70	80
Factor of safety	5.92	4.49	3.92	3.7	4.18	4.87	5.72	**6.12**	6.95	10.7
No. of contributed nail	The bottom 3	The bottom 4	The bottom 5	The bottom 7	All	All	All	**All**	All	All

ii. Sliding stability

$FS_{SL} = \frac{\sum R}{\sum D}$;

Where;

$$\sum R = cb\ B_L\ +\ [W + Q_D + PA\ sin\alpha]\ tan\ \varphi_b$$

B_L=8xcos 15= 7.73 m
W= γxHxB_L =20.8x8x7.73= 1285.84 kN/m

$\sum D = PAcos\alpha$

$K_a = \frac{1-sin\varphi}{1+sin\varphi}$ =0.238

P_A= $0.5\gamma H^2 K_a$=0.5x20.8x8^2x0.238=158.413kN/m

$FS_{SL} = \frac{cb\ BL\ +\ [W+Q_D+PA\ sin\alpha]\ tan\ \phi_b}{PAcos\alpha}$,

$FS_{SL} = \frac{\left(\frac{2}{3}\right)x7.2x\ 7.73\ +\ [1285.84+0+158.413x\ sin0]\ tan\ (\frac{2}{3}x38)}{158.413xcos0}$,

FS_{SL}= 4.08

Similarly, for other lengths to wall height ratio and nail inclination, the result is as shown in Table 3.6.

Table 3.6: Sliding Safety Factor for Different Nail Inclination

Nail inclination (^{0})	0	5	10	15	20
L/H	FS_{SL}				
0.5	2.11	2.10	2.08	2.04	1.98
0.7	2.96	2.94	2.91	2.86	2.78
0.9	3.8	3.78	3.74	3.67	3.57
1.1	4.64	4.63	4.57	4.49	4.36

ii. The factor of safety of nail pull out and nail tensile strength

The maximum axial force at each nail level is given by

$T_z[kN] = Ka(Q + \gamma z)s_h s_v$

$T_z[kN] = 0.238(0 + 20.8xz)$x1x1, shown in table 3.7

Nail pull out safety factor for each nail = $FS_p\ = \left(\frac{R_p}{T}\right)_z$

Nail pullout capacity from table 3.4, 365.51 kN

$FS_p\ = \frac{365.51}{37.11}$ = 9.85

Nail tensile strength factor of safety for each nail:

$FS_p\ = \left(\frac{R_T}{T}\right)_z$

Nail tensile strength = $R_T\ = A_t f_y$ = $A = \frac{\pi d^2}{4} fy$ = $\frac{\pi 0.0254^2}{4} 415x1000 = 210.28$ kN

$FS_T\ = \frac{210.28}{37.11}$ = 5.67

Similarly, for other nails, the factor of safety against nail pullout failure and nail tensile strength shown in Table 3.7.

Table 3.7: Nail Tensile and Pullout Factor of Safety for all nails

Nail No. (from top)	Depth of nail z (m)	Nail pullout capacity RP (kN)	Maximum axial force Tz (kN)	The factor of safety against pullout failure FSp	The factor of safety against nail tensile strength failure FS_T
1	0.5	218.58	2.47	very large	very large
2	1.5	239.57	7.42	very large	very large
3	2.5	260.56	12.37	very large	very large
4	3.5	281.55	17.32	very large	12.14
5	4.5	302.54	22.27	13.59	9.44
6	5.5	323.53	27.21	11.89	7.73
7	6.5	344.52	32.16	10.71	6.54
8	7.5	365.51	37.11	9.85	5.67

The maximum axial force developed in the wall,

$T_z[kN] = Ka(Q + \gamma z)s_h s_v$

$T_z[kN] = T_{max-s}[kN] = 0.238(0 + 20.8x8)$x1x1 = 39.58 kN

The maximum tensile force developed on the face of the wall will be:

$T_o = T_{max-s}\,[0.6 + 0.2(S_{max}[m] - 1)]\,,\, S_{max} = 1\ m$

$T_o = 39.58[0.6 + 0.2(1 - 1)]$ = 23.75 kN

Use welded wire mesh size of WWM 102 x 102 – MW19 x MW19 and waler bar of 2x10mm diameter, (f_y= 415 MPa, A_{vw}=A_{hw}=2x78=156 mm^2) in both directions

Area of Reinforcement in vertical a_{vm} and horizontal a_{hm} directions in mid-span: a_{vm} = a_{hm} = 184.2 mm2/m for WWM 102 x 102 – MW19 x MW19 (Lazarte, 2003)

Area of Reinforcement in vertical a_{vn} and horizontal a_{hn} directions around soil nail head: use the same amount of reinforcement in both directions:

$a_{vn} = a_{hn} = a_{vm} + \frac{A_{vw}}{\text{Sh}}$ = 184.2 + $\frac{156}{1}$ = 340.2

$\rho_{min}[\%] = 20\frac{\sqrt{f'_c[MPa]}}{f_y[MPa]}$ = $20\frac{\sqrt{20}}{415}$ = 0.216

$\rho_{max}[\%] = 50\frac{f'_c[MPa]}{f_y[MPa]}\left(\frac{600}{600+f_y[MPa]}\right)$ = $0.5\frac{20}{415}\left(\frac{600}{600+415]}\right)$ = 1.42

Reinforcement ratio at nail head and mid-span in the vertical direction

$\rho_n[\%] = \frac{\text{an}}{0.5\text{h}}x100 = \frac{340.2/1000}{0.5\text{x}150}x100$ = 0.45

$\rho_n[\%] = \frac{am}{0.5\text{h}}x100 = \frac{184.2/1000}{0.5\text{x}150}x100$ = 0.25

ρ_n and ρ_m are within the allowable value (0.21 and 1.42)

The facing flexural resistance is given by,

$R_{FF}[kN] = \frac{C_F}{265} x(a_{vn} + a_{vm}) \left[\frac{mm^2}{m}\right] x \left(\frac{S_h h[m]}{S_v}\right) x f_y [MPa]$, C_F=1.5 for 150 mm shotcrete thickness and $a_{vn} + a_{vm}$ = 184.2 +340.2 = 524.4

$R_{FF}[kN] = \frac{1.5}{265} x(524.4) x \left(\frac{1x0.15}{1}\right) x415$ = 184.78

$FS_{FF} = \frac{R_{FF}}{T_0}$ = $\frac{184.78}{23.75}$ = 7.78 >1.5 safe

3.5.2. Soil Nail Wall Analysis Using Slope/W

The design of soil nail wall has traditionally been conducted using limit equilibrium method. This method ensures the static equilibrium with the global factor of safety. Slope/W is based on limit equilibrium concept and soil nails can be modeled using this tool, by considering the pull out resistance between the nail and soil. The Input parameters for slope/W tools are listed in Table 3.8.

Table 3.8: Input Parameters for Slope/W Software

Nail length (m)	8
Inclination (degree)	15
Pullout resistance (kPa)	100
Bond diameter (m)	0.15
Bond safety factor	2
Nail spacing (m)	1
Tensile capacity (kN)	210
Tensile bar safety factor	1.8

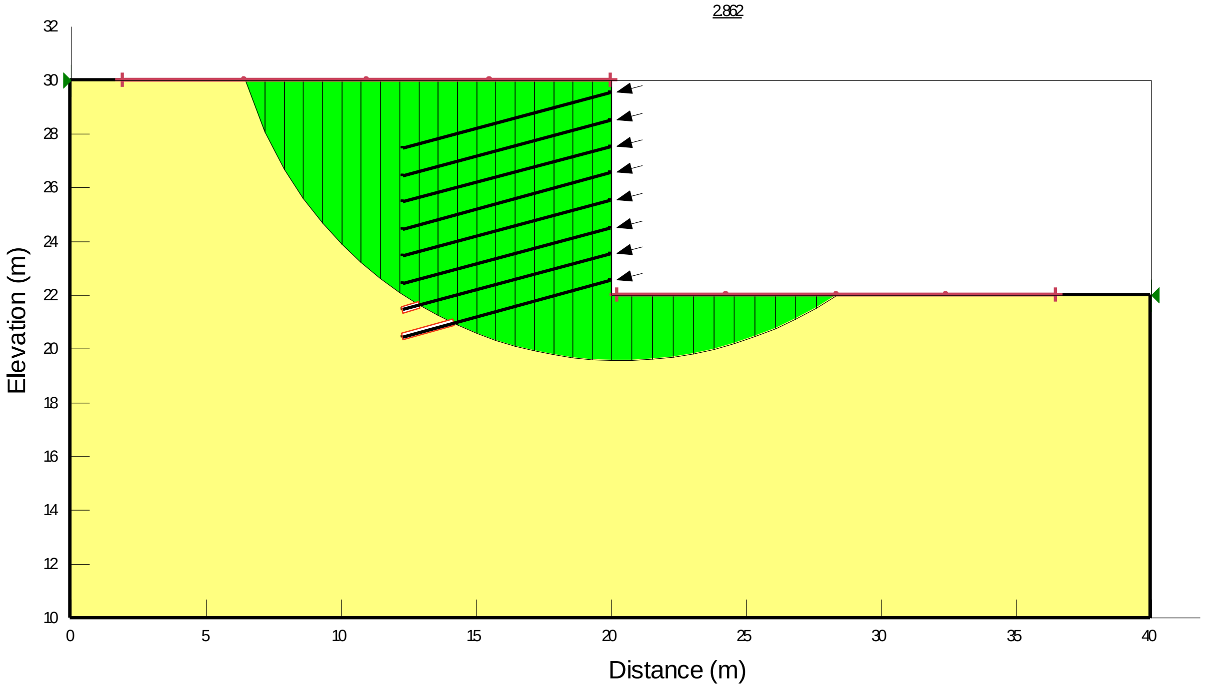


Figure 3.9: Failure Surface of Soil Nail Wall Using Slope/W

From Figure 3.9, using Morgenstern–Price method the factor of safety obtained is 2.86. Here contributing nails are the bottom two only.

4. FINITE ELEMENT ANALYSIS OF SOIL NAIL WALL AS A DEEP EXCAVATION SUPPORT

4.1. Introduction

Limit equilibrium has its own drawback because of a lack of deformation estimation and not considering the deformation needed to mobilize resisting forces in the ground. It is found that finite difference and finite element software are powerful tools for investigating the behavior of a wall stabilized by soil nail. Therefore finite element and finite difference methods are preferable than the limit equilibrium (Fernández, 2014). From the two numerical modelings, FEM (PLAXIS) results are in good agreement with the field measurements. Even though the pattern of deformations is same, the FDM (FLAC) gives relatively high deformation value. However, a good agreement is observed between the values of the factor of safety, which are calculated for different stages of excavation (Ahmad S., 2013). Therefore PLAXIS 2D, based on the finite element method, is used in this study to simulate soil nail wall due to its time-efficient and comparatively easy user-friendly environment. However, the accuracy of the analysis and result is highly dependent on the user's understanding of the problem and computational tool.

4.2. Soil-Structure Interaction

Structures in civil engineering involve more than one material contact. Each material owns specific behavioral characteristics; however, after in contact, the behavior of the composite system is influenced by the response of each material as well as the interaction or coupling between the materials. That is why the need for soil structure interaction arises.

It is possible to develop analytical solutions for engineering problems based on the equations of equilibrium and compatibility; however, it is necessary to make certain simplifying assumptions regarding material behavior, geometry, and boundary conditions. Numerical procedures such as Finite difference, Finite element, Boundary element, and energy methods can be developed for the solution of such problems by reducing the simplifying assumptions. It is believed that the Finite element method possesses certain advantages and generality compared to other methods. (Zaman, 2014)

The stability of many earth structures depends not only on the properties of the structure and surrounding soil but also on the adhesion and friction at the soil-structure interface, while interface friction itself depends on the pressures between the structure and soil. (Michael Carter, 2016)

From the numerical predictions, the overall response of the nailed structure is more sensitive to the soil, nail and facing interface characteristics, but less to the end conditions of the nails. This is due to the maximum nail force mobilized at the middle portion of the nail, with very little relative movement and, hence nail forces, being mobilized at the two ends. (Y.D. Zhou, 2009)

4.2.1. Interfaces and Interface Elements

Interfaces are composed of interface elements. Interface elements are imaginary elements used to connect the structural element and the soil, to evaluate the friction between them. In PLAXIS when 15-node soil element is used, the corresponding interface elements are defined by 5 pairs of nodes and when 6-node soil element is used, the corresponding interface elements are defined by 3 pairs of nodes. Even though the interface element shown in Figure 4.1 has a finite thickness in the drawing, in the finite element formulation the coordinates of each node pair are identical, which tells us that the element has a zero thickness.

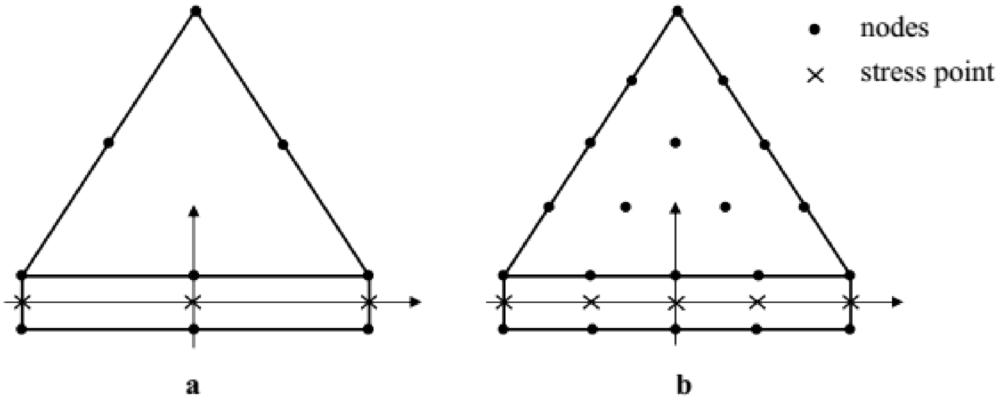


Figure 4.1: Distribution of Nodes and Stress Points in Interface Elements and Their Connection to Soil Elements. (Brinkgreve R.B.J., 2006)

Each interface has is an imaginary dimension called “virtual thickness”, which is used to define the material properties of the interface. The higher the virtual thickness is, the higher the elastic deformation is generated. The virtual thickness is calculated as the virtual thickness factor times the average element size. The element size is determined by the global coarseness of the mesh and the default value of the virtual thickness factor is 0.1.

The interface properties are calculated from the soil properties in the associated data set and the strength reduction factor by applying the following rules: (Brinkgreve R.B.J., 2006)

$$C_{inter} = R_{inter} C_{soil} \tag{4. 1}$$

$$tan(\varphi)_{inter} = R_{inter}\, tan(\varphi)_{soil} \tag{4. 2}$$

Where c and φ are the cohesion and angle of friction respectively, which is used in the material model of the interface material and R_{inter} is the strength reduction factor of the interface element.

4.2.2. Strength Reduction Factor (R_{int})

The strength reduction factor relates the interface strength i.e. wall friction and adhesion to the soil strength of friction angle and cohesion.

Field pullout test studies show that the strength reduction factor, the interface (R_{int}) between the nail and soil is more than unity. According to (Richwien, 2002), to determine the strength reduction factor, there are two types of tests direct shear test and pullout test. The coefficient of friction obtained by the direct shear test is less than obtained in the pullout test. The strength reduction factor from the pullout test, for dense sand, is about 2.01-3 and for medium dense sand 1.55-2.08.

According to (Cheng-Yu Hong, 2013), the apparent coefficient of friction (strength reduction factor for the interface) is mainly dependant on the grouting pressure and the overburden pressure. With constant grouting pressure, the apparent coefficient of friction decreases when the overburden pressure increases and with constant overburden pressure, it increases linearly when the grouting pressure increases. According to their study, the apparent coefficient of friction ranges from 1.92 to 2.83, with constant overburden pressure.

Brinkgreve also suggest the following interaction factors for different materials

Table 4.1: Suggested Interaction Factors, R_{inter} (Brinkgreve R.B.J., 2010)

Interaction between	**R_{inter}**
Sand – Steel	0.6 – 0.7
Clay – Steel	0.5
Sand – Concrete	0.8 – 1.0
Clay – Concrete	0.7 – 1.0
soil/geogrid (grouted body) (interface is not necessary)	1.0

The higher the interaction factor, the higher friction between the structural element and the soil, which in turn increases stability. Similarly, the lower the interface value, Rinter, the larger the bending moment is produced. Therefore, it is important to estimate a reasonably "correct" value for friction reduction factors, R_{inter}. (Tjie-Liong, 2014)

Therefore from the above results, the use of interface elements can be used the default setting of “Rigid Interface” in the material menu for simulation of the soil and nail interface (no need to use interface element). Using a rigid interface means, the interface does not influence the strength of the surrounding soil. However, in the absence of data from the experiment, PLAXIS 2D recommends using an interface factor of 2/3(0.67) in general. A value of R_{int} greater than 1 is not normally used.

Since the grouted type is used in this study, the soil around the grouted body is completely in contact with the grouted body and interaction factor, R_{inter} value of 1 is used (Note: grouted type with pressure should be used instead of gravity grouting to increase the interaction factor)

4.3. Constitutive Models

The constitutive models used in PLAXIS are linear elastic, Mohor coulomb, soft soil model, hardening soil model, soft-soil creep model and jointed rock model. Among these models, Mohor coulomb is appropriate for most soils (Babu, 2010).

4.3.1. Mohr-Coulomb Model (MC)

This model is an elastic perfectly plastic model, which means that the behavior of the soil is linear elastic up to a certain stress limit, after which the soil is perfectly plastic meaning that the strains are irreversible after a stress decrease.

This model requires five input parameters for the soil: Young’s modulus (E), Poisson’s ratio (v), Friction angle (φ), Cohesion (c), Dilatancy angle (ψ)

The values of the stiffness parameter (E), and the cohesion (c) are in general chosen as a representative value that is consistent with the stress level in the soil. In addition, one should take into account the increase in greater depth. It is, however, also possible to model both the stiffness and the cohesion with a linear increase with depth. (Brinkgreve R.B.J., 2006)

4.4. Properties of Soil and Nail Used for PLAXIS 2D

For the purpose of illustration and parametric study of soil nail wall, the values of sand soil property are taken from a Ph.D. thesis (Alhabshi, 2006). The material parameters are listed in Table 4.2.

Table 4.2: Soil Model Parameters and its Value (Alhabshi, 2006)

	Symbol	**Units**	**Soil1**	**Soil2**	**Soil3**	**Soil4**
Unsaturated unit weight	γ_{unsat}	kN/m^3	20.8	20.8	20.8	20.8
saturated unit weight	γ_{sat}	kN/m^3	20.8	20.8	20.8	20.8
Horizontal coefficient of permeability	K_x	m/day	0.305	0.305	0.305	0.305
Vertical coefficient of permeability	K_y	m/day	0.305	0.305	0.305	0.305
Elasticity modulus	E_{ref}	kN/m^2	24000	40700	71800	119000
Poisons ratio	ν	-	0.359	0.347	0.305	0.25
Cohesion	c_{ref}	kN/m^2	7.2	7.2	7.2	7.2
Internal friction angle	φ	^{0}	38	38	38	38
Delatency angle	ψ	-	0	0	0	0
Incremental modulus	E_{inc}	kN/m^2	2400	4070	7180	12000
Reference depth	y_{ref}	m	40	38	36	34
Incremental cohesion	$c_{increment}$	kN/m^2	0	0	0	0

4.4.1. Equivalent Nail Parameters

Soil nail walls are modeled as a plane strain problem in PLAXIS 2D. The plane strain condition of soil nail wall simulation can be modeled as a plate or geo-grid structure. Simulation using plate element accounts both bending stiffness and shear forces, therefore, bending stiffness (EI) and axial stiffness (EA) are the most vital input parameters whereas the geogrid element accounts only axial stiffness (EA). The real soil nail is circular in cross-section but the plate and geogrid elements are rectangular with a thickness d and width of 1m in the out of the plane direction. As a result, it requires determining the equivalent axial and bending stiffness for the real circular cross-section of the soil nail. For calculating the axial and bending stiffness of the grouted nail, the contribution of the mortar was ignored because of a tendency to crack during slight deformation and load mobilization (Chia-Cheng Fan, 2008). As shown in Figure 4.3, the diameter of the nail is 25.4 mm with a modulus of elasticity $2\text{x}10^8$ kN/m^2. The thickness of shotcrete is 0.15 m with an elasticity modulus of $2.1\text{x}10^7$ kN/m^2. Then PLAXIS 2D automatically calculates the plate equivalent thickness d_{eq} using Equation 4.3.

$$d_{eq} = \sqrt{12\left(\frac{EI}{EA}\right)} \tag{4. 3}$$

The axial and bending stiffness of nail and facing parameters are listed in Table 4.3.

Table 4.3: Axial and Bending Stiffness of Nail and Facing

Element	**Axial stiffness**	**Bending stiffness**	**Weight**	**Poisons ratio**
Symbol	EA (kN/m)	EI (kNm^2/m)	w (kN/m/m)	ν
Shotcrete	3.15E+06	5.91E+03	1.59	0.18
Soil Nail	1.01E+05	4.09	0.48	0.2

4.4.2. Staged Construction and Element Simulation of Soil Nails

In real construction cases, the construction sequence is undertaken through stages to excavate the first layer and inserting the nails as a second stage. To simulate these conditions PLAXIS 2D has an option called staged construction. Initially, during geometry formulation and input parameters, each excavation stages (lifts) are drawn as an individual cluster. As a result in the first construction stage (ignoring initial conditions), the upper soil cluster is deactivated. In the second stage, the nail and shotcrete elements (modeled as a plate) is activated. This procedure is continued until the bottom of the excavation. Here the connection between two structural objects is vital for the load transferring mechanism. As stated in (Lazarte, 2003) the connection between the nail and wall facing (shotcrete) is rigid and in PLAXIS 2D the connection of two structural plates is rigid by default.

Soil nails are three-dimensional objects but they can be modeled as a two-dimensional object using plane strain conditions. In this study, soil nails are modeled as a plate and geogrid element while the shotcrete facing is modeled as a plate element. Here the effect of modeling the grouted nail as a plate or geogrid elements was studied.

4.4.3. Mesh, Boundary Geometry, and Fixity Conditions

After the geometry model is fully defined including structural elements and clusters, all the material properties are assigned to the clusters and structures (geogrid and plate). Next to assigning, the mesh is generated. For the finite element mesh generation, the 15-node triangular element is used. In PLAXIS 2D the mesh density alternating from very coarse to very fine option. In addition to the global coarseness, it is possible to refine the meshes around the vicinity of structures (nail and facing). In this study, the effect of mesh coarseness on the lateral deflection and factor of safety was studied and explained.

The boundary conditions at the bottom should be placed on hard soil layer. According to (Lim, 1997), if the depth from the bottom of the excavation to a hard soil layer is not exactly known, they suggested that it can be two to three times the vertical excavation height, H. Based on this suggestion the bottom depth has been taken 4 times the excavation height.

From the previous researches for different excavation height, the domain areas are shown in Table 4.4.

Table 4.4 Domain Area for Different Excavation Height

Depth of excavation (m)	Type of cut	Domain area used (m^2)	Model	Source
20.75	Vertical	70 x 90	2D plane strain	(Fernández, 2014)
10	Slope	15 x 25	2D plane strain	(Gupta, 2016)
10	Vertical	25 x 30	2D plane strain	(Babu, 2010)
8	Vertical	40 x 60	2D plane strain	Current study

In addition to the literature, the effect of the domain to lateral displacement is shown in Figure 4.2. The displacement is nearly the same after the selected domain area (2400 m^2) i.e. the displacement decrement is very small. Even though the accuracy of the result increases for a wide domain area, the time required for calculation increases and it needs a high capacity computer.

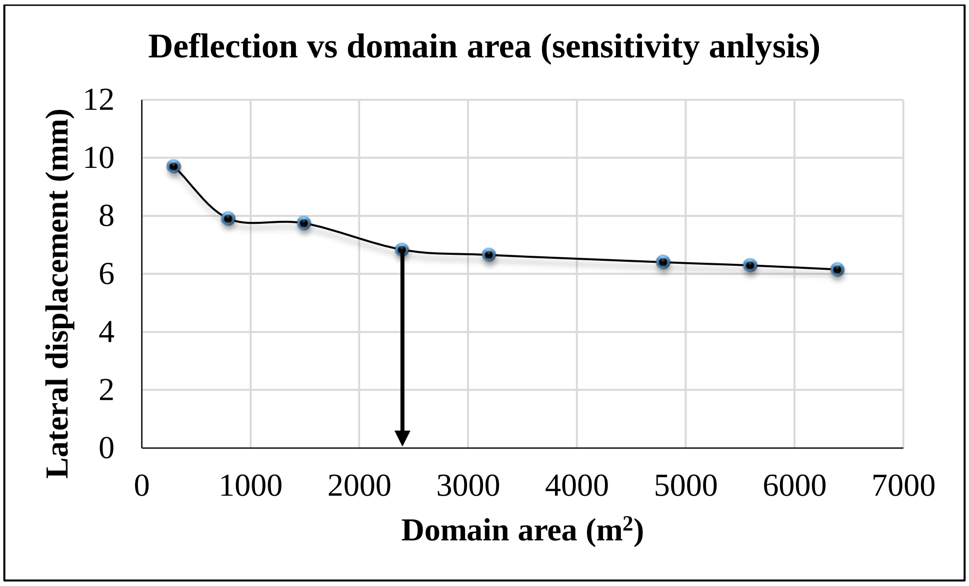


Figure 4.2 Sensitivity Analysis for Domain Selection

The boundary size may affect the result. In this case, plastic points are a good indicator. Plastic points are the stress points where the Mohr stress circle touches the Coulomb failure criterion. If plastic points are found around the boundary line, this indicates that the model is too small. Therefore the model boundary should be enlarged and the calculation should be repeated

(Brinkgreve R.B.J., 2006) and there were no plastic points found in all model around the boundary.

The boundary condition was fixed at the bottom of the horizontal boundary (u_x=0 and u_y=0) and for the vertical boundaries, left and right, only horizontal fixity was applied (u_x=0). PLAXIS 2D gives automatic boundary fixities using standard fixated from the load menu. Based on the sensitivity analysis and the boundary conditions the full model of the soil nail wall is shown in Figure 4.3.

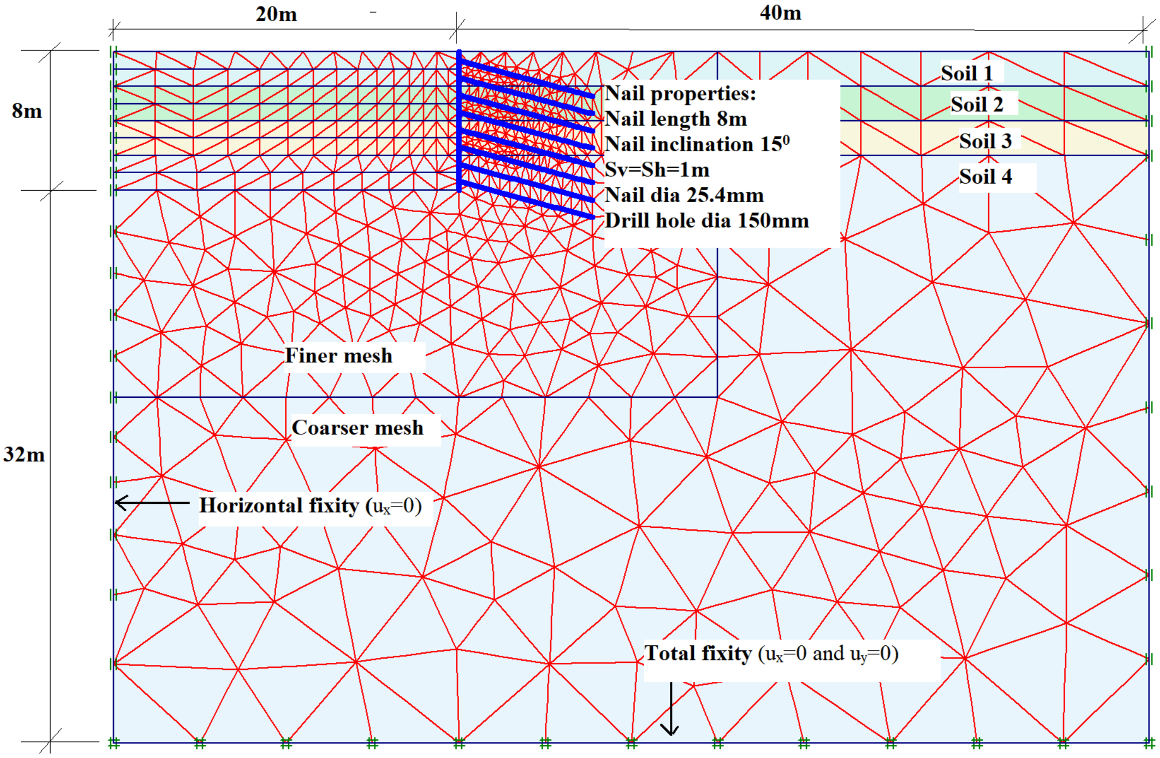


Figure 4.3: Geometry of the Soil Nail Wall

4.4.4. Validation of PLAXIS 2D

Based on the above properties of soil nail wall, the predicted PLAXIS 2D and the measured results are shown in Figure 4.4. It shows that the predicted PLAXIS 2D result is nearly similar to measured, so PLAXIS 2D gives well result similar to the measured data.

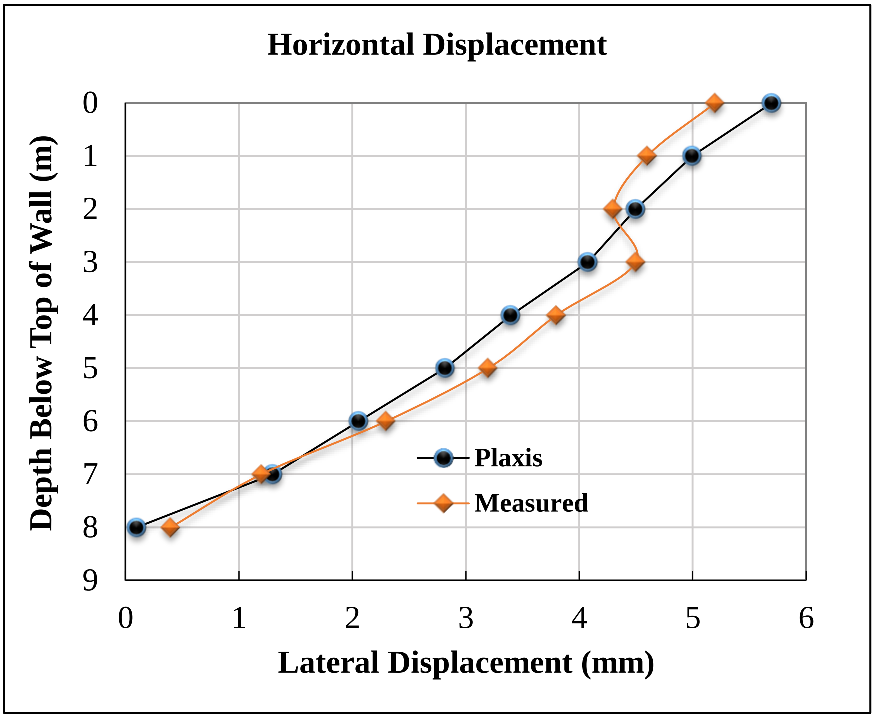


Figure 4.4: Comparison of PLAXIS 2D and Measured Data

4.4.5. Effect of Mesh Properties on Soil Nail Model

PLAXIS 2D has an option to use different mesh density from very coarse mesh to very fine mesh. As the mesh density increases the accuracy of the work increases but it takes time and it needs high-performance computing machine. In this study for the comparison, after the global mesh density is used, the soil cluster around the wall is refined and then nails are refined. For example, if the medium mesh is used globally, the soil cluster around the wall will be fine mesh and the nails again refined. As shown in Table 4.5 and Figure 4.5, the maximum lateral displacement increases from very coarse mesh, 4.6 mm to very fine mesh, 6.3 mm and the global factor of safety decreases from 2.53 to 2.37. Generally, the choice of mesh density depends on the accuracy needed for the work. One can use coarse or medium mesh with refined around the vicinity of the wall.

Table 4.5: Effect of Mesh on Maximum Lateral Displacement and Global Factor of Safety

Mesh type	Number of Element normalized to medium mesh	Maximum lateral displacement (mm)	The global factor of safety
Very coarse	0.4	4.6	2.53
Coarse	0.6	5.1	2.49
Medium mesh	1.0	5.6	2.42
Fine mesh	1.8	6.0	2.38
Very fine	3.1	6.3	2.37

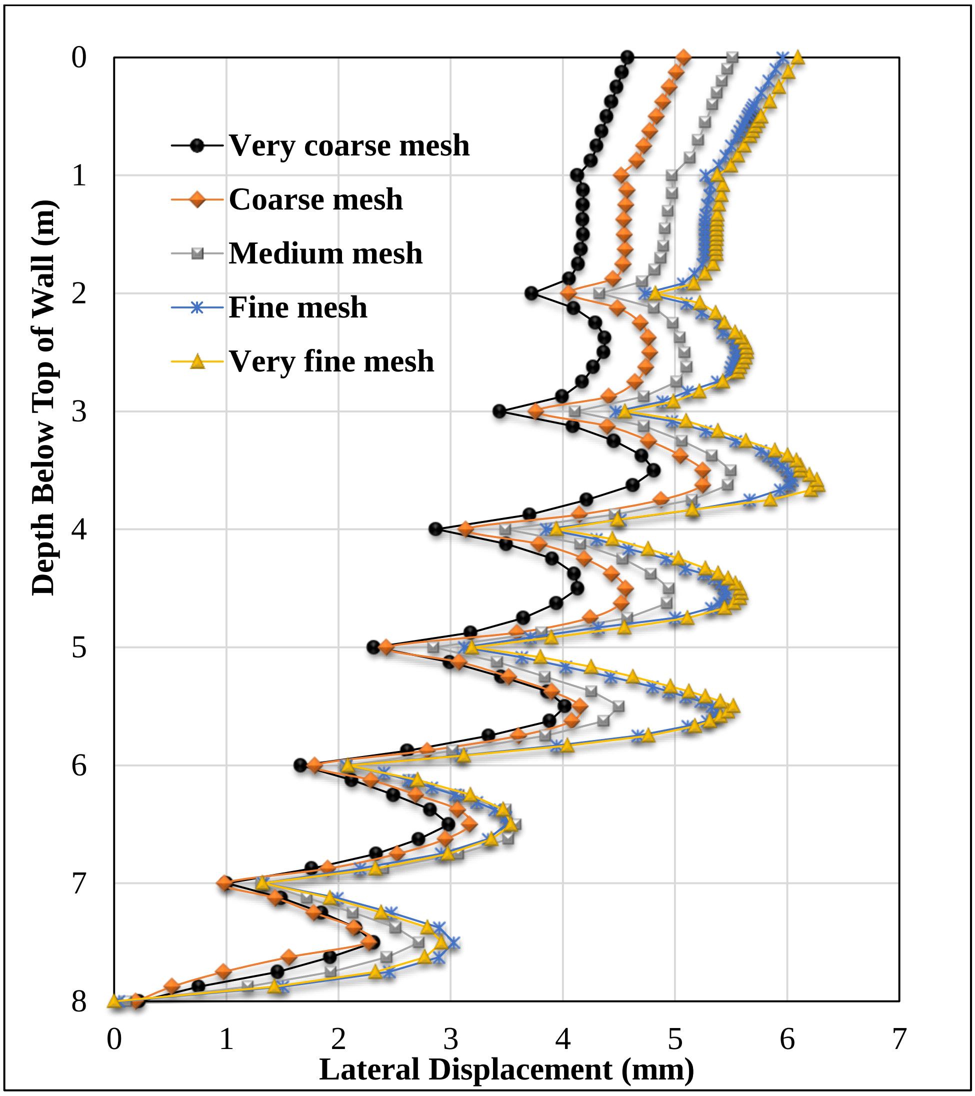


Figure 4.5: Effect of Mesh on Lateral Displacement

4.4.6. Effect of Modeling of Soil Nail Using Plate and Geogrid Elements

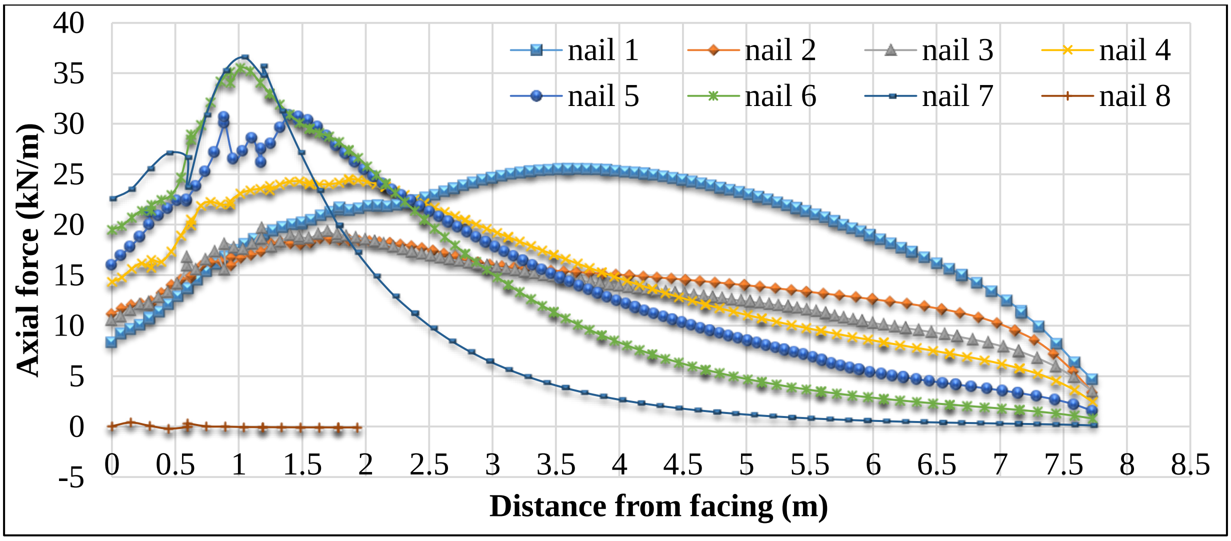


Figure 4.6: Axial Force Distribution of All Nails Using Plate Model

The axial force on first row increases as the construction phase increases (as excavation depth increases). When the first excavation lift is finished and the first nail row is installed, the load from deformation of the upper soil is transferred to the first row through axial forces. In addition, when the second lift excavation is done and the second nail row is installed, the deformation load is transferred to the first and second nail row. Hence, the axial force increases in the first row. In a similar manner for each excavation lift, there is an increment of axial force on the first nail row and this is consistent with (Lazarte, et al., 2015) and shown in Figure 4.7.

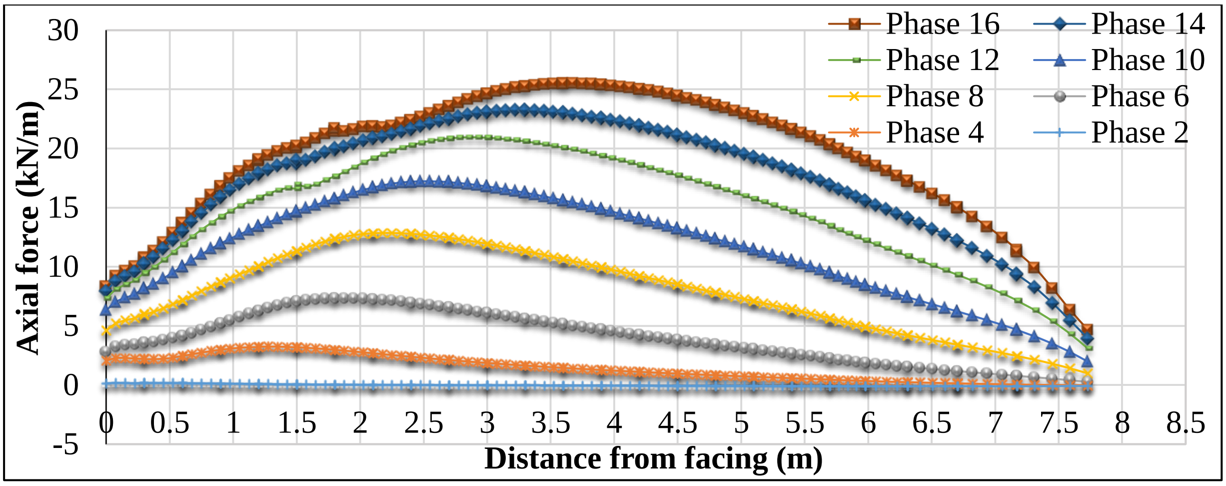


Figure 4.7: Axial Force Distribution of First Nail Row after Facing Installed

It is possible to model soil nail using a plate and geogrid elements. As shown in Figure 4.8, the two right-side curves, the global factor of safety for plate and geogrid elements after the nail and facing installed, at the end of the construction stage, is equal, and there is only 0.5 % difference. This implies that bending stiffness has no contribution to the global factor of safety at the end of the construction stage. However, for the other construction stages, the global factor of safety of the plate is greater than the geogrid element varying from 4.5 % to 34.6 %. This shows that bending stiffness plays an important role during the successive construction stages except for the final stage.

Even if the duration is several hours only, it is required to check the global factor of safety after each excavation lift is done and before the nail and the facing is installed. The global factor of safety of the wall before nail and facing installed is less than after the nail and facing installed. This is shown in Figure 4.8, the left two curves. Similar to the above explanations, bending stiffness has a contribution to the global factor of safety before the facing and nails are installed.

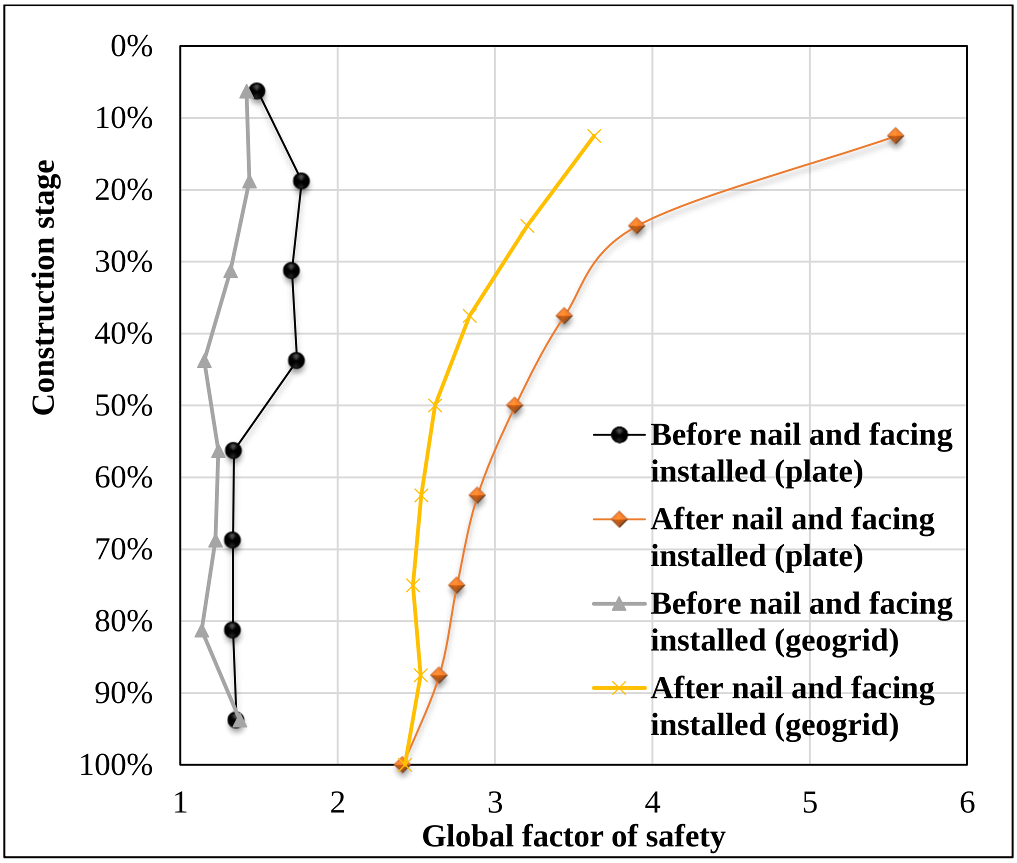


Figure 4.8: Effect of Modeling Using Plate and Geogrid Element on Global Factor of Safety

Similar to the factor of safety, the lateral deflection at the end of the construction stage for plate and geogrid elements is nearly equal and has a 1.6 % difference at the top of the wall. This shows that considering bending stiffness of the nail has no contribution to the lateral displacement of the wall, at the end of the construction stage. Even though the global factor of safety and the lateral deformation of the soil nail wall at the end of the construction stage for plate and geogrid elements are equal, modeling using plate element is reasonable than geogrid elements due to the stability of successive construction stages. The effect of modeling using the plate and geogrid elements on lateral deflection is shown in Figure 4.9

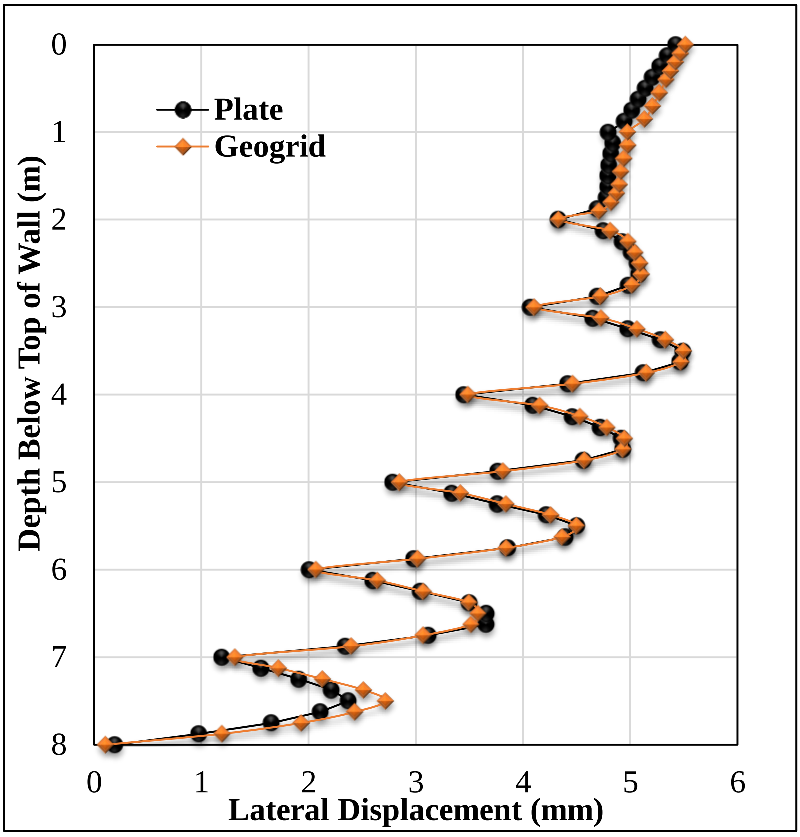


Figure 4.9: Effect of Modeling Using Plate and Geogrid Element on Lateral Displacement

The maximum axial force difference on each nail level is less than 10% which shows that the difference is insignificance for a practical purpose. The maximum axial force at each nail level is shown in Figure 4.10

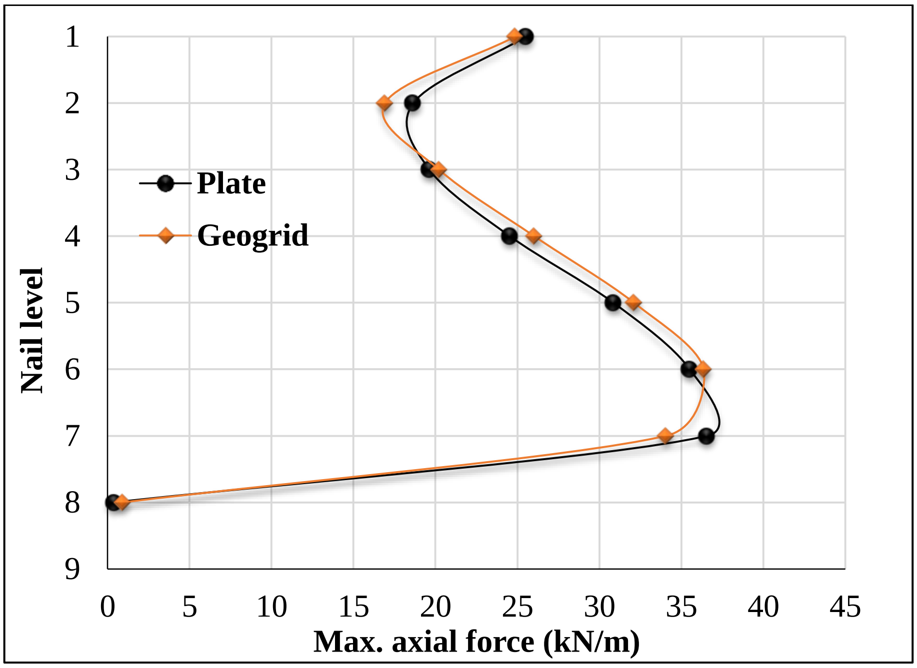


Figure 4.10: Maximum Axial Force on Each Nail

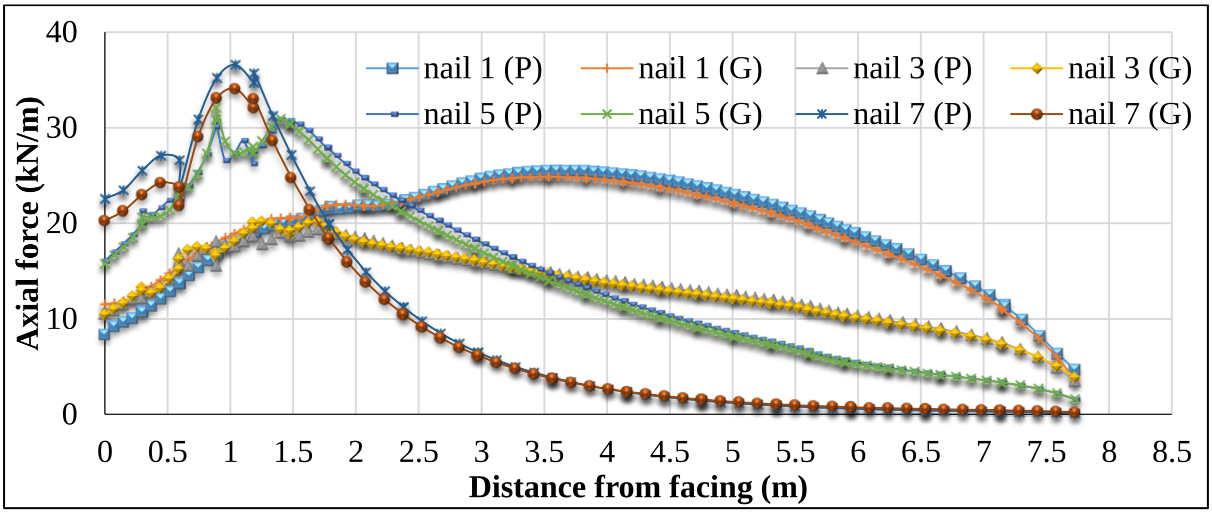


Figure 4.11: Axial Force Comparision of Plate and Geogrid Element on Alternate Nails

4.4.7. Effect of Associated and Non-Associated Flow Rule on Soil Nail

An associated flow is a flow rule when the plastic potential is equal to the yield function. This simplifies the calculation and it is better to simulate pressure no sensitive soils (undrained behavior of fine-grained soils). In frictional soils, the dilation angle for the prediction of irreversible volume change during shear and the friction angle are assumed the same ($\varphi = \psi = 38^0$), leading to the prediction of the excessively high rate of dilation. In the non-associated flow rule, the plastic potential is different from the yield function. It is required for frictional materials to avoid the excessive dilation of associated flow rule. In most cases, a non-associated flow rule is used and the dilation set to zero to obtain the realistic failure loads. (Lees, 2016; Briaud, 2013). In this study in all cases, non-associated flow is used unless and otherwise stated.

As shown in Figure 4.12, the associated flow has a significant effect on each construction stage ranging from 14 % to 27 % reduction to that of the non-associated flow of global factor of safety. However, at the end of the construction stage, the difference is small i.e. 3 %. This shows that considering the associated flow for the analysis has a significant effect on the construction stage but little significant at the end of the construction stage.

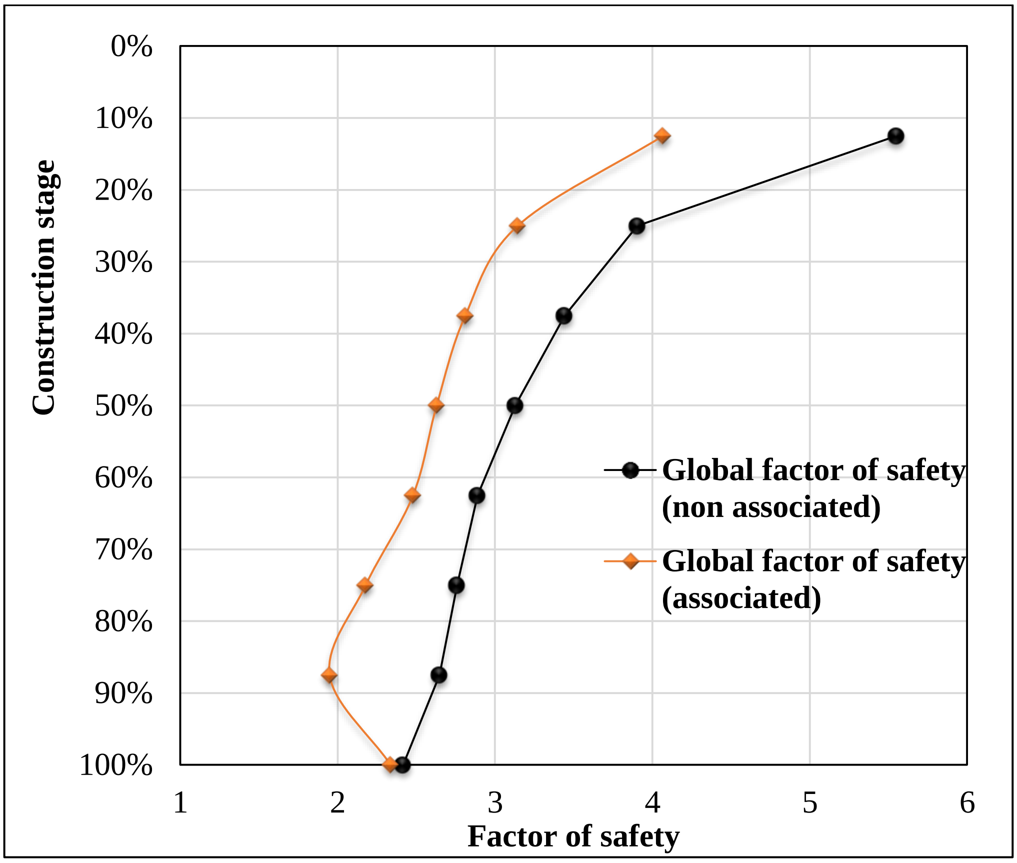


Figure 4.12: Effect of Associated and Non-Associated Properties on Global Factor of Safety

4.4.8. Effect of Lift Excavation on Soil Nail Modeling

When the lift (depth) of excavation increased, the ability of the soil standing without support will be decreased and collapse. In this study, the effect of lift (depth) of excavation has been studied for 0.5 m, 1 m, 1.5 m, and 2 m depth. When the lift excavation was increased to 1.5 m and 2 m depths, the soil collapse. Therefore, if it is intended to use these lift excavations, berms should be used. Lift excavation has no significant effect on the factor of safety i.e. the 0.5 m lift has a reduction of 0 % to 3 % to that of 1 m lift excavation. However, it has a significant effect on the lateral displacement of the wall at the end of the construction stage as shown in Figure 4.14. Using a lift excavation of 0.5 m reduces the lateral displacement by 24 % to that of 1 m lift excavation. Nevertheless, when the lift excavation reduced, the cost and time required for the project will be large compared to larger lift excavations.

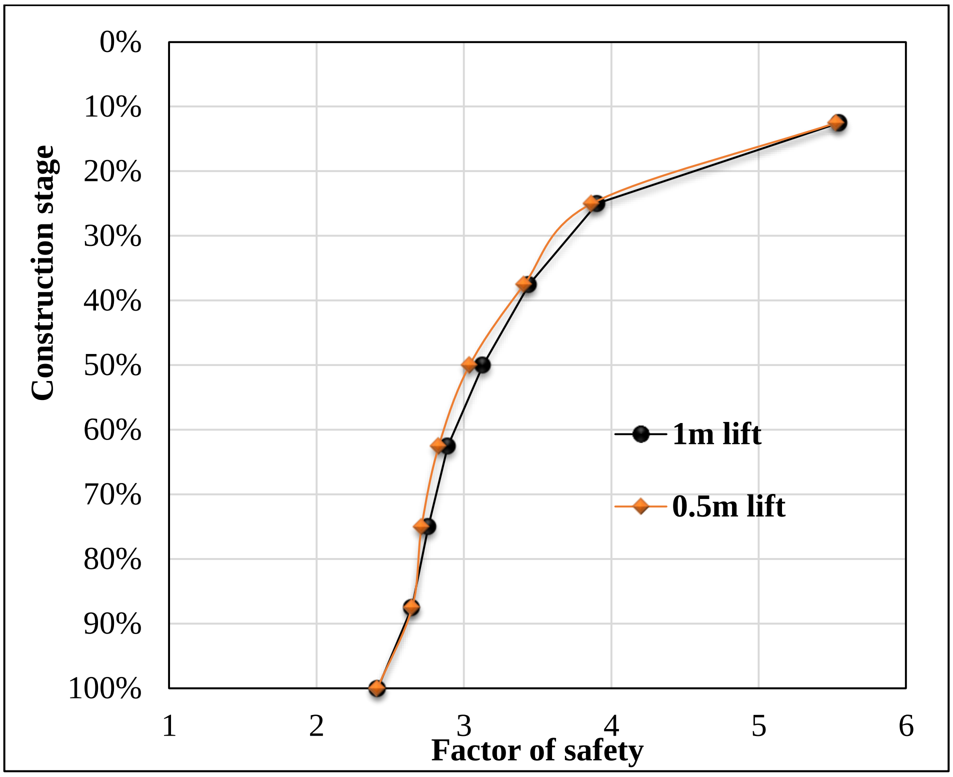


Figure 4.13: Effect of Lift Excavation on Each Construction Stage

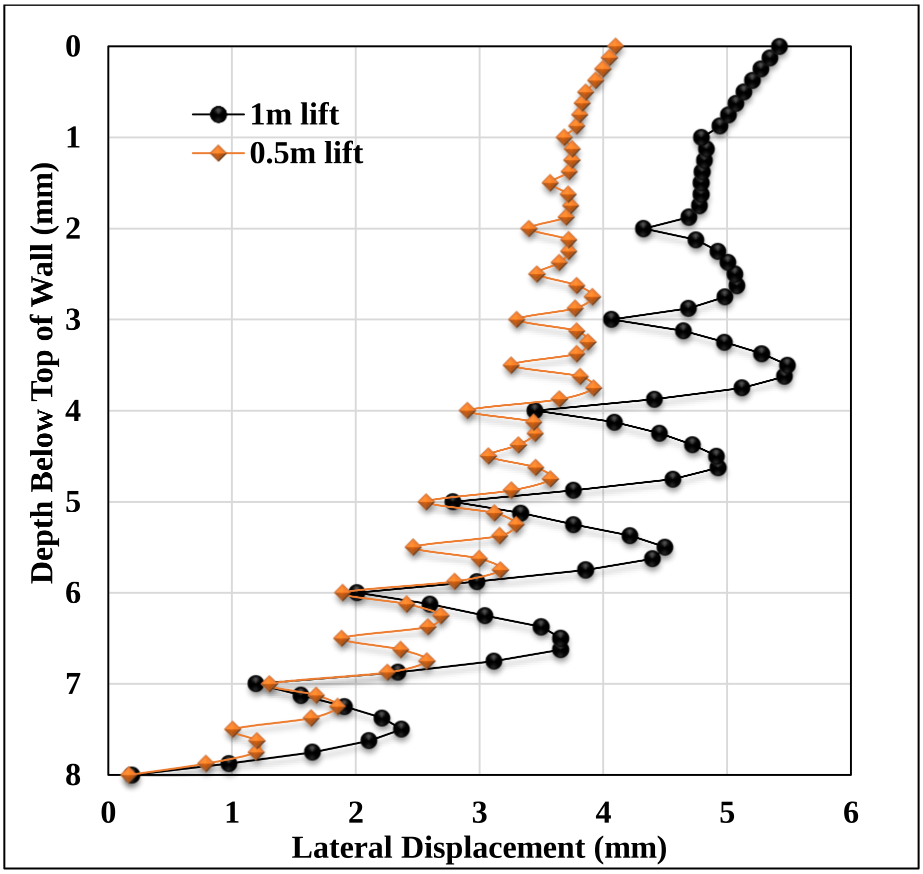


Figure 4.14: Comparison of the Effect of Lift Excavation on Lateral Displacement of the Wall

4.5. Parametric Study of Soil Nail wall

For each nail row installation, there are two methods of simulation of construction stages. The first one is the excavation of the first layer and installation of facing and nail simultaneously as one construction stage which predicts the long-term behavior only. The second method is the excavation of the first layer as one separate excavation stage and installation of soil nail and shotcrete as the second construction stage. The latter is based on the actual construction stage and simulates the real construction behavior of the nail for both short terms (after the excavation of the soil and before facing and the nail is installed) and long term (after installation of facing and nail). Therefore, for this study, the second method is applied throughout.

For all parametric studies, the global factor of safety and the lateral displacement, which is the extreme displacement usually found at the top of the wall, were reported at the end of excavation stage (final configuration of the wall). The horizontal and vertical spacing was assumed to equal, to make the analysis easy. The material properties are listed in Table 4.2 with a constant wall height of 8m. The axial force and the bending stiffness for the different spacing of nails are described in Table 4.6.

Table 4.6: Values of Axial and Bending Stiffness for Different Nail Spacing

Nail spacing	**Axial stiffness**	**Bending stiffness**	**Weight**	**Poisons ratio**
Sv=Sh (m)	EA (kN/m)	EI (kNm^2/m)	w (kN/m/m)	ν
0.5	2.03E+05	8.17	0.48	0.2
1	1.01E+05	4.09	0.48	0.2
1.5	6.76E+04	2.72	0.48	0.2
2	5.07E+04	2.04	0.48	0.2

The mesh size conducted is, based on three steps. The first step is meshing globally using medium-mesh size; the second step is to refine cluster in the vicinity of the wall using refine cluster command and the last step is refine the plate (the nail) using refine line command from the mesh menu.

Generally, for the parametric study the geometry of the model, mesh properties, boundary conditions are shown in Figure 4. 15.

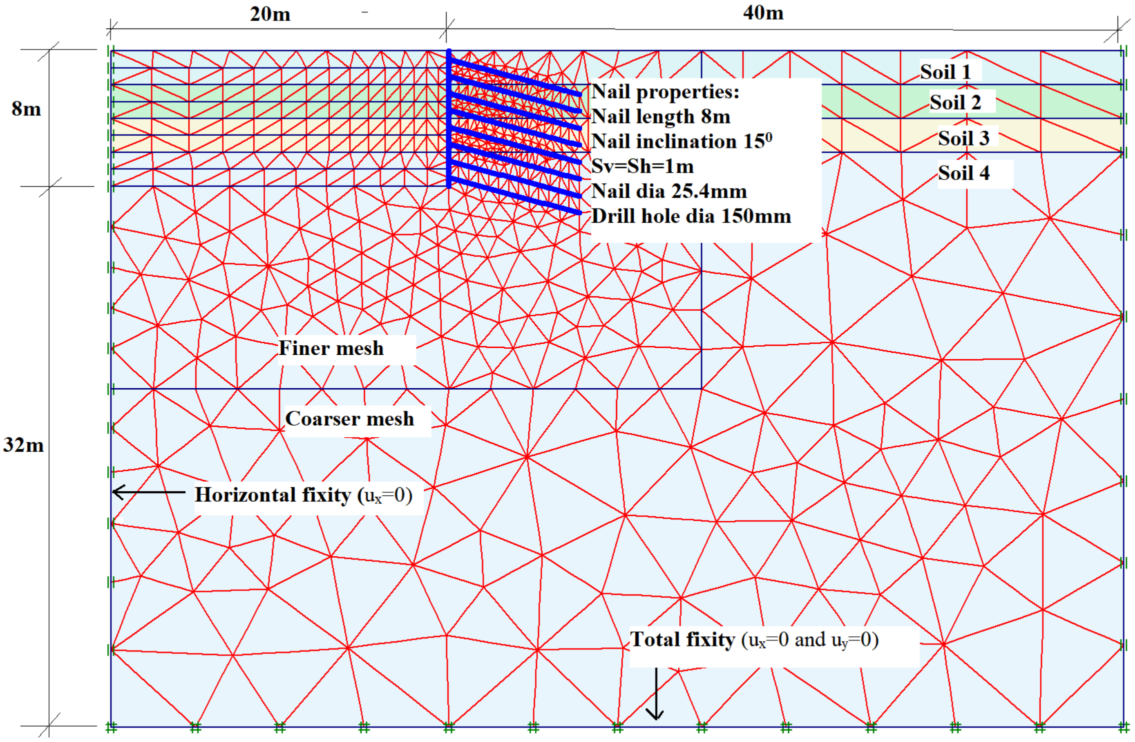


Figure 4. 15: Geometry of the Soil Nail Wall Used for the Parametric Study

4.5.1. Influence of Vertical and Horizontal Spacing

The effect of nail spacing is shown in Figure 4.16. As nail spacing increases the lateral displacement of the wall increases. However, when the nail inclination and the length to wall height ratio increases, the lateral displacement decreases for soil nail spacing of 2m (Figure A-3 and Figure A-4).

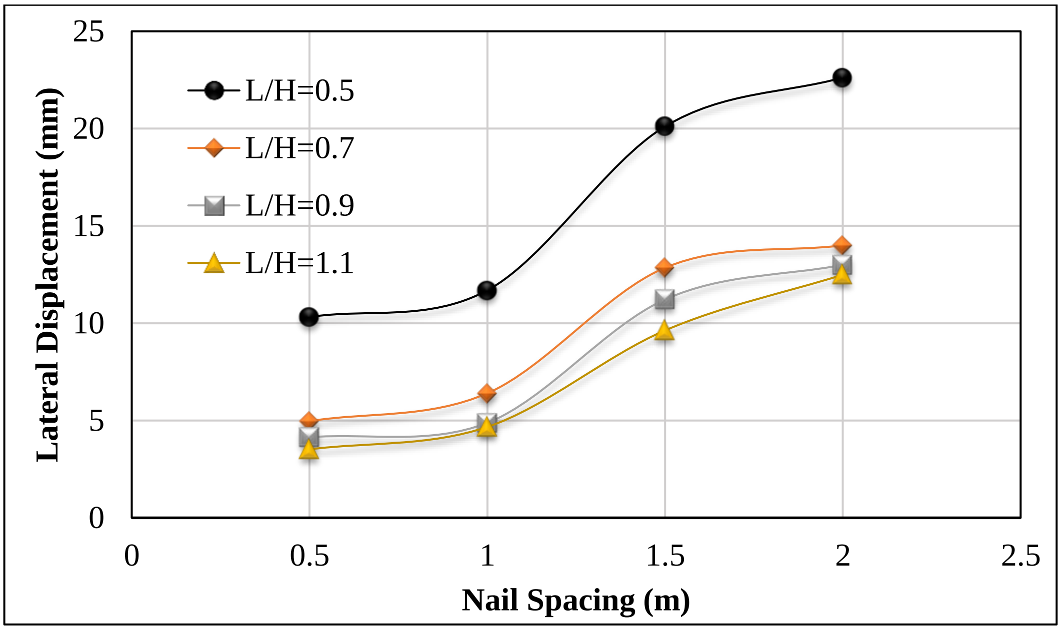


Figure 4.16: Influence of Nail Spacing on Lateral Displacement (Nail Inclination 10^0)

What is shown in Figure 4.16 is so for the following reasons. As the nail spacing increases, the number of nails that contribute to deformation resistance will decrease for the same wall geometries. This in turn will have an effect in increasing the load share on each nail. As a result, there will be an increase in axial deformation as well as pullout deformation of the nails.

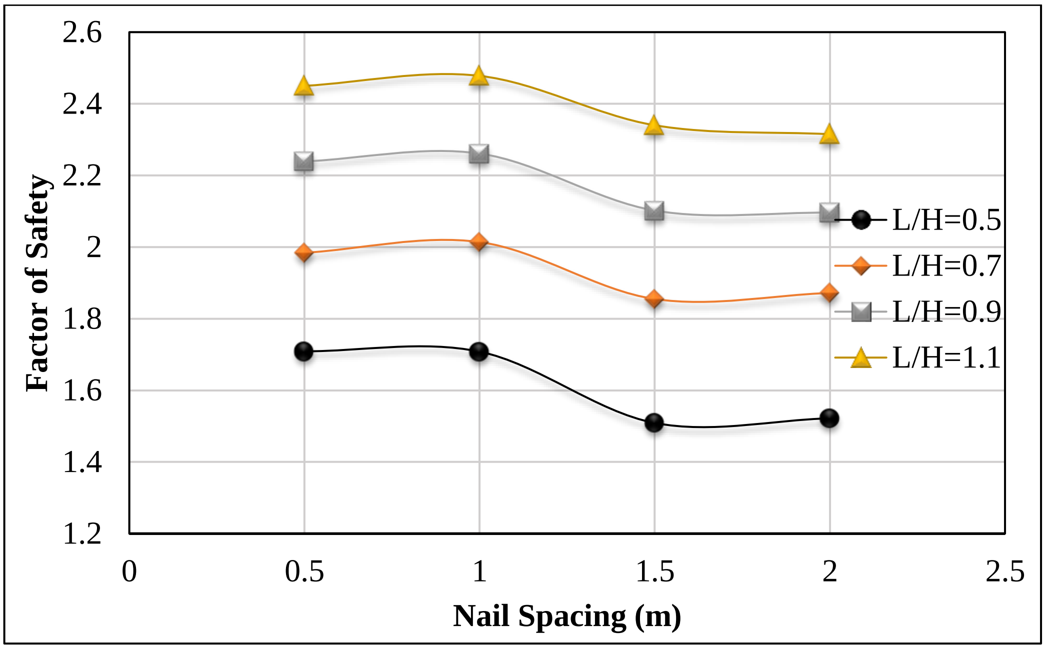


Figure 4.17: Influence of Nail Spacing on Factor of Safety (Nail Inclination 10^0)

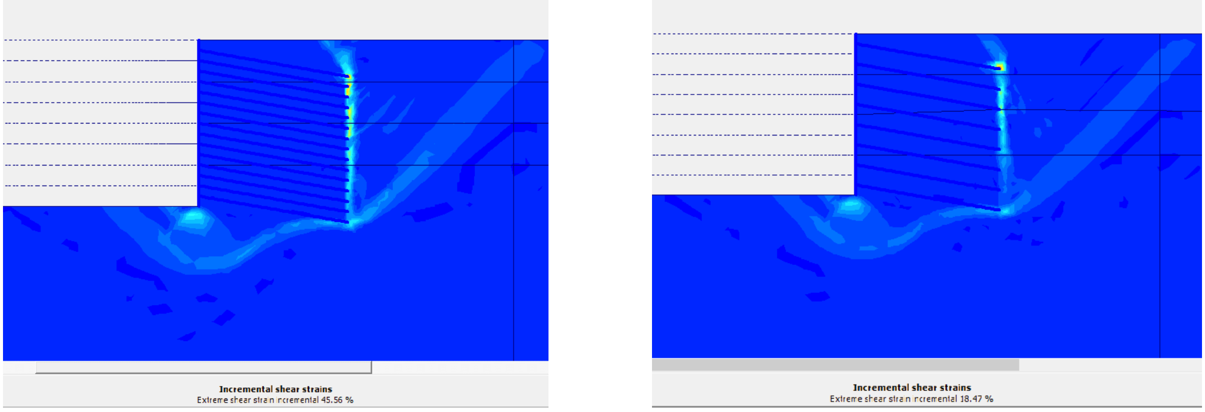


Figure 4.18: Failure Mechanisms of 0.5 m Nail Spacing (Left) and 1 m Nail Spacing (Right) for L/H=0.9 and Nail Inclination 10^0

In contrary to that of lateral displacement, as the nail spacing increases, the factor of safety decreases as shown in Figure 4.17. However, the factor of safety of 0.5 m nail spacing is less than or nearly equal to 1 m nail spacing. This is maybe due to the same failure mechanisms of both of the wall models. As shown in Figure 4.18, the failure mechanism is nearly the same i.e. the failure plane does not cross any nails in both modeling so increasing the number of nails has no effect on the failure plane but it increases the block mass. Therefore, if the block mass increases, the driving force increases and the factor of safety decreases.

4.5.2. Influence of Length to Height ratio (L/H)

The height of the wall is constant and the length was varied. As the L/H ratio increases the lateral displacement of the soil nail wall decreases. From L/H=0.5 to L/H=0.7 the lateral displacement decreases drastically but beyond that, the decrement of the lateral deflection seems small.

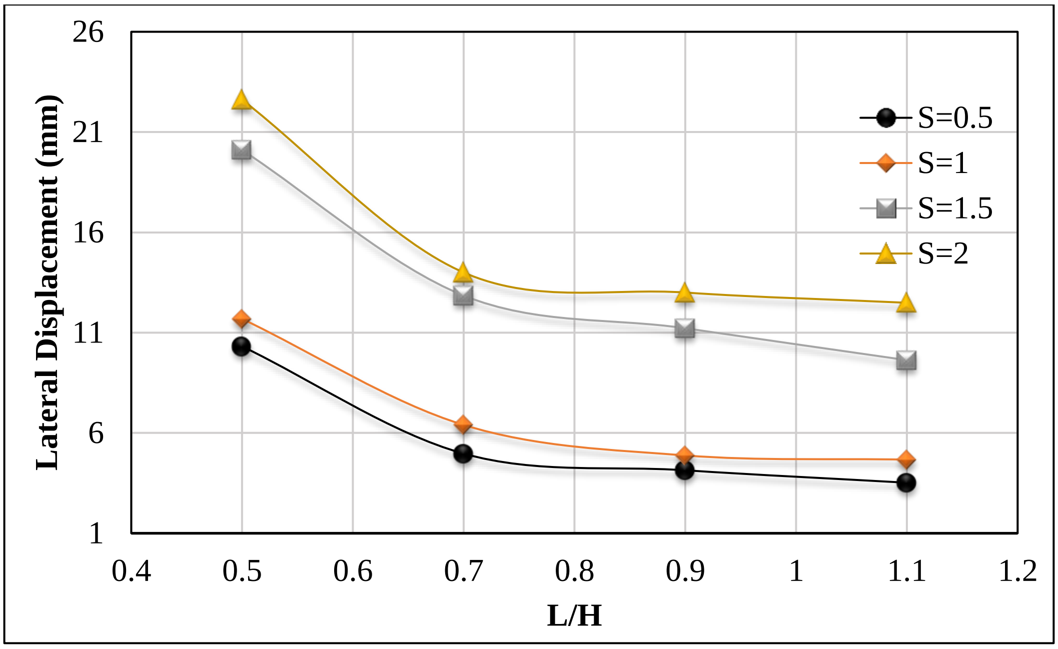


Figure 4.19: Influence of L/H on Lateral Displacement (Nail Inclination 10^0)

As the L/H increases, the resistance to soil deformation increases. Here the lateral deformation of the active wedge is resisted by mainly through the length by its bond strength with the soil. However, later on when the L/H increases the failure surface crosses all or some of the nails. After that increasing the length will not affect the lateral displacement for some of the L/H ratio and decrease for higher L/H with higher nail spacing and inclination.

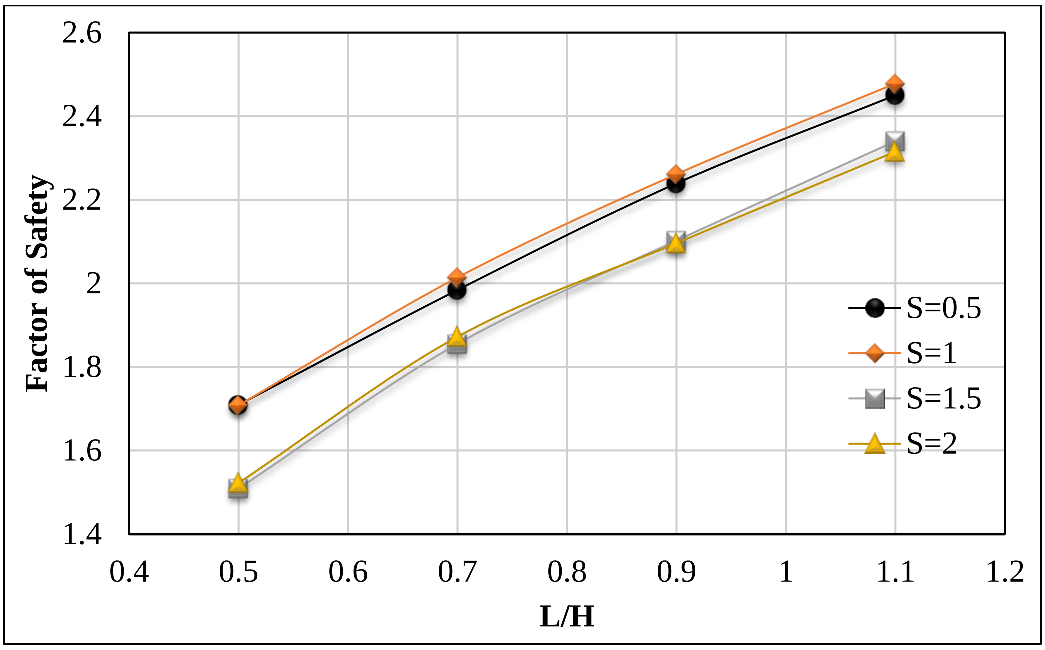


Figure 4.20: Influence of L/H on Global Factor of Safety (Nail Inclination 10^0)

As the L/H ratio increases the global factor of safety increases almost linearly. For each L/H ratio, the failure surface is found at the back of the wall without crossing any nails. Here as the length increases, the length of the failure surface behind the nails increases. Therefore as the length of the failure surface increase, the global factor of safety increases.

4.5.3. Influence of Nail Inclination (λ)

Nail inclination was varied from 0^0 to 20^0 as shown in Figure 4.21 and Figure A-17 through Figure A-19. From the result, as the nail inclination increases the lateral displacement of the wall increases.

As the nail inclination increases, the resistive (horizontal) force component of the nail decreases. Therefore, the lateral wall displacement increase. Nail inclination less than 15^0 has no significant change on lateral displacement but when the inclination increases from 15^0 to 20^0, the lateral displacement increases drastically.

This shows that using nail inclination between 5^0 and 15^0 is an appropriate choice for projects concerned deflection. Even though for some L/H ratio, the lateral displacement at 0^0 inclination is less than that of other inclination, using 0^0 inclination in actual site construction is impossible because of difficulties in pouring the mortar.

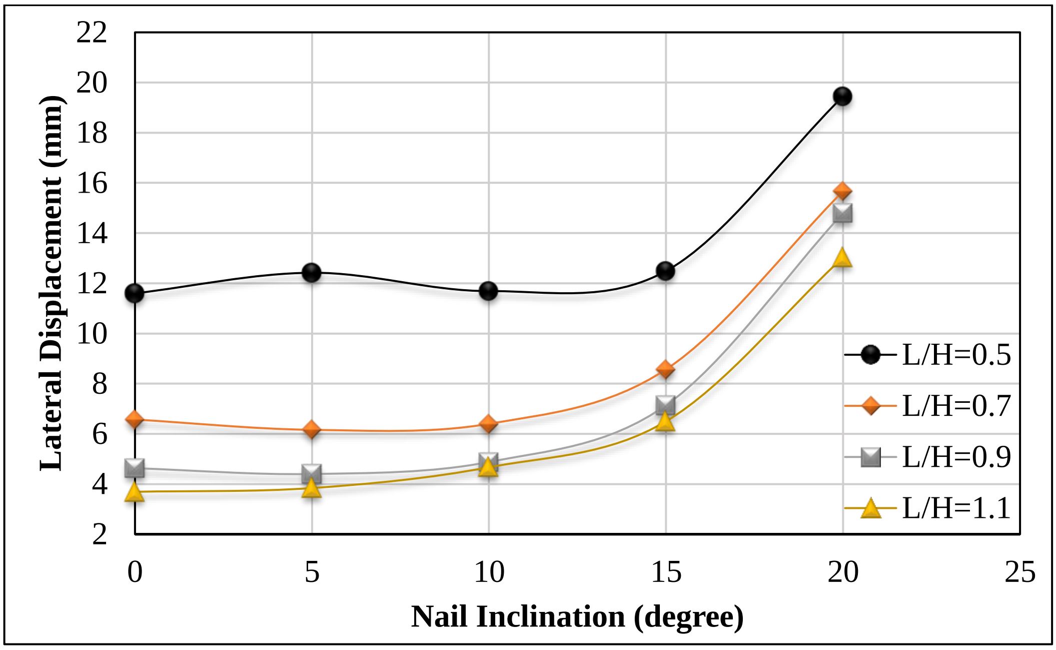


Figure 4.21: Influence of Nail Inclination on Lateral Displacement (Nail Spacing 1m)

As the nail inclination increases, the resistive (horizontal) force component of the nail decreases. Therefore the lateral wall displacement increase.

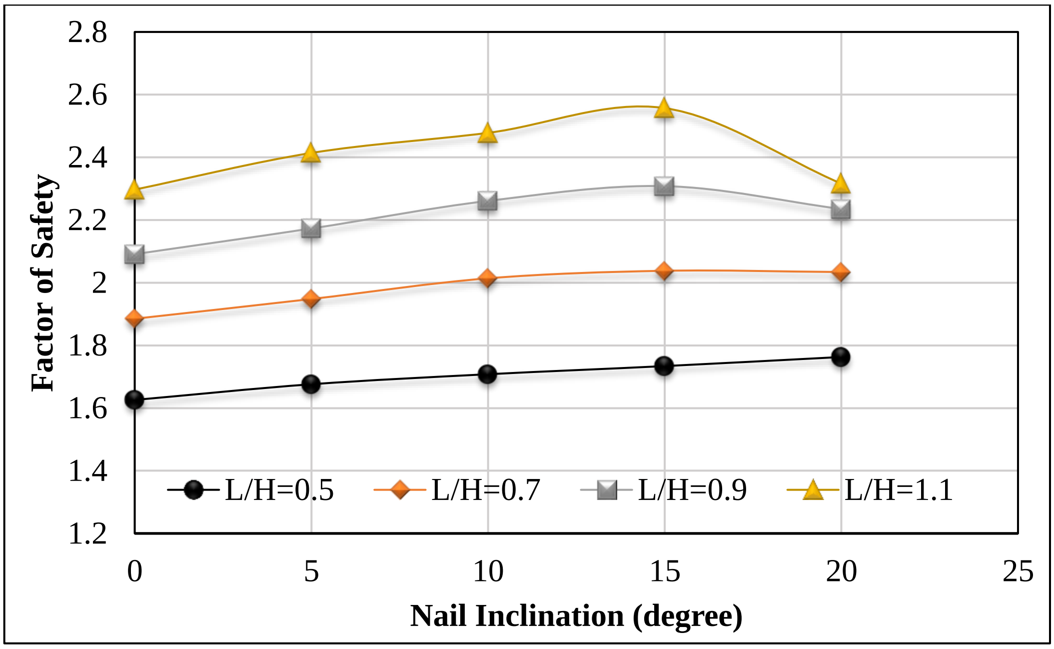


Figure 4.22: Influence of Nail Inclination on Global Factor of Safety (Nail Spacing 1m)

As shown in Figure 4.22 and Figure A-20 through Figure A-22, the global factor of safety almost linearly increases with nail inclination increases except for some L/H=1.1 and L/H=0.9 decrease from 15^0 inclination to 20^0 inclination. This is because as the nail inclination increases, the length of the nail at the bottom of the excavated soil level increases. This is true for the bottom nails. As the length below the bottom of the nail increase, the failure surface is still behind the nails (left of Figure 4.23) and the length of failure surface increases. Therefore, the global factor of safety increases. However, for higher L/H ratio, the failure surface crosses all the nails and afterwards increasing the inclination decreases the global factor of safety (right side of Figure 4.23).

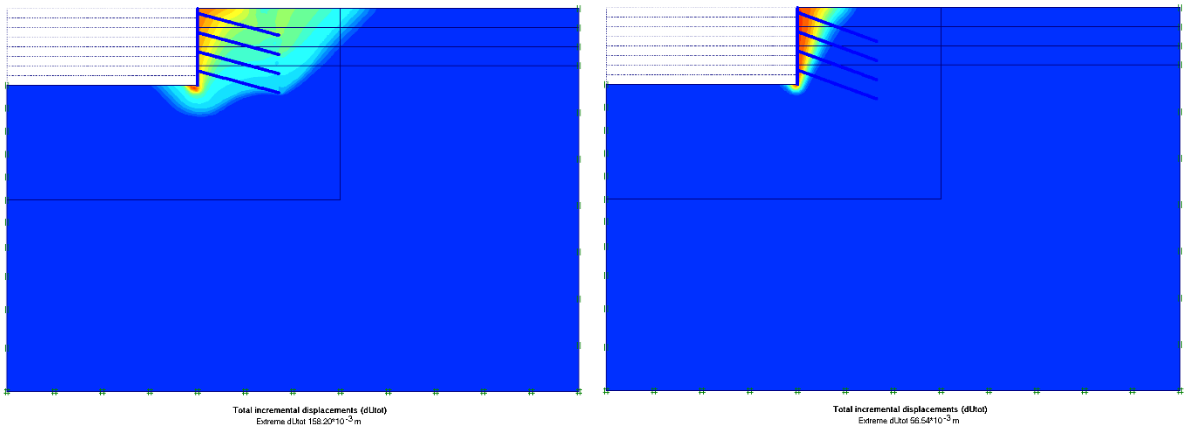


Figure 4.23: For L/H=1.1 Nail Spacing of 2m, Inclination of 15^0 (Left) and for L/H=1.1 Nail Spacing of 2m, Nail Inclination of 20^0 (Right)

4.6. Comparison of Limit Equilibrium and Finite Element Analysis Results

For comparison of the two methods, 8 m nail wall height and 8 m nail length with an inclination of 15^0 have been used. In limit equilibrium both the wedge method using excel program and method of the slice using slope/w has been implemented. However, the failure surfaces used in wedge method and method of the slice are different i.e. for wedge method the failure surface is a straight line starting from the bottom end of the soil nail wall but for a method of slice it a circular deep-seated to the bottom of final excavation stage. Therefore, it is difficult to compare the two methods. Instead of comparing the stability using slope/w and PLAXIS 2D has been found important. Using slope/w a global factor of safety 2.86 was obtained and that of PLAXIS 2.41.

Generally, the factor of safety calculation using finite element is through successive shear strength reduction (SRF), in which the deformation undergoes an increment due to the reduction of the shear strength of the soil. Like that of the limit equilibrium method, no defined path for failure surface, instead of at a place where shear strength is minimum.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusion

From this study, the following conclusions can be drawn:

- The global factor of safety found in limit equilibrium is greater than the global factor of safety found in a finite element.
- The horizontal deflection of the wall is in a good agreement with the measured field data. Therefore, finite element based software is a good and appropriate tool for soil nail wall analysis.
- It is possible to model soil nails using a plate and geogrid elements. Bending stiffness has no contribution to the lateral displacement and to the global factor of safety at the end of the construction stage (final layout of the wall). However, it has a significant effect on the global factor of safety varying from 4.5 % to 34.6 % during the construction process. Generally, plate element can be used for soil nail model.
- Depth of lift excavation has no effect on the global factor of safety but it significantly affects the lateral displacement. As the depth of the lift excavation increase, the lateral displacement at the final construction stages of the wall increases.
- For the parametric studies, as the nail spacing increases, the lateral displacement increases and the factor of safety decreases.
- The global factor of safety increases linearly with nail inclination except for some L/H=1.1 and L/H=0.9 decrease from 15^0 inclination to 20^0 inclination.
- Based on the parametric study, 15^0-nail inclination gives an optimum soil-nail performance.

5.2. Recommendations

The limitation of this study was only one type of soil used and it should be checked for other type of soils. The following points are recommended:

- In this study, the analysis is done only during constriction and at the end of the construction stages. It does not cover the long-term effect on the behavior of the soil nail wall. Therefore, the long-term effect should be studied.
- The vertical and horizontal spacing of the nails is assumed the same value for the parametric study considered. The individual effect on deflection and factor of safety should be studied and verified.

- This study mainly focuses on soil nailing as deep excavation support for railway track and for deep excavation of length to width ratio of large. Therefore, only 2D plane strain model was applied. For the deep excavations of smaller length to width ratio less than about ten (which are common in multi-story buildings excavations), 3D model should be studied.

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APPENDIX A: PARAMETRIC STUDY RESULT

1. Influence of Vertical and Horizontal Spacing

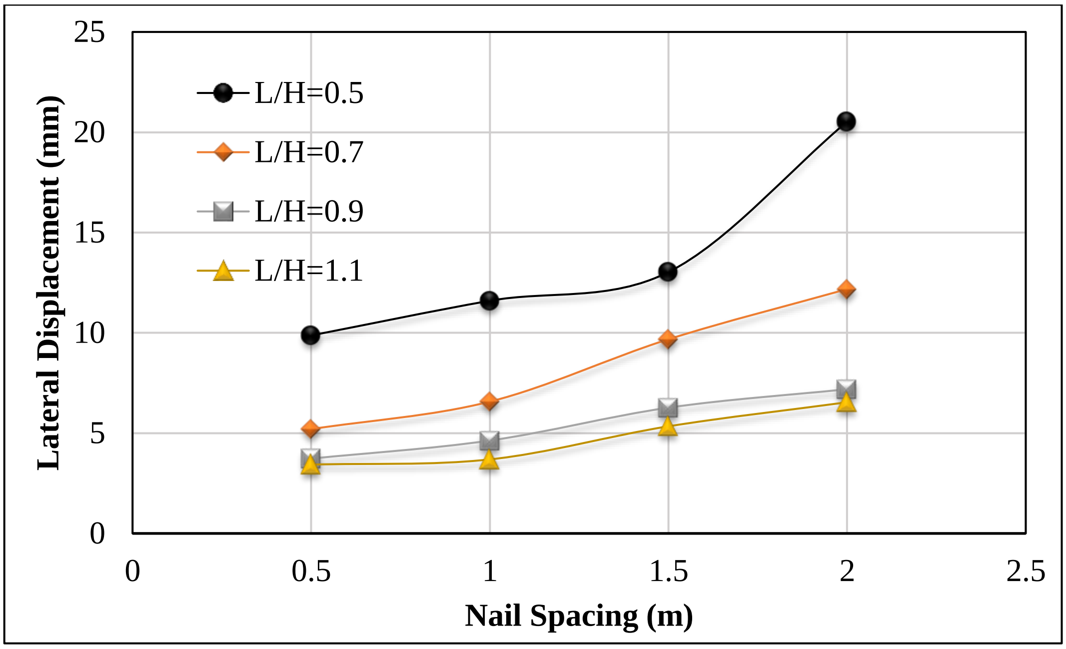

Figure A-1: Influence of Nail Spacing on Lateral Displacement (Nail Inclination 0^0)

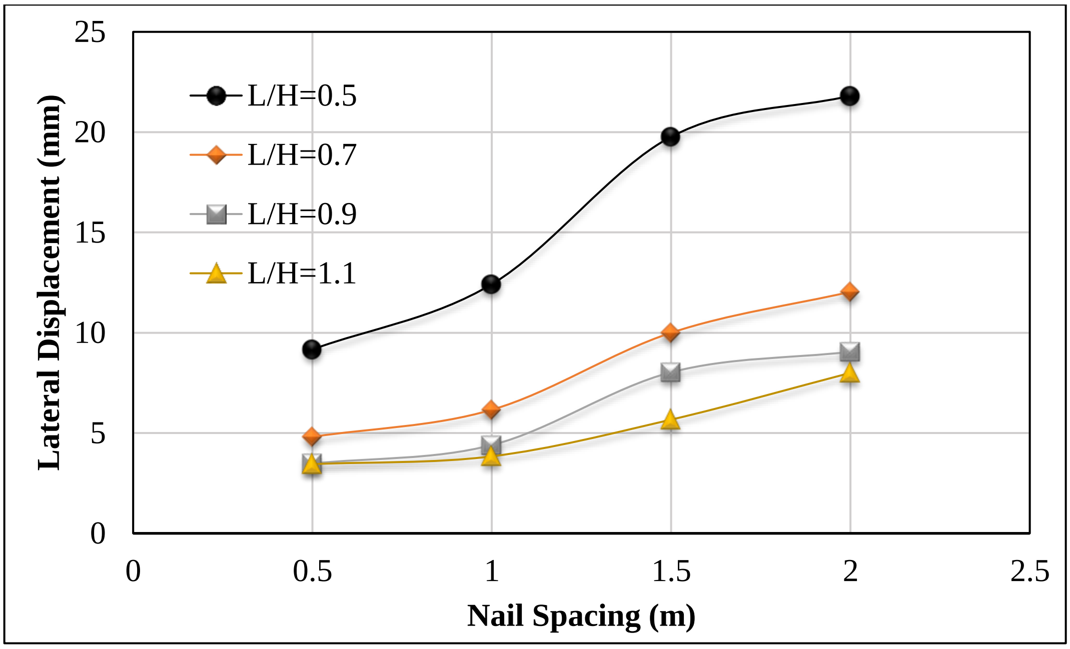

Figure A-2: Influence of Nail Spacing on Lateral Displacement (Nail Inclination 5^0)

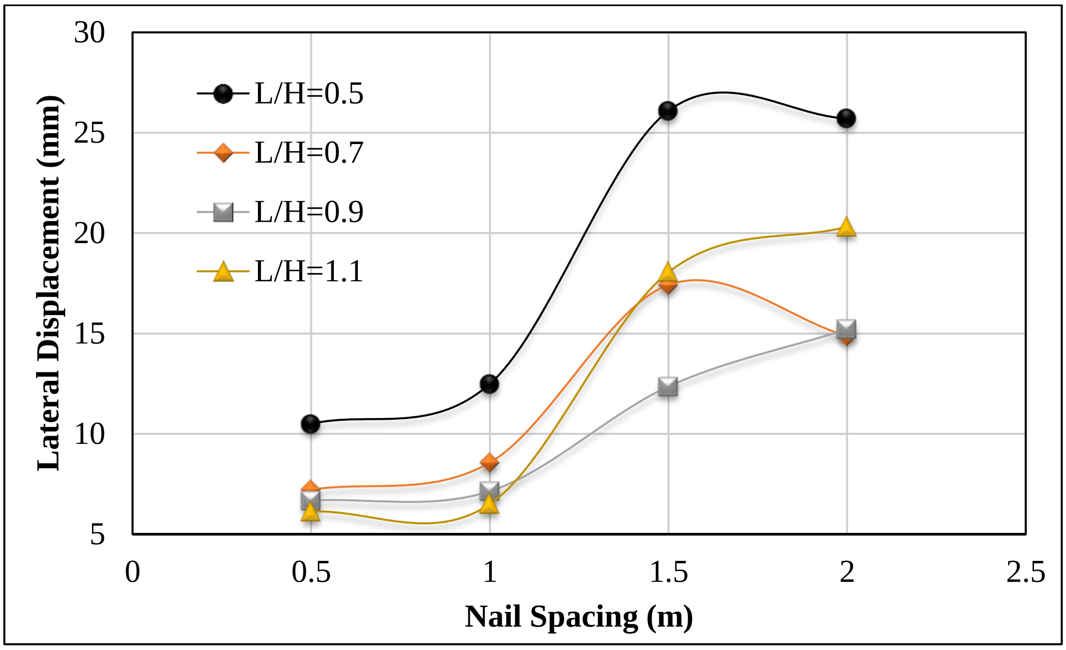

Figure A-3: Influence of Nail Spacing on Lateral Displacement (Nail Inclination 15^0)

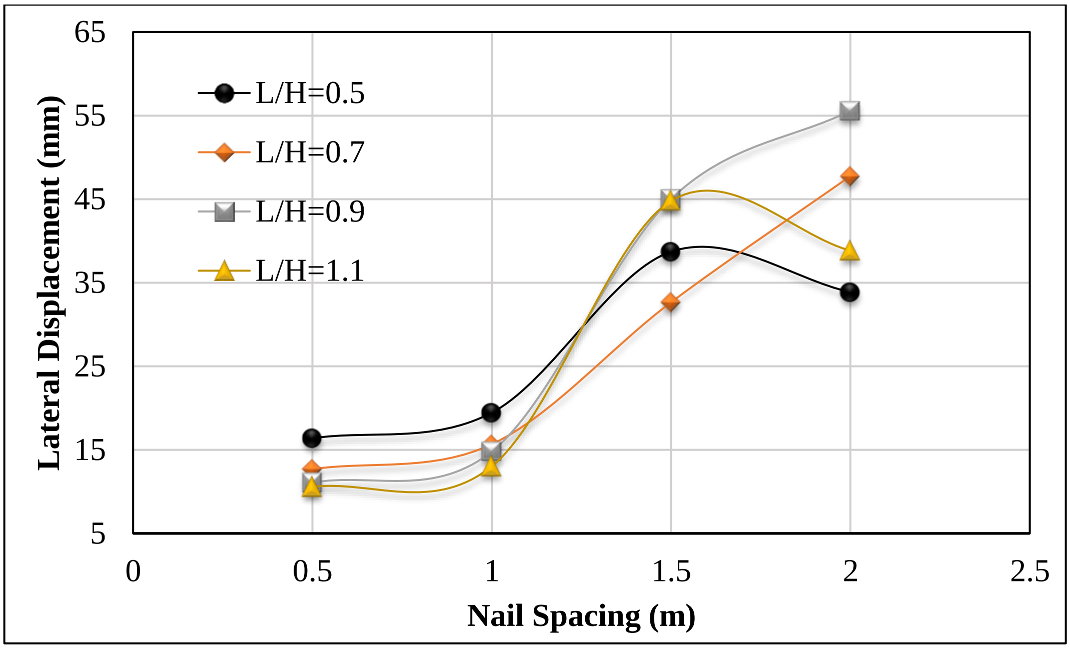

Figure A-4: Influence of Nail Spacing on Lateral Displacement (Nail Inclination 20^0)

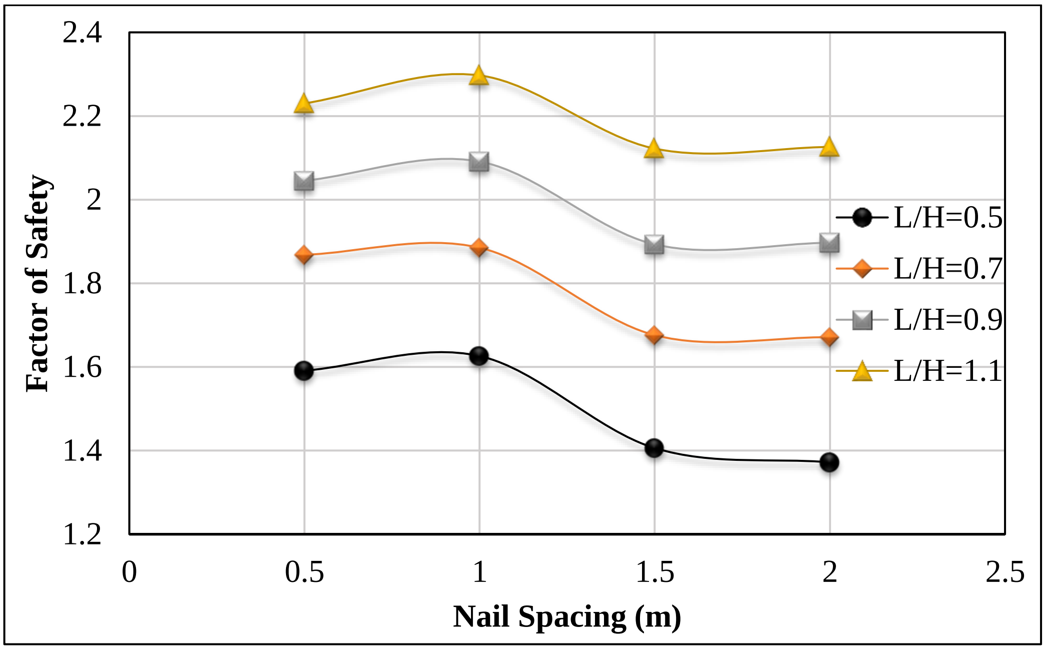

Figure A-5: Influence of Nail Spacing on Global Factor of Safety (Nail Inclination 0^0)

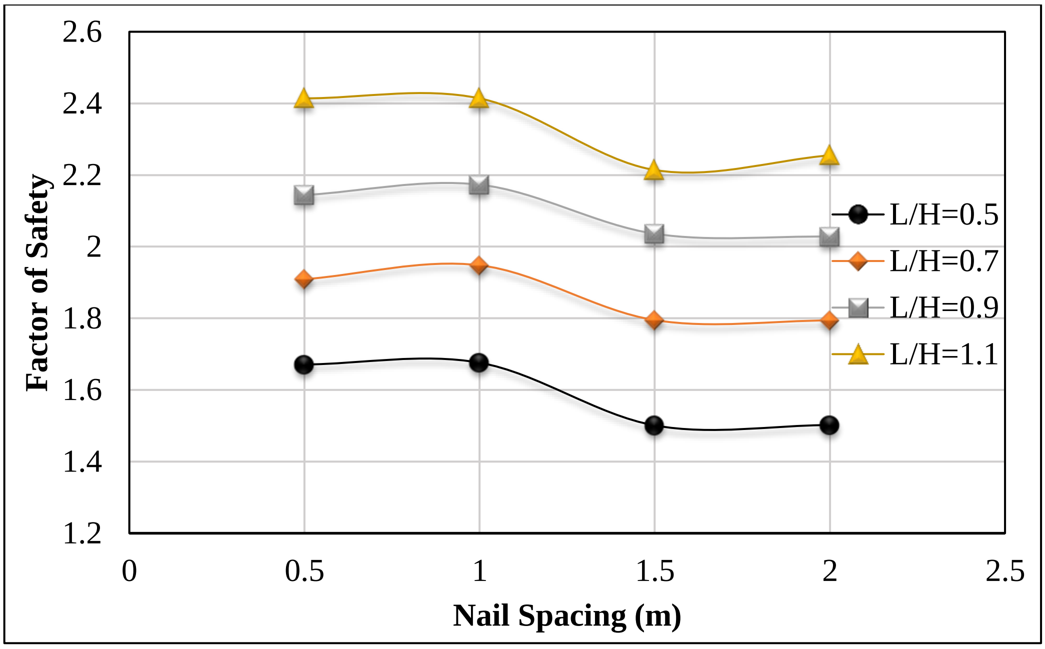

Figure A-6: Influence of Nail Spacing on Global Factor of Safety (Nail Inclination 5^0)

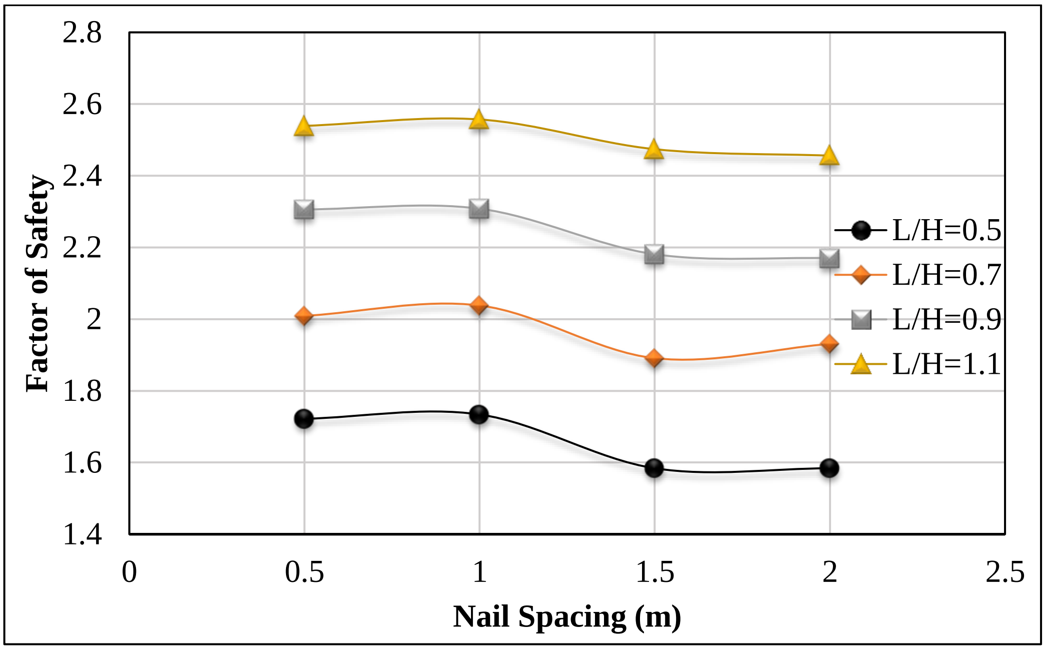

Figure A-7: Influence of Nail Spacing on Global Factor of Safety (Nail Inclination 15^0)

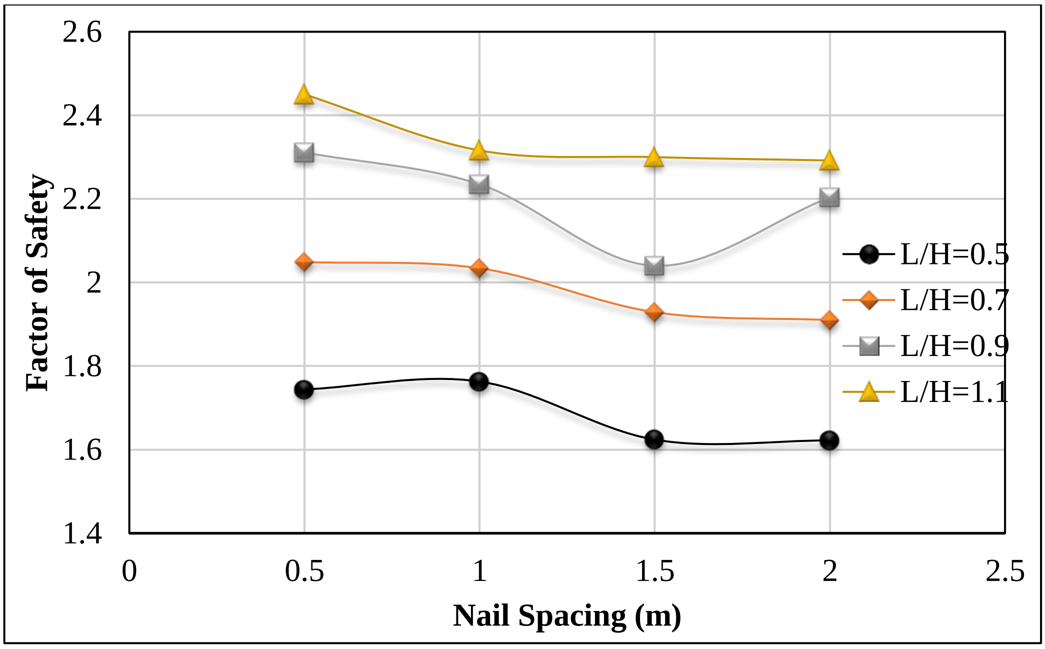

Figure A-8: Influence of Nail Spacing on Global Factor of Safety (Nail Inclination 20^0)

2. Influence of Length to Height Ratio (L/H)

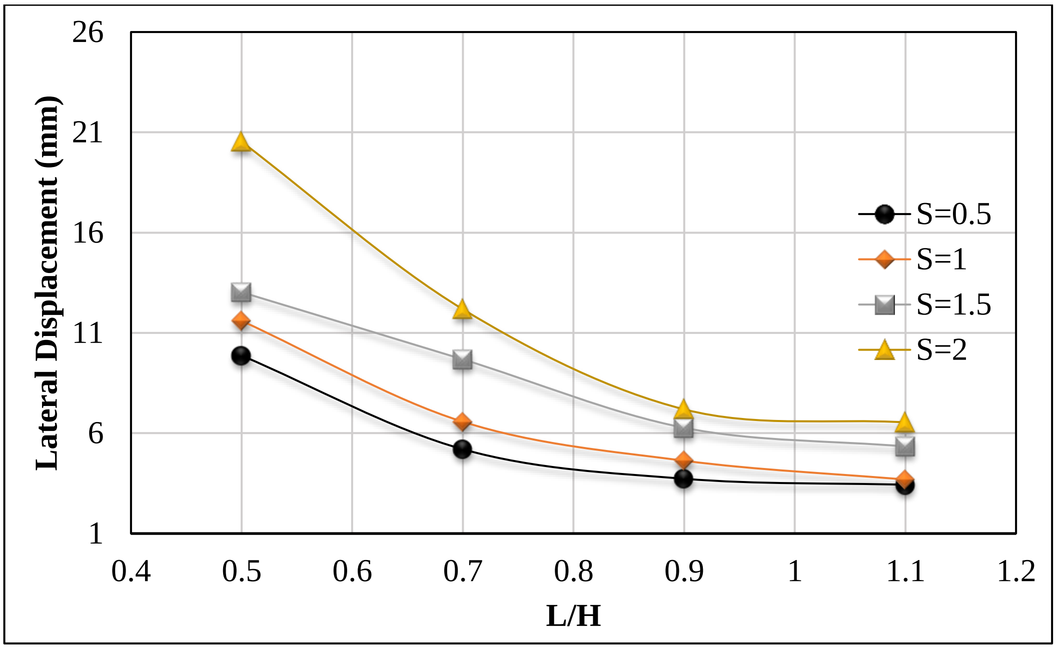

Figure A-9: Influence of L/H on Lateral Displacement (Nail Inclination 0^0)

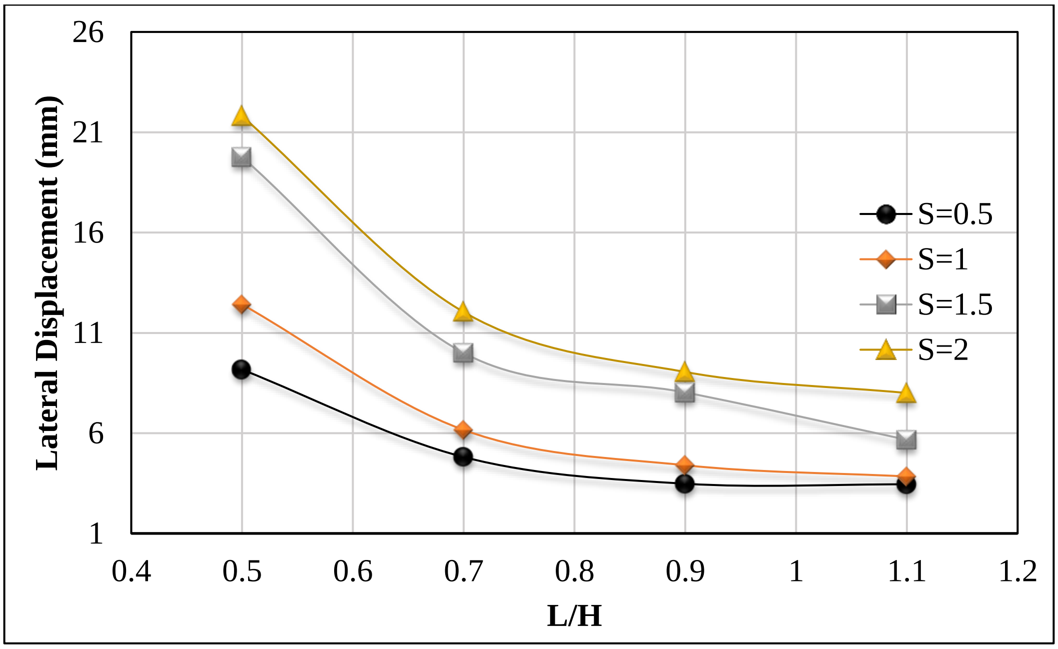

Figure A-10: Influence of L/H on Lateral Displacement (Nail Inclination 5^0)

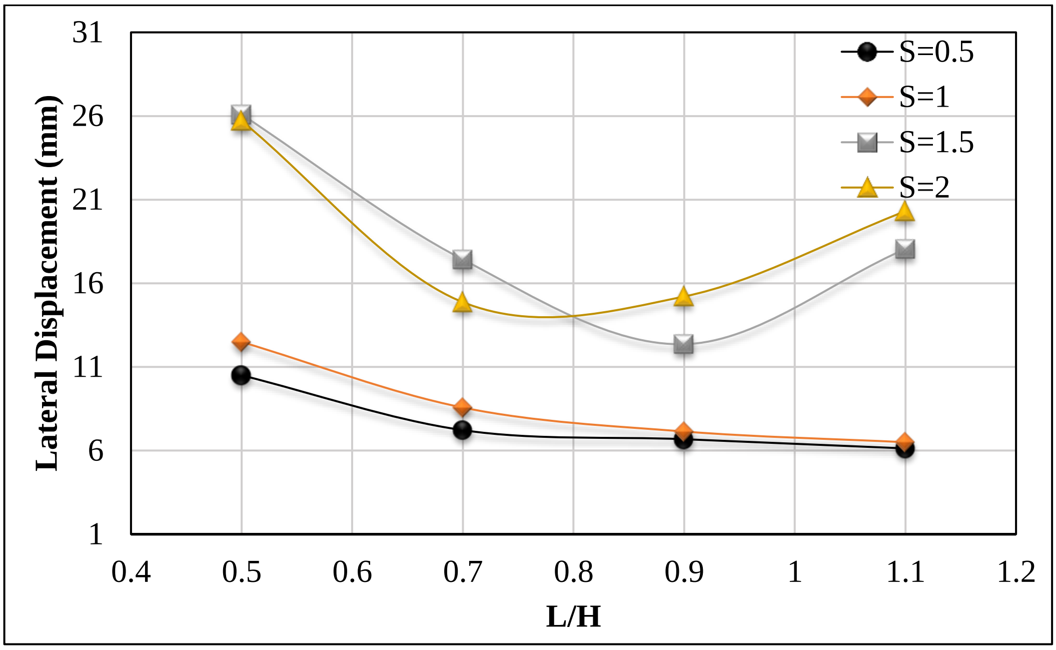

Figure A-11: Influence of L/H on Lateral Displacement (Nail Inclination 15^0)

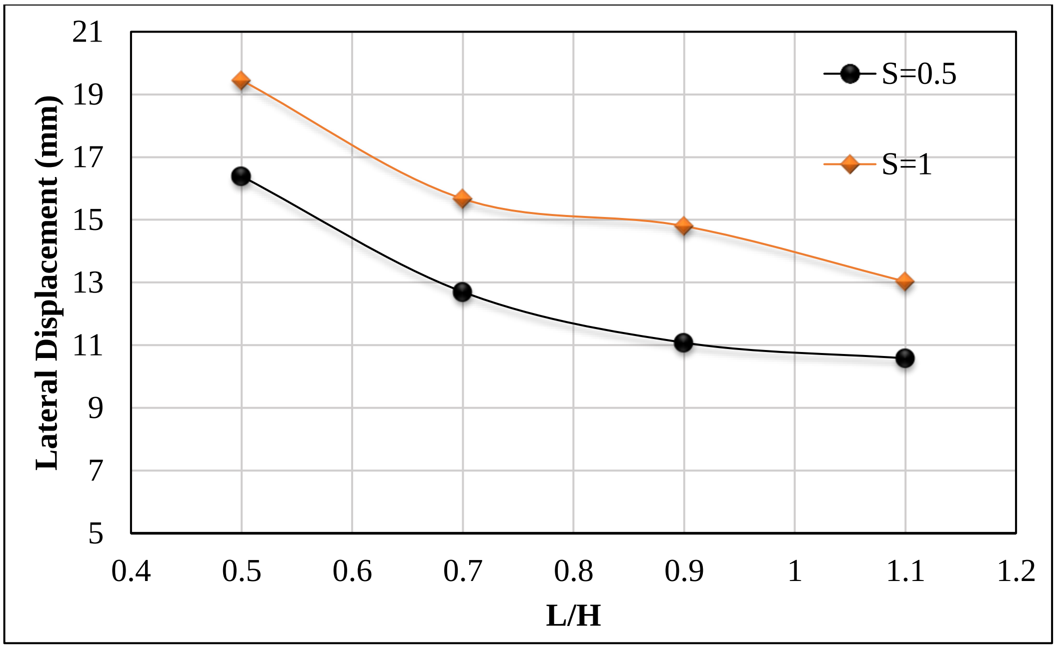

Figure A-12: Influence of L/H on Lateral Displacement (Nail Inclination 20^0)

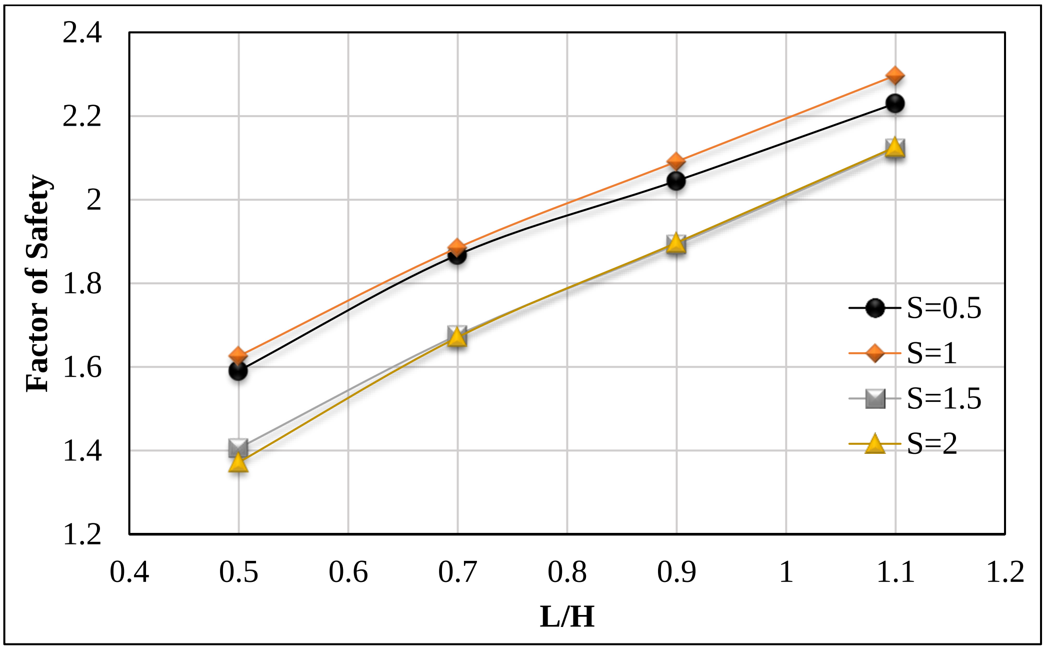

Figure A-13: Influence of L/H on Global Factor of Safety (Nail Inclination 0^0)

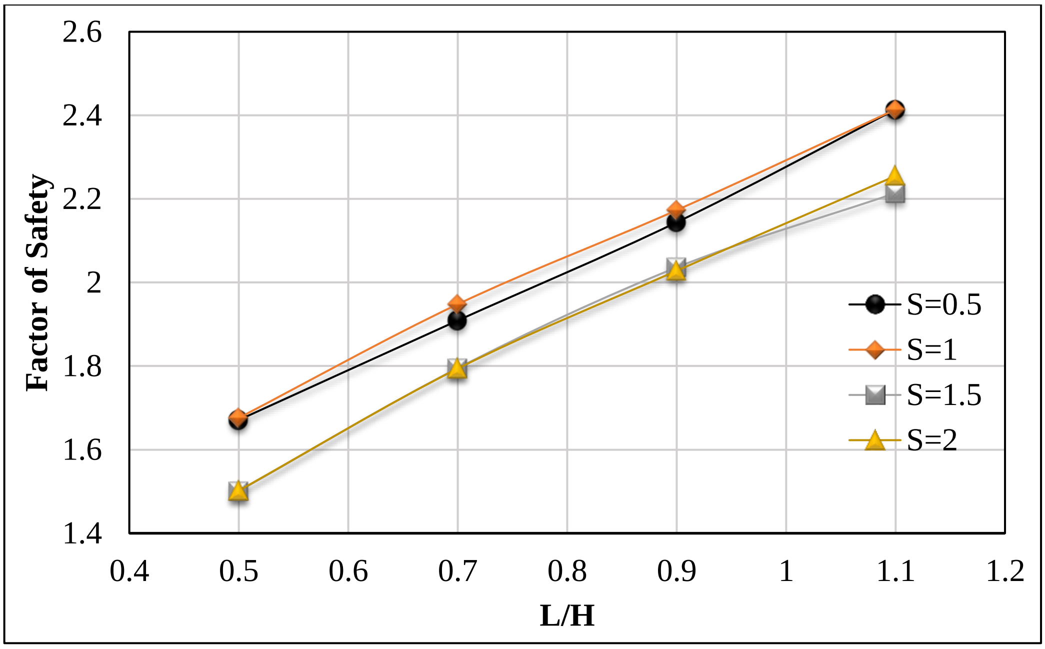

Figure A-14: Influence of L/H on Global Factor of Safety (Nail Inclination 5^0)

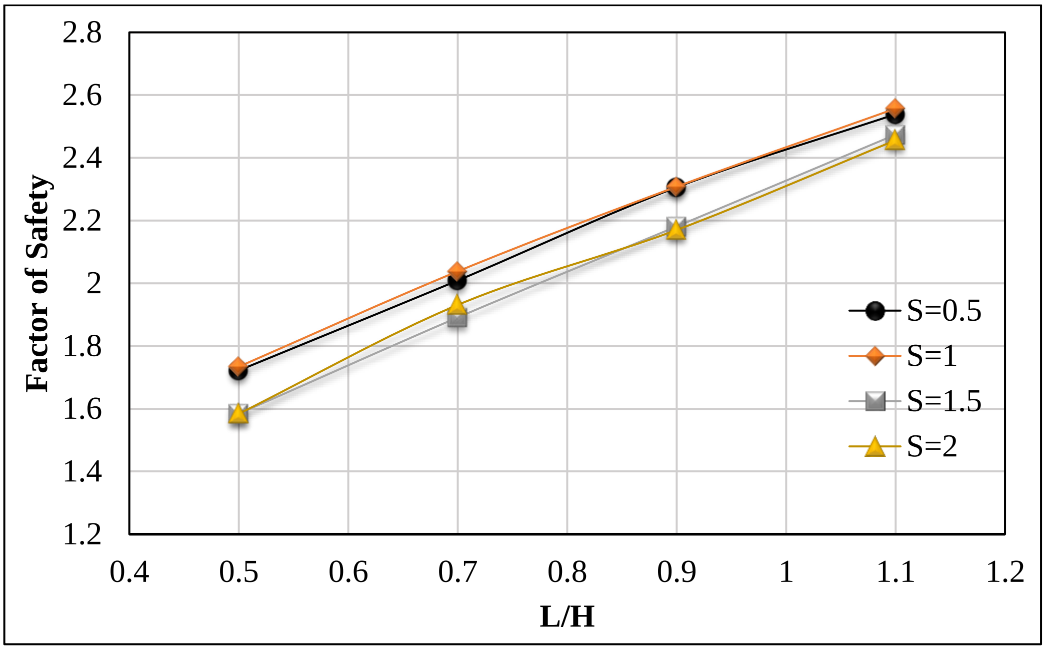

Figure A-15: Influence of L/H on Global Factor of Safety (Nail Inclination 15^0)

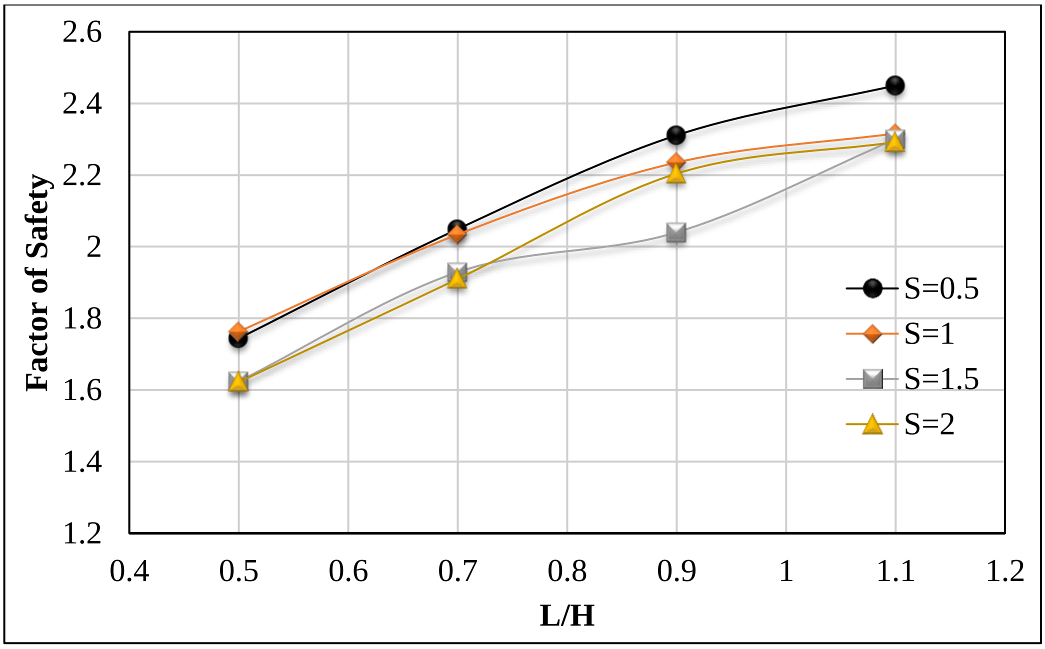

Figure A-16: Influence of L/H on Global Factor of Safety (Nail Inclination 20^0)

3. Influence of Nail Inclination (λ)

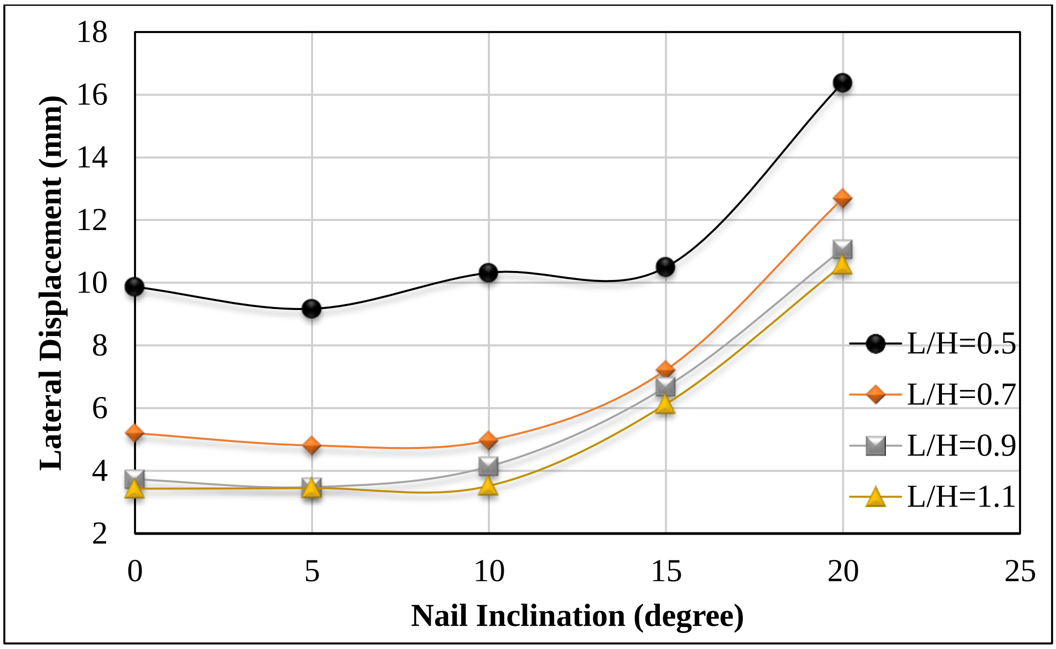

Figure A-17: Influence of Nail Inclination on Lateral Displacement (Nail Spacing 0.5 m)

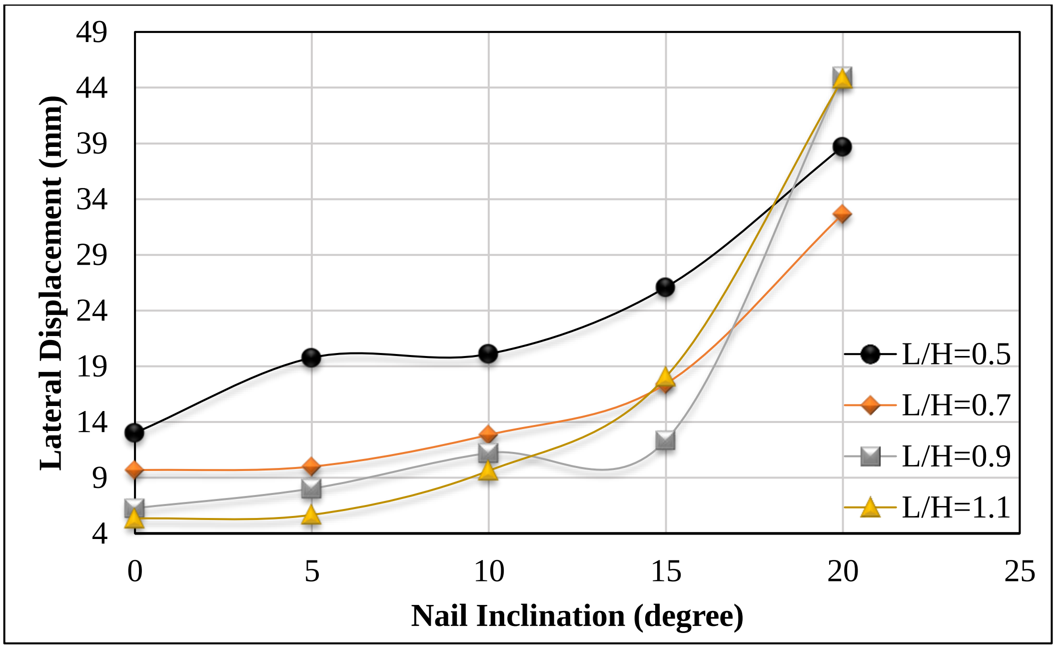

Figure A-18: Influence of Nail Inclination on Lateral Displacement (Nail Spacing 1.5 m)

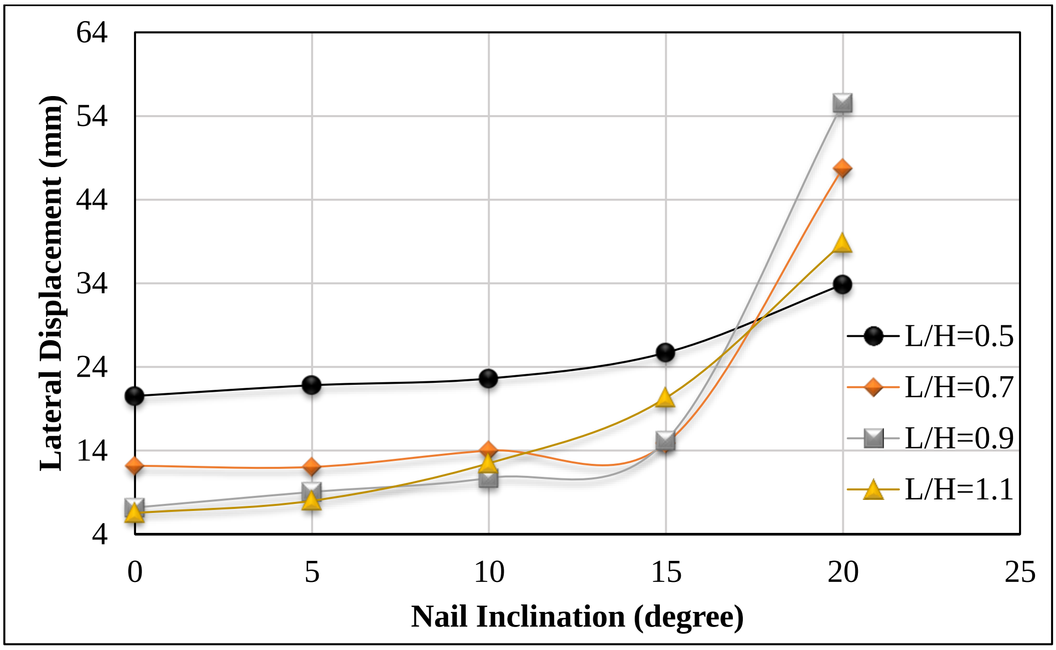

Figure A-19: Influence of Nail Inclination on Lateral Displacement (Nail Spacing 2 m)

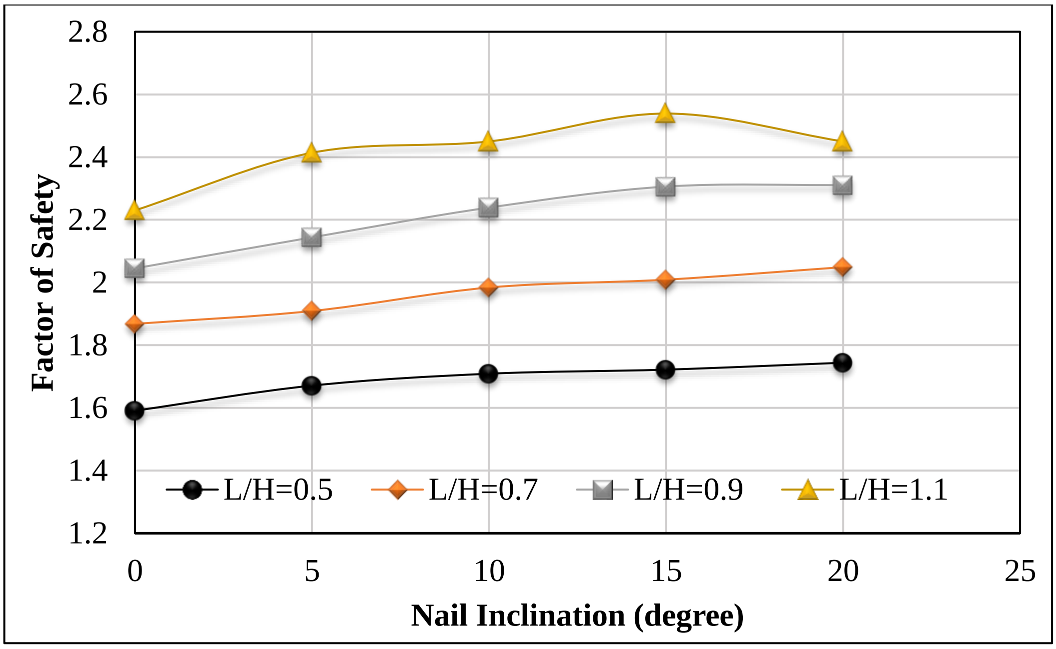

Figure A-20: Influence of Nail Inclination on Global Factor of Safety (Nail Spacing 0.5 m)

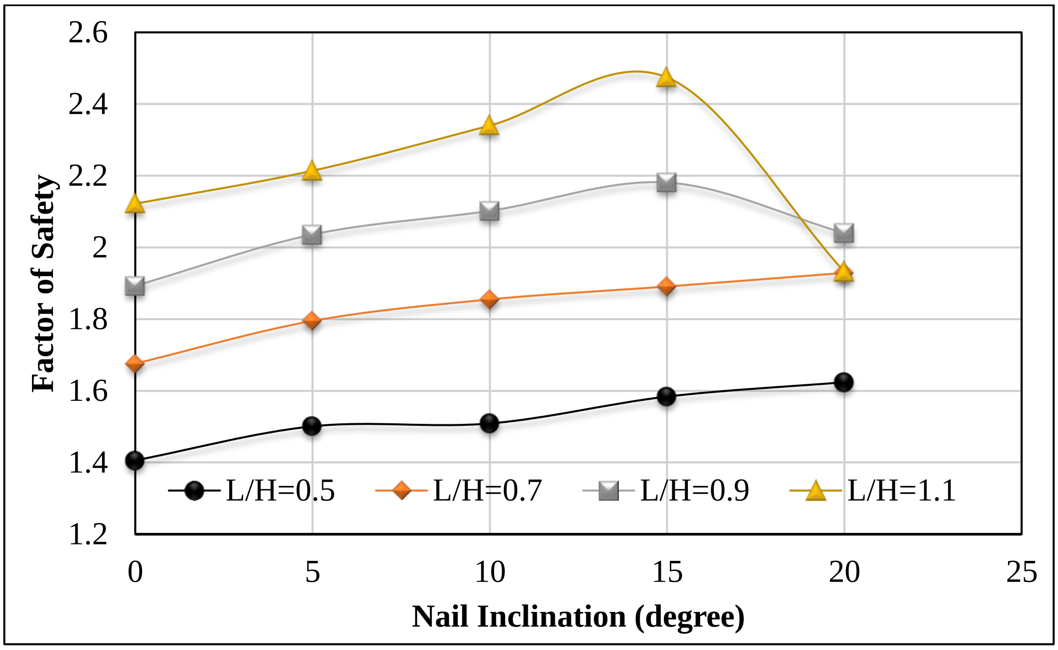

Figure A-21: Influence of Nail Inclination on Global Factor of Safety (Nail Spacing 1.5 m)

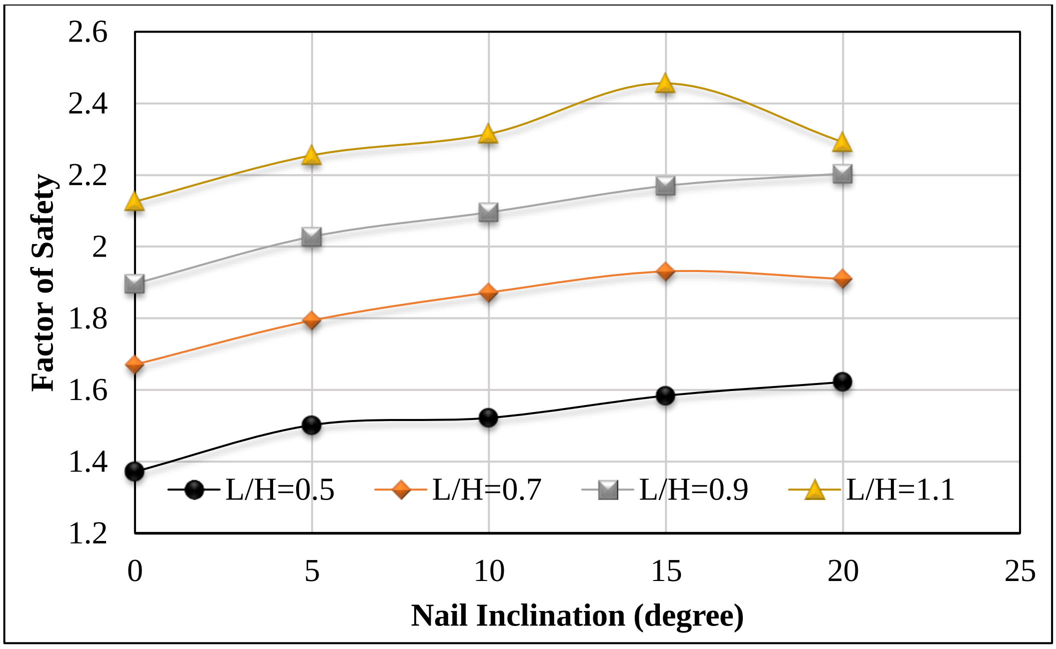

Figure A-22: Influence of Nail Inclination on Global Factor of Safety (Nail Spacing 2 m)