LANDSLIDE MITIGATION USING SOIL NAILING

A

PROJECT REPORT

Submitted in partial fulfillment of the requirements for the award of the degree

of

BACHELOR OF TECHNOLOGY

IN

CIVIL ENGINEERING

Under the supervision

of

Mr. Niraj Singh Parihar

And

Dr. Amardeep

(Assistant Professor)

by

Tenzin (171672)

Bikash Subba (171676)

Kinzang Peldon (171677)

to

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

WAKNAGHAT, SOLAN – 173234

HIMACHAL PRADESH, INDIA

May- 2021

STUDENT'S DECLARATION

I hereby declare that the work presented in the Project report entitled **"Landslide Mitigation Using Soil Nailing"** submitted for partial fulfillment of the requirements for the degree of Bachelor of Technology in Civil Engineering at **Jaypee University of Information Technology, Waknaghat** is an authentic record of my work carried out under the supervision of **(Mr. Niraj Singh Parihar and Dr. Amardeep)**. This work has not been submitted elsewhere for the reward of any other degree/diploma. I am fully responsible for the contents of my project report.

Tenzin (171672)

Bikash Subba (171676)

Kinzang Peldon (171677)

Department of Civil Engineering

Jaypee University of Information Technology, Waknaghat, India

17/05/2021

CERTIFICATE

This is to certify that the work which is being presented in the project report titled **"Landslide Mitigation using Soil Nailing"** in partial fulfillment of the requirements for the award of the degree of Bachelor of Technology in Civil Engineering submitted to the Department of Civil Engineering, **Jaypee University of Information Technology, Waknaghat** is an authentic record of work carried out by **Tenzin (171672), Bikash Subba (171676) and Kinzang Peldon (171677)** during a period from August, 2018 to May, 2019 under the supervision of **Mr. Niraj Singh Parihar and Dr. Amardeep**, Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat.

The above statement made is correct to the best of our knowledge.

Date: 17/05/2021…………………

Mr. Niraj Singh Parihar

Dr. Amardeep
Assistant Professor
Department of Civil Engineering
JUIT, Waknaghat

HOD
CE DEPT

Dr. Ashok Kumar Gupta
Professor and Head
Department of Civil Engineering
JUIT, Waknaghat

External Examiner

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ABSTRACT

Bhutan has may mountainous area, fills and slopes where many buildings, resorts and highway are being constructed. However numerous slopes are unstable and need extra attention. Among the major disaster, slope is one that affects both economic and transportation services in Bhutan which need engineers to work on to avoid the occurrence of landslide with suitable method to stabilize the slope. Soil nailing is one of the effective method to mitigate the slope with simple and economically system. In this present work, GeoSlope software was used for the analysis of slope to find out the most appropriate factor of safety. Both static and dynamic considerations were taken into account. The factor of safety was determined for both without and with soil nailing by finite slope stability methods available in the software namely Morgenstern-Price, Spencer, Janbu, Bishop and Ordinary and comparison has been done. The software results showed that the slope is instable without soil nailing and it stabilize with the help of soil nail and thus soil nailing can be use as retrofitting to stabilize the slopes. Both software and manual calculation for external and internal stability were done and the factor of safety is greater than the recommended FOS.

[Key Words] Bhutan, slope stability, Soil Nail, Factor of Safety, Sorchen, Slope failure

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LIST OF ACRONYMS & ABBREVIATIONS

ASD	Allowable stress design method
GEC	Geotechnical engineering circular
M-O	Mononobe-Okabe method
ROW	Right-of-way
W	Weight of failure wedge
WWM	Welded wire mesh
a_{vm}	Vertical reinforcement cross-sectional area per unit width at mid-span
a_{vn}	Vertical reinforcement cross-sectional area per unit width at nail head
A	Normalized peak ground acceleration coefficient
A_H	Cross-sectional area of stud head
A_m	Normalized horizontal acceleration at center of block
A_{SH}	Cross-sectional area of the headed-stud shaft
A_S	Cross-sectional area of waler bars
A_t	Nail bar cross-sectional area
B	Tangent of angle to calculate orientation of failure plane behind wall
B_L	Length of horizontal failure surface along base
C	Soil effective coefficient
c_b	Soil cohesion along base of failure surface
C_F	Factor to consider non-uniform soil pressures behind the facing
D_{DH}	Average or effective diameter of the drill hole
D_H	Diameter of the stud head
D_S	Diameter of the headed-stud shaft
f_{ck}	Concrete compressive strength
f_y	Nail bar yield strength
$F_{in\ h}$	Horizontal component of force F_{in}
$F_{in\ v}$	Vertical component of force F_{in}

FS_{FF}	Factor of safety against facing flexure failure
FS_{FP}	Factor of safety against facing punching shear failure
FS_G	Factor of safety against global failure
FS_{HT}	Factor of safety against headed-stud tensile failure
FS_P	Factor of safety against pill out failure
FS_{SL}	Factor of safety against sliding stability
FS_T	Factor of safety against soil nail tensile failure
h	Thickness of facing
H	Wall height
H_1	Effective height over which the earth pressure acts
i	Nail inclination
k_h	Horizontal seismic coefficient
k_v	Vertical seismic coefficient
K_A	Active earth pressure coefficient
K_{AE}	Total (static and dynamic) active pressure coefficient
L	Total nail length
L_{BP}	Bearing plate length
L_P	Pullout length or nail length behind the failure surface
L_S	Headed-stud length
N	Total number of nails in a given wall section
N_H	Number of headed-studs (usually 4)
P_A	Active lateral earth pressure
P_{AE}	Total active thrust
PGA	Peak ground acceleration
q_s	Distributed surface loading
q_u	Ultimate bond strength
Q_{ALL}	Allowable pullout capacity per unit length
Q_T	Surcharge load

R_F	Shear force due to punching in facing
R_{FF}	Facing flexure capacity
R_{FP}	Facing punching shear capacity
R_{HT}	Nail head capacity against tensile failure of the headed studs
R_P	Nail pullout resistance
R_T	Nail bar tensile capacity
S_H	Soil nail horizontal spacing
S_{HS}	Headed-stud spacing
S_{max}	Maximum soil nail spacing
S_V	Soil nail vertical spacing
t_H	Headed-stud head thickness
tp	Plate bearing thickness
T	Tensile force along the nail bar
T_{max}	Maximum tensile force along the nail bar
W	Weight of block considered for stability
α	Wall batter angle (from vertical)
β	Back slope angle
γ	Total unit weight of soil mass
δ	Wall-soil interface friction angle
Δk_{AE}	Coefficient of dynamic increment
Δp_{AE}	Dynamic active thrust increment
Φ	Effective angle of internal friction
ρ	Reinforcement ratio
ρ_{vm}	Vertical reinforcement ratio at mid-span
ρ_{vn}	Vertical reinforcement ratio at the nail head
ρ_{tot}	Total reinforcement ratio
ω	Angle of orientation of failure plane pseudo-static slope stability analysis
ψ	Inclination of failure plane

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Slopes can be non-natural such as cuttings and embankments for highways and railways, landscaping operations, temporary excavations, and earth dams, for development of sites, etc. Slopes will also be natural, as in hillside and valleys, coastal and river cliffs and so on. In these kinds of cases, forces exist which have a tendency to cause the soil to transport from higher points to lower points. The essential forces which cause instability of slope consists of the pressure of gravity, the pressure of seepage and earthquake and accordingly inflicting landslides (Ranjan & Rao, 2000). Landslides are one of the most dangerous geomorphological process on earth causing damages to infrastructures, economics, agricultural land and importantly causing casualties. When there is inclined surface of soil mass, the possibility of landslide occurrence is high and sliding of slope from higher point to lower point exist all the time. Instability related issues in engineered and landslides are common challenges to both researchers and professionals. Thus, this has attracted more and more research attention. Numerous guidelines and methodology has been developed by many researchers to reduce the landslide.

The landslide occurrences within the Himalayan region was remarkably high with 75% in the year 2004 to 2016 as expressed by one of the studies during an international database of landslides [1]. Bhutan is no exception to the worst landslides, and it is still obvious in many places. Earthquakes, explosive glacial lakes, heavy rains and floods are some of the contributive factors that cause landslides in Bhutan. Many rivers in Bhutan are well known and will overflow. They can cause severe floods and landslides due to rain, block national highways and disrupt transportation and communications. In early Auguts 2000, over 200 individuals were killed as many land stricken several villages within three days. One of the hardest-hit areas is the Sorkhen Bypass, 17.5 kilometers from the Phuentsholing-Thimphu highway. Kherbandi landslide, that is found 5 km on the highway has proved downside for over two decades. The slope instability problem at Jumja settled at 41 km on the road is another area affected by a landslide [2] (Figure 1.1).

Figure 1.1 Jumja Landslide (Left) (landlside.net), A frontal view of Kherbadi Landslide (Right) (Source: SAARC Disaster Management [2])

Hence, it is requisite that we understand the behavior and property of soil of that particular area to implement suitable method of stabilizing the slope. Stability of slope can be attained with multiple approaches and soil nailing is one of them in which steel reinforcement bars are inserted into the existing soil to resist the force against slope failures.

Historical origins of Soil Nail Walls [3]

The installation and pouring of metal reinforcements and the use of sprayed concrete to the ground are based on a system developed to support excavation in rock formations. This system is called as "New Austrian Tunneling Method" (Rabcewiz 1964a, 1964b and 1965). In 1972 soil nails had been used to stabilize an about 60-feet excessive cut-slope in sand and for a railroad-extending project close to Versailles in France (Rabejac and Toudic 1974). The use of soil nails has become common in France and different European international locations since the completion of the Versailles project. The first use of soil nails in earth maintaining structures in Germany came about in 1975 (Stocker et al. 1979) [3, p. 1]

1.2 CONSTRUCTION SEQUENCE [3]

The sequence of construction for a soil nail wall is briefly shown below [3, pp. 7-9] and schematically given in Figure 1.2.

Step 1. *Excavation*: The first excavation is carried out at a depth of 1m to 2 m.

Step 2. *Drilling Nail Holes*: Drill the hole to a certain (specified) length, diameter, inclination and a specified horizontal distance from the platform being dug.

Step 3. *Nail Installation and Grouting*: Nail bars are fixed in the pre-drilled hole and a grout pipe is also inserted in the drill hole at this time.

Step 4. *Construction of Temporary Shotcrete Facing:* Before the next temporary concrete is excavated, a 100 mm thick temporary reinforced concrete system should be constructed to support the ground.

Step 5. *The subsequent construction phase of this Levels:* For the next remaining earthwork and excavation, repeat the same process as the previous phase. Drain rods are installed on each mine lifts. Place the new WWM panel so that it overlaps with at least one complete grid cell.

Step 6. *Construction of a Final, Permanent Facing.* The final cladding is constructed and can be made of cast-in-place reinforced concrete, reinforced concrete or prefabricated parts.

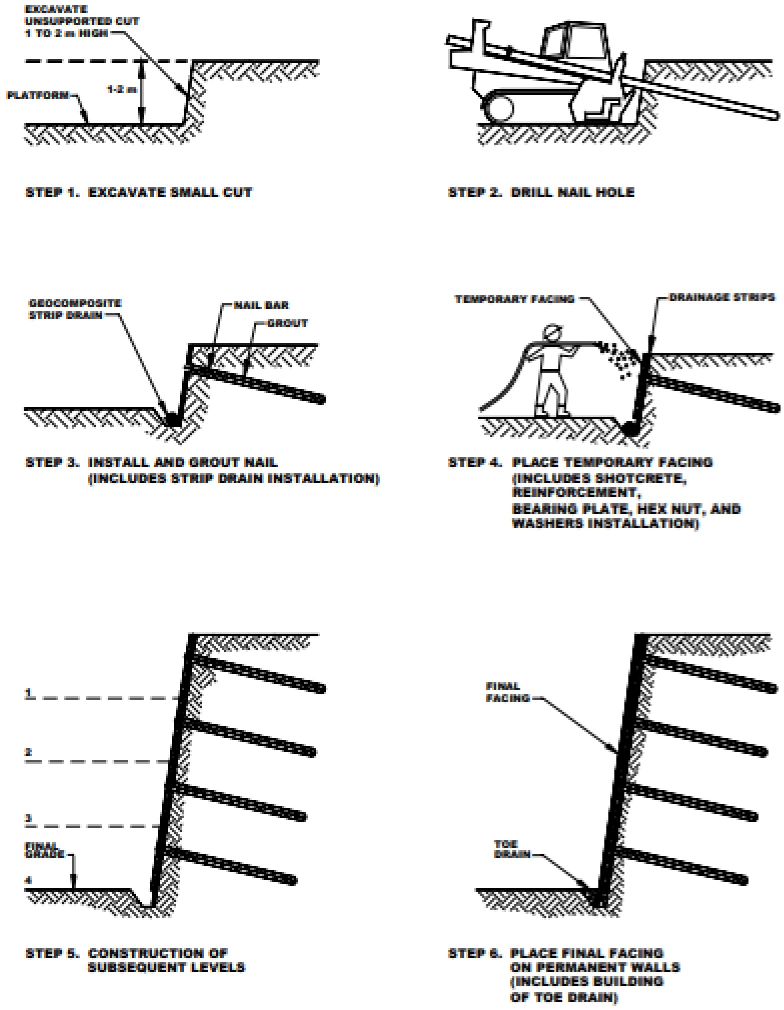

Figure 1.2 Soil Nail Wall Construction Sequences [3]

1.3 PRINCIPAL MODE OF FAILURE OF SOIL NAIL [3]

- External failure mode
- Internal failure mode
- Facing failure mode

1.4 EXTERNAL MODE OF FAILURE [3]

External failure modes refer to the development of potential failure surfaces passing through or behind the soil nails (i.e., failure surfaces that may or may not intersect the nails). The figure given in Figure 1.2 shows the external failure mode. The following external failure modes are considered in the analysis of soil nail wall systems:

- Global failure mode;
- Sliding failure mode (shear at the base); and
- Bearing failure mode (basal heave)

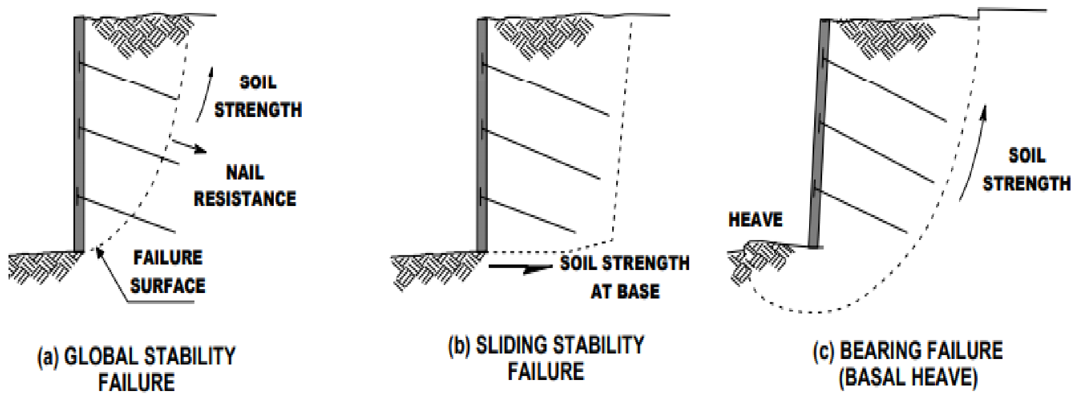

Figure 1.3 External mode of failure [3]

1.4.1 GLOBAL STABILITY [3]

Global stability refers to the overall stability of the wall quality which is enhanced with soil nail walls. In this failure mode, when the nails intersect, the retained mass exceeds the soil resistance along the sliding surface and the nail. This happens when the nail is too short to pass through the sliding surface. Where the reinforcement does not connect the ground to the stable area below [3, pp. 68-71]. The minimum factor of safety recommended is given in Table 1.1

To illustrate the elements of a global stability analysis for soil nail walls, a simple, single-wedge failure mechanism is shown in Figure 1.3.

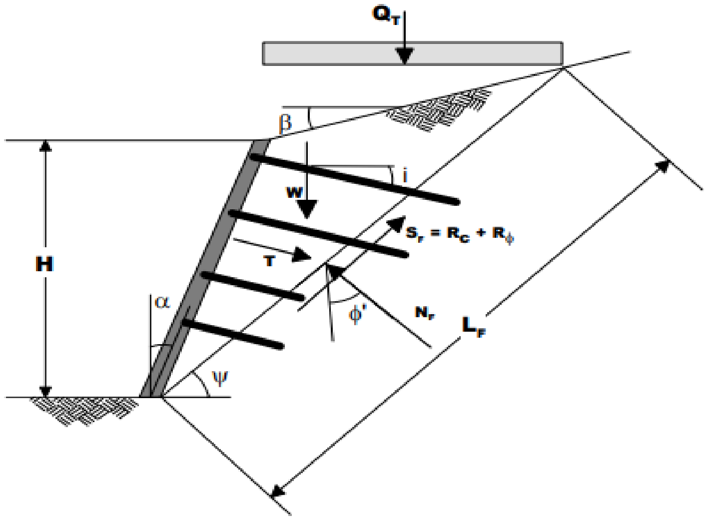

Figure 1.4 Global stability analysis of soil nail wall using a single-wedge failure mechanism [3].

The factor of safety against global failure (FS_G) is expressed as the ratio of the resisting and driving forces, which act tangent to the potential failure plane [3]:

$$FS_G = \frac{\Sigma \text{ resiting forces}}{\Sigma \text{ driving force}}$$

$$FS_G = \frac{cLF + Teq\cos(\psi - i) + [(W + QT - Fv)\cos\psi + Teq\sin(\psi - i) Fh\sin\psi]\tan\phi}{(W + QT - Fv)\sin\psi + Fh\cos\psi}$$

$T_{eq} = \frac{1}{Sh}\sum_j^n (Tall)$, T is minimum of R_T or R_P in kN/m

$$\psi = 45 + \frac{\phi}{2}$$

$$(R_P)_z = \frac{\pi DLpqu}{1000}$$

$$(R_T)_z = \frac{0.25\pi d2fy}{1000}$$

Seismic Effects on Global Stability [3]

The impact of earthquakes on slope stability needs to be assessed. Since Bhutan is located between the IV and V seismic zones, the impact of the earthquake is very large. Therefore, in order to analyze the impact of earthquakes, we need to calculate the factor of safety. For the seismic stability analysis of soil nail walls the pseudo-static method is used where an equivalent, pseudo-static force (F_{in}) act at the center of gravity of the analyzed block. The horizontal and vertical components $F_{in\ h}$ and $F_{in\ v}$ (input parameters in computer program) respectively are define as [3, pp. 78-79]:

$$F_{in\ h} = k_h W$$

$$F_{in\ v} = k_v W$$

Where;

W = weight of the block.

K_h = a non-dimensional horizontal seismic coefficient

K_v = a non-dimensional vertical seismic coefficient

A_m is a function of the normalized peak ground acceleration coefficient (A), which is the actual peak ground acceleration normalized by the acceleration of gravity (g) and is defined as [3]:

$$A_m = (1.45 – A)\ A$$

Where;

A_m = normalized horizontal acceleration.

A = peak ground acceleration coefficient (A depends on regional tectonic setting and are obtained from seismic maps)

For wall height greater than approximately 15 m (45 ft.) and the peak ground acceleration coefficient $A \geq 0.3$, a seismic coefficient for soil nail is given by:

$$K_h = 0.5\ A_m \text{ to } 0.67\ A_m$$

Table 1.1 Minimum recommended factor of safety for global stability, FSG [3]

Temporary Walls		Permanent Walls	
Static	Seismic	Static	Seismic
1.35	1.10	1.35	1.10

1.4.2 SLIDING STABILITY [3]

The sliding stability analysis considers the potential of the soil nail wall to withstand the lateral soil pressure behind the soil nail and to resist sliding along the base of the restraint system. If the additional lateral soil pressure caused by the excavation exceeds the sliding resistance along the foundation, slip damage may occur (Figure 1.4). This is usually not the case, but the possibility of determining weak soil layers should be considered. The minimum recommended factor of safety for sliding stability is given in the Table 1.2.

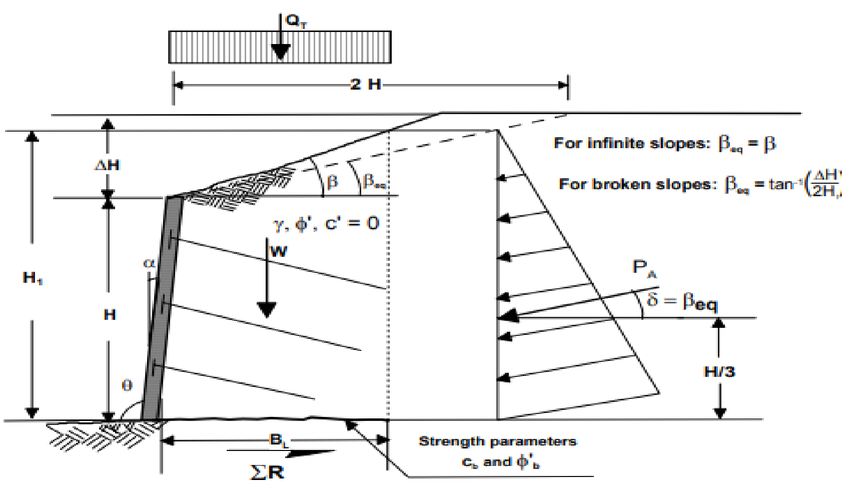

Figure 1.5 Sliding stability of a soil nail wall [3]

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

$$FS_{SL} = \frac{CbBL+(W+QT-Fv+P\sin\beta eq)\tan\phi b}{Fh+p\cos\beta eq}$$

Table 1.2 Minimum recommended factor of safety for sliding stability, FS_{SL} [3]

Temporary walls		Permanent walls	
Static	Seismic	Static	Seismic
1.30	1.10	1.50	1.10

Seismic Effects on Sliding Stability [3]

The sliding stability of soil-nailed walls under seismic loads takes into account the total active thrust (PAE) generated during the earthquake due to the earth pressure behind the soil block. This force is a combination of dynamic and static active lateral ground pressure generated by inertial force.

In general, the total active thrust, P_{AE}, given by (Ebiling and Morrison, 1992) is expressed as:

$$P_{AE} = \frac{\gamma H1^2}{2} K_{AE} (1-k_v)\{1+\frac{2qs}{\gamma H1}[\frac{\cos\alpha}{\cos(\beta-\alpha)}]\}$$

Where;

γ = total unit weight of soil behind block

H_1 = effective height of soil mass that considers sloping ground

K_{AE} = total (static and dynamic) active pressure coefficient

Q_s = distributed surface loading

In general case of a wall, the active pressure coefficient is calculated using M-O formula which is given by:

$$K_{AE} = \frac{\cos^2(\phi-\omega-\alpha')}{\cos\omega \cos^2\alpha' \cos(\alpha'+\delta+\omega)D}$$

Where;

Φ = angle of internal friction of soil behind wall

α' = batter angle (from vertical) of wall internal face

β = back slope angle

δ = wall-soil interface friction angle

ω = an angle relating the horizontal and vertical seismic coefficient

$$\omega = 1/\tan(\frac{kh}{1-kv})$$

$$D = [1+\sqrt{\frac{(\sin\phi+\delta)\sin(\phi-\omega-\beta)}{\cos(\delta+\alpha^2+\omega)\cos(\beta-\alpha')}}]$$

Note: Since we have horizontal backfill slope, the following formula is used to calculate total dynamic active thrust [3].

a) Calculate inertia forces:

$$H_1 = [1 + (\frac{L}{H})\tan\beta]\ H$$

$$B = [(\frac{L}{H}) + \tan\alpha\ ]\ H$$

Where;
H = wall height
L/H = nail length to height ratio
β = back slope angle
α = wall batter angle

The equivalent, pseudo-static forces (F_I and F_{II}) from lower block (W_1) and upper block(W_1) are given by:

$$F = F_I + F_{II} = \frac{H^2\gamma}{2} A_m\ [0.5\tan\alpha + (\frac{L}{H}) + 0.5(\frac{L}{H})^2\tan\beta\ ]$$

The vertical seismic coefficient is neglected as $k_v = 0$ and assumes a horizontal seismic coefficient equal to one-half of the design coefficient of acceleration (i.e., $k_h = 0.5\ A_m$)

b) Calculate seismic active forces:

The dynamic active thrust increment, P_{AE}, which acts behind the wall-nailed soil block for horizontal backfill is given by:

$$\Delta P_{AE} = 0.375\,\frac{\gamma H1^2}{2} A_m$$

Where;

γ = total unit weight of soil behind block.

H_1 = effective height of soil mass that considers sloping ground.

K_v = the vertical seismic coefficient.

K_{AE} = total (static and dynamic) active pressure coefficient.

q_s = distributed surface loading.

The coefficient of dynamic increment recommended by Seed and Whitman (1970) for horizontal backfills is calculated as [3]:

$$\Delta K_{AE} = \frac{3}{4}\,k_h$$

Thus, the total active force is the combination of static and dynamic active thrust increment:

$$P_{AE} = P_A + \Delta P_{AE}$$

The total active pressure coefficient K_{AE} can also be determined from graph given by M-O solution.

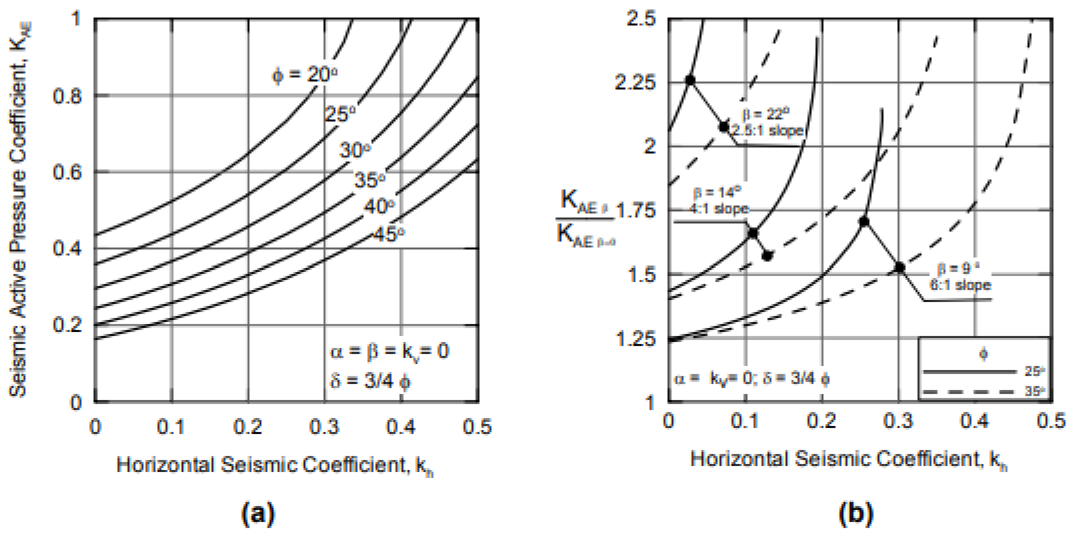

Figure 1.6 Total active pressure coefficients: (a) Horizontal back slope; (b) Non-horizontal back slope [3]

1.4.3 BEARING STABILITY [3]

Although the bearing capacity is low, problems may arise when soil nails are bored in soft, fine-grained soil. Since the wall facing does not reach below the bottom of the excavation (unlike the piles welded to the cantilever anchor wall or welded to the ground), the unequal and unstable load inflicted by the excavation will cause the soil to move and affect the load-bearing capacity of the foundation.

1.5 INTERNAL FAILURE MODES [3]

The internal failure mode refers to the failure of the load transfer mechanism between soil, nails, and mortar. When the nail system deforms from the soil during the excavation process, the soil nails will mobilize the bond strength between the mortar and the surrounding soil. The binding force is gradually mobilized in the entire soil nail in a certain distribution, which is affected by many factors. When the bond strength on the nail is mobilized, it will produce tension. The types of internal failure modes related to the soil nail are listed below:

1.5.1 NAIL PULLOUT FAILURE [3]

"Nail pullout failure is a failure along the soil-grout due to insufficient intrinsic bond and/or insufficient nail length" (Carlos, 2015).

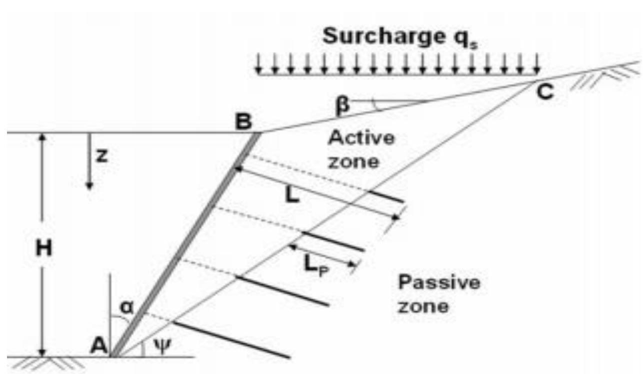

Figure 1.7 Effective length [4]

Nail soil pullout failure is calculated as below:

$$(FS_P)_z = \frac{(Rp)z}{(Tmax)z} = \frac{(QuLp)z}{(Tmax)z}$$

$$(Tmax)_z = K(q_s+ɣz)S_H S_V$$

$$Q_u = \pi q_u D_{DH}$$

$$(L_P)z\ [m] = L - [\frac{(H-z)\cos(\psi-\alpha)}{\cos\alpha \sin(\psi+i)}]$$

Table 1.3 Minimum recommended factor of safety for nail pullout failure, FS_p [3]

Temporary wall		Permanent wall	
Static	Seismic	Static	Seismic
2.00	1.50	2.00	1.50

1.5.2 SLIPPAGE OF THE BAR-GROUT INTERFACE

The sliding resistance along the interface between the mortar and the steel rod is mainly provided by the mechanical bonding between the protrusions and "depressions" of the mortar on the surface of the nail rod. When using threaded bars, mechanical locking provides considerable strength, while for smooth rods it can be ignored [3, p.83].

1.5.3 TENSILE FAILURE OF THE NAIL

If the tensile strength is insufficient, the nail may fail in tension or stress. [3, p. 83].

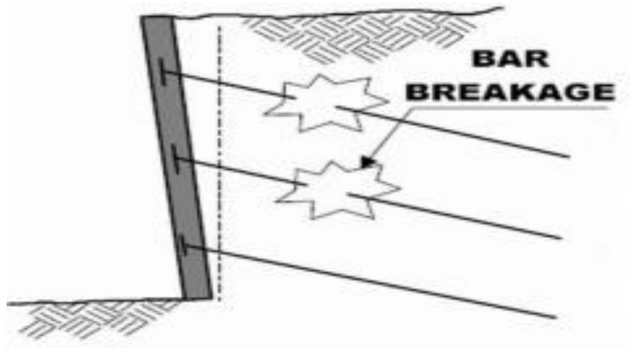

Figure 1.8 Nail Tensile Failure [4]

Nail tensile strength is calculated is below [5]:

$(FST)z = \frac{(RT)z}{(Tmax)z}$

Where:

$R_T = A_t f_y$ = maximum axial tensile load capacity of nail

A_t = c/s area of nail

f_y = yield strength of nail

Table 1.4 Minimum recommended factor of safety for tensile failure of nail [3]

Temporary wall		Permanent wall	
Static	Seismic	Static	Seismic
1.80	1.35	1.80	1.35

1.6 FACING MODE OF FAILURE [3]

The most common potential failure modes at the facing-nail head connection are:

- *Flexural Failure*: This is a failure mode due to excessive bending beyond the facing's bending capacity. For temporary facings and permanent facings, this type of failure should be considered separately. ·

- *Punching Shear Failure:* Around the nails, temporary and permanent coverage inspections should be performed.
- *Headed-Stud Tensile Failure:* This is a failure of the headed studs in tension. Therefore, the failure mode is only important for permanent coatings.

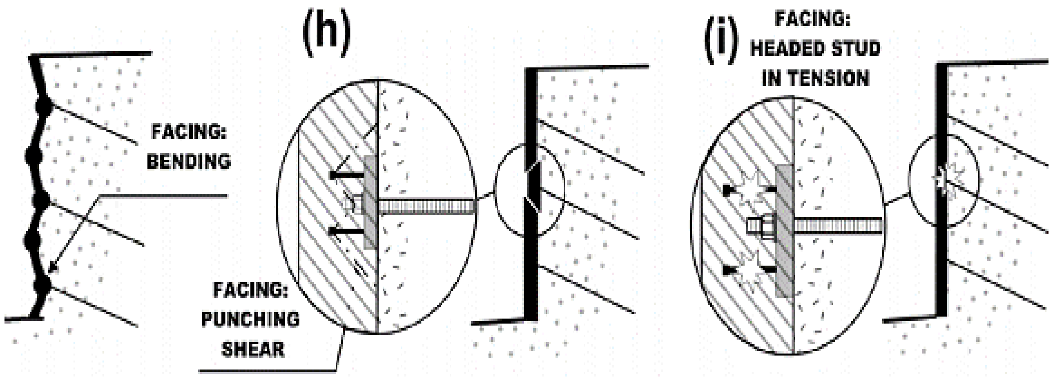

Figure 1.9 Facing flexural failure, facing punching shear failure and headed stud in tension [3]

- Facing flexure failure, $FS_{FF} = \frac{\mathrm{RFF}}{\mathrm{To}}$
- Facing punching shear failure, $FS_{FP} = \frac{RFP}{To}$

Table 1.5 Minimum recommended factor of safety [3]

Failure Mode	Static Loading		Seismic Loading
	Temporary walls	Permanent walls	Both temporary and permanent walls
Facing flexure, FS_{FF}	1.35	1.50	1.10
Facing punching shear failure, FS_{FP}	1.35	1.50	1.10

1.7 ADVANTAGES AND LIMITATIONS OF SOIL NAIL

1.7.1 ADVANTAGES [3]

There are numerous benefits of soil nail wall comparing to ground anchors and alternative top-down construction techniques. Some of them are:

- Anchors on the ground require less ROW than nails on the ground, because the nails on the ground are usually shorter.
- Compared with other construction methods, it has less interference to traffic and less harm to the environment. ·Drilling holes in the ground is less digging than digging.
- It is relatively quick to install the wall with ground nails. ·
- When encountering obstacles such as rocks, stakes or underground utilities, you can easily adjust the inclination and position of the nails.
- The fixing of the soil nail wall is not restricted by air restrictions like welded piles. This advantage is especially important when building in the under bridge.
- At further places, fixing the floor with nails can reduce costs because smaller equipment is easier to move.
- Soil-nailed walls are fairly flexible and can withstand larger overall and changing movements.
- In highway construction, under proper construction supervision, the measured deformation of the nail wall is usually within an acceptable range. ·
- During an earthquake, a wall with nails on the ground works well. ·
- A wall with spikes on the ground is more flexible and can withstand different sediments.
- Soil nail walls are economical and less costly.

1.7.2 LIMITATIONS [3]

- Walls with soil nails may not be suitable for applications that require very strict control of structural deformation.
- The connection behind the wall may limit the position, inclination, and length of the nails (especially the upper row).
- If a huge quantity of groundwater seeps into the pit, it is not suitable·
- Permanent walls made of nails necessitate permanent underground easement. ·
- Professional and experienced contractors are needed to build soil nail walls.

1.8 APPLICATIONS OF SOIL NAIL WALL [3]

Soil nail walls are most suitable for excavation in soil conditions that need vertical or near-vertical cutting, such as roads, retaining structures and embankments. Some uses of soil nails are listed below: ·

- Excavation of road works.
- Widen the road under and at the end of the existing bridge.
- Repair and rebuild the existing retaining structure.
- Temporary or permanent excavation in urban areas.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

This chapter includes the critical analysis of different books, articles and journals about the stability of slope using different software. Several literature has been collected and evaluated for better understanding of slope stabilization using soil nail wall. Through many paper, it has helped us in knowing how slope is being stabilized by increasing factor of safety using soil nail wall. The summary of different literature review is shown below.

2.2 REVIEW OF LITERATURE

1. **Melo and Sharma [6]**. Parametric studies were conducted to analyze the impact of earthquakes on embankments and slopes with strong ground motion. The dynamic behavior of the slope model is used to calculate the seismic coefficients time histories of critical failure suface from the final equilibrium analysis. The study had tried to find the approach for selecting seismic coefficients for pseudo-static analysis.

2. **Babu and Singh [7].** The study has performed laboratory test on soil nail wall under seismic conditions and it was said that the soil nail wall rendered greatly under seismic conditions. Moreover, several studies like maximum lateral displacement, failure modes and development of nail forces were studied under both static and seismic conditions.

3. **Nadher Hassan AI-Baghdadi [8].** The study used the commercial software "SLIDE 6" to conduct a parameter study, which uses different method to solve the problem of slope stability. The Bishop method was used to analyze slopes without nails and with nails for granular soil, different heights and angles of inclination. Various parameters of the nails were checked: the position of the nail, the length of the nail, the angle of the nail and the distance between the nails. It was found that the nail inclination between 10-15 degrees

gives the best optimum value from the horizontal line, but it was related to the angle of inclination. It has been found that the distance between the nails is 1 m, which provides the best improvement in terms of factor of safety.

4. **Vivek, et al., [9]**. The project was analyzed for the slope stability in Jammu, Himachal and Punjab region soil using Geo Studio 2007 slope/w software. As the slope failure become most frequent geological catastrophes along the road network in the hilly terrain of Himalayan regions that lead to huge loss of life, property and above all the environment. Thus using the Geo Studio 2007 slope/w software, slope stability was determined. The results showed that the factor of safety of the slope stability increases with an increase in cohesion and internal friction angle,

5. **Rawat and Gupta [10]** experimented to see the behavior of unreinforced and reinforced with soil nailing under gradual increasing surcharge load at an angle of 45˚ and 60˚ of soil slope with soil nail at an inclination of 0˚, 15˚ and 30˚. The failure pattern and load-settlement plots for the various unreinforced and reinforced slopes were analyzed using finite element software PLAXIS 3D and a comparable study with the experimental data were carried out. It was reported that the failure mechanism for the various slopes were similar to the failure pattern from the model testing. It was reported that the soil nail at an angle 15° gives the best slope stabilization where the factor of safety increases from 0° to 15° but decrease from 15° to 30°

6. **Sarkar, et al., [4].** This document aimed to create the possibility of constructing walls with nails in Sorchen, 17.5 km from Phuentsholing, along Phuentsholing-Thimphu highway in Bhutan. After evaluating the site, they proposed an applicable design of soil nail wall at Sorchen bypass which was verified by SNAP_2 software developed by Federal Highway Administration of United States Department of Transportation.

7. **Singh and Srivastava, A.K [11]** conducted an experiment to see the response of the unreinforced and soil nailed slopes under different static surcharge load for 60˚ slope of sand size soil. These soil slopes were then reinforced by installing soil nails at three

different inclinations of 0˚, 15˚ and 30˚ with the horizontal plane keeping the horizontal and vertical spacing = 0.1m. The effect of soil nail pattern in the soil slope was also analyzed i.e., nails were installed in square, diamond and staggered arrangements. It was reported that nails inserted at 0˚ were more efficient in providing the stability to the slopes as compared to nails inserted at 15˚ and 30˚. Also nails installed in staggered pattern were found to be most efficient.

8. **Nalgire, et al.,** [**12].** Stability analysis was performed by Talha Nalgire’s group for the dump slope of WLC Makardhokara-2 open cast mine in Umred, District, Maharastra, India which was reported dump as failure. Slope stability analysis by seven finite slope stability methods namely Morgenstern-Price method, Spencer, Sarma, Bishop, Janbu and Ordinary method was done using GeoSlope software for the dump slope in Markardhokara site. They have analyzed for both conditions i.e. unreinforced and reinforced with soil nail. It was observed that with the increase in slope angle, the slope stability gets affected. With varying soil nail diameter FOS doesn’t affect much however, nail spacing has its effect on the FOS of the slope.

9. **Rawat and Chatterjee** [**13]** has experimented the seismic stability of slopes using soil nails at angle of 10°, 15° and 30° which was subjected to tensile stresses. With the application of loading, the global factor of safety was calculated. They have reported that with increase in seismic acceleration coefficients and nail inclination, the factor of safety decreases.

10. **Abbas and Mutiny [14].** In this study, the factor of safety of upstream slope stability for number of exist earth dams was assessed using GeoSlope SLOPE/W program. They have taken into account the case of dry condition and gradually rise of water level in upstream part for those dams. It was seen that the slope stability of earth dams increased in case of dry condition. In addition, the slope stability increased when the water reached to maximum value. The factor of safety calculated with different methods didn’t have insignificant differences for all methods.

11. **Dewedree and Jusoh [15].** In this paper, different soil nail inclinations were applied at two different locations (that is Genting Highlands and Pahang) and factor of safety (FOS) was evaluated using SLOPE/W software. The results showed that the inclination of soil nails has an important influence on the stability of the slope. For slopes with an inclination of 30, 45° and 60°, the best factor of safety for inclined nails were 60, 50° and 40° respectively. When the inclination of soil nails is between 5° and 20°, the impact on factor of safety is small.

12. **GEO-SLOPE International Ltd [16]**. The journal shows that soil nails are a kind of reinforcement that can be modeled with SLOPE/W. It has been shown that it is best to distribute the nail forces amongst all of the slices interconnected by the the line of action, resulting in better convergence. A user-defined resistance reduction factor can be used to consider the nail pull based on the calculated safety factor.

2.3 SUMMARY OF LITERATURE REVIEW

Soil nailing inherently comprise of reinforcing the slopes and retaining walls to strengthen the soil stability by insertion of soil nail. To enhance stability of vertical cut faces or natural slope, application of soil nail is increasing for the past decades.

Many research paper has been performed to study the behavior of the unreinforced and reinforced with soil nail for both static loading or dynamic loading with various software. All the parametric study and experiment has been done to analyze the unstable slope, embankment and vertical cut with from top-down construction. Numerous previous paper has tried with different approaches to find out the best factor safety. It includes variation of loading, variation of nail length and diameter, change in slope angle and nail inclination and variation in soil nail spacing.

All most all the papers showed that the change in those data greatly vary its factor of safety. The optimum factor of safety almost always lies between the angle 0°, 10° and 15° and beyond 30°, there is no effect of soil nail. Moreover, it was observed that the factor of safety of the slope stability increases with the increase in length and diameter of the nail.

Though multiple approach to stabilize the slope has been carried out by the earlier researcher, the present studies attempt to make the slope stable with optimum angle of soil nail and make it more economical and cost effective.

2.4 OBJECTIVES OF THE STUDY

The objectives of the study are:

i. To find the stability of slope with or without application of soil nail.
ii. To analyze the effect of soil nailing using GEOSLOPE software.
iii. To find out the optimize angles of nail installations.

2.5 SCOPE OF THE STUDY

The scope of the study is to analyze the behavior of the slope with reinforcement and without reinforcement (soil nail) to determine the stability of slope as soil nailing is broadly used as a remedies to make the slope stable. Additionally, different angle of nail inclination will be performed and find out the optimum nail angle that stabilize the slope. It also includes the study of behavior of slope under seismic effects.

CHAPTER 3

METHODOLOGY

3.1 GENERAL

This chapter presents the detail method on soil nail wall design using GeoSlope software. Since the in-situ and field investigation is impossible due to pandemic, the necessary data required to compute the important mode of failure of soil nail and to compute factor of safety is collected from the past research papers. Thorough calculation and design are presented here.

3.2 METHODOLOGY

1) The project is all based on the previous data available for the given site and it involves in determining some correlation data. To proceed the work, numerous literatures were reviewed for better understanding the process involved in designing the soil nail wall, its application and method available for soil nailing.
2) The available data that are essential for computing the factor of safety for Sorchen site was collected from past research paper (Raju Sarkar, et al., 2017) and (Sangay Dema, et al., 2017).
3) Computation of unavailable data by using the relation available for the input data in Geo-Slope software.
4) Analyzing the slope stability of Sorchen by finding the factor of safety with different methods including Morgenstern-Price method, Spencer method, Janbu method, Bishop method and Fellenius method and comparing with FOS given by each method in software.
5) Preparation of detailed project report which includes the result and its analysis from the result of software.

3.3 DATA COLLECTION

Field reconnaissance was done for the unstable slope of Sorchen to determine drainage pattern, landslide pattern and vegetation by past researchers. The important data required to compute the soil nail design are abstracted from previous research and thus soil profile of the site obtained is given in the figure 3.1 and figure 3.2.

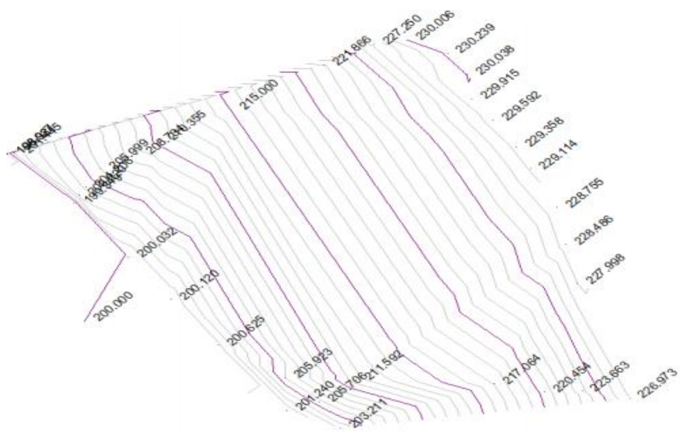

Figure 3.1 Profile of study area [4].

Depth(m)	Resistivity(ohm-m)	Type of soil
Top		
1	1067.60	Varying proportion of sand/gravel
2	1632.80	Gravely silt
3	2505.72	Gravely silt
Middle		
1	910.60	Varying proportion of sand/gravel
2	653.12	
3	866.64	
4	678.24	
5	659.4	
6	640.56	
7	615.44	
Toe		
1	558.92	Varying proportion of sand/gravel
2	628	
3	546.36	
4	628	
5	590.32	
6	547.8672	

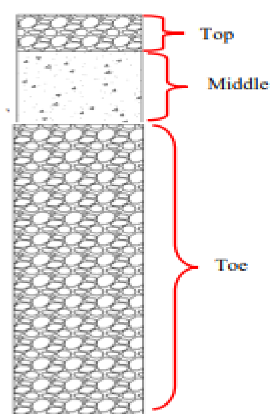

Figure 3.2 Soil Profile of the site [4].

The following data were collected for input data [4]:

- Height of the slope = 30 m
- Angle of the slope = 60˚

The stability analysis for the site is to be performed in two parts where the factor of safety without soil nailing and with soil nailing is to be determined. In order to calculate the dimensional stability of soil nail wall, we need the parameters of soil. The geo-parameters required for the computation in software are given in Table 3.1 obtained from past research paper of College of Science and Technology [4]. Additional data required to compute the seismic factor of safety, the normalized peak ground acceleration coefficient is collected from one online Kuensel map which shows earthquake hazard for Bhutan with 2% probability in the next the years. Bhutan currently use (A=0.36g) for building code in entire country. Figure 4.3 shows the map of earthquake hazard for Bhutan.

Table 3.1 The soil properties obtained from field [4].

Soil Depth(m)	Soil Layer	Internal Friction (ϕ)	Cohesion (c)	Unit Weight kN/m^3
7	Top Layer	20.75	17.32	21.5
14	Middle Layer	17.82	2.86	19.6
9	Bottom Layer	19.67	1.49	19.4

3.4 DESIGN CALCULATION

Soil Nail Wall Design Consideration:

Table 3.2 Design Parameters

Sl. No.	Parameters	Recommendation as per Soil Nail walls reference manual (GEC 07)	Parameters adopted in the design
1	Height of wall, H	Field data	30 m
2	Nail length, L	0.6H to 1.2H	0.7*30 = 21 m
3	Nail inclination, i	10 to 20	10°

4	Drill hole diameter, D_{DH}	100mm to 200mm	150 mm
5	Cantilever distance	0.61m to 1.1m	1 m
6	Nail spacing	1.22m to 1.83m	1 m
7	Ultimate bond strength, q_u	103-179 kN/m	139 kN/m
8	Cohesion, c	Field data (CST-RUB)	20.7
9	Internal angle of friction, ɣ	Field data	19.41°
10	Unit weight	-	20.5
11	Slope angle	-	60°
12	Batter angle	-	0
13	Back slope angle	-	0
14	Surcharge load	Min. surcharge load = 250psi	12 kN/m^2
15	Temporary facing thickness	100 mm to 200 mm	100 mm
16	Permanent facing thickness	150 mm to 250 mm	150 mm
17	Concrete strength, f_{ck}	21 MPa to 28 MPa	28 N/mm^2
18	Bearing plate geometry	200*200 mm to 19 mm - 25 mm thick	200*200*25
19	Horizontal seismic coefficient, k_h	-	0.36 (Source: https://kuenselonline.com/bhutans-earthquake-hazard-and-damage-probabilities/)
20	Vertical seismic coefficient, k_v		0

The deign calculation is based on the parameters that are obtained from trial and error tests.

Design assumptions are:

- 10 m height each of vertical stepped (3 steps).
- Implement top to down construction procedure.
- Adopt rotary drill technique to drill hole.
- Take temporary (initial) facing (h_i) of 100 mm thick.

- Provide final facing (h_f) of 150 mm thick.
- Provide threaded solid bars of yield strength 415 MPa.
- Consider no ground water was encountered in any of the borings.
- No corrosion protection is taken into consideration.

3.4.1 PRELIMINARY DESIGN

a) Determine the maximum axial force, T_{max}

T_{max} (kN) = $k_a(q_s+\gamma H)S_H S_V$

Where;

$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin26.75}{1+\sin26.75} = 0.379$

T_{max} = 0.379 (12+20.5*10) *1*1 = 82.243 Kn

b) Determine minimum nail length, L and nail diameter, d

Factor of safety against nail tensile failure, FS_T = 1.80 [3]

The required cross-sectional area A_t of the nail bar is determined as:

$A_t = \frac{TmaxFST}{fy} = \frac{82.243*1.80*1000}{415} = 356.72\ mm^2$

We know:

$$A_{ST} = \frac{\pi.d^2}{4}$$

$$d = \sqrt{\frac{A_{ST}*4}{\pi}}$$

$$d = \sqrt{\frac{356.72*4}{\pi}}$$

d =21.3 mm

$$A_{ST(Provided)} = \frac{\pi*(25)^2}{4} = 490.87 > 356.62\ mm^2 \text{ (okay)}$$

Hence;

Adopt nail diameter, d = 25 mm

Nail length, L = 10 m (each steps)

3.4.2 FINAL DESIGN

1. External Failure Mode

I. Global Stability

$$FS_G = \frac{\sum R}{\sum D} = \frac{T_{eq}\cos(\psi\text{-}i)+[(W+QT)\cos\psi+Teq\sin(\psi\text{-}i)]\tan\phi}{(W+QT)\sin\psi}$$

$FS_G \geq 1.5$

Where:

T_{eq} = Equivalent nail force

$T_{eq} = = \frac{1}{S_h}\sum_{j=1}^{n}(Tall)j$

W = Weight of failure wedge

$W = 0.5\gamma H^2\cot(\psi)$

$R_P = \pi d L_P q_u$

$\psi = 45° + \frac{\phi}{2} = 45° + \frac{26.75}{2} = 58.375°$

$L_P = L - [\frac{(H\text{-}z)\cos\psi}{\sin(\psi+i)}]$

$R_T = \frac{\pi d^2 f_y}{4*1000} = \frac{\pi *25^2*415}{4*1000} = 203.71$ KN

Where:

R_P = Nominal pullout resistance

L_P = Pullout length or soil nail or sol nail length behind slip surface

ϕ = Friction angle of soil

z = Distance from the top to first nail

R_T = Nominal tensile resistance of tendon

Allowable axial force carrying capacity T_{all} of all nail embedded at depth z is the minimum of R_P and R_T. The estimated ultimate bond shear strength for soil nails in Coarse-Grained is given in Appendix Table A.5.

Table 3.3 Allowable axial forces carrying capacity of the nail T_{all}

Nail No. j (From Top)	Depth of nail, z (m)	Effective pullout length, L_P (m)	Nail pullout capacity, R_P (kN)	Nail tensile capacity, R_T (kN)	Allowable axial forces carrying capacity of the nail, Tall (kN)
1	1	15.92	172.49	203.71	172.49
2	2	16.49	178.66	203.71	178.66
3	3	17.05	184.73	203.71	184.73
4	4	17.62	190.91	203.71	190.91
5	5	18.18	196.97	203.71	196.97
6	6	18.74	203.04	203.71	203.04
7	7	19.17	207.69	203.71	207.69
8	8	19.87	215.28	203.71	215.28
9	9	20.44	221.46	203.71	221.46
				$\sum Tall =$	1771.23

From table 5.2, T_{max} = 1,771.23 kN

a) *First wall*

$$T_{eq} = \frac{1771.23}{1} = 1771.23 \text{ kN}$$

$W = 0.5*21.5*10^2*\cot(58.375) = 661.99$ kN/m

BL= Length of horizontal slip surface (sliding stability)

$QT = q_s*B_L=12*21 = 252$ kN/m

$$FS_G = \frac{1771.23\cos(58.375-10)+[(661.99+252)\cos58.375+1771.23\sin(58.375-10)]\tan20.75}{(661.99+252)\sin(58.375)}$$

= 2.7 > 1.5 (Okay)

b) *Second wall*

$T_{eq} = 1771.23$kN

$W = 0.5*19.6*10^2*\cot(58.375) = 603.49$ kN/m

$Q_T = 10*19.6+12 = 208$ kN/m^2

$$FS_G = \frac{1771.23\cos(58.375-10)+[(603.49+208)\cos58.375+1771.23\sin(58.375-10)]\tan17.82}{(603.49+208)\sin(58.375)}$$

$= 2.52 > 1.5$(okay)

c) *Third wall*

T_{eq}= 1771.23 kN

$W = 0.5*19.4*10^2*\cot(58.375) = 597.33$

$Q_T = 10*19.4+208 =402$ kN/m2

$$FS_G = \frac{1771.23\cos(58.375-10)+[(402+597.33)\cos58.375+1771.23\sin(58.375-10)]\tan19.67}{(402+597.33)\sin(58.375)}$$

= 2.16 > 1.5 (okay)

Seismic Effects on Global Stability [3]

The seismic effects on global stability is mainly computed in software to find the factor of safety. The value of normalized peak ground acceleration coefficient (A) is obtained from the past data of Bhutan (Refer Table 5.1)

$A_m = (1.45 – A)\ A$

$A_m = (1.45 – 0.36)\ 0.36 = 0.3924$

Therefore;

$k_h = 0.5\ A_m$

$k_h = 0.5 * 0.3924 = 0.1962$

Where;

A = Normalized peak ground acceleration coefficient

A_m = Normalized horizontal acceleration at center of block.

k_h = Horizontal seismic coefficient

II. Lateral Sliding Stability

$$FS_{SL} = \frac{c_b B_L + (W+QT+PA\sin\beta)\tan\phi_b}{P_A\cos\beta} \geq 1.5$$

Where:

C_b=Cohesion of soil along base of soil block (sliding stability)

W= Weight of failure wedge

P_A = Active earth pressure (KN/m)

$P_A = \frac{1}{2}\ Ka\gamma H^2$

W = Unit weight $*$ Area of sliding wedge

$Q_T = q_s * B_L$

a) First wall

$P_A = \frac{1}{2}\ Ka\gamma H^2 = \frac{0.379 * 21.5 * 10^2}{2} = 407.425 KN/m$

W = 21.5*(21*10) = 4,515 KN/m

$Q_T = q_s * B_L = 12*21 = 252\ kN/m^2$

$$FS_{SL} = \frac{0*21+(4515+252+407.425+\sin 0°)*\tan 20.75}{407.425\cos 0°} = 4.81 > 1.3 \text{ (Okay)}$$

b) *Second wall*

$P_A = 0.5*0.379*19.6*10^2 = 371.42$ KN/m

W = 19.6*(21*10) = 4,116 KN/m

$Q_T = 10*19.6+12 = 208$ kN/m^2

$$FS_{SL} = \frac{0*21+(4116+208+371.42+\sin 0°)*\tan 17.8}{371.42\cos 0°} = 4.06 > 1.3 \text{ (Okay)}$$

c) *Third wall*

$P_A = 0.5*0.379*19.4*10^2 = 367.63$ kN/m

W = 19.4*(21*10) = 4074 kN/m

QT = 10*19.4+208 = 402 kN/m^2

$$FS_{SL} = \frac{0*21+(4074+402+367.63+\sin 0°)*\tan 19.67}{367.63\cos 0°} = 4.71 > 1.3 \text{ (Okay)}$$

Seismic Effects on Sliding Stability

a) Calculate inertia force

$H_1 = [1 + (\frac{L}{H}) \tan\beta]$ H

$H_1 = [1 + \frac{21}{10} \tan(0)]*10 = 10$ m

$B = [(\frac{L}{H}) + \tan\alpha]$ H

$B = [\frac{21}{10} + \tan(0)]*10 = 21$ m

Where;

H = Wall height

L/H = Nail length to height ratio

β = Back slope angle

α = Wall batter angle

Calculate the equivalent, pseudo-static inertia forces:

$$F = F_I + F_{II} = \frac{H^2\gamma}{2} A_m [0.5\tan\alpha + (\frac{L}{H}) + 0.5(\frac{L}{H})^2\tan\beta]$$

$F = \frac{10^2*20.5}{2}$ *0.3924*[0.5*tan0+ (21/10)+0] = 845.29 kN/m^2 (Neglect k_v as k_v = 0)

b) Calculate seismic active force

$\Delta P_{AE} = 0.375 \frac{\gamma H1^2}{2} A_m$

$\Delta P_{AE} = 0.375*\frac{21.5*10^2}{2}*0.3924 = 158.19$ kN/m^2

Where;

γ = total unit weight of soil behind block.

H_1 = effective height of soil mass that considers sloping ground.

K_v = the vertical seismic coefficient.

K_{AE} = total (static and dynamic) active pressure coefficient.

q_s = distributed surface loading.

First wall

$P_{AE} = P_A + \Delta P_{AE} = 407.425 + 158.19 = 565.61$ kN/m^2

$$FS_{SL} = \frac{0*21+(4515+252+565.61+\sin 0°)*\tan 20.75}{565.61\cos 0°} = 3.57 > 1.1 \text{ (Okay)}$$

Second wall

$\Delta P_{AE} = 0.375*\frac{19.6*10^2}{2}*0.3924 = 144.207$ kN/m^2

$P_{AE} = P_A + \Delta P_{AE} = 371.42 + 144.207 = 515.627$ kN/m^2

$$FS_{SL} = \frac{0*21+(4116+208+515.627+\sin 0°)*\tan 17.82}{515.627\cos 0°} = 3.06 > 1.1 \text{ (Okay)}$$

Third wall

$\Delta P_{AE} = 0.375*\frac{19.4*10^2}{2}*0.3924 = 142.74$ kN/m^2

$P_{AE} = P_A + \Delta P_{AE} = 367.63 + 142.74 = 510.37$ kN/m^2

$$FS_{SL} = \frac{0*21+(4074+402+510.37+\sin 0°)*\tan 19.67}{510.37\cos 0°} = 3.49 > 1.1 \text{ (Okay)}$$

2. **Internal Failure Mode**

I. Soil Nail Pullout Failure

$(FS_P)_z = (\frac{Rp}{T})_z$

$(FS_P)_z \geq 2$

$R_P = \pi d L_p q_u$

$L_p = L - \frac{(H-z)\cos\psi}{\sin(\psi+i)}$

R_p = Nail pull out capacity (kN)

T = Maximum tensile force of the nail at depth z from the top of the wall (kN)

L_p = Effective pull out length (m)

L = length of soil nail (m)

z = Height of the nail from top of the wall (m)

H = height of the wall (m)

$T = K_a * (q_s + \gamma H) * S_H S_V = 0.379*(12+20.5*9) *1*1 = 69.92$ kN

$(FS_P)_{z=9} = \frac{221.46}{69.92} = 3.17 > 2$ (Okay)

II. Soil Nail Tensile Strength Failure

$R_T = \pi d^2 f_y/(4*1000) = \frac{\pi*(25^2)*415}{4*1000} = 203.71$ kN

$(FS_T)_{z=9} = (R_T/T)_z = \frac{203.71}{69.92} = 2.91 > 1.8$ (Okay)

Table 3.4 Factor of safety against soil nail pullout failure, FS_P and nail tensile strength failure, FS_T

Nail No. j (From top)	Depth of nail z (m)	Factor of safety against pullout failure, FS_P	Factor of safety against nail tensile strength failure, FS_T
1	1	14.00	16.54 (Very high)

2	2	8.89	10.14 (Very high)
3	3	6.63	7.32 (Very High)
4	4	5.36	5.72
5	5	4.54	4.69
6	6	3.97	3.98
7	7	3.52	3.45
8	8	3.23	3.05
9	9	2.97	2.74

3. Facing Design

T_o = Tensile force at the nail head

T_{max} = Maximum nail force

$S_{v(m)}$ = Maximum spacing of soil nails

C_f = Factor to consider non-uniform soil pressures behind facing

H_i = Thickness if initial facing (mm)

L_{BP} = Sixe of a square bearing plate

a_{vm} = Cross=sectional area of vertical reinforcement per unit width at mid-span

a_{vn} = Cross-sectional area of vertical reinforcement per unit width at nail head

a) Calculate design nail head tensile force at the face (T_o):

$$T_o = T_{max}[0.6+0.2(S_V[m]-1)]$$

$$T_o = 82.243(0.6+0.2*(1-1) = 49.346 \text{ KN}$$

b) Facing Materials

Table 3.5 Facing Main Features

Elements	**Description**	**Temporary Facing**	**Permanent Facing**
General	Thickness (h)	100 mm	150 mm
	Facing Type	Shotcrete	Reinforced Shotcrete
	Compressive Strength, f_{ck}	21 N/mm^2	28 N/mm^2

Reinforcement	Type	WWM	Steel Bars Mesh
	Grade	415 N/mm^2	415 N/mm^2
	Denomination	102×102-MW19×MW19	No. 13 @ 300 mm (each way)
Other Reinf.	Type	Waler Bars 2*10 mm	-
Bearing Plate	Type	4 Headed-Studs $\frac{3}{8} \times 4\frac{1}{8}$	-
	Steel	250 N/mm^2	-
	Dimensions	Length: L_p = 225 mm	-
		Thickness; t_p = 25 mm	-
Headed Studs	Dimensions	-	Nominal length, L_s = 105 mm
		-	Head Diameter, D_H = 19.1 mm
		-	Shaft Diameter, D_s = 9.7 mm
		-	Head Thickness, t_H = 7.1 mm
		-	Spacing, S_{SH} = 150 mm

c) Facing Tensile Flexural Failure (R_{FF})

For Temporary Facing:

a) Check for facing reinforcement ratios

The minimum reinforcement ratio is calculated as:

$$\rho_{min}\,[\%] = 20 * \frac{\sqrt{fck}}{fy} = 20 * \frac{\sqrt{21}}{415} = 0.22\%$$

The maximum reinforcement ratio is calculated as:

$$\rho_{max}\,[\%] = 50 * \frac{fck}{fy} * \left(\frac{600}{600+fy}\right) = 50 * \frac{21}{415} * \left(\frac{600}{600+415}\right) = 1.49\%$$

b) Select reinforcement

Welded wire mesh (temporary facing): WMM 102 × 102 – MW19 × MW19 from Table A.1.

$$a_{vm} = 184.2 \text{ mm}^2/\text{m}$$

c) Total reinforcement area per unit length around the nails is:

Horizontal and vertical waler rebar: Adopt 10 mm, from Table A.2.

$$A_{VN} = A_{HW} = \frac{2*\pi*10^2}{4} = 157.1 \text{ mm}^2 \text{ (In both directions)}$$

Therefore;

$$a_{vn} = a_{vm} + \frac{Avn}{SH} = 184.2 + \frac{157.1}{1} = 341.3 \text{ mm}^2/\text{m}$$

d) Verify minimum and maximum reinforcement ratios

$\rho_m = a_{vm}/0.5h_i = 184.2/(0.5*100*12) = 0.31\%$

$\rho_n = a_{vn}/0.5h_i = 341.3/(0.5*100*12) = 0.57\%$

$\rho_m = 0.31\% > 0.22\%$

$\rho_m = 0.31\% < 1.49\%$

$\rho_n = 0.57\% > 0.22\%$

$\rho_n = 0.57\% < 1.49\%$

$\rho_{tot} = \rho_n + \rho_m = 0.31\% + 0.57\% = 0.88\%$; $\rho_n/\rho_m = 1.84 < 2.5$

e) Select Factor C_F

Adopt $C_F = 2$ for temporary facing from Table A.3.

f) Flexural capacity

$$R_{FF} = \frac{CF}{265}*(a_{vn} + a_{vm})*(\frac{SH}{SV}h)*f_y$$

$$R_{FF} = [\frac{2}{265}*(341.3+157.1)*(\frac{1}{1}*0.10)*415] = 164.59 \text{ kN}$$

Therefore;

$$FS_{FF} = R_{FF}/T_o = \frac{164.59}{49.346} = 3.33 > 1.35 \text{ (Okay)}$$

For Permanent Facing;

a) Check for facing reinforcement ratios

The minimum reinforcement ratio is calculated as:

$$\rho_{min}[\%] = 20 * \frac{\sqrt{fck}}{fy} = 20 * \frac{\sqrt{28}}{415} = 0.26\%$$

The maximum reinforcement ratio is calculated as:

$$\rho_{max}[\%] = 50 * \frac{fck}{fy} * (\frac{600}{600+fy}) = 50 * \frac{28}{415} *(\frac{600}{600+415}) = 1.99\%$$

b) Select reinforcement

Use a reinforcement mesh made of No. 13 metric bars at 300 mm (Table A.2) center-to-center each way. No waler bars are used.

$a_{vm} = a_{vn} = 129 * 1000/300 = 429.57\ mm^2/m$

c) Reinforcement ratos

$\rho_n = \rho_m = 429.57/12/\ (0.5*150) = 0.48\%$ (Satisfies both ρ_{min} and ρ_{max})

$\rho_{tot} = \rho_n + \rho_m = 0.96\%$

d) Select factor C_F

$C_F = 1.5$ (from Table A.3)

e) Flexural capacity

$$R_{FF} = \frac{CF}{265}*(a_{vn}+a_{vm})*(\frac{SH}{SV}h)*f_y$$

$$R_{FF} = \frac{1.5}{265}*(341.3+184.2)*(1*0.10)*415 = 123.44\ kN$$

$$FS_{FF} = R_{FF}/T_o = \frac{123.44}{49.346} = 2.5 > 1.35\ \text{(Okay)}$$

d) Facing Punching Failure (Temporary)

D_C = Effective equivalent diameter of conical slip surface at soil nail head

L_{BP} = Size of a square bearing plate

R_{FP} = Nominal resistance of facing for punching shear

f_{ck} = Compressive strength of concrete

$R_{FP} = 330\sqrt{f_{ck}} *\pi D_c*h$

$D_c = L_{BP} + h$

$FS_{FP} = R_{FP}/T_o >1.35$

For Temporary Facing

$D_C = 225 + 100 = 325\ mm = 0.325\ m$

$T_o = 82.243(0.6+0.2*(1-1) = 49.346\ KN$

$R_{FP} = 330*\sqrt{21} *\pi *0.325*0.10 = 154.4\ kN$

$$FS_{FP} = \frac{154.4}{49.346} = 3.13 > 1.35\ \text{(Okay)}$$

For Permanent Facing

D_C = minimum of (S_{SH} + h_c) or $2h_c$

$h_c = L_s + t_p + t_{SH}$ = 105 + 25 -7.1 = 122.9 mm

Therefore;

D_c = min [(150 + 122.9) or (2*122.9)] = 245.8 mm

T_o = 82.243(0.6+0.2*(1-1) = 49.346 KN

$R_{FP} = 330*\sqrt{28}*\pi*0.2458*0.1229$ = 165.72 kN

$FS_{FP} = \frac{165.72}{49.346}$ = 3.36 >1.35 (Okay)

e) Facing Headed Stud Resistance (R_{FH}) – Permanent Facing [3]

a) Calculate the facing headed stud tensile resistance as:

$R_{FH} = N_H\ A_S\ f_y$

b) Verify that capacity is higher than nail head tensile force:

$R_{FH} > FS_{ST}\ T_o$

Where;

N_H = cross-sectional area of the stud head

A_S = cross-sectional area of the stud shaft

t_H = head thickness

D_H = diameter of the stud head

D_S = diameter of the headed-stud shaft

$R_{FH} = N_H\ A_{SH}\ f_y$

$R_{FH} = 4 * (\pi\ D_{SC}^2/4) * 415 = 4 * (\pi * 9.7^2/4) * 415$ = 122.7 kN

$FS_{HT} = R_{FH}/To$ =122.7/49.346 =2.49 > 1.5 (Okay)

3.5 DETAILING OF SOIL NAIL WALL

The detail layout of soil nail wall is given below.

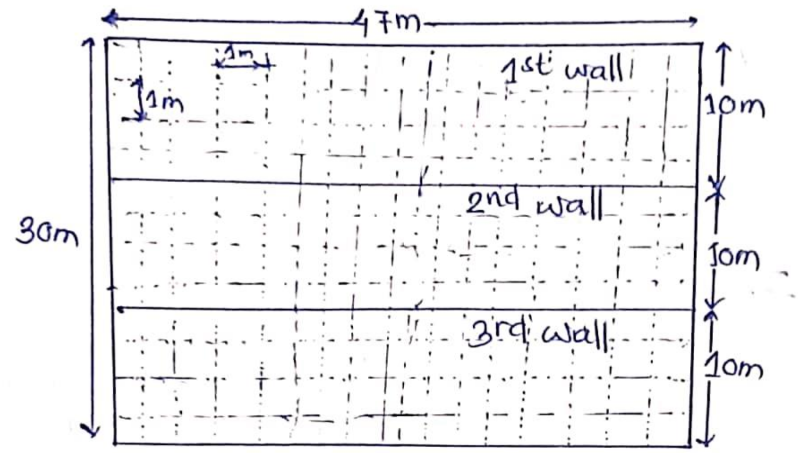

Figure 3.3 Front view of wall (30×27)

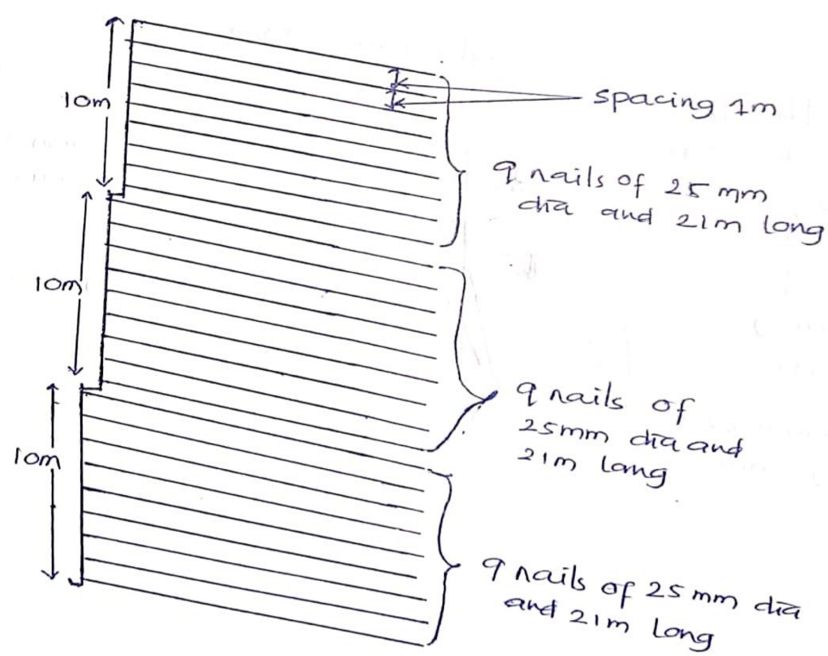

Figure 3.4 View of soil nail in all layer

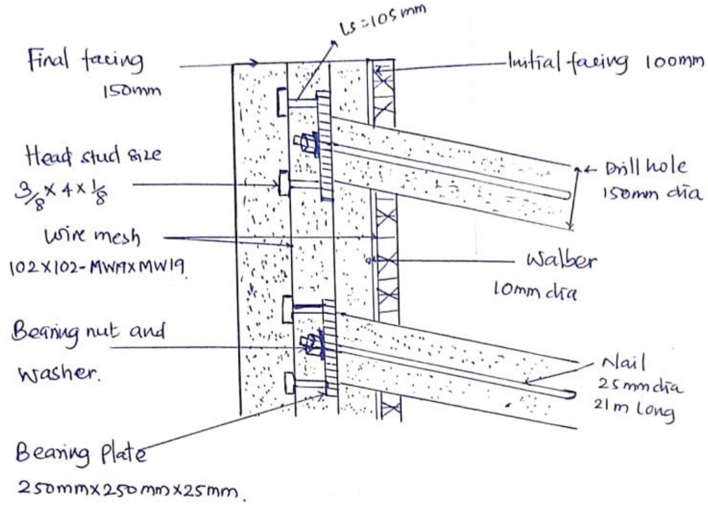

Figure 3.5 View of detail of soil nail

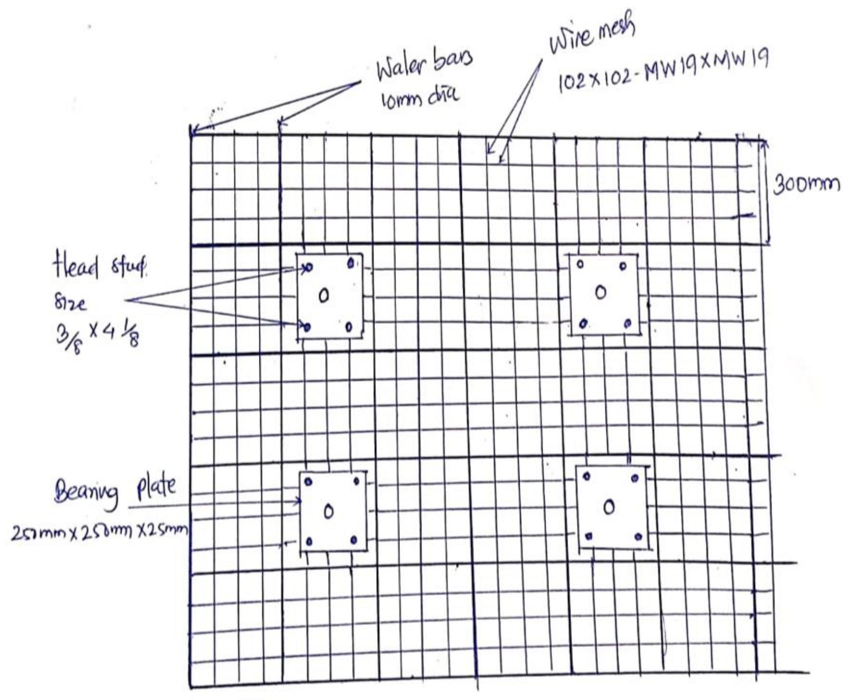

Figure 3.6 View of wire mesh with bearing plate

3.6 INTRODUCTION TO GEOSLOPE [16]

The slope stability of the site is checked by numerical analysis and simulate the effects of all the parameters. The finite element analysis software used for finding out the factor of safety in this study is GeoStudio 2018 R2.

SLOPE / W is a software product that utilizes limit equilibrium theory to calculate the safety factor of soil and rock slopes. The limit equilibrium stability method is the oldest and most famous numerical method in geotechnical engineering. The software uses different safety factor calculation methods to analyze both complex and easy slope stability problems. The limit equilibrium method is based on two factors of safety equations with regard to force and moment equilibrium. SLOPE/W can be used to analyze and plan geotechnical, civil engineering, and mining engineering projects.

This study uses SLOPE/W for the analysis of factor of safety which have different methods available to analyze its stability. Further pseudo-static analysis has been done to analyze the seismic response of the slope. The stability analysis was done for the height of 30 m and 47 m wide and the slope angle of 60° for two cases as 1) with soil nail and 2) without soil nail. The different method used for analysis of slope stability with or without reinforcement is shown below.

a) Morgenstern-Price

Morgenstern and Price (1965) advanced a method similar to Spencer but allows different user-defined interslice force functions such as half-sine, constant, trapezoidal, and data-points specified. The Morgenstern and Price process comprises both shear and normal interslice force which fulfills both moments and forces equilibrium. [16, pp. 40-42].

b) Spencer Method

Spencer (1967) established two factor of safety equations: one is related to moment equilibrium and the other is related to horizontal force balance. He assumed a constant relationship between the shear force and the normal force and used an iterative procedure to change the shear force-normal relationship between the interslice until the two safety factors were equal. Balancing the relationship between these two safety factors means that the equilibrium of moment and force can be observed [16, p.38-40].

Bishop’s Simplified Method

Bishop developed an equation for the normal at the slice base by summing slice forces in the vertical direction and he included interslice normal forces but ignored the interslice shear forces and satisfied moment equilibrium. The consequence of this is that the normal becomes a function of the factor of safety. This in turn makes the factor of safety equation nonlinear and an iterative procedure is consequently required to compute the factor of safety [16, pp. 35-36].

c) Janbu’s Simplified Method

Janbu's Simplified technique is like the Bishop's Simplified Method where normal forces are considered yet that it fulfills just generally horizontal force equilibrium, not overall moment equilibrium [16, p. 37].

d) Ordinary or Fellenius method

In this technique, all interslice forces are neglected which implies it has no interslice shear and normal force. Hence it has a poor force polygon closure and it means the slice is not in force equilibrium. Subsequently, this strategy is never utilized practically and is just kept in SLOPE/W for delineation reasons. The factor of safety is given by total shear strength available along the slip surface divided by the summation of the gravitational driving forces (mobilized shear) [16, pp. 31-35].

CHAPTER 4

RESULTS AND DISCUSSION

4.1 GENERAL

This chapter includes the results obtained after thorough calculation and after performing analysis in software with different nail inclination. The different factor of safety resulted for different mode of failure is all presented in this chapter.

4.2 ANALYSIS WITH GEOSLOPE

4.2.1 Case 1: Without Soil Nail

- **Morgenstern-Price**

  After inserting all the required data in the software, the slip surface and factor of safety versus lambda is given in the figure 4.1 and free body diagram of one section of interslice is shown below in figure 4.2.

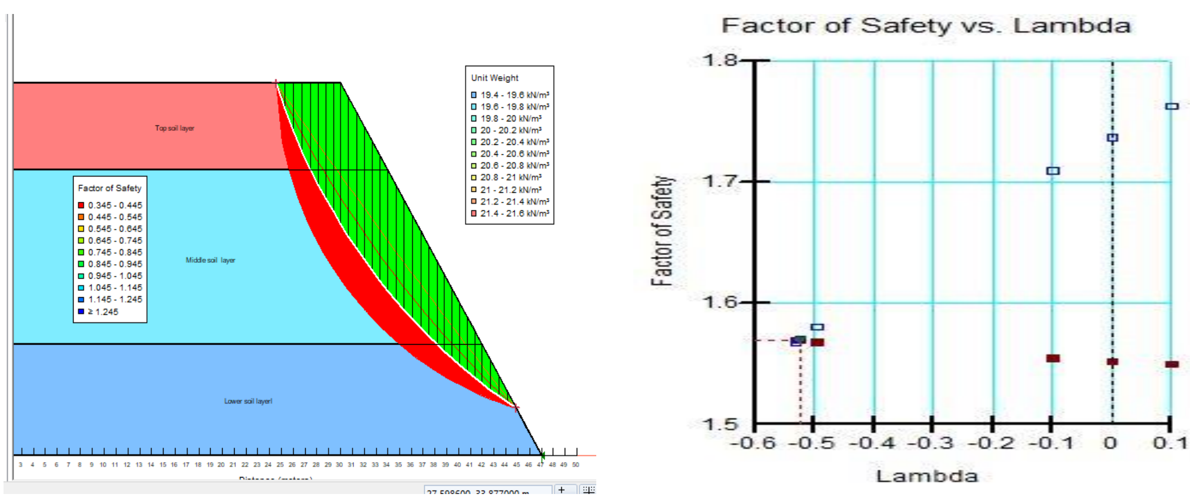

Figure 4.1 Slip surface(Left) and FOS versus lambda (Right) of M-P

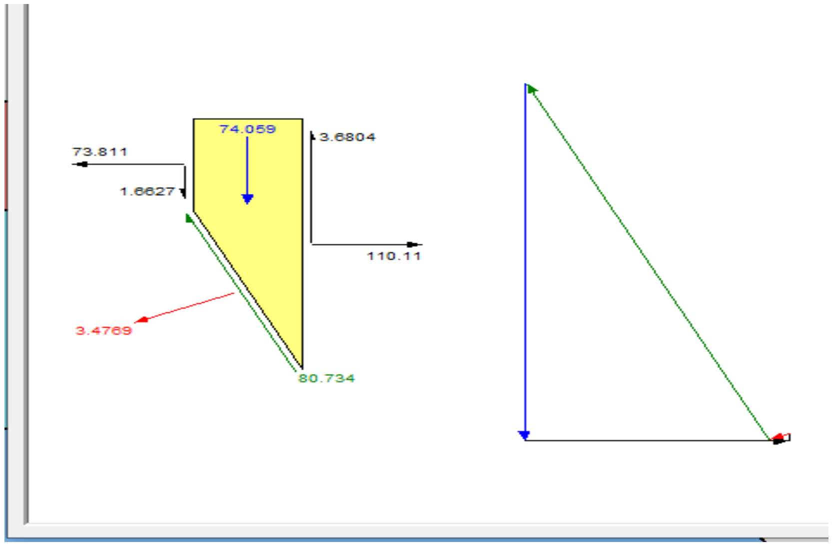

Figure 4.2 Morgenstern-Price free body diagram of interslice

- **Spencer Method**

  The sliding mass have multiple interslice integrated and one of the slice information about shear force and normal force is given in the figure 4.3 and the factor of safety versus lambda is presented in the figure 4.4.

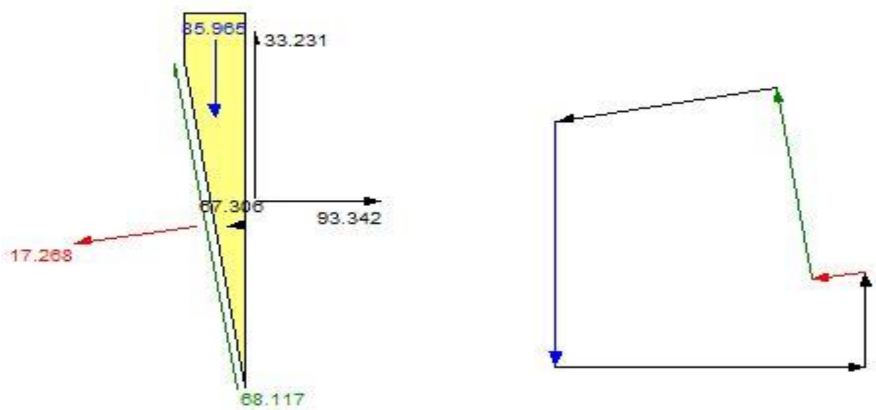

Figure 4.3 Free Body Diagram of Interslice of Spencer

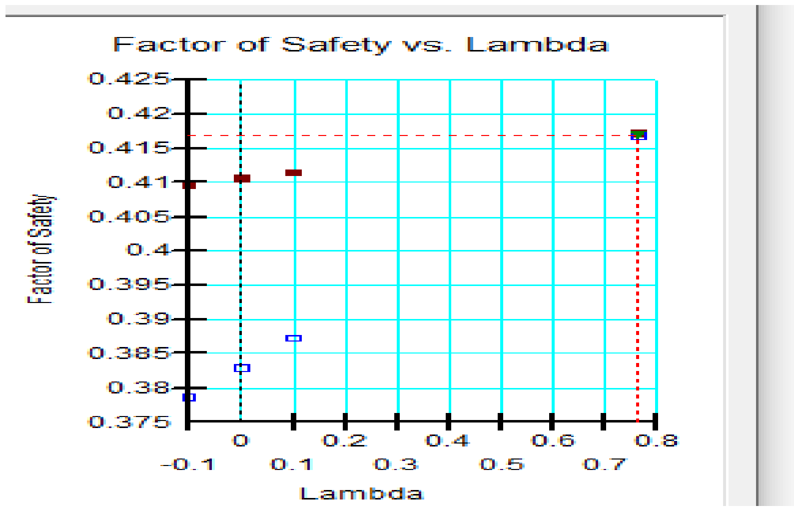

Figure 4.4 FOS versus lambda (Spencer)

- **Bishop Method**

The red colour represents a zone of slip surface with similar factor of safety. The slip surface with factor of safety of range between 1.552 to 2.779. Figure 4.5 represents the slip surface and factor of safety versus slice in Bishop. And also the factor of safety versus lambda and free body diagram of interslice is given in figure 4.6.

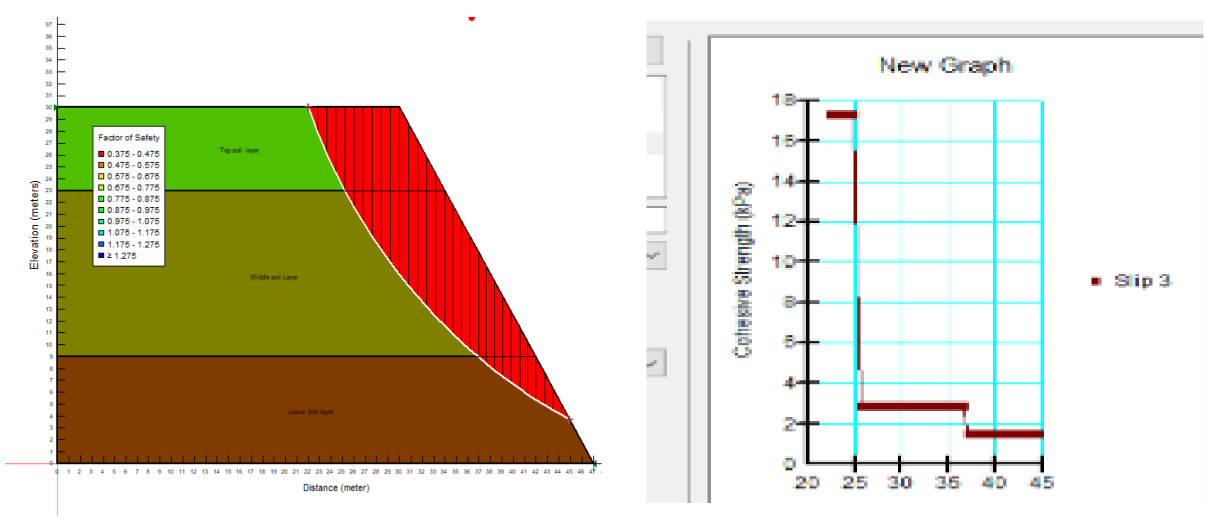

Figure 4.5 slip surface (left) and FOS versus slice (right) (Bishop)

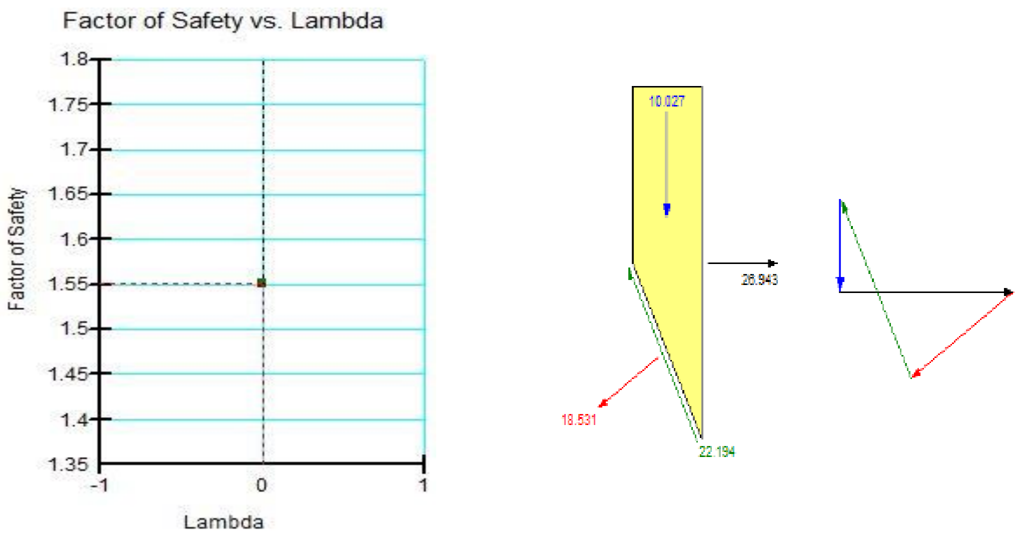

Figure 4.6 FOS versus lamba (left) and Free body diagram of Bishop (right)

- **Janbu Method**

Figure 4.7 can be seen as the interslice with normal forces only and no shear forces are taken into account and also factor of safety versus lambda for critical slip surface are given.

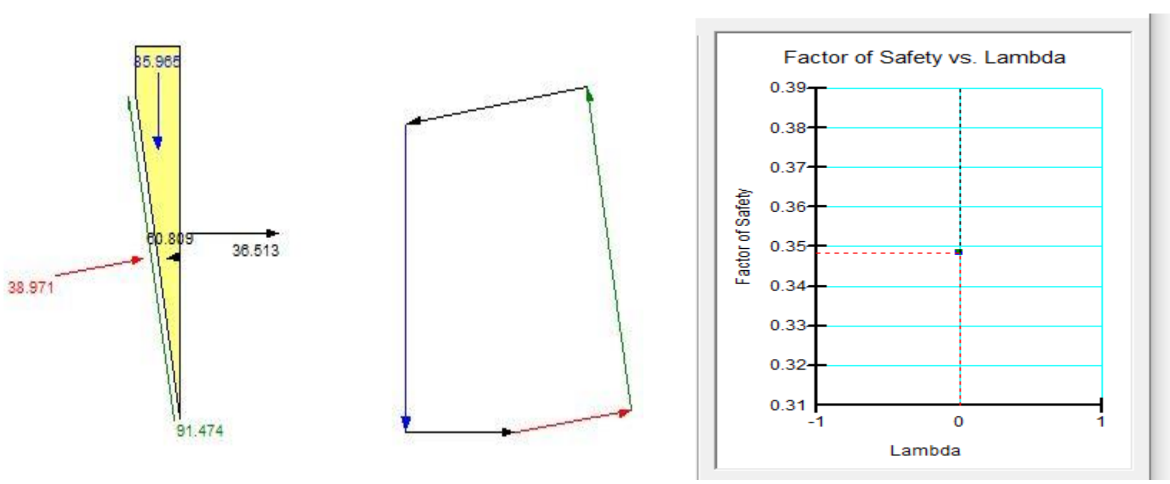

Figure 4.7 FBD of interslice (left) and FOS versus lambda (right) (Janbu)

- **Ordinary Method**

  Figure 4.8 represents the factor of safety versus lambda for ordinary and free body diagram of one interslice of ordinary. It shows that there are no shear force and normal force on the slice.

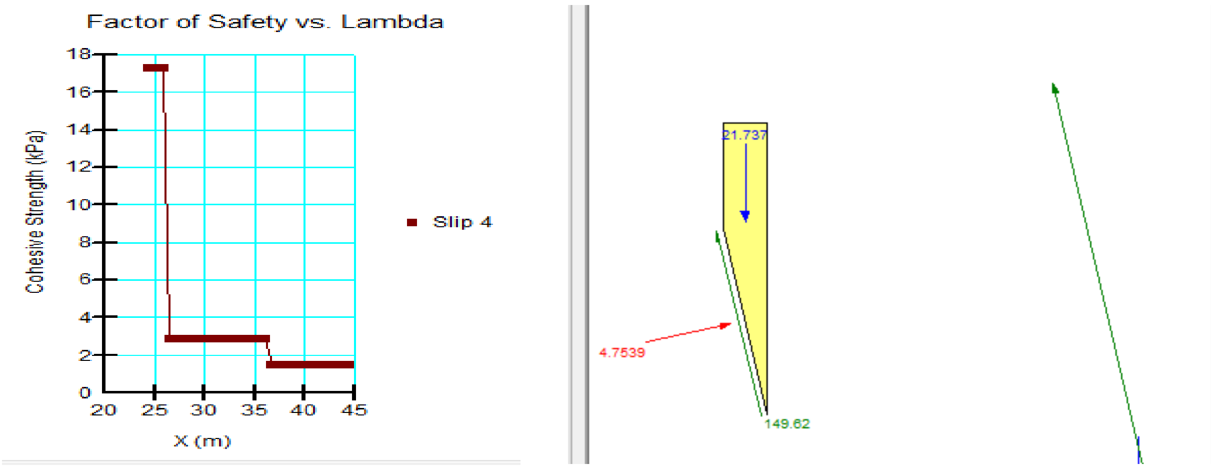

Figure 4.8 FOS versus slice (left) and Free body diagram (right) (Fellenius)

Table 4.1 shows the FOS computed by various methods for slope angle 60˚. It is observed that the slope without soil nail is unstable as the factor of safety is less than 1.5.

Table 4.1 Comparison of Factor of Safety without Soil Nail

Slope Inclination (˚)	Factor of Safety (FOS)				
	Morgenstern Price	Spencer	Bishop	Janbu	Ordinary
60	0.345	0.350	0.376	0.349	0.386

4.2.2 CASE 2: With Soil Nail

Effects of Variation of Nail Inclination

When the angle of soil nailed varied between 10°, 15° and 20°, it is observed that, with the increase in inclination of nail, the factor of safety of decreases. The best FOS is given by 10°. The unstable slope which is reinforced with soil nail at different angle is given in the figure below.

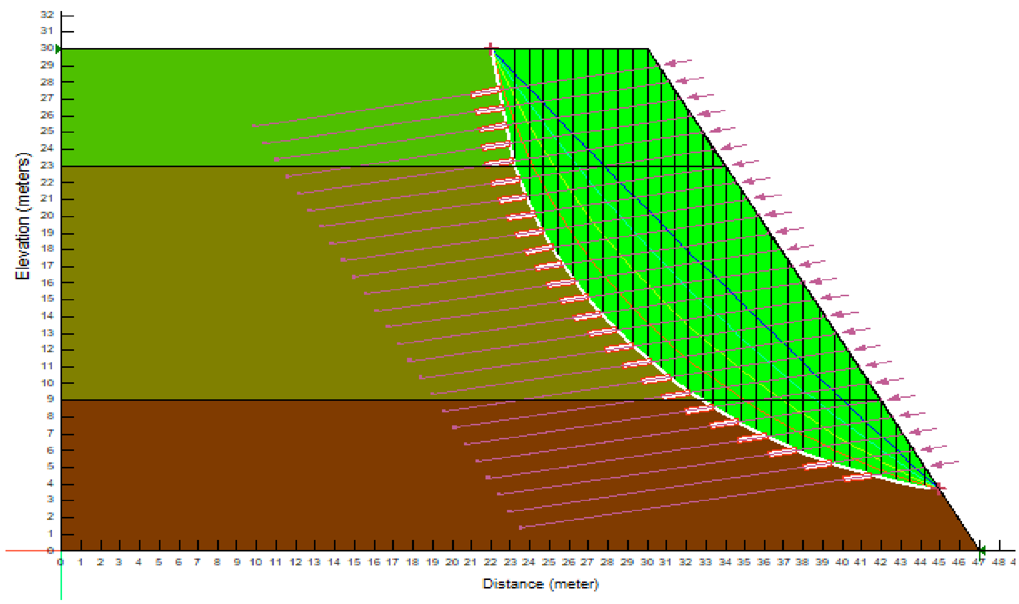

Figure 4.9 A 10° inclined soil nail of M-P Method

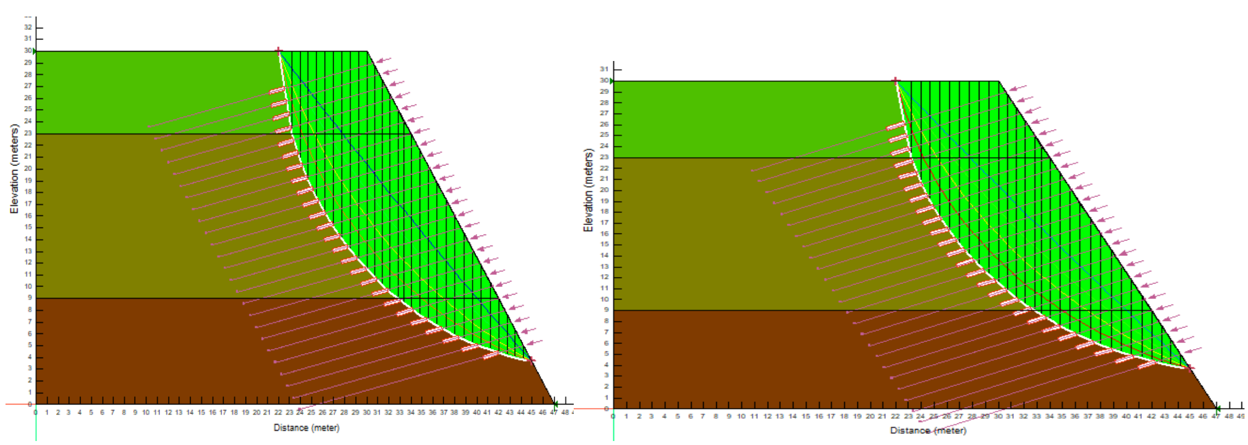

Figure 4.10 15° and 20° nail inclination of M-P Method respectively

In the beginning, the stability analysis was done for unreinforced section and FOS was very low. With the incorporation of nail, it has increased its factor of safety up to greater value. Table 5.2

shows the factor of safety computed for different nail inclination angles. It was observed that the factor of safety is almost same for 10° and 15° and FOS are less for 20° and shown Table 5.6.

Table 4.2 Factor of safety for different nail inclination.

Slope Inclination (°)	Nail Inclination (°)	Factor of Safety (FOS)				
		Morgenstern Price	Spencer	Bishop	Janbu	Ordinary
60	10	1.570	1.569	1.552	1.558	1.737
	15	1.516	1.517	1.502	1.498	1.655
	20	1.458	1.460	1.446	1.434	1.573

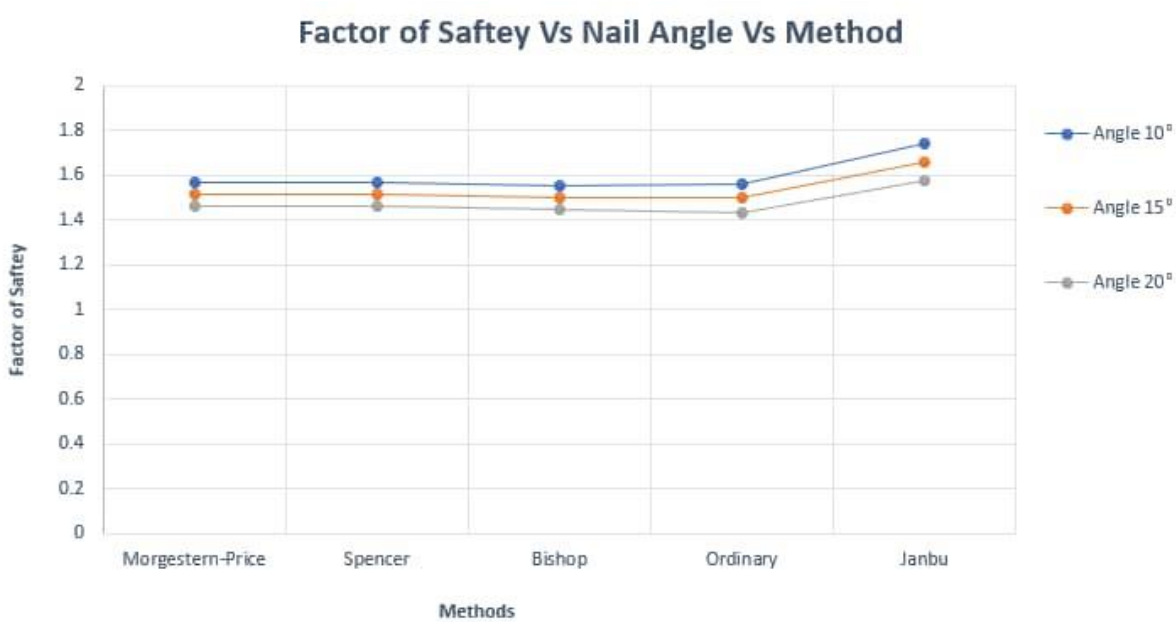

Figure 4.11 Graphical comparison between different FOS for different nail angles and method

Seismic analysis for Pseudo-static

Pseudo-static analysis is one of the easiest way to analyze the seismic response of slope, embankment and hilly areas. The areas that is prone to earthquake uses horizontal and vertical pseudo-static (seismic) coefficients, k_h and k_v respectively to calculate horizontal and vertical forces caused by earthquake. Pseudo-static analysis is suitable to evaluate the performance of embankments constructed of soil (i.e., clayey soil, dry or moist cohesion less soil and dense cohesion less soils) that do not lose significant strength during shaking of earthquake. In this study, we have analyzed the stability of slope with pseudo-static approach determine the factor of safety with incorporation of soil nail in slope. It was resulted that the factor of safety is greater than the minimum recommended factor of safety by Federal Highway Administration guidelines. The free body diagram of interslice of soil mass under seismic condition and seismograph is given below in figure 5.13 and figure 5.14 respectively. Further the factor of safety for global stability is given in figure 5.15.

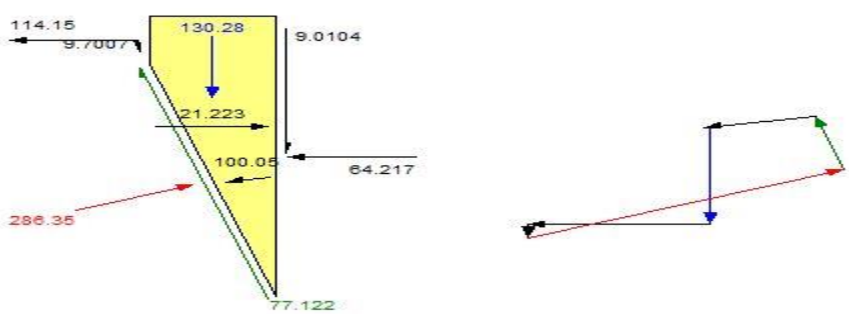

Figure 4.12 Free body Diagram of interslice under seismic effect

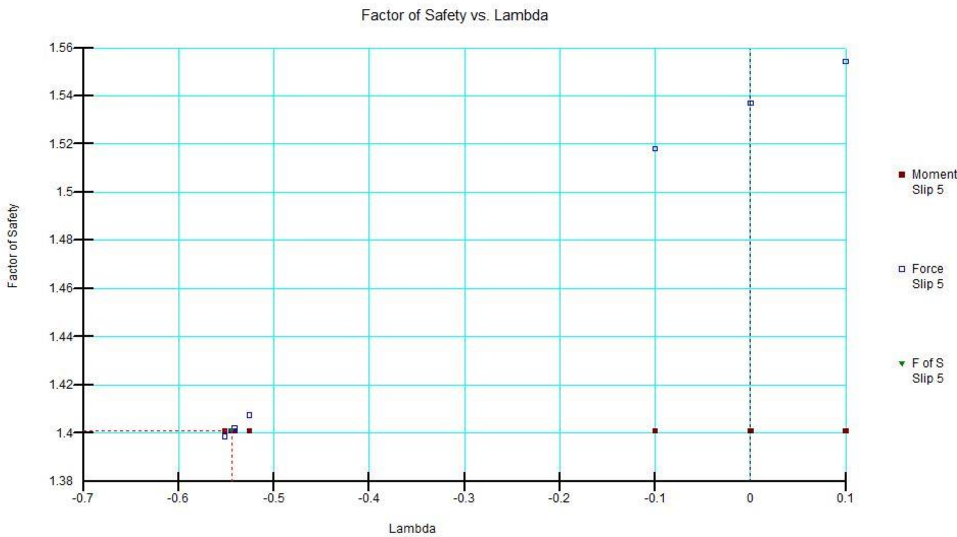

Figure 4.13 Seismograph on Factor of Safety versus lambda

Slip #	F of S	X Center (m)	Y Center (m)	Radius (m)	Details
5	1.401	49.648	30.912	27.623	Critical (analysis)
4	1.521	55.941	36.392	34.498	
3	1.703	67.4	46.37	48.223	
2	1.980	96.701	71.886	85.608	
1	2.439	384.73	322.7	466.07	

Figure 4.14 Factor of safety of pseudo-static condition

CHAPTER 5

CONCLUSION

5.1 GENERAL

In this context, an overall closure and deduction are present. The detail conclusion of this study are presented below.

5.2 CONCLUSIONS

- From the above study, it can be concluded that the slope is unstable and the potential slip surface is very high without any remedies.
- The conventional calculation on different mode of failure are calculated and the factor of safety is obtained higher than the minimum recommended factor of safety by federal highway administration guidelines. It lies between the range of 2.16 to 4.81.
- The unstable slope without reinforcement showed the factor of safety very low (0.345 to 0.386)
- The factor of safety computed from different methods available in software have resulted higher than the recommended one. The factor of safety primarily lies between 1.434 and 1.737.
- The different nail angles (10°, 15° and 20°) with respect to horizontal are analyzed with five different methods (Morgenstern-Price, Spencer, Janbu, Bishop and Ordinary) and factor of safety was obtained. It shows that 10° gives the best factor of safety compare to 15° and 20°.

5.3 FUTURE SCOPE OF THE WORK

1) It is found that the limited field research paper is available for soil nail method to stabilize the slope in Bhutan and is all done relative with respect to different angle of nail. Analyzing different approaches like variation of diameter and length of nail with different software could be done for better outcome.

2) From the past paper for the site selected, it is observed that only few ground investigation is performed. Experimenting some of the test like some of the tests like pullout test, SPT and CPT test and bore hole test would make the test much easier and accurate.

3) Though limit equilibrium computes factor of safety effectively, it does not address displacement and strains. Therefore, the use of finite element would overcome this limitation and give best results.

CHAPTER 6

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APPENDIX A

Table A.1 Welded Wire Mesh Dimensions in Metric (Source: GEC 07)

Mesh Designation [Metric]	**Wire Cross-Sectional Area per Unit Length [Metric]**	**Weight per Unit Area [Metric]**
($mm \times mm$ – $mm^2 \times mm^2$)	**(mm^2/m)**	**(kg/m^2)**
102×102 – MW9×MW9	88.9	1.51
102×102 – MW13×MW13	127.0	2.15
102×102 – MW19×MW19	184.2	3.03
102×102 – MW26×MW26	254.0	4.30
152×152 – MW9×MW9	59.3	1.03
152×152 – MW13×MW13	84.7	1.46
152×152 – MW19×MW19	122.8	2.05
152×152 – MW26×MW26	169.4	2.83

Notes:

1. The first two numbers indicate the mesh opening size, whereas the second pair of numbers following the prefixes indicates the wire cross-sectional area.
2. Prefixes M indicates metric units. Prefix W indicates plain wire. If wires are pre-deformed, the prefix D is used instead of W.
3. This value is obtained by dividing the wire cross-sectional area by the mesh opening size. (Source: FHWA-GEC 07).

Table A.2 Reinforcing Bar Dimensions (English and Metric) (Source: GEC 07)

Bar Designation		**Nominal Diameter**		**Nominal Area**	
English	**Metric**	**in.**	**mm**	**in.^{2}**	**Mm^2**
3	10	0.375	9.6	0.11	71
4	13	0.500	12.7	0.20	129
5	16	0.625	15.9	0.31	199
6	19	0.750	19.1	0.44	284
7	22	0.875	22.2	0.60	387
8	25	1.000	25.4	0.79	510
9	29	1.128	28.7	1.00	645
10	32	1.270	32.3	1.27	819
11	36	1.410	35.8	1.56	1006
14	43	1.693	43.0	2.25	1452
18	57	2.257	57.3	4.00	2581

Table A.3 Factors C_F

Type of Structure	**Nominal Facing Thickness mm (in.)**	**Factor C_F**
Temporary	100 (4)	2.0
	150 (6)	1.5
	200 (6)	1.0
Permanent	All	1.0

Table A.4 Minimum Recommended FOS for the Design of soil Nail Walls using ASD Method

Failure Mode	Resisting component	Symbol	Minimum Recommended Factors of Safety		
			Static Loads		Seismic Loads
			Temporary	Permanent	Temporary and Permanent
External Stability	Global Stability(long-term)	FS_G	1.35	1.5	1.1
	Global Stability(excavation)	FS_G	1.2-1.3		NA
	Sliding	FS_{SL}	1.3	1.5	1.1
	Bearing Capacity	FS_H	2.5	3.0	2.3
Internal Stability	Pullout Resistance	FS_P	2.0		1.5
	Nail Tensile Strength	FS_T	1.8		1.35
Facing Strength	Facing Flexure	FS_{FF}	1.35	1.5	1.1
	Facing Punching Shear	FS_{FP}	1.35	1.5	1.1
	H-Stud Tensile (A307 Bolt)	FS_{HT}	1.8	2.0	1.5
	H-Stud Tensile (A325 Bolt)	FS_{HT}	1.5	1.7	1.3

Table A.5 Estimated Bond Strength for Soil Nails in Coarse-Grained Soils

Drill-Hole Drilling Method	Soil Type	Bond Strength, q_u (psi)
Rotary Drilled	Sand/gravel	15 -26
Rotary Drilled	Silty sand	15 – 22
Rotary Drilled	Silt	9 – 11
Rotary Drilled	Piedmont residual	6 – 17
Rotary Drilled	Fine colluvium	11 – 22
Rotary Drilled	Sans/gravel w/low overburden	28 – 35
Rotary Drilled	Sand/gravel w/high overburden	41 – 62

Rotary Drilled	Dense Moraine	55 – 70
Rotary Drilled	Colluvium	15 – 26
Augered	Silty sand fill	3 – 6
Augered	Silty sand fill	8 – 13
Augered	Silty clayey sand	9 – 20

Table A.6 Headed-Stud Dimensions

Head-Stud Size	**Nominal Length L_S**	**Head Diameter D_H**	**Shaft Diameter D_S**	**Head Thickness t_H**	**Headed Area/Shaft Area**	**Head Thickness/Head or Shaft Diameter**
	mm	**mm**	**mm**	**mm**	**in.**	
1/4×4(1/8)	105	12.7	6.4	0.19	4.0	0.75
3/8×4(1/8)	105	19.1	9.7	0.28	4.0	0.75
3/8×6(1/8)	156	19.1	9.7	0.28	4.0	0.75
1/2×4(1/8)	105	25.4	12.7	0.31	4.0	0.62
1/2×5(5/6)	135	25.4	12.7	0.31	4.0	0.62
1/2×6(6/8)	156	25.4	12.7	0.31	4.0	0.62
5/8×6(9/16)	162	31.8	15.9	0.31	4.0	0.50
3/4×3(11/16)	89	31.8	19.1	0.38	2.8	0.75
3/4×4(3/16)	106	31.8	19.1	0.38	2.8	0.75
3/4×5(3/16)	132	31.8	19.1	0.38	2.8	0.75
3/4×6(3/16)	157	31.8	19.1	0.38	2.8	0.75
7/8×4(3/16)	102	34.9	22.2	0.38	2.5	0.75
7/8×5(3/16	0127	34.9	22.2	0.38	2.5	0.75
7/8×6(3/16)	152	34.9	22.2	0.38	2.5	0.75

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