Final Report

SOIL NAILING FOR STABILIZATION OF STEEP SLOPES NEAR RAILWAY TRACKS

Submitted to

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Prepared by

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Dr. Amit Prashant
Ms. Mousumi Mukherjee
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<tr>
<td>$\alpha$</td>
<td>Inclination of slice base</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Inclination of slice top</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Angle subtended by the slip circle at centre</td>
</tr>
<tr>
<td>$\alpha_\ell$</td>
<td>Inclination of the back slope</td>
</tr>
<tr>
<td>$\beta_\ell$</td>
<td>Slope face angle with respect to the vertical</td>
</tr>
<tr>
<td>$\theta_\ell$</td>
<td>Inclination of failure plane</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Soil effective angle of internal friction</td>
</tr>
<tr>
<td>$c$</td>
<td>Soil effective cohesion</td>
</tr>
<tr>
<td>$L$</td>
<td>Length of failure plane</td>
</tr>
<tr>
<td>$W$</td>
<td>Weight of the sliding mass</td>
</tr>
<tr>
<td>$Q$</td>
<td>Surcharge load</td>
</tr>
<tr>
<td>$N_F$</td>
<td>Normal force on failure surface</td>
</tr>
<tr>
<td>$S_F$</td>
<td>Shear force on failure surface</td>
</tr>
<tr>
<td>$R$</td>
<td>Radius of circular slip surface</td>
</tr>
<tr>
<td>$S_u$</td>
<td>Undrained shear strength</td>
</tr>
<tr>
<td>$\overline{x}$</td>
<td>Horizontal distance between circle centre and the centre of the sliding mass</td>
</tr>
<tr>
<td>$R_c$</td>
<td>Perpendicular distance from the circle centre to shear force</td>
</tr>
<tr>
<td>$L_{arc}$, $L_{chord}$</td>
<td>Lengths of the circular arc and chord defining the failure surface</td>
</tr>
<tr>
<td>$\delta$</td>
<td>Angle of line of action of surcharge with vertical</td>
</tr>
<tr>
<td>$b$</td>
<td>Width of slice</td>
</tr>
<tr>
<td>$h$</td>
<td>Average height of slice</td>
</tr>
<tr>
<td>$S_a$</td>
<td>Available strength</td>
</tr>
<tr>
<td>$S_F$</td>
<td>Mobilized strength</td>
</tr>
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</table>
\( \lambda \)  
Nail inclination of equivalent nail tensile force

\( \lambda_j \)  
Nail inclination of \( j^{th} \) nail

\( n_l \)  
Total number of nail used

\( T_{EQ} \)  
Equivalent nail tensile force

\( T_j \)  
Tensile force in \( j^{th} \) nail

\( T_n \)  
Nail tensile forces for the reinforcement emerging out from the base of \( i^{th} \) slice

\( k_v \)  
Vertical seismic coefficient

\( k_h \)  
Horizontal seismic coefficient

\( N_c, N_\gamma \)  
Bearing capacity factor

\( \gamma \)  
Unity weight of soil behind wall

\( H \)  
Height of the wall (excavation depth)

\( \Delta H \)  
Equivalent overburden

\( H_{eq} \)  
Equivalent wall height

\( B_e \)  
Width of excavation

\( B_i \)  
Width of influence

\( L_e \)  
Length of excavation

\( q \)  
Mobilized bond stress

\( p \)  
Perimeter of the nail

\( d \)  
Diameter of the nail

\( \psi \)  
Mobilized soil-nail interface friction angle

\( h_j \)  
Depth of the midpoint of \( j^{th} \) nail from ground surface

\( l_e \)  
Length of the nail behind the failure surface in case of nailed slope

\( FSG \)  
Factor of safety against global stability

\( FS_{SL} \)  
Factor of safety against sliding stability

\( FS_H \)  
Factor of safety against bearing capacity

\( FS_P \)  
Factor of safety against pullout strength

\( FS_T \)  
Factor of safety against nail-tensile strength

\( FS_{FF} \)  
Factor of safety against facing failure

\( FS_{FP} \)  
Factor of safety against punching failure

\( FS_{HT} \)  
Factor of safety against headed-stud tensile failure
$C_F$  Correction Factor
$eta_{eq}$  Equivalent back slope angle
$K$  Coefficient of lateral active earth pressure
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(b) Simplified Bishop method

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3.9.1 Problem-1

(a) Planar failure surface

(b) Circular arc method

(c) Simplified Bishop method

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(a) Planar failure surface

(b) Simplified Bishop method

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CHAPTER 1
INTRODUCTION

1.1 Nails and soil nailing

Soil nailing is a technique in which soil slopes, excavations or retaining walls are passively reinforced by the insertion of relatively slender elements - normally steel reinforcing bars. Such structural element which provides load transfer to the ground in excavation reinforcement application is called nail (Fig. 1.1). Soil nails are usually installed at an inclination of 10 to 20 degrees with horizontal and are primarily subjected to tensile stress. Tensile stress is applied passively to the nails in response to the deformation of the retained materials during subsequent excavation process. Soil nailing is typically used to stabilize existing slopes or excavations where top-to-bottom construction is advantageous compared to the other retaining wall systems. As construction proceeds from the top to bottom, shotcrete or concrete is also applied on the excavation face to provide continuity. Fig. 1.2 depicts cross section of a grouted nailed wall along with some field photographs of the same in Fig. 1.3. In the present era, soil nailing is being carried out at large in railway construction work for the stabilization of side lopes in existing track-road or laying of new tracks adjoining to an existing one (Fig. 1.4).

Fig. 1.1 Soil nail with centralizers
(www.williamsform.com/Ground_Anchors/Soil_Nails_Soil_Nailing/soil_nail_soil_nailing.html)
Fig. 1.2 Cross-section of a grouted soil nailed wall
(www.williamsform.com/Ground_Anchors/Soil_Nails_Soil_Nailing/soil_nail_soil_nailing.html)

Fig. 1.3 Application of soil nailed wall in
(a) Highway (http://www.classes.ce.ttu.edu/CE5331_013/)
(b) Railway (http://www.geofabrics.com/docs/Tamworth.pdf)
1.2 Various types of soil nailing

Various types of soil nailing methods are employed in the field:

1. **Grouted nail**- After excavation, first holes are drilled in the wall/slope face and then the nails are placed in the pre-drilled holes. Finally, the drill hole is then filled with cement grout.

2. **Driven nail**- In this type, nails are mechanically driven to the wall during excavation. Installation of this type of soil nailing is very fast; however, it does not provide a good corrosion protection. This is generally used as temporary nailing.

3. **Self-drilling soil nail**- Hollow bars are driven and grout is injected through the hollow bar simultaneously during the drilling. This method is faster than the grouted nailing and it exhibits more corrosion protection than driven nail.

4. **Jet-grouted soil nail**- Jet grouting is used to erode the ground and for creating the hole to install the steel bars. The grout provides corrosion protection for the nail.

5. **Launched soil nail**- Bars are “launched” into the soil with very high speed using firing mechanism involving compressed air. This method of installation is very fast; however, it is difficult to control the length of the bar penetrating the ground.
1.3 Elements of nailed structure

Various components of a grouted soil nail are discussed in this section. The cross-section of a nailed wall is presented in Fig. 1.5 along with field photographs of various components in Fig. 1.6

1. **Steel reinforcing bars** – The solid or hollow steel reinforcing bars (with minimum strength of 415 kPa) are the main component of the soil nailing system. These elements are placed in pre-drilled drill holes and grouted in place.

2. **Centralizers** - PVC material, which is fixed to the soil nail to ensure that the soil nail is centered in the drill hole.

3. **Grout** – Grout is injected in the pre-drilled borehole after the nail is placed to fill up the annular space between the nail bar and the surrounding ground. Generally, neat cement grout is used to avoid caving in drill-hole; however, sand-cement grout is also applied for open-hole drilling. Grout transfers stress from the ground to the nail and also acts as corrosion protection to the soil nail. Grout pipe is used to inject the grout.

4. **Nail head** – The nail head is the threaded end of the soil nail that protrudes from the wall facing. It is a square shape concrete structure which includes the steel plate, steel nuts, and soil nail head reinforcement. This part of structure provides the soil nail bearing strength, and transfers bearing loads from the soil mass to soil nail.

5. **Hex nut, washer, and bearing plate** – These are attached to the nail head and are used for connecting the soil nail to the facing. Bearing plate distributes the force at nail end to temporary shortcrete facing.

6. **Temporary and permanent facing** – Nails are connected to the excavation or slope surface by facing elements. Temporary facing is placed on the unsupported excavation prior to advancement of the excavation grades. It provides support to the exposed soil, helps in corrosion protection and acts as bearing surface for the bearing plate. Permanent facing is placed over the temporary facing after the soil nails are installed.

7. **Drainage system** – Vertical geocomposite strip drains are used as drainage system media. These are placed prior to application of the temporary facing for collection and transmission of seepage water which may migrate to the temporary facing.

8. **Corrosion protection** - Protective layers of corrugated synthetic material [HDPE (High Density Polyethylene) or PVC tube] surrounding the nail bar is usually used to provide additional corrosion protection.
Fig. 1.5 Typical cross-section of a drilled soil nail wall
Fig. 1.6 Various elements of soil nailing (Yeung, 2008)
(a) Soil nail reinforcement Bars, (b) Typical Centralizers,
(c) Steel plate and Steel nuts head, (d) Steel plate and Steel nuts head
1.4 **Advantage and disadvantage of soil nailing**

Some advantage and disadvantage of soil nailing procedure are addressed in other literatures (Yeung, 2008, FHWA-SA-96-069R, FHWA-IF-03-017) and presented in this section.

1.4.1 **Advantage of soil nailing**

Soil nailing has several advantages over other ground anchoring and top to down construction techniques. Some of the advantages are described below:

- Less disruptive to traffic and causes less environmental impact than other construction techniques.
- Installation of soil nail walls is relatively faster and uses typically less construction materials. It is advantageous even at sites with remote access because smaller equipment is generally needed.
- Easy adjustments of nail inclination and location can be made when obstructions (e.g., cobbles or boulders, piles or underground utilities) are encountered. Hence, the field adjustments are less expensive.
- Compared to ground anchors, soil nails require smaller right of way than ground anchors as soil nails are typically shorter. Unlike ground anchor walls, soldier beams are not used in soil nailing, and hence overhead construction requirements are small.
- 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.
- It provides a less congested bottom of excavation, particularly when compared to braced excavations.
- 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.
- 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) soil nailing can result in a cost saving of 10 to 30 percent when compared to tieback walls. Shotcrete facing is typically less costly than the structural facing required for other wall systems.
1.4.2 Disadvantage of soil nailing

Some of the potential disadvantages of soil nail walls are listed below:

- In 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.
- Soil nail walls are not well-suited for grounds with high groundwater table for difficulty in drilling and excavation due to seepage of ground water into the excavation, corrosion of steel bars and change in grout water ratio.
- Soil nails are not suitable in cohesionless soils, because during drilling of hole, the un-grouted hole may collapse. However, in such a case drilling can be conducted by providing casing during the drilling process.
- Soil nails are drilled inside the slope wherein they might contain utilities such as buried water pipes, underground cables and drainage systems. Therefore, they should be placed at a safe distance, if possible, by changing its inclination or length or spacing to achieve this distance.
- Construction of soil nail walls requires specialized and experienced contractors.

1.5 Various issues affecting soil nailed slope

There are several factors that affect the feasibility and stability of soil nailing in slopes or excavations. As mentioned earlier, construction of soil nailing is subjected to favorable ground conditions. There are also various internal and global stability factors for soil nailed slopes.

- **Favorable ground condition** - Soil nailing is well suited for Stiff to hard fine-grained soils which includes stiff to hard clays, clayey silts, silty clays, sandy clays, sandy silts, and combinations of theses. It is also applicable for dense to very dense granular soils with some apparent cohesion (some fine contents with percentage of fines not more than 10-15%). Nailing is not suitable for dry, poorly graded cohesionless soils, soils with cobbles and boulder (difficult to drill and increases construction cost), highly corrosive soil (involves expensive corrosion protection), soft to very soft fine-grained soils, and organic soil (very low bond stress or soil nail interaction force leading to excess nail length). Soil nailing is also not recommended for soils with high ground water table.
• **External stability**- The external or global stability of nailed slope includes stability of nailed slope, overturning and sliding of soil-nail system, bearing capacity failure against basal heave due to excavation. Sometimes long-term stability problem also come into picture, e.g., seasonal raining. In such cases, though ground water table may be low, the seeping water may affect the stability of nailed slope without facing or proper drainage system.

• **Internal stability**- It comprises of various failure modes of nailed structure e.g. nail soil pull-out failure, nail tensile failure, and facing flexural or punching shear failure.

Such issues may be overcome by

- Conducting adequate ground investigation and geotechnical testing for identification of soil parameters and ground characterization.
- Performing in-situ test for soil nail interaction and nail strength.
- Effective design of nailed slope system.

Stability analysis is a major part in design of nailed slope structure. It involves proper evaluation of nail-soil interaction forces (bond stress) and nail strength which further requires interpretation from respective in-situ tests (nail pull-out capacity, nail tensile capacity test etc).

### 1.6 Construction procedure of nailed structure

Soil nailed structures are generally constructed in stages and it involves following steps:

- Excavation till the depth where nails will be installed at a particular level
- Drilling nail holes
- Nail installation and grouting
- Construction of temporary shotcrete facing

Subsequent levels are then constructed and finally permanent facing is placed over the wall. The details of the construction methodology and equipments are described in chapter 6. Some of the field photographs of soil nail construction procedure are presented in Fig. 1.7
Fig. 1.7 Construction of soil nailing

(a) Excavation (http://www.wmplanthire.com/slope_stabilisation.htm)

(b) Mobile drilling rig, (c) Steel bar Installation, (d) Grouting Process (Yeung 2008)

(e) Stage construction (http://www.keller-ge.co.uk/engineering/case-studies)
1.7 Testing and inspection

Soil nailing for slope or excavation involves various tests and monitoring at different stage of construction.

- **Before construction** - As mentioned earlier, ground exploration and geotechnical testing is conducted before commencement of excavation. It includes boring, sampling, field testing (SPT, CPT and ground water level determination), and lab experiments (grain size distribution, Atterberge limits, moisture content, consolidation, unconfined compression and triaxial tests). Test nails (5% of total nails required in construction) are used for nail pull-out test or ultimate test prior to the installation of nails for estimation of bond strength. Apart from ultimate test, some verification tests are also carried out on test nails.

- **During construction** - A minute inspection should be performed for quality control of the construction materials (storage and handling of nail tendons, reinforcements, cement, drainage material and checking of their required specification). Construction works do also need to be monitored properly at various stages (excavation, soil nail hole drilling, tendon installation, grouting, structural wall facing and drainage).

- **Performance monitoring** - It is important to monitor the performance of nailed slopes for improvement in future construction and design of such structures. Hence, some of the nailed slopes are instrumented for their performance monitoring. The parameters monitored are
  - Horizontal and vertical movement of wall face, surface and overall structure
  - Performance of any structure supported by the reinforced ground
  - Deterioration of facing and other soil nailing elements
  - Nail loads and change of distribution with time
  - Drainage behavior of ground

Slope inclinometer, electronic distance measuring equipments are installed at various survey positions on the nailed structure, and load cells are installed at nail head for such monitoring purpose.
1.8 Scope of the document

All the above discussed issues are technically addressed in details within this document. The outline of the document is as follows:

**Chapter 1- Introduction**- A brief description is presented on various types of soil nailing, its elements, construction methods, testing and field inspection method. Advantage, disadvantage and applicability of soil nailing are also discussed.

**Chapter 2- Geotechnical investigation and testing**- It includes ground profiling and inspection, various geotechnical field and lab tests for soil characterization, and tests for soil nail strength and interaction with soil.

**Chapter 3- Back ground theory**- In this chapter, the back ground theory of stability analysis methods for unreinforced and nailed slopes is discussed along with examples. Calculation procedure for bearing capacity analysis against heave and bond strength is also given.

**Chapter 4- Design of nailed soil slopes**- Detailed soil nailing design method is discussed with nail slope failure modes. Necessary design parameters and guideline for their specifications are also mentioned.

**Chapter 5- Example problems on nailed slope design**- Example problems on design of nailed slopes with different slope geometries and soil properties are presented in this chapter. Stability analysis results are first presented for the same slopes with lower slope angle when no nails are applied. Then they are designed with higher slope face angle by applying nails.

**Chapter 6- Construction procedure of nailed slope and construction equipments**- Construction method of nailed slopes are discussed along with construction equipments related to soil nailing.

**Chapter 7- Inspection and monitoring**- Monitoring of various parameters of nailed slope and their instrumentation method is given.
2.1 Introduction

Geotechnical investigation is one of the most crucial aspects for identifying technical feasibility and cost effectiveness of any geotechnical construction work. It includes ground characterization and laboratory testing for determination of soil parameters. After site reconnaissance and evaluation of existing surface profile, boring and sampling is conducted for detailed characterization of ground stratification and collection of soil samples. Laboratory experiments are performed on the collected soil samples according to the Indian Standard recommendation. The soil parameters generally provided in the geotechnical investigation include soil classification, unit weight, shear strength and compressibility. Another important parameter in nailed slope design is the position and seasonal variation of ground water table. Apart from its effect on long term stability of slope, presence of high water table creates problem during construction of nailed slopes and thus, render them to be unsuitable. Hence, geotechnical investigation for nailed slopes also includes a minute examination in order to determine the position of ground water table. Soil nail pull-out tests are also performed in the field to find out soil-nail interaction, i.e., bond strength which is a critical parameter in design of nailed system. General information regarding geotechnical field investigation, lab test and pull-out tests are presented in this chapter which are required for proper evaluation of ground condition and effective design of soil nailed structure.

2.2 Site reconnaissance

Before commencing the field investigation, work site reconnaissance is carried out in any geotechnical project. It includes

- Review of existing regional, site and subsurface information from existing geotechnical reports prepared for the site or nearby areas, topographic maps, site plans, geologic maps, air photos, construction plans, surveys and geological data.
- Gathering information regarding groundwater level near the project site and data on seismic aspects, such as ground motion, liquefaction potential, and site amplification.
• Collection of data on the performance of existing engineered structures (including soil nail walls or comparable systems such as cuts, slopes and excavations) in the area.
• Visual inspection of site and collecting data regarding site accessibility, overhead space limitation, identification of underground utilities, nature of above ground structures, traffic condition and control during investigation and construction.
• Inspection for surface settlement, drainage and erosion patterns.

After a detail review of existing ground information and site reconnaissance, the necessary subsurface investigation scheme is prepared considering existing information and additional project requirement.

2.3 Subsurface investigation

The objective of subsurface investigation is to identify the subsurface condition and its variation in lateral and special direction. Any geotechnical subsurface investigation consists of the following steps

1. In-situ testing of soil properties
2. Retrieval of soil samples for visual identification and lab testing
3. Characterization of stratification
4. Identification of ground water level

Determination of location and nature of ground water table is one of the most crucial factor for soil nail projects. These systems are difficult to construct and more costly when the groundwater is high.

Subsurface investigations are performed in accordance with the Indian Standard recommendation (IS 1892: 1979 Code of practice for subsurface investigations for foundations). The following sections present different aspects of subsurface investigations generally used in soil nailing projects and Table 2.1 depicts a brief outline regarding their applicability, activity outcomes and specific IS standards.
<table>
<thead>
<tr>
<th>Field procedures and tests</th>
<th>IS standard</th>
<th>Field activity outcome</th>
<th>Note on suitability</th>
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<tr>
<td>Subsurface exploration</td>
<td>IS 1892: 1979</td>
<td>Site soil stratification, soil property</td>
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<tr>
<td>Sampling</td>
<td></td>
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<td></td>
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<tr>
<td>1. Thin walled tube</td>
<td>IS 2132 : 1986</td>
<td>Undisturbed sample collection</td>
<td>Clays and silts</td>
</tr>
<tr>
<td>sampling</td>
<td></td>
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<tr>
<td>2. Undisturbed sampling of</td>
<td>IS 8763 : 1978</td>
<td></td>
<td>Sands and sandy soils</td>
</tr>
<tr>
<td>sands and sandy soils</td>
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<tr>
<td>Field Tests</td>
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<tr>
<td>Test (SPT)</td>
<td>IS 2131 : 1981</td>
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<tr>
<td>2. Cone Penetration Test</td>
<td>IS 4968 : Part 3 : 1976</td>
<td>Continuous stratification, soil type, strength, relative density, K₀, pore pressures; no sample collection</td>
<td>Sand, silt and clay; not applicable for gravelly soil</td>
</tr>
<tr>
<td>(CPT)</td>
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</tbody>
</table>
2.3.1 Boring

Boring is the first step carried out during field testing and sampling process. It provides

- Disturbed and undisturbed soil samples
- Ground water table location
- SPT N-values for characterization of soil sample and identification of soil stratification

Boring type, number, location, and depth of borings are mainly selected based on the project stage (i.e., feasibility study, preliminary, or final design), availability of existing geotechnical data, variability of subsurface conditions and other project constraints. Fig. 2.1 presents a preliminary guideline for selection of number, location, and frequency of borings for soil nailed structure (FHWA-0-IF-03-017). For soil nail walls more than 30 m long, borings should be spaced between 30 to 60 m along the proposed centerline of the wall. For walls less than 30 m long, at least one boring is necessary along the proposed centerline of the wall. Borings are also necessary in front and behind the proposed wall. Borings behind the wall should be located within a distance up to 1 to 1.5 times the height of the wall behind the wall and should be spaced up to 45 m along the wall alignment. If the ground behind the proposed wall is sloping, the potentially sliding mass behind the wall is expected to be larger than for horizontal ground. Therefore, borings behind the proposed wall should be located farther behind the wall, up to approximately 1.5 to 2 times the wall height. Borings in front of the wall should be located within a distance up to 0.75 times the wall height in front of the wall and should be spaced up to 60 m along the wall alignment.

The depth of boring are selected based on the depth of excavation or wall height and variation in subsurface profile. For railway projects blasting or excavation methods are carried at the initial stage to obtain a suitable ground profile and subsequent laying of new or extension of existing railway tracks,. Soil nailing is then applied for stabilization of side slopes adjacent to the rail-tracks. Hence, for such cases boring can be conducted before or after the blasting or excavation process. In case of borings before excavation, boring should extend at least twice of the slope height from the ground level. For boring after excavation, boring depth should extend up to one full wall or slope height below the bottom of the excavation (Fig. 2.1) or till hard stratum reached. Boring should be deeper when highly compressible soils (i.e., soft to
Fig. 2.1 Preliminary geotechnical boring layout for soil nailed wall
very soft fine-grained soils, organic silt, and peat) occur at the site behind or under the proposed soil nail wall. The required boring depths for soil nail wall projects may be greater if deep loose, saturated, cohesionless soils occur behind and under the proposed soil nail wall and the seismic risk at the site require that the liquefaction potential be evaluated. The subsurface investigation depths may need to be deep at proposed sites of soil nail walls where seismic amplification is of concern, particularly in deep, soft soils.

2.3.2 Field testing
Field tests are conducted for identification of stratification, characterization of soil property and collection of disturbed samples. Such tests are carried out as per the guidelines provided in SP 36: Part 2: 1988 of Indian Standard. Standard Penetration Test (SPT) and Cone Penetration Test (CPT) are the most widely used field tests in soil nailed projects.

SPT provides the SPT N-value, which is the measured number of blows required to drive a standard split-spoon sampler a distance of 300 mm at the bottom of boreholes. SPT tests are carried out as per the instructions given in IS 2131: 1981 and corrections for SPT N-values are also applied according to the guidelines mentioned in the code. Several correlations between SPT N-values and engineering properties are available and thus it can be used for soil characterization. The SPT is also used to obtain disturbed samples from the subsurface, typically spaced at vertical intervals of 1.5 and 3 m. In layers with loose or soft soil, or when other features of interest are encountered (e.g., soil lenses and highly inhomogeneous conditions), sampling should be continuous. The SPT provides a good measurement of the relative density of cohesionless soils (Table 2.2). With limitations, the SPT can also provide an estimate of the consistency of fine grained soils (Table 2.3).

CPT tests are comparatively rapid and cost effective and are performed according to IS 4968: Part 3: 1976. Because of continuous soil profiling, this technique permits the identification of thin soil layers that would be otherwise difficult to detect within a relatively homogeneous soil mass. Such identification of the presence of thin layers of weak soil is useful, as it may initiate instability behind the proposed soil nail wall. The major disadvantage of this technique is that no sample is recovered. Additionally, CPT cannot be performed in soils with gravel and boulders.
Table 2.2 Cohesionless soil density prediction from SPT N-values  
(Meyerhof, 1956)

<table>
<thead>
<tr>
<th>Relative density</th>
<th>SPT N-values</th>
<th>Friction angle $\phi$ (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose</td>
<td>&lt; 4</td>
<td>25-30</td>
</tr>
<tr>
<td>Loose</td>
<td>4-10</td>
<td>27-32</td>
</tr>
<tr>
<td>Medium dense</td>
<td>10-30</td>
<td>30-35</td>
</tr>
<tr>
<td>Dense</td>
<td>30-50</td>
<td>35-40</td>
</tr>
<tr>
<td>Very dense</td>
<td>&gt; 50</td>
<td>38-43</td>
</tr>
</tbody>
</table>

Table 2.3 Fine grained soil consistency prediction from SPT N-values  
(Ranjan and Rao, 2004)

<table>
<thead>
<tr>
<th>Consistency</th>
<th>SPT N-values</th>
<th>Unconfined compressive strength $C_u$ (kg/cm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very soft</td>
<td>&lt; 2</td>
<td>&lt; 0.25</td>
</tr>
<tr>
<td>Soft</td>
<td>2-4</td>
<td>0.25-0.50</td>
</tr>
<tr>
<td>Medium stiff</td>
<td>4-8</td>
<td>0.50-1.00</td>
</tr>
<tr>
<td>Stiff</td>
<td>8-16</td>
<td>1.00-2.00</td>
</tr>
<tr>
<td>Very stiff</td>
<td>16-32</td>
<td>2.00-4.00</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt; 32</td>
<td>&gt; 4.00</td>
</tr>
</tbody>
</table>
For some large projects, the phased use of CPT and conventional borings may be applied for comparatively more geotechnical information at costs that are comparable than with conventional borings alone. In the first phase, the CPT soundings may allow rapid depiction of the soil stratigraphy and early identification of layers with potential deficiencies (e.g., low strength or high compressibility) that may have an impact in the design. An initial CPT-based stratigraphy can help determine the location of zones where undisturbed soil samples should be obtained. In the second phase, conventional borings can be used and samples are obtained only at the depths of interest. Using this two-phase investigation strategy, sampling can be optimized and the number of samples can be reduced.

2.3.3 Sampling

Both disturbed and undisturbed samples are collected from the field during field investigation. Samples obtained with the SPT sampler are disturbed and they are only adequate for soil classification and some laboratory tests such as particle gradation (sieve analysis), fines content, natural moisture content, Atterberg limits, specific gravity of solids, organic contents, unconfined compressive strength test (UC) and unconsolidated-undrained triaxial compression (UU). SPT samples are not used for strength or compressibility testing. Soil samples are disturbed excessively as the SPT sampler has a large wall thickness/diameter ratio. As the shear strength and compressibility of fine-grained soils are heavily affected by sample disturbance, samples obtained with the SPT standard split-spoon sampler are unsuitable for laboratory testing of shear strength and compressibility of fine-grained soils. Samples obtained from cuttings in borings, test pits, and test cuts can also be used for soil classification and laboratory determination of index parameters, as long as they are sufficiently representative and the in situ moisture content was preserved during sampling and transportation.

Undisturbed thin-walled samplers, including the Shelby tube sampler with an outer diameter (OD) range of 76-100 mm, are used to obtain samples of fine-grained soil for laboratory testing of shear strength and consolidation. The method of undisturbed sampling by thin-walled samplers are conducted as per the code IS 2132: 1986 and IS 8763: 1978 for clayey and sandy soils respectively.
2.3.4 **Stratification**

After completion of boring and field testing, it is important to examine the soil stratigraphy of the construction site and to identify presence of any significant spatial variability of subsurface conditions which may affect the design the construction of soil nailed structure. Development of soil site stratification is critical for soil nail walls because the nature, extent, and distribution of the various layers dictate the type of drilling equipment and methods, control the size of the potential sliding soil mass behind the wall, and have an impact on the soil nail lengths. The identification of varying subsurface conditions on plan view is particularly important in long walls, where soil conditions are more likely to vary considerably.

Soil stratifications are initially assessed based on visual logging or in-situ testing results during the site investigation and then subsequently corroborated or adjusted through laboratory testing results.

2.3.5 **Ground water table position**

The presence of water (unsaturated or saturated conditions) in the soil may affect various aspects of the design and long-term performance of a soil nail wall. These aspects include stability of temporarily unsupported cuts, soil strength and bond strength, corrosion potential, pressure on the facing, drillhole stability, grouting procedures, drainage, and other construction considerations. Therefore, the presence of a groundwater table and/or perched groundwater zones must be identified during the subsurface investigation program.

Groundwater depth should be obtained from borings during drilling and should be monitored for at least 24 hours after drilling. If drilling fluid is used during boring advancement, it may not be possible to locate the groundwater in borings. For soils exhibiting relatively high fines content, the groundwater levels obtained during drilling do not commonly represent stabilized levels of the groundwater table, as the observed levels of water are most likely affected by the relatively low permeability of the surrounding soil. In these soils, it is recommended to measure the groundwater level a few times over the course of a few hours or days to allow groundwater to reach its equilibrium level. In soils with very low permeability, more extended periods of time, up to several weeks or months, may be necessary for the groundwater level to stabilize. For these cases, some of the exploratory borings may be converted into piezometers. It is desirable to obtain (or estimate) the seasonal (high and low) groundwater levels from piezometers or other sources (e.g., existing nearby wells).
Improper elevation of groundwater during a field investigation can have serious consequences for any earth retaining system. This is an important matter of concern for soil nailed structures as these systems are not particularly suited to high groundwater conditions, as discussed in the previous chapter.

2.4 Laboratory testing of soil sample

Laboratory tests are conducted on the soil samples collected during the site investigation to produce soil classification, index properties, unit weight, strength, and compressibility. General laboratory testing of soil samples are carried out in accordance to the recommendations provided in SP 36: Part 1: 1987 of Indian Standard. Table 2.4 presents laboratory tests commonly used to develop index parameters and other engineering properties of soils that may be necessary for the design of a soil nail wall. It also presents the relevant codes of the soil testing along with their applicability to specific type of soil.

2.4 Field test for pull-out capacity

Field pull-out capacity tests are carried out to determine the bond strength which is necessary for design of soil nailed structures. Sometimes such tests are also carried out during construction of soil nails to verify the construction performance and uniformity of installation.

Fig. 2.2 presents the schematic diagram of the instrumentation process for the pull-out test. A center-hole hydraulic jack and hydraulic pump are used to apply a test load to a nail bar. The axis of the jack and the axis of the nail must be aligned to ensure uniform loading. Typically, a jacking frame or reaction block is installed between the shotcrete or excavation face and the jack. The jacking frame should not react directly against the nail grout column during testing. Once the jack is centered and aligned, an alignment load is applied to the jack to secure the equipment and minimize the slack in the set-up. The alignment load should not be permitted to exceed 10 percent of the maximum test load. The movement of the nail head is measured with at least one, and preferably two, dial gauges mounted on a tripod or fixed to a rigid support that is independent of the jacking set-up and wall. The use of two dial gauges
### Table 2.4 Various laboratory experiments for soil property characterization

<table>
<thead>
<tr>
<th>Property</th>
<th>Test name</th>
<th>Standard</th>
<th>Applicability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Classification</td>
<td>Classification of soil according to Indian Standard</td>
<td>IS 1498: 1970</td>
<td>All soils</td>
</tr>
<tr>
<td>Index parameters</td>
<td>Particle-Size Analysis (with sieves)</td>
<td>IS 2720: Part 4: 1985</td>
<td>All soils</td>
</tr>
<tr>
<td></td>
<td>Particle-Size Analysis for fraction passing 75 μm</td>
<td>IS 2720: Part 4: 1985</td>
<td>Fine grained soil</td>
</tr>
<tr>
<td></td>
<td>Moisture content</td>
<td>IS 2720: Part 2: 1973</td>
<td>All soils</td>
</tr>
<tr>
<td></td>
<td>Atterberg limits</td>
<td>IS 2720: Part 5: 1985</td>
<td>Fine grained soil</td>
</tr>
<tr>
<td></td>
<td>Specific gravity</td>
<td>IS 2720: Part 3/Sec 1 and Sec 2: 1980</td>
<td>All soils</td>
</tr>
<tr>
<td>Strength</td>
<td>Unconfined Compressive Strength (UC)</td>
<td>IS 2720: Part 10: 1991</td>
<td>Fine grained soil</td>
</tr>
<tr>
<td></td>
<td>Unconsolidated-Undrained Triaxial Compression (UU)</td>
<td>IS 2720: Part 11: 1993</td>
<td>Fine grained soil</td>
</tr>
<tr>
<td>Compressibility</td>
<td>One-Dimensional Consolidation</td>
<td>IS 2720: Part 15: 1986</td>
<td>Fine grained soil</td>
</tr>
</tbody>
</table>

**Note:** In case of soil nailing, UC test is generally performed to obtain the shear strength parameter. Whereas, UU test data are reliable for designing nailed structures constructed at sites with considerable seasonal variation in ground water table.
Fig. 2.2 Schematic diagram of the instrumentation process for the pull-out test

Fig. 2.3 Typical force-displacement curves of pull-out test (Su et al., 2008)
provides: (1) an average reading in case the loading is slightly eccentric due to imperfect alignment of the jack and the nail bar, and (2) a backup if one gauge malfunctions. The dial gauges should be aligned within 5 degrees of the axis of the nail, and should be zeroed after the alignment load has been applied. The dial gauges should be capable of measuring to the nearest 0.02 mm. The dial gauges should be able to accommodate a minimum travel equivalent to the estimated elastic elongation of the test nail at the maximum test load plus 25 mm or at least 50 mm.

The hydraulic jack is used to apply load to the nail bar while, a pressure gauge is used to measure the applied load. The jack, pressure gauge, and load cell are calibrated prior to testing. The pull out tests is continued till pull out failure of the nail bar takes place. Pullout failure is defined as the load at which attempts to further increase the test load increments results in continued pullout movement of the tested nail and the load at this stage is called the pull out capacity. Some typical force-displacement curves of pull-out test are presented in Fig. 2.3 (Su et all, 2008).

2.5 Summary
This chapter summarizes the geotechnical investigation procedure involved with the soil nailing projects. Site reconnaissance, commonly used in-situ tests, sampling procedure and details of the laboratory test program are illustrated in this respect along with the reference to their specific Indian Standards. Details of field pull-out test of the soil nails have also been discussed. Such tests are carried out to identify the bond strength or the nail-soil interaction.
CHAPTER 3
BACKGROUND THEORY

3.1 Introduction
This chapter pertains to the background theory for analyzing stability of nailed slope structures. This includes global slope stability, bearing capacity against heave and interpretation of bond strength for calculating nail pull-out capacity. In this respect, first some common methods for slope stability analysis without nails are discussed. Next the same methods are extended for analyzing stability of nailed slopes. Bearing capacity failure under heave is also addressed in this chapter. This is an important aspect for soil nail wall excavated in fine-grained soft soils. Finally, a method for calculating nail pull-out capacity is presented accounting for bond strength. Illustrative examples are presented at the end of each section.

3.2 Slope stability without nails
Global stability of soil nailed slope is generally analyzed using two-dimensional limit-equilibrium principles which are similar to conventional slope stability methods. In limit-equilibrium analysis, the potentially sliding mass is modeled either as a rigid block or series of slices, global force and/or moment equilibrium is established and a stability factor of safety that relates the stabilizing and destabilizing effect is calculated. In case of slope stability for static analysis, the allowable factor of safety is usually considered to be 1.5. Various potential failure surfaces are evaluated until the most critical surface corresponding to lowest factor of safety is obtained. Different shapes of the failure surface have been considered in various methods to analyze the global stability of slopes with or without soil nailing. In this section, some of the commonly used slope stability analysis methods (Abramson et al., 1996) are first discussed. In the subsequent section, those methods are further used in analysis of nailed slopes.

3.2.1 Single-wedge with planar surface
A simple, single-wedge failure mechanism is shown in Fig. 3.1.
Fig. 3.1 Slope failure as single-wedge with planar surface

Where,

$\alpha_i$ = Inclination of the back slope

$\beta_i$ = Slope face angle with respect to the vertical

$\theta_i$ = Inclination of failure plane

$\phi$ = Soil effective angle of internal friction

$c$ = Soil effective cohesion

$L$ = Length of failure plane

$W$ = Weight of the sliding mass

$Q$ = Surcharge load

$N_F$ = Normal force on failure surface

$S_F$ = Shear force on failure surface

The destabilizing forces consist of the driving components of the weight ($W$) and the surcharge load ($Q$) and the stabilizing force along the failure surface is the mobilized shear
force \( (S_r) \). 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:

\[
FS_g = \frac{\sum \text{resisting forces}}{\sum \text{driving forces}} \quad (3.1)
\]

The weight of the wedge can be determined from the geometry using

\[
L = \frac{H}{\sin \beta_i} \left[ \frac{\sin(\beta_i - \alpha_i)}{\sin(\theta_i - \alpha_i)} \right] \quad (3.2)
\]

\[
W = \frac{1}{2} \gamma H^2 \left[ \frac{\sin(\beta_i - \theta_i)}{\sin^2 \beta_i}, \frac{\sin(\beta_i - \alpha_i)}{\sin(\theta_i - \alpha_i)} \right] \quad (3.3)
\]

The normal and tangent forces on the failure plane are:

\[
\sum \text{Normal Forces} = (W + Q) \cos \theta - N_F = 0 \quad (3.4)
\]

\[
\sum \text{Tangent Forces} = (W + Q) \sin \theta - S_F = 0 \quad (3.5)
\]

Where,

\[
S_F = c_m L + N_F \tan \phi_m \quad (3.6)
\]

\[
\tan \phi_m = \frac{\tan \phi}{FS_g} \quad (3.7)
\]

\[
c_m = \frac{c}{FS_g} \quad (3.8)
\]

\( \phi_m \) is the mobilized friction angle, and \( c_m \) is the mobilized cohesion. A single global factor of safety is used for the cohesive and friction strength components of the soil. However, it is possible to select different factor of safety for each strength component. Wedge or block analysis method does not consider the distribution of the normal stress along the failure surface.

### 3.2.2 Circular arc method

The simplest circular slip surface analysis is based on the assumption that a rigid, cylindrical block will fail by rotation about its centre (Fig. 3.2). The method is suitable for total stress analysis where shear strength along the failure surface is defined by the undrained strength \( (\phi = 0, c = S_u) \). The FOS for such slope is analyzed by taking the ratio of the resisting and overturning moments about the centre of circular surface.
If the over turning moment and resisting moment are given by $W_x$ and $S_u LR$ respectively, the factor of safety ($FS_G$) for the slope may be given by

$$FS_G = \frac{S_u LR}{W_x} \quad (3.9)$$

Where,

- $\theta$ = Angle subtended by the slip circle at centre, O
- $L$ = Length of the slip surface = $R\theta$
- $R$ = Radius of circular slip surface
- $S_f$ = Shear force on failure surface
- $S_u$ = Undrained shear strength
- $W$ = Weight of sliding mass
- $\bar{x}$ = Horizontal distance between circle centre, O and the centre of the sliding mass.

### 3.2.3 Friction circle method
Friction circle method is useful for soil with $\phi > 0$ where shear strength depends on normal stress. In other words, the method is useful when both cohesive and frictional component for shear strength have to be considered in the calculation. It is equally suitable for total or effective stress type of analysis in soil. In the stability analysis, failure surface is assumed to be a circular arc.

The method attempts to satisfy the requirement of complete equilibrium by assuming the direction of the resultant of normal and frictional component of strength mobilized along the failure surface (Fig. 3.3). This direction corresponds to a line that forms tangent to the friction circle with a radius, $R_f = R \sin \phi$. This is equivalent to assuming that the resultant of all normal stress acting on the failure surface is concentrated at one point. $\phi$ is calculated from equation Eq.3.7. The cohesive shear stresses along the base of failure surface will have a resultant, $C_m$, that acts parallel to the direction of the chord. Its location may be found by taking moments of distribution and the resultant $C_m$ about the circle centre, and is given by

$$R_c = \frac{L_{arc}}{L_{chord}} R$$

(3.10)

Where,

$R$ = Radius of failure circle

$R_c$ = Perpendicular distance from the circle centre to force, $C_m$

$L_{arc}$, $L_{chord}$ = Lengths of the circular arc and chord defining the failure surface.
Fig. 3.3 Friction circle method for analysis of slope stability

The actual point of application, A is located at the intersection of the effective weight force \( W \), which is resultant to weight \( W \). As the direction of \( C_m \) is known, the force polygon can be closed to obtained the value of the mobilized cohesive force (Fig. 3.3). Next, \( F_c \) is calculated from Eq.3.8 using this calculated \( C_m \) and given soil cohesion value, i.e., \( c \). The final factor of safety \( (FS_c) \) is computed in an iterative way assuming that it will be the same against cohesion \( (F_c) \) and friction \( (F_\phi) \).

\[
FS_c = F_c = F_\phi
\]  

(3.11)

The solution procedure is usually followed graphically.

3.2.4 Simplified Bishop Method

In this method, the mass of the failure slope is analyzed by discretizing it into smaller slices and treating each individual slice as a unique sliding block (Fig. 3.4). A circular slip surface is assumed in the analysis. The normal stress on the base of the slice is considered to be acting at the midpoint. Simplified Bishop method satisfies vertical force equilibrium for each slice and overall moment equilibrium about the center of the circular trial surface. It does not assume any interslice force and fails to satisfy horizontal force equilibrium equation for one slice. Forces acting on a typical slice are shown in Fig. 3.5.
Fig. 3.4 Division of potential sliding mass into slices

Fig. 3.5 Forces acting on a typical slice
The terms in Fig. 3.5 are described below.

\( \alpha = \) Inclination of slice base

\( \beta = \) Inclination of slice top

\( \delta = \) Angle of line of action of surcharge with vertical

\( b = \) Width of slice

\( h = \) Average height of slice

\( FS_G = \) Factor of safety

\( S_a = C + N' \tan \phi = \) Available strength

\( S_F = \frac{S_a}{FS_G} = \) Mobilized strength

\( W = \) Weight of the slice

\( N = \) Normal force

\( Q = \) External surcharge

The overall moment equilibrium of the forces acting on each slice is given by

\[
\sum M_0 = \sum_{i=1}^{n} [W + Q \cos \delta]R \sin \alpha
- \sum_{i=1}^{n} [Q \sin \delta](R \cos \alpha - h)
- \sum_{i=1}^{n} [S_F]R
= 0
\]

(3.12)

The Normal force, acting on the base of the slice, does not affect the moment equilibrium expression since they are directed through the centre of the circle.

If the FOS against shear failure is defined as \( FS_G \), and is assumed to be the same for all slices, the Mohr-Coulomb mobilized shear strength, \( S_F \), along the base of each slice is given by

\[
S_F = \frac{C + N \tan \phi}{FS_G}
\]

(3.13)

Replacing Eq.3.13 in Eq.3.12 and rearranging

\[
FS_G = \frac{\sum_{i=1}^{n} C + N \tan \phi}{\sum_{i=1}^{n} A_1 - \sum_{i=1}^{n} A_2}
\]

(3.14)
Where,
\[ A_i = [W + Q \cos \delta] \sin \alpha \]
\[ A_2 = (Q \sin \delta)(R \cos \alpha - \frac{h}{R}) \]  

(3.15)

Next, from vertical force equilibrium of each slice
\[ \sum F_v = N \cos \alpha + S_r \sin \alpha - W - Q \cos \delta = 0 \]  

(3.16)

The above equation may be arranged for \( N \) as
\[ N = -S_r \sin \alpha + W + Q \cos \delta \]

(3.17)

Again, substituting Eq.3.13 in Eq.3.17
\[ N = \frac{1}{m_\alpha} \left[ W - \frac{C \sin \alpha}{FS_G} + Q \cos \delta \right] \]  

(3.18a)

Where,
\[ m_\alpha = \cos \alpha [1 + \frac{\tan \alpha \tan \phi}{FS_G}] \]  

(3.18b)

Eq.3.12 through 3.18 are the expressions which are involved in the calculation of FOS for circular surfaces according to the simplified Bishop method.

### 3.2.5 Simplified Janbu Method

Simplified Janbu method also uses method of slices to determine the stability of the sliding mass and assumes that there are no interslice shear forces. Forces acting on a typical slice are the same as in simplified Bishop method and presented in Fig. 3.5. This method is applicable to all types of slope analyses irrespective of the assumed shape of trial slip surface. Though the method satisfies both the vertical and horizontal force equilibrium for each slice, it fails to satisfy moment equilibrium.

As derived in the previous case, the normal force obtained from the vertical equilibrium of the slice is given by
\[ N = \frac{1}{m_\alpha} \left[ W - \frac{C \sin \alpha}{FS_G} + Q \cos \delta \right] \]

and \( m_\alpha = \cos \alpha [1 + \frac{\tan \alpha \tan \phi}{FS_G}] \)  

(3.19)

In case of an individual slice \( i \), the horizontal force equilibrium equation is given by
\[ [F_{ij}] = N \sin \alpha - Q \sin \delta - S_F \cos \alpha = 0 \]  
(3.20)

Substituting \( S_F \) from Eq.3.13 and overall horizontal force equilibrium for the sliding mass

\[
\sum_{i=1}^{n} [F_{ij}] = \sum_{i=1}^{n} [N \sin \alpha - Q \sin \delta] - \sum_{i=1}^{n} \left[ \frac{C + N \tan \phi}{FS_G} \cos \alpha \right] = 0
\]  
(3.21)

By rearranging the above equation, the following equation may be obtained

\[
\sum_{i=1}^{n} [N \sin \alpha - Q \sin \delta] = \sum_{i=1}^{n} \frac{1}{FS_G} (C + N \tan \phi) \cos \alpha
\]  
(3.22)

Then if each slice has the same FOS, \( F \)

\[
FS_G = \frac{\sum_{i=1}^{n} [C + N \tan \phi] \cos \alpha}{\sum_{i=1}^{n} A_i + \sum_{i=1}^{n} N \sin \alpha}
\]  
(3.23)

Where \( N \) is given by Eq.3.19 and the term \( A_i \) is given by

\[
A_i = -Q \sin \delta
\]  
(3.24)

The reported Janbu FOS value is calculated by multiplying the calculated \( F \) value by a modification factor \( f_0 \)

\[
FS_{G,Janbu} = f_0 \cdot FS_{G,calculated}
\]  
(3.25)

The modification factor is a function of the slide geometry and the strength parameters of the soil. Fig. 3.6 presents the variation of the \( f_0 \) value as a function of the slope geometry (d and L) and type of soil. The curves were presented by Janbu in an attempt to compensate for the assumption of negligible interslice forces in his formulation for the simplified method. These correction curves were generated after performing analysis on both simplified and rigorous (i.e., satisfying complete equilibrium) methods for the same slopes with homogeneous soil conditions and comparing the obtained FOS values.
Fig. 3.6 Magnitude of modification factor $f_0$ (Source: Abramson et al., 1996)
3.3 Slope stability with nails

In this section, the stability of nailed slopes is discussed with the modified equilibrium equations incorporating the effect of soil nails. Here also the allowable factor of safety is considered as 1.5 for static condition. Only nail tension is considered in the present analysis as bending and shear force of soil nails has a lesser effect in stabilization of nailed slope (Jewell and Pedley, 1992).

Nail tension \( (T_j) \) is calculated based on the available pull-out resistance of the soil nail. The available pull-out resistance is equal to either the bond strength between the soil and reinforcement to be obtained from pull-out test on site or the tensile strength of the reinforcement, whichever is lesser. The in-situ tests for choosing nail pull-out capacity and tensile strength are discussed in chapter 2. Bond strength of soil nails along with the method of calculating pull-out capacity \( (T_c) \) is discussed further in section 3.6.

Fig. 3.7 Slope failure as single-wedge with planar surface

3.3.1 Single-wedge with planar surface

Single-wedge failure mechanism of nailed slope is shown in Fig. 3.7. In this case, the destabilizing forces consist of the driving components of the weight \( (W) \) and the surcharge
load \((Q)\) and the stabilizing forces along the failure surface are the mobilized shear force \((S_r)\) and the equivalent nail tensile force \((T_{EQ})\).

The normal and tangent forces on the failure plane are:

\[
\sum \text{Normal Forces} = (W + Q) \cos \theta + T_{EQ} \cos(\theta - \lambda) - N_F = 0 \tag{3.26}
\]
\[
\sum \text{Tangent Forces} = (W + Q) \sin \theta - T_{EQ} \sin(\theta - \lambda) - S_F = 0 \tag{3.27}
\]

The equivalent nail tensile force \((T_{EQ})\) can be determined by plotting the force polygon as shown in Fig. 3.8.

Fig. 3.8 Force polygon for determining equivalent nail tensile force \((T_{EQ})\)

If all the nails are with same inclination \((\lambda)\) then \(T_{EQ}\) is given by

\[
T_{EQ} = \sum_{j=1}^{nl} T_j \tag{3.28}
\]

Where,
\(\theta = \text{Inclination of failure plane}\)
\(\lambda = \text{Nail inclination of equivalent nail tensile force}\)
\(\lambda_j = \text{Nail inclination of } j^{th} \text{ nail}\)
\(nl = \text{Total number of nail used}\)
\(T_{EQ} = \text{Equivalent nail tensile force}\)
\(T_j = \text{Tensile force in } j^{th} \text{ nail}\)
\(W = \text{Weight of the sliding mass}\)
\(Q = \text{Surcharge load}\)
$N_F$ = Normal force on failure surface

$S_F$ = Shear force on failure surface

Weight of the slice ($W$), mobilized shear force ($S_F$) and the global factor of safety ($FS_g$) are calculated from Eq. 3.1-3.3 and Eq. 3.6-3.8.

![Fig. 3.9 Nailed slope failure as single-wedge with circular slip surface (for $\phi = 0$ soil)](image)

### 3.3.2 Circular arc method

Circular arc failure for nailed slope is shown in Fig. 3.9. The modified equation for such block stability analysis is given by

$$FS_g = \frac{S_u R^2 \theta + (T_{EQ} \cos \lambda)(R \cos \beta)}{Wx + T_{EQ} \sin \lambda R \sin \beta}$$

(3.29)

Where,

$\theta$ = Angle subtended by the slip circle at centre, O

$\beta$ = Angle between horizontal plane and the tangent at point of intersection of nail equivalent force with failure surface

$\lambda$ = Nail inclination of equivalent nail tensile force

$L$ = Length of the slip surface = $R\theta$

$R$ = Radius of circular slip surface

$S_F$ = Shear force on failure surface
\[ S_u = \text{Undrained shear strength} \]
\[ T_{EQ} = \text{Equivalent nail tensile force (determined as mentioned in section 3.3.1)} \]
\[ W = \text{Weight of sliding mass} \]
\[ x = \text{Horizontal distance between circle centre, O and the centre of the sliding mass.} \]

### 3.3.3 Friction circle method

The method for calculation of factor of safety of nailed slope by friction circle method is illustrated in Fig. 3.10.

The method of calculation is similar as described in section 3.2.3. The resultant force of weight of block \((W)\) and equivalent nail force \((T_{EQ})\) passes through the point B. As mentioned in section 3.2.3, the direction of \(C_m\) is parallel to the chord which is shown in dotted line in the Fig. 3.10. Now, the value of mobilized cohesive force is calculated from the closed force polygon (Fig. 3.10). The final factor of safety \((FS_c)\) is computed with the assumption that it will be the same against cohesion \((F_c)\) and friction \((F_\phi)\).

\[ FS_c = F_c = F_\phi \]  \hspace{1cm} (3.30)
3.3.4 Simplified Bishop Method

As mentioned earlier, method of slices with circular failure surface is employed in this method for analyzing stability of nailed slope. Only those nail tensile forces are considered in the equilibrium equations of the slices which are from the reinforcements emerging out of the base of the slices. Slices are selected in such a manner that only one nail emerges out from the mid-point of the base of the slice. Forces acting on a typical slice are presented in Fig. 3.11.

Where,
\( \alpha \) = Inclination of slice base
\( \beta \) = Inclination of slice top
\( \delta \) = Angle of line of action of surcharge with vertical

Fig. 3.11 Forces acting on a typical slice including nail tension

Where,
\( \alpha \) = Inclination of slice base
\( \beta \) = Inclination of slice top
\( \delta \) = Angle of line of action of surcharge with vertical
\( \lambda \) = Nail inclination with horizontal

\( b \) = Width of slice

\( h \) = Average height of slice

\( FS_G \) = Factor of safety

\( S_a = C + N' \tan \phi \) = Available strength

\( S_F = \frac{S_a}{FS_G} \) = Mobilized strength

\( W \) = Weight of the slice

\( N \) = Effective normal force

\( Q \) = External surcharge

\( T_n \) = Nail tensile forces for the reinforcement emerging out from the base of \( i^{th} \) slice

Considering overall moment equilibrium of the forces acting on each slice is given by

\[
\sum M_0 = \sum_{n=1}^{n} [W + Q \cos \delta]R \sin \alpha - \sum_{n=1}^{n} [Q \sin \delta](R \cos \alpha - h) \\
- \sum_{n=1}^{n} [S_F]R - \sum_{n=1}^{n} [(T_n \sin \lambda)(R \sin \alpha) - (T_n \cos \lambda)(R \cos \alpha)]
\]

\[
= 0
\]

After rearranging,

\[
\sum \frac{M_0}{R} = \sum_{n=1}^{n} [W + Q \cos \delta] \sin \alpha - \sum_{n=1}^{n} [Q \sin \delta](\cos \alpha - \frac{h}{R}) \\
- \sum_{n=1}^{n} [S_F] - \sum_{n=1}^{n} [T_n \cos(\alpha + \lambda)]
\]

\[
= 0
\]

Replacing Eq.3.13 in Eq.3.32 and rearranging

\[
FS_G = \frac{\sum_{i=1}^{n} C + N \tan \phi}{\sum_{i=1}^{n} A_i - \sum_{i=1}^{n} A_2 - \sum_{i=1}^{n} A_4}
\]

Where,

\[
A_i = [W + Q \cos \delta] \sin \alpha
\]

\[
A_2 = (Q \sin \delta)(R \cos \alpha - \frac{h}{R})
\]

\[
A_4 = T_n \cos(\alpha + \lambda)
\]
From vertical force equilibrium of each slice
\[ \sum F_y = N \cos \alpha + S_p \sin \alpha - W - Q \cos \delta - T_u \sin \lambda = 0 \]  
(3.35)

The above equation may be arranged for \( N \) as
\[ N = \frac{-S_p \sin \alpha + W + Q \cos \delta + T_u \sin \lambda}{\cos \alpha} \]  
(3.36)

Again, substituting Eq. 3.13 in Eq. 3.36
\[ N = \frac{1}{m_\alpha} \left[ W - \frac{C \sin \alpha}{FS_G} + Q \cos \delta + T_u \sin \lambda \right] \]  
(3.37)

And, \( m_\alpha = \cos \alpha \left[ 1 + \tan \alpha \tan \phi \right] \)

Eq. 3.31 through 3.37 are used for calculating FOS of nailed slopes according to the simplified Bishop method.

### 3.3.5 Simplified Janbu Method

Similar assumptions are also applied in this method for calculating stability of nailed slopes. As mentioned earlier, the method is applicable to all types of slope analysis irrespective of assumed shape of trial slip surface. Forces acting on a typical slice are the same as in simplified Bishop method, as given in Fig. 3.11.

As derived in the previous case, the normal force obtained from the vertical equilibrium of the slice is given by
\[ N = \frac{1}{m_\alpha} \left[ W - \frac{C \sin \alpha}{FS_G} + Q \cos \delta + T_u \sin \lambda \right] \]  
(3.38)

and \( m_\alpha = \cos \alpha \left[ 1 + \tan \alpha \tan \phi \right] \)

In case of an individual slice \( i \), the horizontal force equilibrium equation is given by
\[ [F_h]_i = N \sin \alpha - Q \sin \delta - S_p \cos \alpha - T_u \cos \lambda = 0 \]
(3.39)

Substituting \( S_p \) from Eq. 3.13 and overall horizontal force equilibrium for the sliding mass
\[ \sum_{i=1}^{n} [F_h]_i = \sum_{i=1}^{n} [N \sin \alpha - Q \sin \delta - T_u \cos \lambda] - \sum_{i=1}^{n} \left[ \frac{C + N \tan \phi}{FS_G} \cos \lambda \right] \]
(3.40)

By rearranging the above equation, the following equation may be obtained
\[ \sum_{i=1}^{n} [N \sin \alpha - Q \sin \delta - T_u \cos \lambda] = \sum_{i=1}^{n} \left[ \frac{1}{FS_G} \left( C + N \tan \phi \right) \cos \lambda \right] \]
(3.41)

Then if each slice has the same FOS, \( FS_G \)
\[
FS_G = \frac{\sum_{i=1}^{n} (C + N \tan \phi) \cos \alpha}{\sum_{i=1}^{n} A_i + \sum_{i=1}^{n} N \sin \alpha}
\] (3.42)

Where, \(N\) is given by Eq.3.38 and the term \(A_i\) is given by

\[
A_i = Q \sin \delta - T_n \cos \lambda
\] (3.43)

Next, the modified FOS is calculated from Eq. 3.25.

**Note:** Planner and log spiral failure surface is generally observed in slopes with cohesionless soil. Whereas, circular slip surfaces are encountered for cohesive or variable soil stratum. Simplified Bishop, Friction circle and Circular arc are the common methods used in case of circular slip surface analysis for slopes with cohesive soil. Simplified Janbu method is suitable for analysis of slopes with variable soil stratum. However, Simplified Bishop and Janbu methods are difficult to employ for manual calculation due to rigorous iterative steps. They are normally used through computerized coding process. For manual calculation Friction circle and Circular arc methods are used.

### 3.4 Slope stability under dynamic condition

Table 3.1 presents the modified equations for calculation of factor of safety against slope failure under dynamic condition. Pseudo-static method has been employed for stability analysis of slope assuming equivalent vertical \((k_vW)\) and horizontal \((k_hW)\) forces due to earthquake, acting at the centre of gravity of the sliding soil block or the slice considered.

Where,

- \(k_v\) = Vertical seismic coefficient
- \(k_h\) = Horizontal seismic coefficient
- \(W\) = Mass of the slice

Magnitudes of these coefficients are chosen based on peak normal ground acceleration observed during previous earthquakes, the details of which are illustrated in chapter 4.
Table 3.1 Modified equations for calculation of factor of safety against slope failure under dynamic condition

<table>
<thead>
<tr>
<th>Slope without nail</th>
<th>Simplified Bishop Method</th>
<th>Simplified Janbu Method</th>
<th>Slope with nail</th>
<th>Simplified Bishop Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>$FS_G = \frac{\sum_{i=1}^{n} C + N \tan \phi}{\sum_{i=1}^{n} A_i - \sum_{i=1}^{n} A_2 + \sum_{i=1}^{n} A_3}$</td>
<td>$N = \frac{1}{m_\alpha} \left[ W(1-k_s) - \frac{C \sin \alpha}{FS_G} + Q \cos \delta \right]$</td>
<td>$m_\alpha = \cos \alpha \left[ 1 + \frac{\tan \alpha \tan \phi}{FS_G} \right]$</td>
<td>$A_1 = \left[ W(1-k_s) + Q \cos \delta \right] \sin \alpha$</td>
<td>$A_2 = (Q \sin \delta)(R \cos \alpha - \frac{h}{R})$</td>
</tr>
<tr>
<td></td>
<td>$m_\alpha = \cos \alpha \left[ 1 + \frac{\tan \alpha \tan \phi}{FS_G} \right]$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$A_4 = W k_h - Q \sin \delta$</td>
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</tbody>
</table>
\[ m_a = \cos \alpha \left[ 1 + \frac{\tan \alpha \tan \phi}{FS_G} \right] \]
\[ A_1 = \left[ W(1 - k_c) + Q \cos \delta \right] \sin \alpha \]
\[ A_2 = (Q \sin \delta)(R \cos \alpha - \frac{h}{R}) \]
\[ A_3 = k_h W \left( \cos \alpha - \frac{h}{R} \right) \]
\[ A_5 = T_n \cos (\alpha + \lambda) \]

**Simplified Janbu Method**

\[
FS_G = \frac{\sum_{i=1}^{n} [C + N \tan \phi] \cos \alpha}{\sum_{i=1}^{n} A_b + \sum_{i=1}^{n} N \sin \alpha}
\]

\[
N = \frac{1}{m_a} \left[ W(1 - k_c) - \frac{C \sin \alpha}{FS_G} + Q \cos \delta + T_n \sin \lambda \right]
\]

\[
m_a = \cos \alpha \left[ 1 + \frac{\tan \alpha \tan \phi}{FS_G} \right], \quad A_b = W k_h \sin \delta - T_n \cos \lambda
\]

### 3.5 Bearing capacity of nailed wall

Bearing capacity is an important factor for soil nail wall is excavated in fine-grained soft soils. As in such cases, the wall facing does not extend below the bottom of the excavation, the unbalanced load caused by the excavation may cause the bottom the excavation to heave and trigger a bearing capacity failure of the foundation (Fig. 3.12a).

For purely cohesive soil or under undrained condition of saturated soil \((\phi = 0, c = S_u)\), bearing capacity is given by \(S_u N_c\). The factor of safety to withstand the surcharge of \(H_{eq} \gamma\) for an excavation of depth \(H\) with equivalent overburden \(\Delta H\) is given by

\[
FS_H = \frac{S_u N_c}{H_{eq} \gamma}
\]

For \(c - \phi\) soils the bearing capacity will be \(c N_c + 0.5 \gamma B_N \gamma\) (as no surcharge) and the factor of safety will be

\[
FS_H = \frac{c N_c + 0.5 \gamma B_N \gamma}{H_{eq} \gamma}
\]
Where,
\( \phi = \) Angle of internal friction
\( c = \) Cohesion of soil
\( S_u = \) Undrained shear strength of soil
\( N_c, N_\gamma = \) Bearing capacity factor
\( \gamma = \) Unity weight of soil behind wall
\( H = \) Height of the wall (excavation depth)
\( H_{eq} = \) Equivalent wall height = \( H + \Delta H \)
\( \Delta H = \) Equivalent overburden
\( B' = \) Width of influence = \( B_e / \sqrt{2} \)
\( B_e = \) Width of excavation
\( L_e = \) Length of excavation

Eq. 3.44 and 3.45 are applicable when the width of the excavation \((B_e)\) is very large or the contribution of the shearing resistance \(S_uH\) outside the failure block of width \(B'\) is neglected. These equations are conservative because they neglect the shear contribution of the nails that are intersected by the failure surface shown in Fig. 3.12a.

**Note:** In case of railway excavations \(B_e\) is considerably large and Eq. 3.44 will be the governing criterion against bearing capacity failure under such condition.

For smaller width of the excavation, contribution of the shearing resistance outside the failure block is considered. In such case, the factor of safety against heave \((FS_H)\) (Terzaghi et al., 1996), is given by Eq. 3.46 \((\phi = 0, c = S_u\text{ soil})\) and Eq. 3.47 \((c - \phi \text{ soil})\).

\[
FS_H = \frac{S_uN_e}{H_{eq}(\gamma - S_u / B')}
\]

(3.46)

\[
FS_H = \frac{cN_e + 0.5\gamma B_e N_\gamma}{H_{eq}(\gamma - S_u / B')}
\]

(3.47)

When a strong deposit underlying the soft layer and occurring at a depth \(D_B < B_e / \sqrt{2}\) below the excavation bottom is encountered (Fig. 3.12b), \(B'\) in Eq. 3.46 and Eq. 3.46 is replaced by \(D_B.\)
(a) Deep deposit of soft fine-grained soil

(b) Shallow deposit of soft fine-grained soil underlain by stiff layer

Fig. 3.12 Bearing capacity analysis against heave
Bearing capacity factors \((N_c, N_r)\) are calculated based on the \(c - \phi\) (Terzaghi et al., 1996). These factors are adopted based on the existing geometric conditions, and for \((\phi = 0, c = S_u)\) soils \(N_c\) values are given in Fig 3.13. For very wide excavations (typical case for a soil nail wall), \(H / B_e\) can be considered conservatively equal to 0. For very long walls, it is conservative to adopt \(B_e / L_e = 0\) and \(N_c = 5.14\). Factors of safety against heave for soil nail walls are selected in such a way that they are consistent with those typically used for heave analysis at the bottom of excavations. In general, \(FS_h\) is adopted as 2.5 and 3 for temporary and permanent walls, respectively.

![Fig. 3.13 Bearing capacity factor \((N_c)\) for analysis against heave in \(\phi = 0, c = S_u\) soils](Source: Terzaghi, et al. 1996)
### 3.6 Bond strength

The pull-out capacity a soil nail installed in a grouted nail hole is affected by the size of the nail (i.e., perimeter and length) and the ultimate bond strength, $q_u$. The bond strength is the mobilized shear resistance along the soil-grout interface. The bond strength is rarely measured in the laboratory and there is no standard laboratory testing procedure that can be used to evaluate bond strength. Therefore, designs are typically based on conservative estimates of the bond strength obtained from field correlation studies and local experience in similar conditions. As a result of this dependency on local conditions, contract specifications include a strict requirement that some percentage of the soil nails be load tested in the field to verify bond strength design. The testing procedure is described in Chapter 2.

It has been noticed from field tests that for drilled and grouted nails, the bond strength is affected by:

- Ground conditions around the nail (soil type and conditions);
- Size of the grouted zone.
- Soil nail installation including:
  - drilling method;
  - grouting procedure;
  - grout nature;
  - grout injection (e.g., gravity or under pressure)

Now, considering a single nail segment subjected to a tensile force, $t_0$, at one end, and applying equilibrium of forces along the differential length ($dx$) of the nail shown in Fig. 3.14, the tensile force ($dt$) can be related to the interface shear stress as:

$$dt = p.q.dx = \pi d.q.dx$$  \hspace{1cm} (3.48)

Where,

$q$ = Mobilized bond stress

$p$ = Perimeter of the nail = $\pi d$

$d$ = Diameter of the nail
The above equation represents the transfer mechanism between the stresses on the nail-soil interface to tensile forces in the nail bar. If the mobilized bond shear stress is assumed to be uniform, the tensile force \( t \) at a distance “\( x \)” along the bar is

\[
t(x) = \int_0^x \pi d q \, dx = \pi d q \, x
\]

(3.49)
Actual distributions of mobilized bond shear stress are not uniform, as illustrated in Fig. 3.14, and depend on various factors including nail length, magnitude of applied tensile force, grout characteristics, and soil conditions. As a simplification, the mobilized bond strength is often assumed to be constant along the nail. Hence, the nail force at the end of the pullout length ($l_e$),

$$t(l_e) = t_0 = \pi dq l_e$$  \hspace{1cm} (3.50)

The pull-out capacity is mobilized when the ultimate bond strength is achieved. The pull-out capacity ($T_{cm}$) can be estimated by

$$T_{cm} = t_{\max} = \pi dq_{u} l_e = q_u p l_e$$ \hspace{1cm} (3.51)

$q_u$ = Limit bond stress or bond strength of the soil nail interface. It is obtained from pull-out test.

$l_e$ = Pull-out length for the case of pull-out test or length of the nail behind the failure surface in case of nailed slope

Pull-out capacity is generally expressed as per unit length of horizontal spacing

$$T_{cm} = t_{\max} = \pi dq_{u} l_e / s_h = q_u p l_e / s_h$$ \hspace{1cm} (3.52a)

$s_h$ = Horizontal spacing of nails

In absence of in-situ test, pull-out capacity is calculated from the following equation (Su et al. 2008),

$$T_{cm} = (c + \sigma_v \tan \psi) p l_e / s_h$$ \hspace{1cm} (3.52b)

Where,

$\psi$ = Mobilized soil-nail interface friction angle = $\psi = \frac{2}{3} \phi$

$\phi$ = Soil effective angle of internal friction

$c$ = Unit cohesion of the soil

For $j$th nail, $\sigma_v = \gamma h_j + Q$

$h_j$ = Depth of the midpoint of $j$th nail from ground surface

$\gamma$ = Unity weight of slope soil

$Q$ = Surcharge acting on the slope

The overburden pressure ($\sigma_v$) may be calculated at the midpoint of the whole nail length instead of the midpoint of $l_e$. This approximation will not have significant effect on the pullout capacity of nail for small nail inclinations.
During design of nailed structure, the mobilized pull-out capacity value is used after reducing by a factor of safety value to take care of variability in field measurement during in-situ testing and soil property. Hence, design pull-out capacity \( T_c = \frac{T_{cm}}{FOS_r} \) (3.53)

Generally, \( FOS_r = 1.5 \) is applied in most of the cases.

### 3.7 Mobilized nail tension

![Mobilized nail tension](image)

Fig. 3.15 Mobilized nail tension force

Mobilized nail tension (Fig. 3.15) can be calculated using the following equation

\[ T_n = \pi dq_u l_e / FOS_r \] (3.54)

In absence of bond strength value

\[ T_n = (c + \sigma_v \tan \phi) \pi dl_e / FOS_r \] (3.55)

Where,

- \( q_u \) = Bond strength of the soil nail interface. It is obtained from pull-out test.
- \( l_e \) = Length of the nail behind the failure surface in case of nailed slope
- \( d \) = Diameter of the nail
- \( c \) = Unit cohesion of the soil
- \( \phi \) = Soil effective angle of internal friction
\( \psi = \text{Mobilized soil-nail interface friction angle} = \psi = \frac{2}{3} \phi \)

For \( j \) th nail, \( \sigma_v = \gamma h_j + Q \)

\( h_j \) = Depth of the midpoint of \( j \) th nail from ground surface

\( \gamma \) = Unity weight of slope soil

\( Q \) = Surcharge acting on the slope

Factor of safety against nail tension = \( FOS_T = 1.5 \)

Eq. 3.54 and Eq. 3.55 are applicable when \( T_{n,\text{max}} < R_T \) and \( T_{n,\text{max}} < R_f \)

Where,

\[ T_{n,\text{max}} = \pi dq_v l / FOS_T \quad \text{if Eq. 3.54 is used} \quad (3.56) \]

\[ T_{n,\text{max}} = (c + \sigma_v \tan \psi) \pi dl / FOS_T \quad \text{if Eq. 3.55 is used} \quad (3.57) \]

\( l \) = Total nail length

\( R_T \) = Nail tensile strength = \( \frac{\pi d^2 f_y}{4 \ FOS_{RT}} \) \quad (3.58)

(Factor of safety against nail tensile strength \( FOS_{RT} = 1.8 \))

\( f_y \) = Yield strength of steel

\( R_f \) = Facing capacity

(Details of calculation procedure of facing capacity is given in chapter 4)
3.8 Example problems on stability analysis of unreinforced slope

Two example stability problems are solved here for unreinforced slopes. The first one is a slope on homogenous soil and the second one is on layered soil.

3.8.1 Problem-1

Stability analysis of the slope illustrated in Fig. 3.16 with varying face angle ($\beta$) by

(a) Planar failure surface
(b) Circular arc method
(c) Simplified Bishop method

Where, $\beta = 70^\circ, 75^\circ, 80^\circ, 90^\circ$

For the analysis, unit slope width is considered.

Fig. 3.16 Stability analysis of unreinforced homogeneous slope
(a) Planar failure surface

From trigonometric relation (Fig. 3.17) it can be shown that

\[
L = \frac{H}{\sin \theta} \quad \text{and} \quad W = \frac{1}{2} \gamma H^2 \frac{\sin(\beta - \theta)}{\sin \beta \sin \theta}
\]

For stable equilibrium along failure surface

\[
W \sin \theta = c_m L
\]

Where, \( c_m = \frac{c}{F} \)

\( F = \) Factor of safety

Hence, \( c_m = \frac{W \sin \theta}{L} \)

So \( F = \frac{cL}{W \sin \theta} \)
Table 3.2 Factor of safety for different slope face angles considering various failure plane for unreinforced homogeneous slope

<table>
<thead>
<tr>
<th>β (in degree)</th>
<th>θ (in degree)</th>
<th>Weight (W) (kN)</th>
<th>Length (L) (m)</th>
<th>Factor of safety (F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>75</td>
<td>118.20</td>
<td>7.25</td>
<td>2.54</td>
</tr>
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<td>60</td>
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<td>2.11</td>
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<td>280.50</td>
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<td></td>
<td>30</td>
<td>603.30</td>
<td>14.00</td>
<td>1.86</td>
</tr>
</tbody>
</table>
For the given problem,

\[ H = 7 \text{m}, \gamma = 18 \text{kN/m}^3, c = 40 \text{kPa} \text{ and } \phi = 0^\circ \]

\[ L = \frac{7}{\sin \theta} \text{ and } W = \frac{1}{2} \cdot 18 \cdot 7^2 \cdot \frac{\sin(\beta - \theta)}{\sin \beta \sin \theta} = 441 \cdot \frac{\sin(\beta - \theta)}{\sin \beta \sin \theta} \]

For \( \beta = 80^\circ, \theta = 60^\circ \)

\[ L = \frac{7}{\sin 60} = 8.08 \text{m} \text{ and } W = \frac{1}{2} \cdot 18 \cdot 7^2 \cdot \frac{\sin(80 - 60)}{\sin 80 \sin 60} = 176.85 \text{kN/m} \]

\[ F = \frac{cL}{W \sin \theta} = 2.11 \]

Similarly, factor of safety is calculated for different slope face angles considering various failure plane and the results are presented in Table 3.2. It can be observed that factor of safety values are increasing with reduced slope face angle implying higher stability of the slope structure. For a particular slope face configuration, factor of safety value initially decreases with decreased failure wedge angle and then again increases after giving an optimum value. The minimum factor of safety value is generally achieved at critical failure surface.

(b) Circular arc method

![Circular arc method diagram](image)

Fig. 3.18 Stability analysis of unreinforced homogeneous slope by circular arc method
Fig. 3.18 represents the graphical representation (not to scale) of the circular arc analysis method for the slope with face angle $\beta = 90^\circ$. Graphical calculations are employed to get actual values.

For the analysis, $R=18m$ and $\theta=100^\circ$

In graphical method, following areas are defined for easiness of the calculation

$A_1 = \text{Oaf}$, $A_2 = \text{ace}$, $A_3 = \text{Obc}$, $A_4 = \text{Area of the sliding soil}$

Weight of soil above failure surface, $W = \gamma A_4$

$A_4 = \text{Area of circular arc} - (A_1 + A_2 + A_3)$

Now magnitude of these areas are estimated from graphical calculations

Area of $A_1 = 0.5*\text{Oa*Of} = 62.01m^2$

Area of rectangle $A_2 = \text{ac*ae} = 63.00 m^2$

Area of $A_3 = 0.5*\text{Oc*bc} = 76.85 m^2$

Area of circular arc (A) = $\frac{R^2 \theta}{2} = 285.06 m^2$

From above values, $A_4 = 83.20 m^2$

As considering unit width of the slope,

So weight of slope above failure surface $(W) = \gamma A_4 = 1497.67kN$

Now for calculating centre of mass

$A_1 x_1 + A_2 x_2 + A_3 x_3 + A_4 x_4 = Ax$

$x_4 = \frac{Ax - (A_1 x_1 + A_2 x_2 + A_3 x_3)}{A_3}$

$x_4 = -2.51m$

$(x_1, x_2, x_3, x)$ are calculated from mathematical relations and graphical values; their values are measured from the point d (Fig. 3.18) along with proper signs)

Hence, $\bar{x} = x_{centre} - x_4 = 6.49m$

For given problem, $H=7m$, $\gamma =18kN/m^3$, $c = 40$ kPa

$F = \frac{cLR}{Wx} = \frac{cR^2\theta}{Wx} = 2.34$
Similarly, factor of safety is calculated for different slope face angles and presented in Table 3.3.

Table 3.3 Factor of safety calculated for different slope face angles for unreinforced homogeneous slope by circular arc method

<table>
<thead>
<tr>
<th>(\beta) (degree)</th>
<th>R (m)</th>
<th>(\theta) (degree)</th>
<th>(A_d)</th>
<th>Weight ( (W) ) (kN)</th>
<th>(x_4) (m)</th>
<th>(x_{\text{centre}}) (m)</th>
<th>(y_{\text{centre}}) (m)</th>
<th>(-x) (m)</th>
<th>(F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>18</td>
<td>100.82</td>
<td>83.2</td>
<td>1497.67</td>
<td>-2.51</td>
<td>9</td>
<td>14.6</td>
<td>6.49</td>
<td>2.35</td>
</tr>
<tr>
<td>80</td>
<td>18</td>
<td>100.82</td>
<td>78.88</td>
<td>1419.91</td>
<td>-2.67</td>
<td>9</td>
<td>14.6</td>
<td>6.33</td>
<td>2.54</td>
</tr>
<tr>
<td>75</td>
<td>18</td>
<td>100.82</td>
<td>76.64</td>
<td>1379.51</td>
<td>-2.78</td>
<td>9</td>
<td>14.6</td>
<td>6.22</td>
<td>2.66</td>
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<tr>
<td>70</td>
<td>18</td>
<td>100.82</td>
<td>74.29</td>
<td>1337.16</td>
<td>-2.92</td>
<td>9</td>
<td>14.6</td>
<td>6.09</td>
<td>2.80</td>
</tr>
</tbody>
</table>

(c) Simplified Bishop method

![Stability analysis of unreinforced homogeneous slope by Bishop’s method](image)

Fig. 3.19 Stability analysis of unreinforced homogeneous slope by Bishop’s method
Fig. 3.19 represents the graphical representation of the stability analysis of unreinforced homogeneous slope by Bishop method. Graphical calculations are employed to estimate the values. A typical calculation is presented in Table 3.4 for \( \beta = 90^0 \). Similarly, factor of safety is calculated for different slope face angles and presented in Table 3.5.

Slope face angle with respect to the horizontal \( (\beta) = 90^0 \)
Radius of slip surface \( (R) = 17.98 \text{m} \)
Co-ordinate of the centre of slip surface \( (x, y) = (-9, 15.57) \)

Factor of safety for the given case \( F = \frac{\sum_{i=1}^{n} C}{\sum_{i=1}^{n} W \sin \alpha} = \frac{388.29}{317.83} = 1.22 \)

Table 3.4 Factor of safety calculation for unreinforced homogeneous slope with slope face angle \( \beta = 90^0 \) by Bishop’s method

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>Base slope of slice ( (\alpha) ) (degree)</th>
<th>Weight ( (W) ) (kN)</th>
<th>( W\sin \alpha ) (kN)</th>
<th>Length of the base of slice ( (L) ) (m)</th>
<th>Cohesion along base of slice ( (C = cL) ) (kN)</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>38.66</td>
<td>118.80</td>
<td>74.21</td>
<td>1.28</td>
<td>51.22</td>
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</tr>
<tr>
<td>2</td>
<td>34.99</td>
<td>105.30</td>
<td>60.39</td>
<td>1.22</td>
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</tr>
<tr>
<td>3</td>
<td>41.99</td>
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<td>60.81</td>
<td>1.35</td>
<td>53.81</td>
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</tr>
<tr>
<td>4</td>
<td>45.00</td>
<td>73.80</td>
<td>52.18</td>
<td>1.41</td>
<td>56.57</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>50.19</td>
<td>54.00</td>
<td>41.48</td>
<td>1.56</td>
<td>62.48</td>
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</tr>
<tr>
<td>6</td>
<td>56.31</td>
<td>34.56</td>
<td>28.76</td>
<td>2.88</td>
<td>115.38</td>
<td></td>
</tr>
<tr>
<td>( \Sigma )</td>
<td></td>
<td>317.83</td>
<td></td>
<td></td>
<td>388.29</td>
<td>1.22</td>
</tr>
</tbody>
</table>
Table 3.5 Factor of safety calculation for unreinforced homogeneous slope with different slope face angles by Bishop’s method

<table>
<thead>
<tr>
<th>β (degree)</th>
<th>R (m)</th>
<th>x</th>
<th>y</th>
<th>No. of slices</th>
<th>( \sum W \sin \alpha ) (kN)</th>
<th>( \sum C ) (kN)</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>17.98</td>
<td>-9</td>
<td>15.57</td>
<td>6</td>
<td>317.83</td>
<td>388.29</td>
<td>1.223</td>
</tr>
<tr>
<td>80</td>
<td>17.98</td>
<td>-9</td>
<td>15.57</td>
<td>6</td>
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<td>396.09</td>
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<td>-3.74</td>
<td>12.22</td>
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</tr>
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<td>11.26</td>
<td>-1.96</td>
<td>11.09</td>
<td>8</td>
<td>299.51</td>
<td>463.53</td>
<td>1.548</td>
</tr>
</tbody>
</table>

3.8.2 Problem-2

Stability analysis of the slope illustrated in Fig. 3.20 with varying face angle (β) by

(a) Planar failure surface (β= 70°, 75°, 80°, 90°)

(b) Simplified Bishop method (β= 70°, 75°, 80°)

For the analysis, unit slope width is considered.

Fig. 3.20 Stability analysis of unreinforced layered slope

\( \gamma_2 = 18 \text{kN/m}^3 \)
\( c_2 = 20 \text{kPa} \)  \( \phi_2 = 20° \)

\( \gamma_1 = 20 \text{kN/m}^3 \)
\( c_1 = 40 \text{kPa} \)  \( \phi_1 = 15° \)
(a) Planar failure surface

Fig. 3.21 Stability analysis of unreinforced layered slope considering planar failure surface

The length of the failure surface within each layer (Fig. 3.21)

\[
L_1 = \frac{1}{\sin \gamma} \left[ \frac{H_2 \sin(\beta - \alpha_2)}{\sin(\gamma - \alpha_1)} \right]
\]

\[
L_2 = \frac{1}{\sin \gamma} \left[ \frac{H_2 \sin(\beta - \alpha_2)}{\sin(\gamma - \alpha_2)} - \frac{H_1 \sin(\beta - \alpha_1)}{\sin(\gamma - \alpha_1)} \right]
\]

The weight of the sliding block of each layer

\[
W_1 = \frac{1}{2} \gamma_1 H_2 \frac{\sin(\beta - \alpha_2) \sin(\beta - \gamma)}{\sin(\gamma - \alpha_1)}
\]

\[
W_2 = \frac{1}{2} \gamma_2 \left[ \frac{H_2 \sin(\beta - \alpha_2)}{\sin(\gamma - \alpha_2)} - \frac{H_1 \sin(\beta - \alpha_1)}{\sin(\gamma - \alpha_1)} \right]
\]

For stable equilibrium along planner surface

\[
(W_1 + W_2) \sin \theta = \frac{c_1 L_1 + c_2 L_2 + W_1 \cos \theta \tan \alpha_1 + W_2 \cos \theta \tan \alpha_2}{F}
\]

\[
P = \frac{c_1 L_1 + c_2 L_2 + W_1 \cos \theta \tan \alpha_1 + W_2 \cos \theta \tan \alpha_2}{(W_1 + W_2) \sin \theta}
\]
Where, $F = \text{Factor of safety}$

For the given problem,

For layer 1: $\gamma_1=20\text{kN/m}^3$, $H_1=3\text{m}$, $c_1=40\text{kPa}$, $\phi_1=15^\circ$, $\alpha_1=\tan^{-1}(1/3)$

For layer 2: $\gamma_2=18\text{kN/m}^3$, $H_2=8\text{m}$, $c_2=20\text{kPa}$, $\phi_2=20^\circ$, $\alpha_2=\tan^{-1}(1/2)$

For $\beta=80^\circ$, $\theta=45^\circ$

\[
L_1 = \frac{1}{\sin 80^\circ} \left[ \frac{3\sin(80^\circ - \tan^{-1}(1/3))}{\sin(45^\circ - \tan^{-1}(1/3))} \right] = 5.99m
\]

\[
L_2 = \frac{1}{\sin 80^\circ} \left[ \frac{8\sin(80^\circ - \tan^{-1}(1/2)) - 3\sin(80^\circ - \tan^{-1}(1/3))}{\sin(45^\circ - \tan^{-1}(1/2)) - \sin(45^\circ - \tan^{-1}(1/3))} \right] = 22.75m
\]

Considering unit slope width,

\[
W_1 = \frac{1}{2} \cdot 20 \cdot 3^2 \cdot \frac{3\sin(80^\circ - \tan^{-1}(1/3))}{\sin(45^\circ - \tan^{-1}(1/3))} \cdot \frac{\sin(80^\circ - 45^\circ)}{\sin^2 80^\circ} = 104.66kN
\]

\[
W_2 = \frac{1}{2} \cdot 18 \cdot \frac{\sin(80^\circ - 45^\circ)}{\sin^2 80^\circ} \left[ 8^2 \cdot \frac{\sin(80^\circ - \tan^{-1}(1/2))}{\sin(45^\circ - \tan^{-1}(1/2))} - 3^2 \cdot \frac{\sin(80^\circ - \tan^{-1}(1/3))}{\sin(45^\circ - \tan^{-1}(1/3))} \right] = 771.02kN
\]

\[
F = \frac{L_1c_1 + L_2c_2 + W_1 \cos \theta \tan \phi_1 + W_2 \cos \theta \tan \phi_2}{(W_1 + W_2) \sin \theta} = 1.541
\]

Similarly, factor of safety is calculated for different slope face angles considering various failure plane and the results are presented in Table 3.6.
### Table 3.6 Factor of safety for different slope face angles considering various failure plane for unreinforced layered slope

<table>
<thead>
<tr>
<th>$\beta$ (in degree)</th>
<th>$\theta$ (in degree)</th>
<th>$W_1$ (kN)</th>
<th>$L_1$ (m)</th>
<th>$W_2$ (kN)</th>
<th>$L_2$ (m)</th>
<th>Factor of safety ($F$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>70</td>
<td>37.30</td>
<td>3.63</td>
<td>222.70</td>
<td>7.44</td>
<td>1.33</td>
</tr>
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<td>65</td>
<td>49.70</td>
<td>3.92</td>
<td>305.50</td>
<td>8.78</td>
<td>1.20</td>
</tr>
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<td>60</td>
<td>64.30</td>
<td>4.29</td>
<td>409.60</td>
<td>10.71</td>
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<tr>
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<td>50</td>
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<td>5.44</td>
<td>738.30</td>
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<td>1.91</td>
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<td>60</td>
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<td>1609.10</td>
<td>56.60</td>
<td>1.97</td>
</tr>
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</table>
Fig. 3.22 Stability analysis of unreinforced layered slope by Bishop’s method

The graphical representation of the stability analysis of unreinforced layered slope by Bishop method is presented in Fig. 3.22. Graphical calculations are employed to estimate the values. A typical calculation is presented in Table 3.7 for $\beta=90^0$. Similarly, factor of safety is calculated for different slope face angles and presented in Table 3.8.

Slope face angle with respect to the horizontal ($\beta$) = $75^0$
Radius of slip surface (R) = 22.76m
Co-ordinate of the centre of slip surface (x, y) = (-11.05, 19.9)

Factor of safety for the given case:

$$ F = \frac{\sum_{i=1}^{n} [C + N \tan \phi]}{\sum_{i=1}^{n} W \sin \alpha} = \frac{829.78}{547.41} = 1.516 $$
Table 3.7 Factor of safety calculation for unreinforced layered slope with slope face angle $\beta=75^0$ by Bishop’s method

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>Base slope of slice ($\alpha$) (degree)</th>
<th>Weight ($W$) (kN)</th>
<th>$W\sin\alpha$ (kN)</th>
<th>$L$ (m)</th>
<th>$N$ (kN)</th>
<th>$C + N \tan\phi = C + N \tan\phi$ (kN)</th>
<th>$F$</th>
</tr>
</thead>
<tbody>
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<td>26.57</td>
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<td>9.30</td>
<td>0.89</td>
<td>23.26</td>
<td>42.01</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>45.00</td>
<td>11.08</td>
<td>7.83</td>
<td>0.28</td>
<td>15.67</td>
<td>15.51</td>
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<td>30.96</td>
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<td>57.89</td>
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<td>75.09</td>
<td>1.41</td>
<td>150.19</td>
<td>96.81</td>
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</tr>
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<td>1.41</td>
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<td>66.37</td>
<td>1.56</td>
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<td>72.00</td>
<td>58.59</td>
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<tr>
<td>11</td>
<td>57.99</td>
<td>54.00</td>
<td>45.79</td>
<td>1.89</td>
<td>101.89</td>
<td>74.82</td>
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</tr>
<tr>
<td>12</td>
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<td>25.92</td>
<td>24.07</td>
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</tr>
<tr>
<td>$\sum$</td>
<td></td>
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<td></td>
<td></td>
<td>829.78</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.8 Factor of safety calculation for unreinforced layered slope with different slope face angles by Bishop’s method

<table>
<thead>
<tr>
<th>$\beta$ (degree)</th>
<th>R (m)</th>
<th>x</th>
<th>y</th>
<th>No. of slices</th>
<th>$\sum W \sin \alpha$ (kN)</th>
<th>$\sum C + N \tan \phi$ (kN)</th>
<th>$F$</th>
</tr>
</thead>
<tbody>
<tr>
<td>80</td>
<td>25.65</td>
<td>-13.22</td>
<td>22</td>
<td>10</td>
<td>751.93</td>
<td>953.96</td>
<td>1.269</td>
</tr>
<tr>
<td>75</td>
<td>22.76</td>
<td>-11.05</td>
<td>19.9</td>
<td>12</td>
<td>547.41</td>
<td>829.78</td>
<td>1.516</td>
</tr>
<tr>
<td>70</td>
<td>23.12</td>
<td>-10.13</td>
<td>20.79</td>
<td>11</td>
<td>474.39</td>
<td>775.52</td>
<td>1.635</td>
</tr>
</tbody>
</table>
3.9  Example problems on stability analysis of nailed slope

Two example stability problems are solved here for nailed slopes. The same slope geometry has been chosen as in the previous section.

3.9.1 Problem-1

Stability analysis of the slope illustrated in Fig. 3.23 with varying face angle (β) by

(a) Planar failure surface
(b) Circular arc method
(c) Simplified Bishop method

Where, β= 70°, 75°, 80°, 90°

For the analysis, unit slope width and nail horizontal spacing is considered.

Fig. 3.23 Stability analysis of nailed homogeneous slope
(a) **Planar failure surface**

![Planar failure surface](image)

Fig. 3.24 Stability analysis of nailed homogeneous slope considering planar failure surface

Length of failure surface (Fig. 3.24), \( L = H / \sin \theta \)

\[
W = \frac{1}{2} \gamma H^2 \sin(\beta - \theta) \sin \beta \sin \theta
\]

\[
T_{EQ} = \sum_{j=1}^{nl} T_j
\]

\[
T_j = \frac{(c + \sigma_v \tan \psi) p l_t}{s_h}
\]

For stable equilibrium along failure surface, \( W \sin \theta - T_{EQ} \sin \theta = c_m L \)

Where, \( c_m = c / F \) and \( F = \text{Factor of safety} \)

Hence,

\[
c_m = \frac{W \sin \theta - T_{EQ} \sin \theta}{L}
\]

\[
F = \frac{c L}{W \sin \theta - T_{EQ} \sin \theta}
\]

Given,

- \( H = 7 \text{m}, \beta = 90^\circ, \gamma = 18 \text{kN/m}^3, c = 40 \text{kPa}, \phi = 0 \)
- Nail length = 8m

Similarly factor of safety is calculated for different slope face angles considering various failure plane and the results are presented in Table 3.9.
Table 3.9 Factor of safety for different slope face angles considering various failure plane for reinforced homogeneous slope

<table>
<thead>
<tr>
<th>$\beta$ (in degree)</th>
<th>$\theta$ (in degree)</th>
<th>$T_{EQ}$ (kN)</th>
<th>$W$ (kN)</th>
<th>$L$ (m)</th>
<th>Factor of safety (F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>45</td>
<td>22.86</td>
<td>441.00</td>
<td>9.90</td>
<td>1.34</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>20.11</td>
<td>525.56</td>
<td>10.89</td>
<td>1.34</td>
</tr>
<tr>
<td></td>
<td>41</td>
<td>20.69</td>
<td>507.31</td>
<td>10.67</td>
<td>1.34</td>
</tr>
<tr>
<td></td>
<td>42</td>
<td>21.25</td>
<td>489.78</td>
<td>10.46</td>
<td>1.33</td>
</tr>
<tr>
<td></td>
<td>43</td>
<td>21.80</td>
<td>472.91</td>
<td>10.26</td>
<td>1.33</td>
</tr>
<tr>
<td>80</td>
<td>35</td>
<td>16.92</td>
<td>552.05</td>
<td>12.20</td>
<td>1.59</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>20.11</td>
<td>447.80</td>
<td>10.89</td>
<td>1.58</td>
</tr>
<tr>
<td></td>
<td>37</td>
<td>18.26</td>
<td>507.47</td>
<td>11.63</td>
<td>1.58</td>
</tr>
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<td>38</td>
<td>18.89</td>
<td>486.69</td>
<td>11.37</td>
<td>1.58</td>
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<td>19.51</td>
<td>466.83</td>
<td>11.12</td>
<td>1.58</td>
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<td>30</td>
<td>13.12</td>
<td>645.67</td>
<td>14.00</td>
<td>1.77</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>20.11</td>
<td>407.40</td>
<td>10.89</td>
<td>1.75</td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>16.92</td>
<td>511.65</td>
<td>12.20</td>
<td>1.72</td>
</tr>
<tr>
<td></td>
<td>36</td>
<td>17.60</td>
<td>488.82</td>
<td>11.91</td>
<td>1.72</td>
</tr>
<tr>
<td></td>
<td>37</td>
<td>18.26</td>
<td>467.06</td>
<td>11.63</td>
<td>1.72</td>
</tr>
<tr>
<td>70</td>
<td>30</td>
<td>13.12</td>
<td>603.32</td>
<td>14.00</td>
<td>1.90</td>
</tr>
<tr>
<td></td>
<td>32</td>
<td>14.73</td>
<td>545.24</td>
<td>13.21</td>
<td>1.88</td>
</tr>
<tr>
<td></td>
<td>33</td>
<td>15.49</td>
<td>518.57</td>
<td>12.85</td>
<td>1.88</td>
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<td>34</td>
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<td>12.52</td>
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<td></td>
<td>35</td>
<td>16.92</td>
<td>469.30</td>
<td>12.20</td>
<td>1.88</td>
</tr>
</tbody>
</table>
(b) Circular arc method

Fig. 3.25 Stability analysis of reinforced homogeneous slope by circular arc method

Given,
H=7m, β = 80°, γ = 18kN/m³, c = $S_u = 40kPa$, φ=0

Nail length=8m

Weight of soil above failure surface (W) (Fig. 3.25)

$A = \text{Area of circular arc} (A_5) - (A_1+A_2+A_3+A_4)$

Now magnitude of these areas are estimated from graphical calculations

Area of $\Delta Oaf = A_1 = 0.5*Oa*Of = 13.33m^2$
Area of $\Delta Obc = A_2 = 0.5*Oc*bc = 47.85 m^2$
Area of rectangle acde = $A_3 = ac*ae = 45.36 m^2$
Area of $\Delta deg = A_4 = 0.5*de*eg = 4.32 m^2$
Area of circular arc = $A_5 = R^2\theta / 2 = 224.58 m^2$

From above values, $A=113.71 m^2$

So weight of slope above failure surface $W = \gamma A = 2046.918kN$ (for unit width)
To calculate centre of mass

\[ A_1 \cdot c_m_{A1} + A_2 \cdot c_m_{A2} + A_3 \cdot c_m_{A3} + A_4 \cdot c_m_{A4} + A_5 \cdot c_m_{A5} = \]

\[ c_m_{A} = \frac{A_1 \cdot c_m_{A1} + A_2 \cdot c_m_{A2} + A_3 \cdot c_m_{A3} + A_4 \cdot c_m_{A4} + A_5 \cdot c_m_{A5}}{A} \]

By graph & geometrical formulas

\[ c_m_{A} = -2.32 \]

From this we can calculate \[ A = \sqrt{(A_{centre} - c_m_{A})^2} = 8.17 \text{m} \]

We have to calculate \( T_{EQ} \) for that

\[ T_{EQ} = T_1 + T_2 + T_3 \]

Where

\[ T_j = \frac{(c + \sigma_t \tan \psi) \rho l_e}{s_h} \]

Here,

\[ \psi = 0^\circ \quad \text{and} \quad s_h = 1 \text{m} \]

\[ T_j = cp l_e \]

\[ T_{EQ} = cp(l_{e1} + l_{e2} + l_{e3}) \]

From graph taking the values of \( T_{EQ} = cp(1.8+3.2+5.2) = 12.05 \text{kN} \)

Now applying moment

\[ T_1*O_k_1 + T_2*O_k_2 + T_3*O_k_3 = T_{EQ}*O_k \]

From graph \( O_k = 4.2 \)

Measuring \( \beta \) from graph, \( \beta = 62.5^\circ \)

Now to find out factor of safety we will use formula,

\[ F = \frac{S_\theta R^2 + T_{EQ} \cos(\lambda) R \cos \beta}{W x + T_{EQ} \sin(\lambda) R \sin \beta} = 2.09 \]

Similarly, factor of safety is calculated for different slope face angles and presented in Table 3.10.
Table 3.10 Factor of safety calculated for different slope face angles for reinforced homogeneous slope by circular arc method

<table>
<thead>
<tr>
<th>β (degree)</th>
<th>R (m)</th>
<th>θ (degree)</th>
<th>A</th>
<th>W  (kN)</th>
<th>T_EQ (kN)</th>
<th>Factor of safety (F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>16.54</td>
<td>141.91</td>
<td>203.37</td>
<td>3660.69</td>
<td>0.53</td>
<td>2.04</td>
</tr>
<tr>
<td>80</td>
<td>13.95</td>
<td>132.31</td>
<td>113.72</td>
<td>2046.92</td>
<td>12.05</td>
<td>2.09</td>
</tr>
<tr>
<td>75</td>
<td>11.89</td>
<td>138.66</td>
<td>92.27</td>
<td>1660.94</td>
<td>13.69</td>
<td>2.11</td>
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<td>11.23</td>
<td>140.81</td>
<td>83.31</td>
<td>1499.57</td>
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<td>2.27</td>
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</table>

(c) Simplified Bishop method

Fig. 3.26 Stability analysis of reinforced homogeneous slope by Bishop’s method
The graphical representation of the stability analysis of reinforced homogeneous slope by Bishop method is presented in Fig. 3.26. Graphical calculations are employed to estimate the values. A typical calculation is presented in Table 3.11 for $\beta=90^\circ$. Similarly, factor of safety is calculated for different slope face angles and presented in Table 3.12.

Slope face angle with respect to the horizontal ($\beta$) = $90^\circ$

Radius of slip surface (R) = 7.49m

Co-ordinate of the centre of slip surface (x, y) = (-0.1, 7.5)

Factor of safety for the given case = $F = \frac{\sum C}{\sum [W \sin \alpha - T_n \cos (\alpha + \lambda)]}$ = $\frac{443.44}{293.4633}$ = 1.51

Table 3.11 Factor of safety calculation for unreinforced layered slope with slope face angle $\beta=90^\circ$ by Bishop’s method

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>$\alpha$ (degree)</th>
<th>W (kN)</th>
<th>Wsin$\alpha$ (kN)</th>
<th>$T_n$ (kN)</th>
<th>$T_n \cos (\alpha + \lambda)$ (kN)</th>
<th>L (m)</th>
<th>$C = cL$ (kN)</th>
<th>$W \sin \alpha$ $T_n \cos (\alpha + \lambda)$ (kN)</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11.30</td>
<td>124.20</td>
<td>24.40</td>
<td>0.00</td>
<td>0.00</td>
<td>1.02</td>
<td>40.79</td>
<td>24.36</td>
<td>1.51</td>
</tr>
<tr>
<td>2</td>
<td>11.30</td>
<td>120.60</td>
<td>23.70</td>
<td>0.00</td>
<td>0.00</td>
<td>1.02</td>
<td>40.79</td>
<td>23.65</td>
<td>1.51</td>
</tr>
<tr>
<td>3</td>
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<td>42.80</td>
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<td>0.00</td>
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<td>43.08</td>
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<td>0.00</td>
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<td>43.08</td>
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<tr>
<td>5</td>
<td>38.70</td>
<td>97.20</td>
<td>60.70</td>
<td>4.70</td>
<td>3.10</td>
<td>1.28</td>
<td>51.23</td>
<td>57.62</td>
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</tr>
<tr>
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<td>79.20</td>
<td>60.80</td>
<td>1.70</td>
<td>0.83</td>
<td>1.56</td>
<td>62.48</td>
<td>60.01</td>
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<td>7</td>
<td>69.80</td>
<td>47.88</td>
<td>44.90</td>
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<td>0.00</td>
<td>4.05</td>
<td>161.99</td>
<td>44.93</td>
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</tr>
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<td></td>
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<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>
Table 3.12 Factor of safety calculation for unreinforced layered slope with different slope face angles by Bishop’s method

<table>
<thead>
<tr>
<th>$\beta$ (degree)</th>
<th>R (m)</th>
<th>x</th>
<th>y</th>
<th>No. of slices</th>
<th>$\sum [W \sin \alpha - T_n \cos (\alpha + \lambda)]$ (kN)</th>
<th>$\sum C$ (kN)</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>7.49</td>
<td>-0.1</td>
<td>7.50</td>
<td>7.00</td>
<td>293.46</td>
<td>443.44</td>
<td>1.51</td>
</tr>
<tr>
<td>80</td>
<td>8.60</td>
<td>-0.3</td>
<td>8.60</td>
<td>8.00</td>
<td>306.24</td>
<td>467.66</td>
<td>1.53</td>
</tr>
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<td>75</td>
<td>9.50</td>
<td>0.20</td>
<td>9.50</td>
<td>8.00</td>
<td>300.64</td>
<td>477.35</td>
<td>1.59</td>
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<td>70</td>
<td>10.12</td>
<td>0.33</td>
<td>10.10</td>
<td>9.00</td>
<td>285.13</td>
<td>503.23</td>
<td>1.76</td>
</tr>
</tbody>
</table>

3.9.2 Problem-2

Stability analysis of the slope illustrated in Fig. 3.27 with varying face angle ($\beta$) by

(a) Planar failure surface
(b) Simplified Bishop method

Where, $\beta = 70^\circ, 75^\circ, 80^\circ, 90^\circ$

For the analysis, unit slope width and nail horizontal spacing is considered.

Fig. 3.27 Stability analysis of nailed layered slope
(a) Planar failure surface

Fig. 3.28 Stability analysis of nailed layered slope considering planar failure surface

Failure surface lengths (Fig. 3.28),

\[ L_1 = \frac{1}{\sin \theta} \left[ \frac{H_1 \sin(\beta - \alpha_1)}{\sin(\theta - \alpha_1)} \right] \]

\[ L_2 = \frac{1}{\sin \theta} \left[ \frac{H_2 \sin(\beta - \alpha_2)}{\sin(\theta - \alpha_2)} \right] \]

\[ W_1 = \frac{1}{2} \gamma_1 H_1 \sin(\theta - \alpha_1) \frac{\sin(\theta - \theta)}{\sin \theta} \]

\[ W_2 = \frac{1}{2} \gamma_2 H_2 \sin(\theta - \alpha_2) \left[ \frac{\sin(\beta - \alpha_2)}{\sin(\theta - \alpha_2)} - \frac{\sin(\beta - \alpha_1)}{\sin(\theta - \alpha_1)} \right] \]

For stable equilibrium along planar surface

\[ (W_1 + W_2) \sin \theta - T_{EQ} \sin(\theta - \lambda) = \frac{c_1 L_1 + c_2 L_2 + W_1 \cos \theta \tan \phi_1 + W_2 \cos \theta \tan \phi_2 + T_{EQ} \cos(\theta - \lambda)}{F} \]

Where, F= Factor of safety

Hence,

\[ F = \frac{c_1 L_1 + c_2 L_2 + W_1 \cos \theta \tan \phi_1 + W_2 \cos \theta \tan \phi_2 + T_{EQ} \cos(\theta - \lambda)}{(W_1 + W_2) \sin \theta - T_{EQ} \sin(\theta - \lambda)} \]
Given,
\( \gamma_1 = 20\text{kN/m}^3, \ H_1 = 3\text{m} \)
\( c_1 = 40\text{kPa}, \ \phi_1 = 15^\circ, \ \alpha_1 = \tan^{-1}(1/3) \)
\( \gamma_2 = 18\text{kN/m}^3, \ H_2 = 8\text{m} \)
\( c_2 = 20\text{kPa}, \ \phi_2 = 20^\circ, \ \alpha_2 = \tan^{-1}(1/2) \)

For,
\( \beta = 80^\circ, \ \theta = 45^\circ \)

\[
L_1 = \frac{1}{\sin 80^\circ} \left[ \frac{3 \sin(80^\circ - \tan^{-1}(1/3))}{\sin(45^\circ - \tan^{-1}(1/3))} \right] = 5.99\text{m}
\]

\[
L_2 = \frac{1}{\sin 80^\circ} \left[ \frac{8 \sin(80^\circ - \tan^{-1}(1/2)) - 3 \sin(80^\circ - \tan^{-1}(1/3))}{\sin(45^\circ - \tan^{-1}(1/2)) - 3 \sin(45^\circ - \tan^{-1}(1/3))} \right] = 22.75\text{m}
\]

Considering unit slope width,

\[
W_1 = \frac{1}{2} \times 20 \times 3^2 \times \frac{3 \sin(80^\circ - \tan^{-1}(1/3))}{\sin(45^\circ - \tan^{-1}(1/3))} \times \frac{\sin(80^\circ - 45^\circ)}{\sin^2 80^\circ} = 104.66\text{kN}
\]

\[
W_2 = \frac{1}{2} \times 18 \times \frac{\sin(80^\circ - 45^\circ)}{\sin^2 80^\circ} \times \left[ \frac{8^2 \sin(80^\circ - \tan^{-1}(1/2)) - 3^2 \sin(80^\circ - \tan^{-1}(1/3))}{\sin(45^\circ - \tan^{-1}(1/2)) - 3 \sin(45^\circ - \tan^{-1}(1/3))} \right] = 771.02\text{kN}
\]

\( T_{\text{EQ}} = 44.30 \text{kN} \)

\[
F = \frac{c_1 L_1 + c_2 L_2 + W_1 \cos \theta \tan \phi_1 + W_2 \cos \theta \tan \phi_2 + T_{\text{EQ}} \cos(\theta - \lambda)}{(W_1 + W_2) \sin \theta - T_{\text{EQ}} \sin(\theta - \lambda)} = 1.37
\]

Similarly factor of safety is calculated for different slope face angles considering various failure plane and the results are presented in Table 3.13.
Table 3.13 Factor of safety for different slope face angles considering various failure plane for reinforced layered slope

<table>
<thead>
<tr>
<th>$\beta$ (in degree)</th>
<th>$\theta$ (in degree)</th>
<th>$T_{EQ}$ (kN)</th>
<th>$W_1$ (kN)</th>
<th>$L_1$ (m)</th>
<th>$W_2$ (kN)</th>
<th>$L_2$ (m)</th>
<th>Factor of safety (F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>45</td>
<td>29.53</td>
<td>135.00</td>
<td>6.36</td>
<td>1030.50</td>
<td>25.63</td>
<td>1.34</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>39.00</td>
<td>64.34</td>
<td>4.29</td>
<td>409.60</td>
<td>10.70</td>
<td>1.30</td>
</tr>
<tr>
<td></td>
<td>55</td>
<td>36.31</td>
<td>86.29</td>
<td>4.89</td>
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<td>614.94</td>
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<td>29.28</td>
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<td>1191.26</td>
<td>35.74</td>
<td>1.65</td>
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<td>50</td>
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<td>5.12</td>
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<td>15.97</td>
<td>1.56</td>
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<td>47</td>
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<td>92.95</td>
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<td>660.46</td>
<td>19.56</td>
<td>1.56</td>
</tr>
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<td></td>
<td>48</td>
<td>35.74</td>
<td>87.64</td>
<td>5.42</td>
<td>612.82</td>
<td>18.23</td>
<td>1.55</td>
</tr>
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<td></td>
<td>49</td>
<td>36.40</td>
<td>82.64</td>
<td>5.26</td>
<td>569.38</td>
<td>17.04</td>
<td>1.56</td>
</tr>
<tr>
<td>75</td>
<td>40</td>
<td>31.45</td>
<td>125.68</td>
<td>7.05</td>
<td>1027.24</td>
<td>33.03</td>
<td>1.77</td>
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<td>35.54</td>
<td>90.00</td>
<td>5.80</td>
<td>649.34</td>
<td>20.97</td>
<td>1.72</td>
</tr>
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<td></td>
<td>42</td>
<td>33.30</td>
<td>109.67</td>
<td>6.48</td>
<td>846.55</td>
<td>27.13</td>
<td>1.74</td>
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<td>46</td>
<td>36.24</td>
<td>84.33</td>
<td>5.60</td>
<td>597.11</td>
<td>19.41</td>
<td>1.72</td>
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<td></td>
<td>44</td>
<td>34.83</td>
<td>96.08</td>
<td>6.01</td>
<td>707.53</td>
<td>22.75</td>
<td>1.72</td>
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<tr>
<td>70</td>
<td>38</td>
<td>31.69</td>
<td>126.34</td>
<td>7.47</td>
<td>1085.04</td>
<td>37.59</td>
<td>1.94</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>33.69</td>
<td>108.61</td>
<td>6.80</td>
<td>867.39</td>
<td>30.03</td>
<td>1.91</td>
</tr>
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<td></td>
<td>41</td>
<td>34.51</td>
<td>100.87</td>
<td>6.52</td>
<td>781.44</td>
<td>27.11</td>
<td>1.90</td>
</tr>
<tr>
<td></td>
<td>42</td>
<td>35.29</td>
<td>93.75</td>
<td>6.26</td>
<td>706.73</td>
<td>24.63</td>
<td>1.90</td>
</tr>
<tr>
<td></td>
<td>44</td>
<td>36.78</td>
<td>81.10</td>
<td>5.80</td>
<td>583.17</td>
<td>20.63</td>
<td>1.91</td>
</tr>
</tbody>
</table>
Fig. 3.29 Stability analysis of reinforced layered slope by Bishop’s method

The graphical representation of the stability analysis of reinforced layered slope by Bishop method is presented in Fig. 3.29. Graphical calculations are employed to estimate the values. A typical calculation is presented in Table 3.14 for $\beta=90^0$. Similarly, factor of safety is calculated for different slope face angles and presented in Table 3.15.

Slope face angle with respect to the horizontal ($\beta$) = $90^0$

Radius of slip surface (R) = 13.02m

Co-ordinate of the centre of slip surface ($x, y$) = (-4.28, 12.32)

Factor of safety for the given case $F = \frac{\sum_{i=1}^{n=10} [C + N \tan \phi]}{\sum_{i=1}^{n=10} [W \sin \alpha - T_y \cos(\alpha + \lambda)]} = \frac{960.83}{657.6} = 1.46$
Table 3.14 Factor of safety calculation for reinforced layered slope with slope face angle $\beta=90^\circ$ by Bishop’s method

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>$\alpha$ (degree)</th>
<th>$W$ (kN)</th>
<th>$T_n$ (kN)</th>
<th>$N$ (kN)</th>
<th>$L$ (m)</th>
<th>$C + N \tan \phi = cL + N \tan \phi$ (kN)</th>
<th>$W \sin \alpha - T_n \cos(\alpha + \lambda)$ (kN)</th>
<th>$F$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>21.80</td>
<td>150.9</td>
<td>0.00</td>
<td>162.52</td>
<td>1.08</td>
<td>86.63</td>
<td>56.04</td>
<td>1.46</td>
</tr>
<tr>
<td>2</td>
<td>30.96</td>
<td>150.6</td>
<td>0.00</td>
<td>175.63</td>
<td>1.17</td>
<td>93.71</td>
<td>77.48</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>30.96</td>
<td>148.2</td>
<td>0.00</td>
<td>172.83</td>
<td>1.17</td>
<td>92.96</td>
<td>76.25</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>38.66</td>
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<td>13.96</td>
<td>184.28</td>
<td>1.28</td>
<td>100.60</td>
<td>80.67</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>38.66</td>
<td>137.6</td>
<td>0.00</td>
<td>176.21</td>
<td>1.28</td>
<td>98.44</td>
<td>85.94</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>50.19</td>
<td>127.2</td>
<td>6.65</td>
<td>198.69</td>
<td>1.56</td>
<td>115.72</td>
<td>94.42</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>57.99</td>
<td>57</td>
<td>0.00</td>
<td>107.55</td>
<td>0.94</td>
<td>66.55</td>
<td>48.34</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>57.99</td>
<td>51.75</td>
<td>0.00</td>
<td>97.64</td>
<td>0.94</td>
<td>54.41</td>
<td>43.88</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>67.38</td>
<td>81.9</td>
<td>1.12</td>
<td>212.94</td>
<td>2.6</td>
<td>129.50</td>
<td>75.36</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>81.03</td>
<td>19.44</td>
<td>0.00</td>
<td>124.65</td>
<td>3.85</td>
<td>122.31</td>
<td>19.20</td>
<td></td>
</tr>
<tr>
<td>$\sum$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>960.83</td>
<td>657.6</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.15 Factor of safety calculation for reinforced layered slope with different slope face angles by Bishop’s method

<table>
<thead>
<tr>
<th>$\beta$ (degree)</th>
<th>R (m)</th>
<th>x</th>
<th>y</th>
<th>No. of slices</th>
<th>$\sum[C + N \tan \phi]$ (kN)</th>
<th>$\sum[W \sin \alpha - T_n \cos(\alpha + \lambda)]$ (kN)</th>
<th>$F$</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>13.02</td>
<td>-4.28</td>
<td>12.32</td>
<td>10</td>
<td>960.83</td>
<td>657.6</td>
<td>1.46</td>
</tr>
<tr>
<td>80</td>
<td>12.7</td>
<td>-2.32</td>
<td>12.44</td>
<td>10</td>
<td>1006.75</td>
<td>631.51</td>
<td>1.59</td>
</tr>
<tr>
<td>75</td>
<td>12.55</td>
<td>-1.42</td>
<td>12.44</td>
<td>11</td>
<td>1082.28</td>
<td>661.16</td>
<td>1.64</td>
</tr>
<tr>
<td>70</td>
<td>12.47</td>
<td>-0.56</td>
<td>12.44</td>
<td>11</td>
<td>1054.83</td>
<td>580.44</td>
<td>1.82</td>
</tr>
</tbody>
</table>
3.10 Summary

This chapter presents the background theory for analysis of soil nailed slopes. Various methods for analyzing slope stability are discussed first. Next the same methods are used for analyzing nailed slope. Modified equations for calculation of slope stability under dynamic condition are also illustrated for both unreinforced and nailed slopes. Necessary equations for calculating bearing capacity against heave and pull-out capacity from bond strength are presented. Some example problems are solved for better understanding of the slope stability analysis methods for both unreinforced slope and slopes reinforced with nails.
CHAPTER 4
DESIGN OF NAILED SOIL SLOPE

4.1 Introduction
This chapter describes the design procedure of nailed slopes. Some example of the slope stability problems were worked out in the previous chapter with nail and without nail. The results obtained from the analysis show that providing reinforcement in the slope improves the stability of slopes and embankments, making it possible to construct slopes and embankment steeper and higher than it would otherwise be possible. In this chapter, detailed discussion of the design procedure of nailed slope is provided step by step.

All the failure modes of the nail is been considered in the analysis. As limit equilibrium analyses provide valid indication for factor of safety and failure mechanisms for reinforced slopes therefore limit equilibrium analysis is used as the basis of the analysis procedure.

4.2 Design requirement
Installation of nailing along the slope face increases the resisting force against the driving force of the soil mass in the failure zone. Hence, it can be regarded as a slope stabilization method. The fundamental principle of soil nailing is the development of tensile force in the soil mass and renders the soil mass stable. Although only tensile force is considered in the analysis and design, soil nail also resist bending and shear force in the slope. Through finite element analysis by Cheng (1998), has demonstrated that the bending and shear contribution to the factor of safety is relatively insignificant and the current practice in soil nail design (of considering only tensile force) should be good enough for the most cases. Nails are usually constructed at an angle of inclination from 10° to 20°. Depending upon the climate of a particular region some sort of thickness of corrosion zone is assumed for an ordinary steel bar soil nail. As in Hong-Kong practice, a thickness of 2 mm is assumed as the corrosion zone so that the design bar diameter is totally 4mm less than the actual diameter of the bar. The nail is usually protected by galvanization, paint, epoxy and cement grout. Alternatively, fiber reinforced polymer (FRP) and
carbon fiber reinforced polymer (CFRP) may be used for soil nails which are currently under consideration.

There are several practices in the design of soil nails. The effective nail load is usually taken as the minimum of (a) the bond strength between cement grout and soil, (b) the tensile strength of the soil nail and (c) the bond stress between grout and the nail.

The following sub-sections describe the design of soil nail from two aspects- geometry of nailed slopes and soil characteristics.

4.2.1 Geometry of nailed slope
Nails are usually constructed at an angle of inclination from 10° to 20°. Steeper nail inclinations may be required, particularly for the upper row of nails if there is a significantly stronger soil zone located at a greater depth. However, it is recommended that nail should not be inclined at an angle of less than 10°, otherwise there is a possibility of creating voids in the grout increases. Depending upon the climate of a particular region some sort of thickness of corrosion zone is assumed for an ordinary steel bar soil nail.

The slopes without nail are flatter in most cases whereas, the slopes with nail can be constructed steeper. Nailed slopes can be inclined even at an angle of 70°-80°. While constructing the nailed slope the right-of-way, should be considered. The nail length should be chosen in such a manner that it should not cross the right-of-way. The following recommendations are made concerning soil nail length –

- Select uniform length pattern wherever possible.
- Select longer nails than required by the target factor of safety as a means to reduce wall deformation in the upper portions of the wall.
- Avoid the use of short nails in top portion of wall.
- Non-uniform nail length pattern may be used if soil layers with very dissimilar conditions are encountered
Fig. 4.1 shows the location of installing nails in the cross-section of an embankment. The fig. depicts that in a particular row, nails are aligned at same horizontal layer. The numbers of nails get reduced as it moves to the side of the embankment.

4.2.2 Soil characteristics

It is observed from different projects that soil nailing is cost effective than the other techniques for certain favorable ground conditions. However, in other ground conditions using soil nailing technique are too costly when compared to the other techniques.

Following are the in-situ conditions considered favorable for the prospective use of soil nailing technique.

(a) Soil shall be able to stand unsupported to a depth of about 1 m – 2 m high vertical or nearly vertical cut for 24-48 hours.

(b) Groundwater table shall be sufficiently below level of the lower most soil nail at all cross-sections.

(c) Favorable soils: Stiff to hard fine–grained soils, dense to very dense granular soils with some apparent cohesion, weathered rock with no weak planes, and glacial soils. Generally dry poorly graded cohesionless soils, strata with high ground water, soil with cobbles and boulders, soft to very soft fine grained soils, organic soils are not favorable for soil nailing.

Therefore, in order to determine whether a particular slope is good enough for providing nails or not, the following basic properties of soil should be determined from the field investigation and laboratory testing of the backfill of the slope:
1. Soil Classification (all soils);
2. Sieve Analysis (cohesionless soils);
3. Fines Content (mixed fine and coarse grained soils);
4. Natural Moisture (mostly fine grained soils);
5. Atterberg limits (fine grained soils); and
6. Organic content (fine grained soils)

4.3 Specifications

While designing nailed slope, we calculate the factor of safety against the failure modes. Table 4.1 suggests what should be the factor of safety against the static case and for seismic case, for both type of facings (temporary and permanent). The grout that fills the entire drill hole must have the properties as recommended in Table 4.2. To determine the pullout-capacity of the soil nail ultimate bond strength of the nail is to be known. Based on the construction method and the soil type ultimate bond strength of the soil nail is tabulated in Table 4.3. Once the soil type is identified properly, these values can be directly used in the design process. Table 4.4 tabulates the welded wire mesh dimension that is used in facing design. To account the non-uniform soil pressures behind facing, correction factor should be incorporated in the design process. Correction factor for facing flexure design for different facing thickness is recommended in Table 4.5. Another aspect in soil–nail design is to determine the area of reinforcement required per run for a particular spacing. In this context, Table 4.6 can be referred. Table 4.7 prescribes the dimension of the headed-stud permanent facing. Table 4.8 listed the values of different variable parameters.

<table>
<thead>
<tr>
<th>Failure mode</th>
<th>Resisting component</th>
<th>Symbol</th>
<th>Static</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Temporary structures</td>
</tr>
<tr>
<td>External Stability</td>
<td>Global Stability</td>
<td>FS_G</td>
<td>1.35</td>
</tr>
<tr>
<td></td>
<td>Sliding stability</td>
<td>FS_SL</td>
<td>1.3</td>
</tr>
</tbody>
</table>
### Table 4.2 Recommendation of Grout Properties

<table>
<thead>
<tr>
<th></th>
<th>Minimum compressive strength at 7 days:</th>
<th>Minimum 25 MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water cement ratio:</td>
<td></td>
<td>Not exceeding 0.45</td>
</tr>
<tr>
<td>Consistency:</td>
<td></td>
<td>Free from lumps and undispersed cement</td>
</tr>
<tr>
<td>Bleeding:</td>
<td></td>
<td>Not to exceed 4% of the initial volume. All bleed water shall be reabsorbed after 24 hours</td>
</tr>
<tr>
<td>Volume change after 24 hours:</td>
<td></td>
<td>within the range 0% to +5%</td>
</tr>
</tbody>
</table>

### Table 4.3 Estimated Bond Strength of Soil Nails in Soil and Rock

<table>
<thead>
<tr>
<th>Material</th>
<th>Construction Method</th>
<th>Soil/Rock type</th>
<th>Ultimate bond strength $q_u$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock</td>
<td>Rotary Drilled</td>
<td>Marl/Limestone</td>
<td>300-400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Phyllite</td>
<td>100-300</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Chalk</td>
<td>500-600</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Soft dolomite</td>
<td>400-600</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fissured dolomite</td>
<td>600-1000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Weathered sandstone</td>
<td>200-300</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Weathered Shale</td>
<td>100-150</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Weathered Schist</td>
<td>100-175</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Basalt</td>
<td>500-600</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Slate/hard shale</td>
<td>300-400</td>
</tr>
<tr>
<td></td>
<td>Method</td>
<td>Soil Type</td>
<td>Range</td>
</tr>
<tr>
<td>------------------</td>
<td>---------------</td>
<td>---------------------------------------</td>
<td>-----------</td>
</tr>
<tr>
<td><strong>Cohesionless soils</strong></td>
<td>Rotary Drilled</td>
<td>Sand/gravel</td>
<td>100-180</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Silty Sand</td>
<td>100-150</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Silt</td>
<td>60-75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Piedmont residual</td>
<td>40-120</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fine colluviums</td>
<td>75-150</td>
</tr>
<tr>
<td></td>
<td>Driven casing</td>
<td>Sand/gravel</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Low overburden</td>
<td>190-240</td>
</tr>
<tr>
<td></td>
<td></td>
<td>High overburden</td>
<td>280-430</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dense moraine</td>
<td>380-480</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Colluvium</td>
<td>100-180</td>
</tr>
<tr>
<td></td>
<td>Augered</td>
<td>Silty sand fill</td>
<td>20-40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Silty fine sand</td>
<td>55-90</td>
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<td></td>
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<td>Silty clayey sand</td>
<td>60-140</td>
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<tr>
<td></td>
<td>Jet grouted</td>
<td>Sand</td>
<td>380</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sand/gravel</td>
<td>700</td>
</tr>
<tr>
<td><strong>Fine-grained soils</strong></td>
<td>Rotary drilled</td>
<td>Silty clay</td>
<td>35-50</td>
</tr>
<tr>
<td></td>
<td>Driven casing</td>
<td>Clayey silt</td>
<td>90-140</td>
</tr>
<tr>
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<td>Augered</td>
<td>Loess</td>
<td>25-75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Soft clay</td>
<td>20-30</td>
</tr>
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<td>Stiff clay</td>
<td>40-60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Stiff clayey silt</td>
<td>40-100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Calcareous sandy clay</td>
<td>90-140</td>
</tr>
</tbody>
</table>
### Table 4.4 Welded Wire Mesh Dimensions

<table>
<thead>
<tr>
<th>Mesh Designation(1),(2)</th>
<th>Wire cross-sectional area per unit length(3) (mm²/m)</th>
<th>Weight per unit area(kg/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>102x102-MW9xMW9</td>
<td>88.9</td>
<td>1.51</td>
</tr>
<tr>
<td>102x102-MW13xMW13</td>
<td>127.0</td>
<td>2.15</td>
</tr>
<tr>
<td>102x102-MW19xMW19</td>
<td>184.2</td>
<td>3.03</td>
</tr>
<tr>
<td>102x102-MW26xMW26</td>
<td>254.0</td>
<td>4.30</td>
</tr>
<tr>
<td>152x152-MW9xMW9</td>
<td>59.3</td>
<td>1.03</td>
</tr>
<tr>
<td>152x152-MW13xMW13</td>
<td>84.7</td>
<td>1.46</td>
</tr>
<tr>
<td>152x152-MW19xMW19</td>
<td>122.8</td>
<td>2.05</td>
</tr>
<tr>
<td>152x152-MW26xMW26</td>
<td>169.4</td>
<td>2.83</td>
</tr>
</tbody>
</table>

**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. Prefix M indicates metric units and prefix W indicates plain wire. If wires are pre-deformed, the prefix D shall be used instead of W.
3. This value is obtained by dividing the wire cross-sectional area by the mesh opening size.

### Table 4.5 Correction Factors C_F

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Table 4.6 Area of reinforcement bars at given Spacing (Values in cm² per Meter width)

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\frac{3}{4} \times 6 \frac{3}{16} & 157 & 31.8 & 19.1 & 9.5 \\
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\frac{7}{8} \times 5 \frac{3}{16} & 127 & 34.9 & 22.2 & 9.5 \\
\hline
\frac{7}{8} \times 6 \frac{3}{16} & 152 & 34.9 & 22.2 & 9.5 \\
\hline
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\]

\(D_H\) and \(t_H\) denoted the depth and the thickness of the head respectively; \(D_S\) and \(L_S\) denoted the depth and the length of the stud respectively (Ref. to Fig.4.2)

![Fig. 4.2 Reference Fig. for dimension of headed stud](image)

D_H and t_H denoted the depth and the thickness of the head respectively; D_S and L_S denoted the depth and the length of the stud respectively (Ref. to Fig.4.2)
Table 4.8 Variable Parameters

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4.4 Failure modes of soil nail wall

Fig. 4.3 Different failure modes of soil nail wall
The ability of the soil nail wall to act as a coherent gravity mass is a function of the vertical and horizontal spacing of the nails, the long-term allowable strength of the nails, the stress transfer between the reinforced soil and the nail, the connection strength between the nail and the facing, and the flexural strength of the facing. Based on these parameters, there are different failure modes. Broadly these failure modes of soil nail walls can be classified into three distinct groups as external, internal and facing failure modes. Fig. 4.3 shows different failure modes. The stability of nailed slopes for different failure modes are discussed in the subsequent sections.

4.4.1 External stability of nailed slope
External stability of the soil nail wall is concerned with the ability of the reinforced soil mass to withstand the earth pressures and surcharge loads exerted on the composite material from the retained soils. The different modes of failure for external stability are slope stability, sliding and bearing capacity.

4.4.1.1 Slope stability
Refer to section 3.3 for detailed discussion.

4.4.1.2 Sliding stability
When additional lateral earth pressures, mobilized by the excavation, exceed the sliding resistance along the base sliding failure may occur. Sliding stability considers 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 soil nails. Hence, while determining the preliminary length of the soil nails, a check of the reinforced soil mass resistance to sliding at the base of the soil nail wall should be performed.
To evaluate the factor of safety for sliding stability, soil nail wall is considered as a rigid block of width $B_L$ (Ref. to Fig. 4.4) against which lateral earth force from retained soil are applied. It is assumed that the displacements of the soil block along its base are large enough to mobilize the active pressure acting on it. The factor of safety against sliding is determined as follows:

$$FS_{SL} = \frac{\sum \text{Horizontal Resisting Force}}{\sum \text{Horizontal Driving force}}$$  \hspace{1cm} (4.1)$$

Where, the resisting force is the least of the shear resistance along the base of the wall, or of a weak layer near the base of the soil nail wall and the driving force is the horizontal component of the thrust on the vertical plane at the back of the nails.

$$FS_{SL} = \frac{c_b B_L + (W + Q_T - F_V + P \sin \beta_{eq}) \tan \phi_b}{F_h + P \cos \beta_{eq}}$$  \hspace{1cm} (4.2)$$
Where,

\( c_b \) and \( \phi_b \) = soil strength parameter along the base of rigid sliding block (AD);

\( \beta_{eq} \) = equivalent back slope angle [for broken slopes \( \beta_{eq} = \tan^{-1}(\Delta H/2H_1) \), for infinite slopes \( \beta_{eq} = \beta \)]

\( B_L[m] = L + H \tan \alpha \) = base width of the rigid sliding block (AD); 

\( H_1[m] = H + \Delta H = H + L \tan \beta \) = effective height over which earth pressure acts (CD); 

\( Q_T[kN/m] = q_s L \) = total surcharge load;

\( W[kN/m] = W_{ABF} + W_{BCE} + W_{BEDF} = \frac{1}{2} \gamma H^2 \left[ \tan \alpha + 2 \left( \frac{L}{H} \right) + \left( \frac{L}{H} \right)^2 \tan \beta \right] \) = total weight of the rigid sliding block (ABCD);

\( F_h[kN/m] = k_h (W + Q_T) \) = horizontal seismic inertia force

\( F_v[kN/m] = k_v (W + Q_T) \) = horizontal seismic inertia force

**Determination of total active thrust \( P \)**

\[ P = \frac{\gamma H^2}{2} K (1 - k_v) \left\{ 1 + \frac{2q_s}{\gamma H_1} \left[ \frac{\cos \alpha}{\cos(\beta - \alpha)} \right] \right\} \]  \hspace{1cm} (4.8)

Where, \( K \) = coefficient of lateral active earth pressure which can be determined using Eq. 4.9

\[ K = \frac{\cos^2(\phi - \alpha - \omega)}{\cos \omega \cos^2 \alpha \cos(\alpha + \beta + \omega) \left[ 1 + \frac{\sin(\phi + \beta) \sin(\phi - \beta - \omega)}{\cos(\alpha + \beta + \omega) \cos(\beta - \omega)} \right]^2} \]  \hspace{1cm} (4.9)

Where: \( \omega \) [degrees] = an angle relating the horizontal and vertical seismic coefficients such that \( \phi - \beta \geq \omega \), given by Eq. 4.10

\[ \omega = \tan^{-1} \left( \frac{k_h}{1 - k_v} \right) \]  \hspace{1cm} (4.10)

**4.4.1.3 Bearing capacity failure**

Refer to section 3.5 for detailed discussion.
### 4.4.2 Internal stability of nailed slope

Internal stability of the soil nail wall is concerned with the ability of the nails carrying tensile forces and transferring them by friction, friction and adhesion, or friction and bearing. During excavation, while the soil nail wall system deforms, it mobilizes bond strength between the grout and the surrounding soil. This bond strength is mobilized progressively along the entire soil nail which subsequently causes tensile forces to develop. Due to insufficient bond strength or inadequate tensile strength of the nail or slippage of the grout and steel bar interface failure occurs in the load transfer mechanisms between the soil, the nail and the grout. These failure modes are denoted as internal failure modes.

The most common internal failure mechanisms include:

- **Nail pull-out failure**: failure along the soil-grout interface due to insufficient intrinsic bond strength or insufficient nail length.
- **Tensile failure of the nail**: inadequate tensile strength
- **Slippage of the bar-grout interface**
- **Bending and shear of the nails**

As the common and recommended design practice is to use threaded bars and relatively high-strength grout, the potential slippage between nail and grout can be avoided and therefore, disregarded. Due to relatively ductile behavior of the mild steel reinforcements and no strength contribution assigned to the grout, the shear and bending strengths of the soil nails are conservatively disregarded in most current design methods. Generally, the two remaining mechanisms i.e. nail pull-out failure and nail tensile failure are the two failure criteria. These two mechanisms are discussed in the following sub-sections. Before discussing that, tensile force distribution in nails are discussed..

Fig. 4.5 describes the tensile force distribution from the anchoring zone to the facing. The tensile force at the end of the nail is zero. It starts increasing at rate $Q_a$ which is the pull-out capacity per unit length as discussed in next section. At some point which is not necessarily the failure surface it reaches a maximum value $T_{\text{max}}$ which determines nail tensile strength. Then again it starts decreasing at the same rate $Q_a$ till it reaches $T_D$ at the nail head which determines facing capacity. The value $T_{\text{max}}$ is bounded by three limiting conditions: the pullout capacity,
the tensile capacity, and the facing capacity. The minimum of these three capacities controls the value of $T_{max}$.

![Fig. 4.5 Nail tensile force distribution](image)

### 4.4.2.1 Nail pull-out failure

Nail pullout failure is a failure along the soil-grout or soil-nail (in case of driven nails) interface due to insufficient intrinsic bond strength and/or insufficient nail length. It is the primary internal failure mode in a soil-nail wall. The pull-out capacity of the soil nails is governed by the following factors:

- The location of the critical slip plane of the slope.
- The size (diameter) of the grouted hole for soil nail.
- The ground-grout bond stress (soil skin friction).

In general, the mobilized pullout per unit length, $Q$ (also called the load transfer rate) can be expressed as:

$$Q = \pi q d$$  \hspace{1cm} (4.11)

Where:

- $q =$ mobilized bond shear stress acting around the perimeter of the nail-soil interface; and
- $d =$ average or effective diameter of the drill hole (Ref. to Fig. 3.14).
In reality, mobilized bond shear stress and subsequently the load transfer rates are not uniform (as it depends on various factors e.g. soil conditions, grout characteristics, nail length etc.); however, for simplification mobilized bond strength is often assumed to be constant along the nail resulting in a constant load transfer rate, $Q$. Therefore, the nail force at the end of the pullout length, $L_p$, is given as eq.

$$ T_0 = Q L_p $$  \hspace{1cm} (4.12)

$L_p$ is the length of the nail beyond the failure surface. The stability contribution of the lower soil nail is more significant than the upper one because the nail length behind the failure surface ($L_p$) is more in the lower nails compare to the upper ones. The higher the value of $L_p$ the higher it will develop the pullout capacity of the soil nail.

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

$$ R_p = T_{\text{max}} = Q_u L_p $$ \hspace{1cm} (4.13a)

$$ Q_u = \pi q_u d $$ \hspace{1cm} (4.13b)

Where:

$Q_u =$pullout capacity per unit length; and
\( q_u = \text{ultimate bond strength.} \)

The value of \( q_u \) can be used either from the Table 4.3 (where \( q_u \) is specified for various soils and drilling method) or can be derived from nail pullout tests.

To minimize the effect of some of the uncertainties regarding bond strength and soil-grout interaction, the value of \( Q_u \) in design (allowable \( Q_u \)) is used as given by

\[
Q_{u, \text{allowable}} = \frac{Q_u}{F_{S_p}}
\]

and subsequently,

\[
R_{p, \text{allowable}} = \frac{R_p}{F_{S_p}}
\]

Where, \( F_{S_p} \) is the factor of safety against pullout failure. A minimum factor of safety of 2 is recommended against pullout failure.

For any particular nail embedded at depth \( z \) from the ground surface, \( F_{S_p} \) can be determined as given by

\[
(F_{S_p}) = \left( \frac{R_p}{T} \right)_z
\]

where, \( R_p \) is determined from Eq. 4.13 and max. axial force \( T \) at depth \( z \) can be obtained from Eq. 4.17.

\[
(T)_z [kN] = K (q_s + \gamma z) S_h S_v
\]

\( k \) can be obtained from Eq. 4.9.

### 4.4.2.2 Nail tensile failure

Tensile failure of a nail takes place when the axial force along the soil nail \( T \) is greater than the nail tensile capacity which may lead to the break-up of the tensile member. The location of maximum tensile force \( T_{\text{max}} \) plays a crucial role in determining the tensile capacity of soil nails. Fig. 4.7 shows tensile force distribution along three soil nails, installed at different height of the
facing. It is observed that the contribution of tensile forces to global stability varies from nail to nail. Here, tensile force is considered to be maximum at the bottom nail. Tensile forces may increase moderately (e.g. generally 15%) in the time period between end of construction condition and the long-term, steady condition. This increase in tensile force occurs due to post construction soil creep and stress relaxation. However, this additional load is not calculated in the analysis procedure. It is taken into consideration in the design of soil nail wall by means of factor of safety.

Factor of safety against nail tensile strength failure $FS_T$ for any nail embedded at depth $z$ can be calculated as given in Eq. 4.19

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

where:

- $(R_T)_z$ [kN] = maximum axial tensile load capacity of nail = $(0.25 \pi d^2 f_y) / 1000$
- $(T)_z$ = maximum axial force developed in the nail at depth $z$ as given in Eq. 4.17

Fig. 4.7 Schematic location of soil nails maximum tensile forces
4.4.3 Facing design

Soil nail walls are generally provided with two types of facings: (a) temporary facing and (b) permanent facing. Temporary facing is usually constructed by providing reinforcement in the form of welded wire mesh throughout the wall face, and by additional bearing plates and waler bars at the nail heads; which is, subsequently shotcreted. On the other hand, permanent facing is usually constructed as cast-in-place RCC. However, reinforcement in the permanent facing may be adopted in the form of welded wire mesh or reinforcement bars in either direction. Most of the times temporary facing resists major portion of the loads transferred from soil nails at nail head at the wall face, while permanent facing serves the purpose of improving aesthetic of the wall face. Connection between temporary facing and permanent facing is usually provided by means of headed-studs (usually four numbers per plate) welded on the bearing plates.

In a nutshell, the facing of the soil nail serves the following functions:

- it provides lateral confinement of the soil at the face of the excavation
- it prevents or minimizes the deterioration of the soil's shear strength associated with exposure to the elements
- it may support external loads (e.g. facing panels used for decorative purposes)
- it prevents sloughing between nails.

4.4.3.1 Tensile force at the Wall facing

When the soil nails are connected to a facing structure at the slope surface, the whole nail system behaves like an earth retaining structure anchored with soil nails. A potential flow slide is initiated by the strain-softening of the filling material; the movement of sliding mass is restricted due the presence of facing. As the soil pushes the facing away from the slope, axial forces and bending moments are induced in the facing. Simultaneously, tensile forces are mobilized in the soil nails. The development of the nail forces and structural forces in the facing indicates that the stabilizing mechanism in the nailed loose fill slope relies on the structural facing, which resists the forces generated from the potential failure mass. The tensile force developed at the wall facing \((T_0)\) depends on many parameters including the reinforcement spacing \((s_v\) and \(s_h)\), the rigidity of the facing, and the stiffness of the connection between the reinforcement and the facing. McGown et. al.(1987) showed that a reduction in the vertical spacing between the
reinforcements produces horizontal stresses on the facing. This consequently leads to a decrease in the tensile forces mobilized in the reinforcements, in the vicinity of the wall facing. Due care should be taken during the design process that the maximum tensile forces in the reinforcements do not occur at the wall facing.

**Fig. 4.8 Soil-nail Stress Transfer mechanism**

### 4.4.3.2 Facing Failure modes

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

- Flexure Failure
- Punching Shear Failure
- Headed-Stud Tensile Failure

For each of these failure modes, the nail head and facing must be designed properly so that it should have the capacity more than the maximum nail head tensile force ($T_0$) at the wall face. In order to achieve the design capacities with adequate factor of safety for all, potential failure modes, appropriate dimensions, strength, and reinforcement of the facing and suitable nail head hardware (e.g. bearing plate, nut, and headed studs) must be provided. In the following sections the facing failure modes are discussed briefly.

*(i) Flexure Failure:*
To analyze the facing in flexure we assume it to be a continuous reinforced concrete slab. The lateral earth pressure acts as the load and nail tensile forces act as the support. These induce positive moments (i.e. tension on the outside of the section) in the mid-span between nails and negative moments (i.e. tension on the inside of the section) around the nails. High value of these moments results in a flexure failure of the shotcrete. As the lateral pressure increases, fractures grow and deflections ($\delta$) and nail tensile forces increase. Fig. 4.9a and 4.9b show the fractures in the facing and its deflection pattern respectively. The stiffness of the facing plays an important role in the pressure distribution of the facing. Thin facings (as typical temporary facing) have low stiffness. This causes the facing to deform in the mid-span section and subsequently resulting in a relatively lower value of soil pressure in the mid-span section. Thicker facing have more stiffness and thus results in a lower deformation. The tensile force that is obtained around the nail head due to the soil pressure that causes facing failure is known as facing flexural 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.

(ii) *Punching Shear Failure:*

![Fig. 4.9 (a) Fracture and (b) Deflection pattern of facing subjected to flexure failure](image)
Punching shear is a type of failure of reinforced concrete slabs subjected to high localized forces. As in the nailed slope, the facing experience the concentrated load where the nails are located, therefore, there is a probability of this type of failure around the nail head. Hence, the nail-head capacity must be assessed in consideration of punching shear capacity which is designated as $R_{FP}$. With increase of nail head tensile force to a critical value, fractures can form a local failure mechanism resulting in a conical failure surface around the nail head. This failure surface extends beyond the bearing plate connection (used in temporary facings) or headed studs connection (used in permanent facings) and punches through the facing at an angle of $45^\circ$. Fig. shows the punching shear failure modes in different nail-face connection. The size of the cone depends on the facing thickness and the type of the nail-face connection. The factor of safety against punching shear failure ($FS_{FP}$) is defined as the ratio of $T_0$ and $R_{FP}$. Generally for static loads, a minimum factor of safety of 1.35 (in temporary walls) and 1.5 (in permanent load) is adopted.

![Fig. 4.10 Punching Shear Failure](image)

(iii) Headed-stud tensile failure:
This is failure of the headed studs in tension. Unlike the other two failure modes this failure mode is only a concern for permanent facings. Fig.4.11b shows the details of the headed-stud connector.

For static loads, the allowable min. factor of safety (FOS) against tensile failure along the headed-stud depends on the nail-face connection and the yield strength of the steel that is used in the construction. Mostly, FOS is taken within 1.5-2.

It is required to provide sufficient anchorage to headed-stud connectors and extended them at least to the middle of the facing section and preferably behind the mesh reinforcement in permanent facing. Also, another requirement is to provide a minimum 50mm of cover over headed-studs.
4.4.3.3 Facing design procedure

Step-1: Determining design nail head tensile force at the wall face $T_0$

$$T_0[kN] = T_{\text{max}} [0.6 + 0.2 (S_{\text{max}} - 1)]$$

Where: $T_{\text{max}} [kN] = \text{max. axial force developed in the soil nails}$; and

$$S_{\text{max}} [m] = \text{max. of } S_v \text{ and } S_h$$

Step-2: Selecting facing thickness:

Temporary facing thickness $h_2$: [e.g., 100, 150, 200 mm].

Permanent facing thickness $h_1$: [e.g., 200 mm]

Step-3: Selecting appropriate facing materials

(a) Adopt steel reinforcement grade Fe 415 (or Fe 500) i.e. characteristic strength $f_y = 415$ MPa (or 500 MPa)

(b) Adopt suitable welded wire mesh (WWM) and reinforcement bar (see Table 4.4 and 4.6)

(c) Adopt suitable concrete/shotcrete grade between M20 to M30 i.e. characteristic compressive strength $f_{ck}$ between 20MPa to 30MPa.

(d) Adopt suitable headed-stud characteristics (see Table 4.7)

(e) Adopt bearing plate geometry: minimum size 200mm x 200mm x 19mm.

Step-4: Check for minimum and maximum reinforcement requirements

(a) Calculate the minimum and maximum reinforcement requirement ratio as given in Eq.4.21 and Eq. 4.22. At any section of the facing, reinforcement ratio is defined as the ratio of the effective area of the reinforcement to the effective area of the concrete.
Fig. 4.12 Reinforcement in Facing

Fig. 4.12 shows the reinforcement area in facing. ‘a’ denotes the area of the reinforcement and the subscript ‘v’ and ‘h’ denotes the vertical and horizontal direction and subscript ‘n’ and ‘m’ denotes nail head and mid-span region respectively.

\[
\rho = \frac{\sigma_{ck} \times 100}{f_y} \quad \text{(4.21)}
\]

\[
\rho = \frac{\sigma_{ck} \times 100}{f_y} \left( \frac{600}{600 + f_y} \right) \quad \text{(4.22)}
\]

The placed reinforcement should be within \( \rho_{\text{min}} \leq \rho \leq \rho_{\text{max}} \). Another issue should be taken care of, that the ratio of the reinforcement in the nail head and mid-span zones should be less than 2.5 to ensure comparable ratio of flexural capacities in these areas.

(b) Select reinforcement area/unit length of WWM for temporary/permanent facing (see Table 4.4) at the nail head \( a_n \) and at mid-span \( a_m \) in both the vertical and horizontal directions. Usually, the amount of reinforcement at the nail head is adopted same as the
amount of reinforcement at the mid-span (i.e., \( a_n = a_m \)) in both vertical and horizontal
directions. However, for temporary facing, if waler bars are used at the nail head in
addition to the WWM, recalculate the total area of reinforcement at the nail head in the
vertical direction and horizontal direction using Eqs. (4.23) and (4.24) respectively.

\[
\begin{align*}
    a_{vn} &= a_{vm} + \frac{A_{v_w}}{S_h} \\
    a_{hn} &= a_{hm} + \frac{A_{h_w}}{S_v}
\end{align*}
\]  

(4.23)  

(4.24)

where: \( a_{vn} \) and \( a_{hn} \) are the reinforcement cross sectional areas per unit width in the
vertical and horizontal directions at the nail head respectively; \( a_{vm} \) and \( a_{hm} \) are the
reinforcement cross sectional area per unit width in the vertical and horizontal directions
at the mid-span respectively; and \( A_{v_w} \) and \( A_{h_w} \) are the total cross sectional area of waler
bars in the vertical and horizontal directions respectively.

(c) Calculate the reinforcement ratio at the nail head and the mid-span as:

\[
\rho_n[\%] = \frac{a_n}{0.5h} \times 100
\]

(4.25)

\[
\rho_m[\%] = \frac{a_m}{0.5h} \times 100
\]

(4.26)

(d) Verify that the reinforcement ratio of the temporary and permanent facing at the mid-
span and the nail head are greater than the minimum reinforcement ratio (i.e. \( \rho_{min} \leq \rho \))
otherwise increase the amount of reinforcement (\( a_n \) and/or \( a_m \)) to satisfy this criterion.

(e) Verify that the reinforcement ratio of the temporary and permanent facing at the mid-
span and the nail head are smaller than the maximum reinforcement ratio (\( \rho \leq \rho_{max} \)),
otherwise reduce the amount of reinforcement (\( a_n \) and/or \( a_m \)) to satisfy this criterion.

**Step-5: Verifying facing flexural resistance \( R_{FF} \)**

(a) Calculate facing flexural resistance \( R_{FF} \) for the temporary and permanent facing as the
minimum of:

\[
R_{FF}[kN] = \frac{C_F}{265} \times (a_{vn} + a_{vm}) \left[ \frac{mm^2}{m} \right] \times \left( \frac{S_h}{S_v} \right) [m] \times f_y [MPa]
\]

(4.27)
\[
R_{FF}[kN] = \frac{C_F}{265} \times (a_{hm} + a_{hm}) \left[ \frac{mm^2}{m} \right] \times \left( \frac{S}{S_h} \right) h[m] \times f_y[MPa]
\]

(4.28)

\(C_F=\) Correction factor that considers the non-uniform soil pressure behind the facing. For permanent facing \(C_F\) is adopted taken equal to 1, whereas, for temporary facings with thickness: 100 mm, 150 mm and 200m, \(C_F\) shall be adopted as 2.0, 1.5 and 1.0 respectively.

(b) Determine the safety factor against facing flexural failure (\(FS_{FF}\)) using Eq. (4.29) and if minimum recommended factor of safety against facing flexural failure is not achieved, redesign the facing with increased thickness of facing, steel reinforcement strength, concrete strength, and/or amount of steel and repeat the facing flexural resistance calculations.

\[
FS_{FF} = \frac{R_{FF}}{T_0}
\]

(4.29)

**Step 6: Verify facing punching shear resistance \(R_{FP}\)**

(a) For practical purposes, punching shear capacity \(R_{FP}\) is assessed similar to the concrete structural slabs subjected to concentrated loading and is evaluated as:

\[
R_{FP}[kN] = 330 \sqrt{f_{ck}[MPa] \pi D_c[m] h_c[m]}
\]

(4.30)

Where:

\(D_c\) = effective diameter of conical failure surface at the center of section (i.e., considering an average cylindrical failure surface)

\(h_c\) = effective depth of conical surface

(b) For temporary facing (see Fig. 4.10)

\[D_c = L_{BP} + h \quad \text{and} \quad h_c = h; \quad \text{and} \quad L_{BP} = \text{length of bearing plate and} \quad h = \text{thickness of temporary facing}

For permanent facing (see Fig. 4.11)

\[D_c = \text{minimum of} \ (S_{hs} + h_c \quad \text{and} \quad 2h_c) \quad \text{and} \quad h_c = L_S - t_H + t_p\]
Where: $S_{hs} =$ headed-stud spacing, $L_s =$ headed-stud length, $t_h =$ headed-stud head thickness, and $t_p =$ bearing plate thickness (see Table 4.7).

(c) Determine the safety factor against facing flexural failure ($FS_{FP}$) using Eq. (4.31) and if capacity for the temporary/permanent facing is not adequate, then implement, larger elements or higher material strengths and repeat the punching shear resistance calculations.

\[
FS_{FP} = \frac{R_{FP}}{T_0}
\]  

(4.31)

**Step-7: Verify headed-stud resistance $R_{HT}$ (Permanent facing)**

(a) The tensile capacity of the headed-studs connectors providing anchorage of the nail into the permanent facing (also providing connection between both temporary and permanent facings) must be verified. The nail head capacity against tensile failure of the headed-studs $R_{HT}$ can be computed as:

\[
R_{HT} = N_H A_{SH} f_y
\]  

(4.32)

where:

$N_H =$ number of headed-studs in the connection (usually 4);

$A_{SH} =$ cross-sectional area of the headed-stud shaft and

$f_y =$ tensile yield strength of the headed-stud.

(b) Knowing the nail head capacity against tensile failure of the headed-studs $R_{HT}$ and the axial force at nail head $T_0$, factor of safety against the tensile failure of the headed-studs $FS_{HT}$ can determined as:

\[
FS_{HT} = \frac{R_{HT}}{T_0}
\]  

(4.33)

(c) Also verify that compression on the concrete behind headed-stud is within tolerable limits by assuring that: :

\[
A_h \geq 2.5 A_{SH} \quad ; \quad t_h \geq 0.5(D_h - D_s)
\]  

(4.34)
Step 8: Other facing design considerations:

To minimize the likelihood of a failure at the nail head connection: (1) bearing plates should be mild steel with a minimum yield stress $f_y$ equal to 250 MPa, (2) nuts should be the heavy-duty, hexagonal type, with corrosion protection, and (3) beveled washers (if used) should be steel or galvanized steel.

4.5 Seismic consideration for stability analysis of nailed slope

In comparison to other flexible retaining structure, soil nail walls have an intrinsic flexibility which makes the soil-nailed systems to have an inherent satisfactory seismic response.

In areas of high seismic exposure, soil nail walls have generally performed well in contrast to the generally poor performance of gravity retaining structures. The design method that has been prescribed so far, involves high level of conservatism into it. This also makes the soil-nailed system safe against the ground motion to some extent. However, some seismic design method should be adopted while constructing the nailed-wall system in the region of frequent earthquakes as the shear strength of the soil reduces due to earthquake loading. Seismic design methods have been based on the concept of limit-equilibrium where seismic forces are substituted with static loads by performing various laboratory and field experiments. The following section discusses some of the analyses needed to assess seismic effects on global and sliding stability.

4.5.1 Selection of seismic coefficients

In order to consider the effect of seismic responses on ground nailed-wall system, seismic coefficient related to the seismically-induces wall displacement should be incorporated in the design section. If the tolerable lateral movement of wall by seismic force is considered to be $d_c$, horizontal seismic coefficient can be expressed as,

$$k_h = 1.66A_m\left(\frac{A_m}{d_c\text{ (mm)}}\right)^{0.25}$$

(4.35)

where:
\[ A_m = \text{normalized horizontal acceleration, which acts at the centroid of the wall-soil mass. It is a function of the normalized peak ground acceleration (PGA) coefficient (A), which is the ratio of PGA and acceleration of gravity (g), and is defined as shown in the Eq.4.36} \]

\[ A_m = (1.45 - A)A \quad (4.36) \]

\[ d_e = \text{It depends on the wall type and the service the wall provides. The Eq.4.35 is valid only for } 25 \leq d_e \leq 200. \text{ It shows that smaller the value of } d_e \text{ higher will be } k_h \text{ and will results in larger nail length.} \]

It is recommended that the Eq. 4.35 should not be used when:

- the PGA coefficient A is \( \geq 0.3 \);
- the wall has a complex geometry (i.e. the distribution of mass and/or stiffness is abrupt); and
- the wall height is greater than approximately 15m

Specific site response dynamic analyses are required for the soft soils where significant ground acceleration amplification and non-linear site response may take place. When the simple pseudo-static method describes above is not applicable, specifically for large walls subjected to strong ground motions, dynamic and deformation analyses may be necessary.

### 4.5.2 Seismic effects on sliding stability

The total active thrust \( P_{AE} \) which is the combination of the static and dynamic active lateral earth pressure must be considered while analyzing sliding stability of soil nail wall under seismic loads. To induce the seismic response in the design procedure this increased lateral earth force must be incorporated in the denominator portion of the Eq. This lateral earth force is evaluated using the Mononobe-Okabe (M-O) method, which is an extension of the Coulomb theory. The assumption of the M-O method are:

- the total active thrust \( P_{AE} \) acts behind the wall;
- the wall and the nailed soil are considered a rigid block (the ground acceleration is fully transmitted to the system);
- the wall movement induces active earth pressure conditions behind the block (the soil behind the soil nail wall system is “yielding”); and
the soil behind the soil nail wall system is drained, (i.e. neither excess pore pressures nor hydrodynamic effects are considered, which is typical for soil nail walls). The total active thrust, \( P_{ae} \), acting behind the wall-nailed soil block is expressed as:

\[
P_{ae} = \frac{\gamma H_1^2}{2} K_{AE} \left(1 - k_v\right) \left\{1 + \frac{2q_s}{\gamma H_1} \left[\cos \alpha \right]\right\}
\]

(4.37)

where:

\( \gamma = \) 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; and
\( q_s = \) distributed surface loading

In the general case of a wall (Fig. 3.1), the total active pressure coefficient can be calculated using the M-O formulation:

\[
K_{AE} = \frac{\cos^2 \left(\varphi - \omega - \alpha'\right)}{\cos \omega \cos^2 \alpha' \cos \left(\alpha' + \delta + \omega\right) D}
\]

(4.38)

where:

\( \varphi = \) angle of internal friction of soil behind wall;
\( \alpha' = \) batter angle (from vertical) of wall internal face;
\( \beta = \) backslope angle;
\( \delta = \) wall-soil interface friction angle; and

\[
\omega = \tan^{-1} \left(\frac{k_h}{1 - k_v}\right)
\]

(4.39)

\[
D = 1 + \frac{\sin \left(\varphi + \delta\right) \sin \left(\varphi - \omega - \beta\right)}{\cos \left(\delta + \alpha' + \omega\right) \cos \left(\beta - \alpha'\right)}
\]

(4.40)
Fig. 4.13 shows the parameters that should be considered in the wall geometry in the Mononobe Okabe method. The failure plane behind the wall is oriented at an angle $\xi$ from the horizontal which is defined as:

$$\xi = \varphi - \omega + \rho^*$$  \hspace{1cm} (4.41)

where:

$$\rho^* = \tan^{-1} \left[ \frac{\sqrt{A(A^2 + 1)(A + B) - A^2}}{1 + B(A^2 + 1)} \right]$$  \hspace{1cm} (4.42)

with:

$$A = \tan(\varphi - \omega - \beta)$$

$$B = \tan(\delta + \omega + \beta)$$

Fig. 4.13 Generic wall geometry in the Mononobe-Okabe method
From the Eq. 4.38 it can be inferred that the total active pressure coefficient $K_{AE}$ is a function of $k_h$, $k_v$, backslope angle, wall interface angle, friction angle etc. Fig. 4.14a shows the variation of the total active pressure coefficient as a function of the horizontal seismic coefficient and the friction angle for horizontal backslope. Here, vertical seismic coefficient ($k_v$) is considered to be zero. Fig. 4.14b presents a correction for the total active pressure coefficient when the backslope is not horizontal.

Limitations of M-O method:

1) For certain values of the variables, M-O formulation fails to solve the problem. For an example, when the slope of the backslope is greater than 22° M-O formulation does not arrive at a solution.

2) Seismic coefficient used in the M-O method provides a simple approximation and cannot capture the complex deformation response of the soil-nail wall system.
4.5.3 Seismic effects on global stability

One of the earliest procedures of analysis for seismic stability is the pseudostatic procedure, in which earthquake loading is represented by a static force, equal to the soil weight multiplied by the seismic coefficient \( k \). The pseudostatic force is treated as a static force and acts in only one direction, whereas the earthquake accelerations act for only a short time and change direction, tending at certain instances in time to stabilize rather than destabilize the soil. Although the earthquake acceleration has two components, however, the vertical components are usually neglected in the pseudostatic method, and the seismic coefficient is usually represented by a horizontal force. Typical values for seismic coefficient usually range within 0.05 to 0.25.

Application of a seismic coefficient and pseudostatic force in limit equilibrium slope stability analysis is straightforward. The pseudostatic force is assumed to be known force and is included in the various equilibrium equations. Fig. 4.15 shows an infinite slope with the shear strength expressed in terms of total stresses. Resolving the forces perpendicular to slip plane and parallel to slip plane the factor of safety \( F \) for an infinite slope is calculated as given in the Eq. 4.43.
F = shear strength / shear stress

\[
F = \frac{c + (\gamma z \cos^2 \beta - k \gamma z \cos \beta \sin \beta) \tan \phi}{\gamma z \cos \beta \sin \beta + k \gamma z \cos^2 \beta}
\]  

(4.43)

where, c and \(\phi\) are the cohesion factor and internal friction angle of the soil.

Similar equations can be derived for effective stresses and for other limit equilibrium procedures, including any of the procedures of slices discussed in Chapter 3.

An issue that arises in pseudostatic analyses is the location of the pseudostatic force. Terzaghi (1950) suggested that the pseudostatic force should act through the center of gravity of each slice or the entire sliding soil mass. This assumption is slightly conservative for most of the dams or embankment structures.

4.6 Summary

This chapter elaborates the design procedure of the soil nail and the facing. Design of soil nail wall is performed based on external stability mode failure and internal stability mode failure. This chapter also illustrates how the facing failure modes should be incorporated in the design procedure. Seismic forces were also been considered in the design procedure. Specification of the nail, headed-stud, grouting materials, WWM are also mentioned. The final aim of the designing is to achieve the satisfying factor of safety from all aspects.
CHAPTER 5
EXAMPLE PROBLEMS ON NAILED SLOPE DESIGN

5.1 Introduction
Example problems on design of nailed slopes with different slope geometry and soil property are presented in this chapter. Nailed slopes are designed based the recommendations presented in chapter 3 and 4. Such design takes care of the stability against external and internal failure modes. Internal stability includes breaking of nail in tension, pull-out capacity of nail and face failure of the nailed slope; whereas, external stability comprises of stability against slope failure, sliding and bearing capacity failure. First, the slopes have been designed with lower slope face angle without applying nails. Next, the stability analysis has been performed after applying nails and slopes have been designed with higher slope face angle.

5.2 Problem-1
5.2.1 Slope stability without nail
Stability analysis of the slope illustrated in Fig. 5.1

![Fig. 5.1 Geometry of the unreinforced slope of Problem-1](image)

Fig. 5.1 Geometry of the unreinforced slope of Problem-1
Stability analysis of the unreinforced slope has been performed by Bishop’s method (Fig. 5.2). Graphical calculations are employed to get actual values and the detailed calculation is presented in Table 5.1

Radius of slip surface (R) = 43.17m
Co-ordinate of the centre of slip surface (x, y) = (0.81, 43.16)

\[
\text{Factor of safety for the given case} = F = \frac{\sum_{i=1}^{n=20} [C + N \tan \phi]}{\sum_{j=1}^{n=20} W \sin \alpha} = \frac{809.94}{447.94} = 1.808
\]

Fig. 5.2 Stability analysis of unreinforced slope of Problem-1
Table 5.1 Factor of safety calculation for unreinforced slope of Problem-1

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>Base slope of slice (α) (degree)</th>
<th>Weight (W) (kN)</th>
<th>Wsinα (kN)</th>
<th>L (m)</th>
<th>N (kN)</th>
<th>(C + N \tan \phi = cL + N \tan \phi) (kN)</th>
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<td>11.14</td>
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<td>0.51</td>
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<td>64.96</td>
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<td>0.58</td>
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</tr>
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<td>16</td>
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<td>17</td>
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<td>34.78</td>
<td>0.64</td>
<td>71.31</td>
<td>41.76</td>
</tr>
<tr>
<td>18</td>
<td>38.66</td>
<td>42.32</td>
<td>26.44</td>
<td>0.64</td>
<td>54.20</td>
<td>32.66</td>
</tr>
<tr>
<td>19</td>
<td>36.87</td>
<td>24.95</td>
<td>14.97</td>
<td>0.50</td>
<td>31.19</td>
<td>19.58</td>
</tr>
<tr>
<td>20</td>
<td>56.31</td>
<td>5.35</td>
<td>4.45</td>
<td>0.36</td>
<td>9.64</td>
<td>7.28</td>
</tr>
<tr>
<td>(\Sigma)</td>
<td></td>
<td>447.94</td>
<td></td>
<td></td>
<td>809.94</td>
<td></td>
</tr>
</tbody>
</table>
5.2.2 Stability of nailed slope

Geometry of nailed slope is presented in Fig. 5.3.

Soil properties given: $c = 6\text{kPa}$, $\varphi = 28^\circ$, $\gamma = 19.8\text{kN/m}^3$

Let, nail length ($l$) = 8m, diameter ($d$) = 25mm, inclination ($\lambda$) = $10^\circ$

Vertical spacing of the nails ($s_v$) = 1m

Horizontal spacing of the nails ($s_h$) = 1m

![Diagram of the reinforced slope](image)

Fig. 5.3 Geometry of the reinforced slope in Problem-1

5.2.2.1 Internal stability

(a) Nail tensile capacity ($R_T$)

From Eq. 3.58

$$R_T = \frac{\pi d^2 f_y}{4 FOS_{RT}} = \frac{415 \times \pi \times 25^2}{4 \times 1.8} = 113.17\text{kN}$$

Where,

$f_y$ = Yield strength of steel = 415MPa

$d$ = Diameter of the nail = 25mm
(b) Facing design
For facing design, the procedure given in section 4.4.3 is followed accordingly.

**Step-1 Determining \( T_{\text{max}} \)**

\( T \) developed in nail at depth \( z \) is given by Eq 4.17

\[ T_0 = K(q+\gamma z) S_h S_v; \quad K=(1-\sin \phi)/(1+\sin \phi)=0.36 \]

As the backfill is homogenous and the nails are spaced uniformly; therefore, \( T_{\text{max}} \) would be at the last nail from the top surface.

\[ \therefore T_{\text{max}}=0.36(19.8\times10.5)=74.844 \text{kN} \]

**Step-2 Adopting wall facing thickness**

For this problem thickness of facing is taken as:
- Temporary facing=100mm.
- Permanent facing=200mm.

**Step-3 Adopting appropriate facing materials**

i. Steel reinforcement: Grade Fe-415

ii. Concrete/Shotcrete: Grade M-20

iii. WWM (temporary facing): WMM 102x102-MW 19x19

iv. Horizontal and vertical (temporary facing): 2x10mm diameter, \( (f_y=415 \text{ MPa}, A_{vw}=A_{hw}=2x78=156\text{mm}^2) \) in both directions.

v. Bearing plate (temporary facing): Grade 250 \( (f_y =250 \text{ MPa}); \) Shape: Square; Length: \( L_{BP} = 225 \text{ mm}; \) Thickness: \( t_p = 25 \text{ mm}. \)

vi. Reinforcement bar (permanent facing): 16 mm diameter @ 300 mm both ways.

vii. Headed-studs: 4 numbers; Size: \( 1\times4\frac{1}{8}; L_s=100\text{mm}; D_h =25\text{mm}; D_S =13 \text{ mm}; t_H=8\text{mm}; S_{HS}=150\text{mm}. \)

**Step-4 Checking for facing reinforcement**

\[ \rho_{\text{min}} [%] = 20 \sqrt{\frac{f_{ck}}{f_y}} [\text{MPa}] = 20 \sqrt{\frac{20}{415}} =0.21 \]

\[ \rho_{\text{max}} [%] = 50 \frac{f_{ck}}{f_y} [\text{MPa}] \left( \frac{600}{600 + f_y [\text{MPa}]} \right) =1.42 \]
At any section of the facing, reinforcement ratio is defined as the ratio of the effective area of the reinforcement to the effective area of the concrete. The placed reinforcement should be within $\rho_{\min}$ and $\rho_{\max}$.

**Temporary facing**

Area of Reinforcement in vertical $a_{vm}$ and horizontal $a_{hm}$ directions in mid-span (Ref. to Fig.4.12):

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

for WWM 102 x 102 – MW19 x MW19 (see Table 4.4)

Area of Reinforcement in vertical $a_{vn}$ and horizontal $a_{hn}$ directions around soil nail head:

Since the same amount of reinforcement is provided in both directions

$$a_{vn} = a_{hn} = a_{vm} + \frac{A_{vn}}{S_h} = 156$$

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

Reinforcement ratio $\rho$ at nail head and mid-span in vertical direction

$$\rho_n[\%] = \frac{a_{n}}{0.5h}100 = \frac{(288.2 \times 1000)}{0.5 \times 100} \times 100 = 0.58$$

$$\rho_m[\%] = \frac{a_{m}}{0.5h}100 = \frac{(184.2 \times 1000)}{0.5 \times 100} \times 100 = 0.37$$

Both $\rho_n$ and $\rho_m$ are within the allowable limit (i.e. within 0.21 and 1.42).

**Permanent Facing**

Total area of 16 mm diameter @ 300 mm c/c is equal to 670 mm$^2$/m (Ref. Table 4.6).

This area of reinforcement is provided in both vertical and horizontal directions; therefore, $a_{vn} = a_{hn} = a_{vm} = a_{hm} = 670 \text{ mm}^2/\text{m}$ (no waler bars are provided in permanent facing).

Reinforcement ratio $\rho$ at nail head and mid-span in vertical direction

$$\rho_n[\%] = \frac{a_{n}}{0.5h}100 = \frac{(670 \times 1000)}{0.5 \times 200} \times 100 = 0.67$$

(satisfies both the criteria: $\rho$ is within $\rho_{\min}$ and $\rho_{\max}$ and $\frac{\rho_n}{\rho_m} = 1 < 2.5$).

**Step-5 Verify facing flexural resistance $R_{FF}$**

**Temporary facing**

Calculate facing flexural resistance $R_{FF}$ as:

$$R_{FF}[kN] = \frac{C_F}{265} \times (a_{vm} + a_{vn}) \times \left( \left[ \frac{S_h}{S_v} b[m] \right] \times f_y[MPa] \right)$$

For temporary facing with thickness $h = 100 \text{ mm} (= 0.1 \text{ m})$, adopt $C_F = 2.0$
\( (a_m + a_m) = 288.2 + 184.2 = 472.4 \text{mm}^2 / \text{m}; \text{ nail spacing ratio: } S_h/S_v = 1.0. \)

Therefore, \( R_{FF} \text{ [kN]} = \frac{2}{265} \times 472.4 \times (1 \times 0.1) \times 415 = 148 \)

Factor of safety against flexural facing failure \( FS_{FF} : \)

\[
FS_{FF} = \frac{R_{FF}}{T_0} = \frac{148}{63} = 2.35 (> 1.50 \text{ safe})
\]

**Permanent facing**

For permanent facing with thickness \( h = 200 \text{ mm} (= 0.2 \text{ m}) \), adopt \( C_F = 1. \)

\( (a_m + a_m) = 670 + 670 = 1340 \text{mm}^2 / \text{m}; \text{ nail spacing ratio: } S_h/S_v = 1.0 \)

Therefore, \( R_{FF} \text{ [kN]} = \frac{1}{265} \times 1340 \times (1 \times 0.2) \times 415 = 420 \)

Factor of safety against flexural facing failure \( FS_{FF} : \)

\[
FS_{FF} = \frac{R_{FF}}{T_0} = \frac{420}{63} = 6.67 (> 1.50 \text{ safe})
\]

**Step-6 Verify facing punching shear resistance \( R_{FP} \)**

**Temporary facing:**

Check for bearing-plate connection.

Facing punching shear capacity \( R_{FP} \) is given by:

\[ R_{FP} [\text{kN}] = 330 \times \sqrt{20} \times \pi \times 0.234 \times 0.117 \]

\[ = 330 \times \sqrt{20} \times \pi \times 0.325 \times 0.1 = 150 \text{ [where, } D_c = L_{BP} + h = 225 + 100 = 325 \text{ mm} = 0.325 \text{m}] \]

Factor of safety against punching shear failure \( FS_{FP} \) is given by:

\[
FS_{FP} = \frac{R_{FP}}{T_0} = \frac{220.23}{63} = 3.49 (> 1.5 \text{ (safe)})
\]

**Permanent Facing:**

Check for headed-stud connection.
Here: \( f_{ck} = 20 \text{ MPa}; L_S = 100 \text{ mm}; D_H = 25 \text{ mm}; D_S = 13 \text{ mm}; t_H = 8 \text{ mm}; S_{HS} = 150 \text{ mm}; t_p = 25 \text{ mm}; \)

\( h_c = L_S - t_H + t_p = 100 - 8 + 25 = 117 \text{ mm} = 0.117 \text{ m}; \)

\( 2h_c = 0.234 \text{ m}; \)

\( D'_c = \text{minimum of } (S_{HS} + h_c \text{ and } 2h_c) \)

\( S_{HS} + h_c = 150 + 117 = 267 \text{ mm} = 0.267 \text{ m} \)

Therefore, \( D'_c = 0.234 \text{ m} \)

Substituting values of various parameters, permanent facing punching shear capacity \( R_{FP} \) is calculated as:

\[
R_{FP}[kN] = 330\sqrt{f_{ck} [MPa] \pi D'_c [m] h_c [m]} = 330 \times \sqrt{20 \times \pi \times 0.234 \times 0.117} = 127
\]

Factor of safety against punching shear failure \( FS_{FP} \) is given by:

\[
FS_{FP} = \frac{R_{FP}}{T_0} = \frac{127}{63} = 2.01 (>1.50 \text{ and hence, safe})
\]

**Step 7 Verify headed-stud resistances \( R_{HT} \) (only for permanent facing)**

**Tensile capacity of the headed-studs**

For,

\[
N_H = 4
\]

\[
A_{SH} = \frac{\pi D_H^2}{4} = \frac{\pi \times 13^2}{4} = 132.73 \text{ mm}^2
\]

\[
f_y = 0.415 \text{kN} / \text{mm}^2
\]

\[
R_{HT} = N_H A_{SH} f_y = 220.33 \text{kN}
\]

Factor of Safety against headed-stud tensile failure \( FS_{HT} \) is given by:

\[
FS_{HT} = \frac{R_{HT}}{T_0} = \frac{220.23}{63} = 3.49 (>1.50 \text{ and hence, safe})
\]

**Check for tolerable limits of compression on the concrete behind headed-stud**

\[
A_{SH} = \frac{\pi D_H^2}{4} = \frac{\pi \times 13^2}{4} = 132.73 \text{ mm}^2;
\]

\[
A_H = \frac{\pi D_H^2}{4} = \frac{\pi \times 25^2}{4} = 490.87 \text{ mm}^2;
\]

To assure that the compression on the concrete behind headed-stud is within tolerable limits, following two conditions shall be satisfied:
i) \[ A_H \geq 2.5A_{SH} \Rightarrow 490.87 \geq 2.5(132.73) = 331.82 \text{mm}^2 \text{ ...... Hence, ok.} \]

ii) \[ t_h \geq 0.5(D_a - D_e) \Rightarrow 8 \geq 0.5(25 - 13) = 8\text{mm} \geq 6\text{mm} \text{ ........... Hence, ok.} \]

Table 5.2 Summary of facing design (temporary and permanent)

<table>
<thead>
<tr>
<th>Element</th>
<th>Description</th>
<th>Temporary Facing</th>
<th>Permanent Facing</th>
</tr>
</thead>
<tbody>
<tr>
<td>General</td>
<td>Thickness (h)</td>
<td>100</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>Facing Type</td>
<td>Shotcrete</td>
<td>CIP concrete</td>
</tr>
<tr>
<td></td>
<td>Concrete grade</td>
<td>M20</td>
<td>M20</td>
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<tr>
<td>Reinforcement</td>
<td>Type</td>
<td>Welded Wire Mesh WWM)</td>
<td>Steel Bars</td>
</tr>
<tr>
<td></td>
<td>Steel grade</td>
<td>Fe415</td>
<td>Fe415</td>
</tr>
<tr>
<td></td>
<td>Denomination</td>
<td>102x102—MW19xMW19</td>
<td>16(\phi@300) b/w</td>
</tr>
<tr>
<td>Other reinforcement</td>
<td>Type</td>
<td>Waler bars 2-10(\phi) b/w</td>
<td>------</td>
</tr>
<tr>
<td>Bearing plate</td>
<td>Type</td>
<td>Square</td>
<td>4H-Studs1/2x33/8</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
<td>Fe250</td>
<td>------</td>
</tr>
<tr>
<td></td>
<td>Dimensions</td>
<td>225x225x25</td>
<td>------</td>
</tr>
<tr>
<td>Headed Studs</td>
<td>Dimensions</td>
<td>------</td>
<td>Nominal length, (L_s=100)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>------</td>
<td>Head Diameter, (D_H=25)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>------</td>
<td>Shaft Diameter, (D_S=13)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>------</td>
<td>Head Thickness, (t_H=08)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>------</td>
<td>Spacing, (S_{HS}=150)</td>
</tr>
</tbody>
</table>

All dimensions are in mm
(c) Nail pull out resistance for the full nail length \( (T_{n,\text{max}}) \)

\[
T_{n,\text{max}} = T_p \times l
\]

Where, 
\[
T_p = \frac{q_u \pi d}{1.5} = \frac{(c + \sigma_v \tan \psi) \pi d}{1.5}
\]

As mentioned in chapter 3, the overburden pressure is calculated at midpoint of the nails. For nails below berms, two \( T_p \) values are calculated considering two overburden pressure. The first one \( (T_{p1}) \) is at the end of the berm length and the second one \( (T_{p2}) \) at midpoint of length beyond berm. Average value of these two can be considered as the representative \( T_p \) value for the calculation of pullout resistance over the full nail length \( (T_{n,\text{max}}) \). In the present design, overburden pressure for \( T_{p2} \) is calculated at 3m berm length instead of 4m as a conservative analysis.

For the first nail from top (Fig. 5.3),

\[
\sigma_v = 19.8 \times (1.5 + \frac{8}{2} \sin 10^\circ) = 19.8 \times 2.195 = 43.45 \text{kN/m}^2
\]

\[
T_p = \frac{[6 + 43.45 \times \tan(2 \times 28 / 3)] \times 3.14 \times 0.025}{1.5} = 1.083 \text{kN/m}
\]

\[
T_{n,\text{max}} = 1.084 \times 8 = 8.66 \text{kN}
\]

For the first nail below berm (Fig. 5.3),

\[
\sigma_{v1} = 19.8 \times [1.5 + (1.5 + 3) \times \tan 10^\circ] = 19.8 \times 2.293 = 45.411 \text{kN/m}^2
\]

\[
T_{p1} = \frac{[6 + 45.411 \times \tan(2 \times 28 / 3)] \times 3.14 \times 0.025}{1.5} = 1.117 \text{kN/m}
\]

Length of nail beyond berm = \( 8 - (4+1.5)/\cos 10^\circ = 2.415 \text{m} \)

\[
\sigma_{v2} = 19.8 \times [7.2 + (8 - \frac{2.415}{2}) \times \sin 10^\circ] = 19.8 \times 8.38 = 165.92 \text{kN/m}^2
\]

\[
T_{p2} = \frac{[6 + 165.92 \times \tan(2 \times 28 / 3)] \times 3.14 \times 0.025}{1.5} = 3.25 \text{kN/m}
\]

After averaging \( T_{p1} \) and \( T_{p2} \)

\[
T_p = (T_{p1} + T_{p2}) / 2 = (1.117 + 3.249) / 2 = 2.183
\]

\[
T_{n,\text{max}} = 2.183 \times 8 = 17.465 \text{kN}
\]

Similarly \( T_p \) and \( T_{n,\text{max}} \) are calculated for other nails and presented in Table 5.3.
Table 5.3 $T_p$ and $T_{n,\text{max}}$ values for different nails in Problem-1

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{p1}$ (kN/m)</td>
<td></td>
<td></td>
<td></td>
<td>1.12</td>
<td>1.54</td>
<td>1.96</td>
<td>2.38</td>
</tr>
<tr>
<td>$T_{p2}$ (kN/m)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3.26</td>
<td>3.67</td>
<td>4.09</td>
<td>4.51</td>
</tr>
<tr>
<td>$T_p$ (kN/m)</td>
<td>1.80</td>
<td>1.43</td>
<td>1.78</td>
<td>2.18</td>
<td>2.60</td>
<td>3.02</td>
<td>3.44</td>
</tr>
<tr>
<td>$T_{n,\text{max}}$ (kN)</td>
<td>8.66</td>
<td>11.46</td>
<td>14.27</td>
<td>17.47</td>
<td>20.83</td>
<td>24.19</td>
<td>27.55</td>
</tr>
</tbody>
</table>

As for all nails $T_{n,\text{max}} < R_f$ and $T_{n,\text{max}} < R_f$,

i) Nail length and diameters are adequately selected

ii) Nail tensile forces ($T_n$) can be calculated from the nail length beyond the failure surface ($l_e$) using Eq. 3.55. For nails below berms, nail tensile forces are calculated from the following relation

$$T_n = T_{p1} \times l_{e1} + T_{p2} \times l_{e2}$$

Where, $l_{e1}$ = Part of $l_e$ beyond berm region, and $l_{e2}$ = Part of $l_e$ within berm region

5.2.2.2 External stability

(a) Slope stability

Slope stability of the nailed structure is checked using Bishop’s Method. Graphical calculations are employed to get actual values.
Case-1: Slope stability for upper part of nailed structure

The graphical representation of the stability analysis of upper slope is given in Fig. 5.5. Nail tensile force and factor of safety calculation details are given in Table 5.4 and Table 5.5 respectively.

Radius of slip surface (R) = 10.30m
Co-ordinate of the centre of slip surface (x, y) = (8.03, 16.12)

Factor of safety for the given case = \[ F = \frac{\sum_{i=1}^{15} [C + N \tan \phi]}{\sum_{i=1}^{15} [W \sin \alpha - T_n \cos (\alpha + \lambda)]} = \frac{219.18}{123.27} = 1.805 \]
Table 5.4 Calculation of nail tension for case-1 in Problem-1

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_p$ ($kN/m$)</td>
<td>1.80</td>
<td>1.43</td>
<td>1.78</td>
</tr>
<tr>
<td>$l_e$ ($m$)</td>
<td>4.20</td>
<td>4.00</td>
<td>4.00</td>
</tr>
<tr>
<td>$T_n$ ($kN$)</td>
<td>4.55</td>
<td>5.73</td>
<td>7.85</td>
</tr>
</tbody>
</table>

Table 5.5 FOS calculation for case-1 in Problem-1

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>$\alpha$ (degree)</th>
<th>W ($kN$)</th>
<th>$T_n$ ($kN$)</th>
<th>$N$ ($kN$)</th>
<th>L ($m$)</th>
<th>$C + N \tan \phi = cl + N \tan \phi$ ($kN$)</th>
<th>$W \sin \alpha - \frac{C + N \tan \phi}{T_n} \cos(\alpha + \lambda)$ ($kN$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7.13</td>
<td>1.38</td>
<td>0</td>
<td>1.40</td>
<td>0.40</td>
<td>3.16</td>
<td>0.17</td>
</tr>
<tr>
<td>2</td>
<td>16.70</td>
<td>5.20</td>
<td>0</td>
<td>5.42</td>
<td>0.52</td>
<td>6.02</td>
<td>1.49</td>
</tr>
<tr>
<td>3</td>
<td>21.80</td>
<td>8.41</td>
<td>0</td>
<td>9.06</td>
<td>0.54</td>
<td>8.05</td>
<td>3.12</td>
</tr>
<tr>
<td>4</td>
<td>21.80</td>
<td>11.39</td>
<td>0</td>
<td>12.26</td>
<td>0.54</td>
<td>9.75</td>
<td>4.23</td>
</tr>
<tr>
<td>5</td>
<td>21.80</td>
<td>14.35</td>
<td>0</td>
<td>15.46</td>
<td>0.54</td>
<td>11.45</td>
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Case-2: Slope stability for lower part of nailed structure

The graphical representation of the stability analysis of lower slope is given in Fig. 5.6. Nail tensile force and factor of safety calculation details are given in Table 5.6 and Table 5.7 respectively.

Radius of slip surface (R) = 9.29m
Co-ordinate of the centre of slip surface (x, y) = (-1.61, 9.51)

Factor of safety for the given case = \[ F = \frac{\sum_{i=1}^{13} [C + N \tan \phi]}{\sum_{i=1}^{13} [W \sin \alpha - T_i \cos (\alpha + \lambda)]} = \frac{163.97}{61.21} = 2.679 \]

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<td>W (kN)</td>
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<tr>
<td>( \Sigma )</td>
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Case-3: Global slope stability of nailed structure

The graphical representation of the global stability analysis of the nailed slope is given in Fig. 5.7. Nail tensile force and factor of safety calculation details are given in Table 5.8 and Table 5.9 respectively.

Radius of slip surface \((R) = 32.60\text{m}\)

Co-ordinate of the centre of slip surface \((x, y) = (-6.31, 32)\)

Factor of safety for the given case = 

\[
F = \frac{\sum_{i=1}^{n=13} [C + N \tan \phi]}{\sum_{i=1}^{n=11} [W \sin \alpha - T_n \cos (\alpha + \lambda)]} = \frac{764.81}{470.13} = 1.626
\]

Hence, the minimum factor of safety for the designed nailed slope against slope stability in given three cases is 1.626.
Table 5.8 Calculation of nail tension for case-3 in Problem-1

<table>
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<th>5</th>
<th>6</th>
<th>7</th>
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<td>-</td>
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<td>-</td>
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<td>-</td>
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Table 5.9 FOS calculation for case-3 in Problem-1

<table>
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<th>$W$ ($kN$)</th>
<th>$T_p$ ($kN/m$)</th>
<th>$N$ ($kN$)</th>
<th>$L$ ($m$)</th>
<th>$C + N \tan \phi = c/l + N \tan \phi$ ($kN/m$)</th>
<th>$W \sin \alpha - T_n \cos(\alpha + \lambda)$ ($kN$)</th>
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</table>
(b) Stability against sliding

Fig. 5.8 Stability analysis of nailed slope in Problem-1 against sliding

Fig. 5.8 presents the reinforced soil mass considered for sliding stability analysis along with the sliding surface $QR$.

$L_x = l \times \cos \lambda - t \times \tan \alpha$

$l$ = Length of nail = 8m

$t$ = Depth of first nail = 1.5m

$\alpha$ = Angle of slope with horizontal = 45°

$\lambda$ = Nail inclination with horizontal = 10°

$L_x = 8 \cos 10^\circ - 1.5 \tan 45^\circ = 6.38m$

Weight of sliding wedge ($W$) = $\gamma \times$ area of wedge $MNOPQR$

$W = 19.8 \times 172.56 = 3416.69kN$

$B_L$ = width of failing wedge = $(6 + 4 + 6 + 6.38)m = 22.38m$

Total resisting force ($\sum R$) = $W \times \tan \phi + c \times B_L = 1854.76kN$

Total destabilising force ($\sum D$) = $P_d = \frac{\gamma H^2}{2} \times K_d = 514.64kN$
Where,

\[ H = \text{Height of wedge} = 12\text{m} \]

\[ K_d = \tan^2\left(45 - \frac{\phi}{2}\right) = 0.361 \]

\[ \phi = 28^\circ, \ c = 6\text{kPa} \text{ and } \gamma = 19.8\text{kN/m}^3 \]

Factor of safety against sliding (FOS) = \(\frac{\sum R}{\sum D}\) = 3.603 > 3.0

Hence, the slope is stable against sliding failure.

(c) Check for bearing capacity

The factor of safety against bearing capacity failure (\(FS_H\)) is calculated from Eq. 3.45

\[ FS_H = \frac{cN_c + 0.5\gamma B_e N_\gamma}{H_{eq}\gamma} \]

For the given case,

\[ \phi = 28^\circ, \ c = 6\text{kPa} \text{ and } \gamma = 19.8\text{kN/m}^3 \]

\[ H_{eq} = H = \text{Height of the wall} = 12\text{m} \]

\[ B_e = \text{Width of the excavation} = H \text{ (assumed)} = 12\text{m} \]

\[ N_c, N_\gamma = \text{Bearing capacity factor (mostly dependent on } \phi) \]

From standard chart for \(\phi = 28^\circ\)

\[ N_c = 31.61 \text{ and } N_\gamma = 13.70 \]

\[ FS_H = \frac{6 \times 31.61 + 0.5 \times 19.8 \times 12 \times 13.70}{19.8 \times 12} = 7.64 > 3.0 \]

Hence, the slope is stable against bearing capacity failure.
5.3 Problem-2

5.3.1 Slope stability without nail

Stability analysis of the slope illustrated in Fig. 5.9

Stability analysis of the unreinforced slope has been performed by Bishop’s method (Fig. 5.10). The actual figure was drawn on the graph paper to estimate the values and the detailed calculation is presented in Table 5.10

Radius of slip surface (R) = 43.52m
Co-ordinate of the centre of slip surface (x, y) = (-0.47, 43.51)

Factor of safety for the given case = 

$$F = \frac{\sum_{i=1}^{n-1} [C + N \tan \phi]}{\sum_{i=1}^{n-1} W \sin \alpha} = \frac{2391.53}{1522.57} = 1.57$$

Fig. 5.9 Geometry of the unreinforced slope of Problem-2
Fig. 5.10 Stability analysis of unreinforced slope of Problem-2

- $c = 0.05$ kPa
- $\gamma = 22$ kN/m$^3$
- $\phi = 32^\circ$

Slices for analysis
Table 5.10 FOS calculation for unreinforced slope of Problem-2

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<th>Wsinα (kN)</th>
<th>L (m)</th>
<th>N (kN)</th>
<th>( C + N \tan \phi = cL + N \tan \phi ) (kN)</th>
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5.3.2 Stability of nailed slope

Geometry of nailed slope is presented in Fig. 5.11.

Soil properties given: \( c = 0.05 \text{kPa}, \phi = 32^\circ, \gamma = 22 \text{kN/m}^3 \)

Let, nail length \((l) = 8 \text{m}\), diameter \((d) = 20 \text{mm}\), inclination \((\lambda) = 10^\circ\)

Vertical spacing of the nails \((s_v) = 1 \text{m}\)

Horizontal spacing of the nails \((s_h) = 1 \text{m}\)

![Fig. 5.11 Geometry of the reinforced slope in Problem-2](image)

5.3.2.1 Internal stability

(a) Nail tensile capacity \((R_T)\)

From Eq. 3.58

\[
R_T = \frac{\pi d^2 f_y}{4 FOS_{RT}} = \frac{415 \times \pi \times 20^2}{4 \times 1.8} = 72.394 \text{kN}
\]

Where,

\( f_y = \) Yield strength of steel = 415MPa

\( d = \) Diameter of the nail= 20mm

(b) Facing design

Facing details are same as the previous problem.
(c) Nail pull out resistance for the full nail length ($T_{n,\text{max}}$)

Nail pull out resistance for the full length is calculated similar to the previous problem and presented in Table 5.11.

Table 5.11 $T_p$ and $T_{n,\text{max}}$ values for different nails in Problem-2

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{p1}$ ($kN/m$)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.57</td>
<td>0.93</td>
<td>1.29</td>
<td>1.65</td>
<td>2.01</td>
</tr>
<tr>
<td>$T_{p2}$ ($kN/m$)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2.54</td>
<td>2.90</td>
<td>3.26</td>
<td>3.62</td>
<td>3.98</td>
</tr>
<tr>
<td>$T_p$ ($kN/m$)</td>
<td>0.60</td>
<td>0.96</td>
<td>1.33</td>
<td>1.68</td>
<td>2.04</td>
<td>1.55</td>
<td>1.91</td>
<td>2.27</td>
<td>2.63</td>
<td>2.99</td>
</tr>
<tr>
<td>$T_{n,\text{max}}$ ($kN$)</td>
<td>4.78</td>
<td>7.66</td>
<td>10.54</td>
<td>13.42</td>
<td>16.30</td>
<td>12.44</td>
<td>15.32</td>
<td>18.20</td>
<td>21.08</td>
<td>23.96</td>
</tr>
</tbody>
</table>

As for all nails $T_{n,\text{max}} < R_f$ and $T_{n,\text{max}} < R_f$, nail length and diameters are adequately selected.

5.3.2.2 External stability

(a) Slope stability

Slope stability of the nailed structure is checked using Bishop’s Method and the values are estimated from the figures drawn on the graph paper.
Case-1: Slope stability for upper part of nailed structure

Fig. 5.12 Stability analysis of nailed slope for case-1 in Problem-2

The graphical representation of the stability analysis of upper slope is given in Fig. 5.12. Nail tensile force and factor of safety calculation details are given in Table 5.12 and Table 5.13 respectively.

Radius of slip surface (R) = 9.05m
Co-ordinate of the centre of slip surface (x, y) = (7.52, 14.72)

Factor of safety for the given case = 

\[
F = \frac{\sum_{i=1}^{n=12} [C + N \tan \phi]}{\sum_{i=1}^{n=12} [W \sin \alpha - T_i \cos (\alpha + \lambda)]} = \frac{125.99}{61.2} = 2.059
\]

Table 5.12 Calculation of nail tension for case-1 in Problem-2

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_P ) (kN/m)</td>
<td>0.60</td>
<td>0.96</td>
<td>1.33</td>
<td>1.68</td>
<td>2.04</td>
</tr>
<tr>
<td>( l_e ) (m)</td>
<td>7</td>
<td>6.70</td>
<td>6.60</td>
<td>6.80</td>
<td>7.40</td>
</tr>
<tr>
<td>( T_n ) (kN)</td>
<td>4.18</td>
<td>6.42</td>
<td>8.69</td>
<td>11.41</td>
<td>15.08</td>
</tr>
</tbody>
</table>
Table 5.13 FOS calculation for case-1 in Problem-2

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>( \alpha ) (degree)</th>
<th>( W ) (kN)</th>
<th>( T_n ) (kN)</th>
<th>( N ) (kN)</th>
<th>( L ) (m)</th>
<th>( C + N \tan \phi = cl + N \tan \phi ) (kN)</th>
<th>( W \sin \alpha - T_n \cos(\alpha + \lambda) ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11.31</td>
<td>2.20</td>
<td>0</td>
<td>2.24</td>
<td>0.51</td>
<td>1.43</td>
<td>0.43</td>
</tr>
<tr>
<td>2</td>
<td>21.80</td>
<td>6.05</td>
<td>0</td>
<td>6.52</td>
<td>0.54</td>
<td>4.10</td>
<td>2.25</td>
</tr>
<tr>
<td>3</td>
<td>21.80</td>
<td>9.35</td>
<td>15.07</td>
<td>10.07</td>
<td>0.54</td>
<td>6.32</td>
<td>-9.34</td>
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<tr>
<td>4</td>
<td>30.96</td>
<td>12.10</td>
<td>0</td>
<td>14.11</td>
<td>0.58</td>
<td>8.85</td>
<td>6.23</td>
</tr>
<tr>
<td>5</td>
<td>30.96</td>
<td>14.30</td>
<td>0</td>
<td>16.68</td>
<td>0.58</td>
<td>10.45</td>
<td>7.36</td>
</tr>
<tr>
<td>6</td>
<td>34.99</td>
<td>16.23</td>
<td>11.41</td>
<td>19.81</td>
<td>0.61</td>
<td>12.41</td>
<td>1.24</td>
</tr>
<tr>
<td>7</td>
<td>38.66</td>
<td>17.60</td>
<td>0</td>
<td>22.54</td>
<td>0.64</td>
<td>14.12</td>
<td>10.99</td>
</tr>
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<td>8</td>
<td>41.99</td>
<td>18.43</td>
<td>8.69</td>
<td>24.79</td>
<td>0.67</td>
<td>15.52</td>
<td>6.97</td>
</tr>
<tr>
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<td>47.73</td>
<td>18.43</td>
<td>0</td>
<td>27.39</td>
<td>0.74</td>
<td>17.15</td>
<td>13.63</td>
</tr>
<tr>
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<td>52.43</td>
<td>17.33</td>
<td>6.41</td>
<td>28.42</td>
<td>0.82</td>
<td>17.80</td>
<td>10.76</td>
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<td>59.53</td>
<td>11.83</td>
<td>4.18</td>
<td>23.32</td>
<td>0.99</td>
<td>14.62</td>
<td>8.73</td>
</tr>
<tr>
<td>12</td>
<td>65.22</td>
<td>2.15</td>
<td>0</td>
<td>5.12</td>
<td>0.72</td>
<td>3.23</td>
<td>1.95</td>
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<tr>
<td>( \Sigma )</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>125.99</td>
<td>61.20</td>
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</table>
Case-2: Slope stability for lower part of nailed structure

The graphical representation of the stability analysis of lower slope is given in Fig. 5.13. Nail tensile force and factor of safety calculation details are given in Table 5.14 and Table 5.15 respectively.

Radius of slip surface (R) = 11.56m

Co-ordinate of the centre of slip surface (x, y) = (-3.37, 11.06)

Factor of safety for the given case = \[ F = \frac{\sum_{i=1}^{n=16} [C + N \tan \phi]}{\sum_{i=1}^{n=16} [W \sin \alpha - T_i \cos(\alpha + \lambda)]} = \frac{667.93}{466.41} = 1.432 \]

Table 5.14 Calculation of nail tension for case-2 in Problem-2

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_{p1} \text{ (kN/m)} )</td>
<td>0.57</td>
<td>0.93</td>
<td>1.29</td>
<td>1.65</td>
<td>2.01</td>
</tr>
<tr>
<td>( l_{i1} \text{ (m)} )</td>
<td>2.80</td>
<td>3.60</td>
<td>3.90</td>
<td>4.00</td>
<td>4.40</td>
</tr>
<tr>
<td>( T_{p2} \text{ (kN/m)} )</td>
<td>2.54</td>
<td>2.90</td>
<td>3.26</td>
<td>3.62</td>
<td>3.98</td>
</tr>
<tr>
<td>( l_{i2} \text{ (m)} )</td>
<td>1.50</td>
<td>0.40</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>( T_{n} \text{ (kN)} )</td>
<td>5.41</td>
<td>4.51</td>
<td>5.03</td>
<td>6.60</td>
<td>8.84</td>
</tr>
</tbody>
</table>
Table 5.15 FOS calculation for case-2 in Problem-2

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>$\alpha$ (degree)</th>
<th>$W$ (kN)</th>
<th>$T_n$ (kN/m)</th>
<th>$N$ (kN/m)</th>
<th>$L$ (m)</th>
<th>$C + N \tan \phi = cl + N \tan \phi$ (kN)</th>
<th>$W \sin \alpha - T_n \cos (\alpha + \lambda)$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11.31</td>
<td>2.20</td>
<td>0.00</td>
<td>2.24</td>
<td>0.51</td>
<td>1.43</td>
<td>0.43</td>
</tr>
<tr>
<td>2</td>
<td>16.70</td>
<td>6.33</td>
<td>0.00</td>
<td>6.60</td>
<td>0.52</td>
<td>4.15</td>
<td>1.82</td>
</tr>
<tr>
<td>3</td>
<td>16.70</td>
<td>10.18</td>
<td>0.00</td>
<td>10.62</td>
<td>0.52</td>
<td>6.66</td>
<td>2.92</td>
</tr>
<tr>
<td>4</td>
<td>21.80</td>
<td>13.75</td>
<td>0.00</td>
<td>14.81</td>
<td>0.54</td>
<td>9.28</td>
<td>5.10</td>
</tr>
<tr>
<td>5</td>
<td>26.57</td>
<td>16.78</td>
<td>0.00</td>
<td>18.76</td>
<td>0.56</td>
<td>11.75</td>
<td>7.50</td>
</tr>
<tr>
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<td>26.57</td>
<td>19.53</td>
<td>0.00</td>
<td>21.83</td>
<td>0.56</td>
<td>13.67</td>
<td>8.73</td>
</tr>
<tr>
<td>7</td>
<td>30.96</td>
<td>22.00</td>
<td>8.84</td>
<td>25.66</td>
<td>0.58</td>
<td>16.06</td>
<td>4.64</td>
</tr>
<tr>
<td>8</td>
<td>30.96</td>
<td>24.20</td>
<td>0.00</td>
<td>28.22</td>
<td>0.58</td>
<td>17.66</td>
<td>12.45</td>
</tr>
<tr>
<td>9</td>
<td>38.66</td>
<td>25.85</td>
<td>0.00</td>
<td>33.10</td>
<td>0.64</td>
<td>20.72</td>
<td>16.15</td>
</tr>
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<td>10</td>
<td>38.66</td>
<td>26.95</td>
<td>6.60</td>
<td>34.51</td>
<td>0.64</td>
<td>21.60</td>
<td>12.48</td>
</tr>
<tr>
<td>11</td>
<td>45.00</td>
<td>27.50</td>
<td>0.00</td>
<td>38.89</td>
<td>0.71</td>
<td>24.34</td>
<td>19.45</td>
</tr>
<tr>
<td>12</td>
<td>45.00</td>
<td>27.50</td>
<td>5.03</td>
<td>38.89</td>
<td>0.71</td>
<td>24.34</td>
<td>16.56</td>
</tr>
<tr>
<td>13</td>
<td>45.00</td>
<td>99.00</td>
<td>4.51</td>
<td>140.01</td>
<td>1.41</td>
<td>87.52</td>
<td>67.42</td>
</tr>
<tr>
<td>14</td>
<td>54.46</td>
<td>163.35</td>
<td>0.00</td>
<td>281.04</td>
<td>2.58</td>
<td>175.66</td>
<td>132.92</td>
</tr>
<tr>
<td>15</td>
<td>57.99</td>
<td>158.40</td>
<td>5.40</td>
<td>298.87</td>
<td>3.77</td>
<td>186.80</td>
<td>132.30</td>
</tr>
<tr>
<td>16</td>
<td>68.20</td>
<td>27.50</td>
<td>0.00</td>
<td>74.05</td>
<td>2.69</td>
<td>46.30</td>
<td>25.53</td>
</tr>
<tr>
<td>$\Sigma$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>667.93</td>
<td>466.41</td>
</tr>
</tbody>
</table>
Case-3: Global slope stability of nailed structure

The graphical representation of the global stability analysis of the nailed slope is given Fig. 5.14. Nail tensile force and factor of safety calculation details are given in Table 5.16 and Table 5.17 respectively.

Radius of slip surface \((R) = 36.32\)m

Co-ordinate of the centre of slip surface \((x, y) = (-10.02, 34.92)\)

Factor of safety for the given case

\[
F = \frac{\sum_{i=1}^{n=12} [C + N \tan \phi]}{\sum_{i=1}^{n=13} [W \sin \alpha - T_s \cos(\alpha + \lambda)]} = \frac{548.06}{306.94} = 1.786
\]
Table 5.16 Calculation of nail tension for case-3 in Problem-2

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_p (kN/m)$</td>
<td>0.60</td>
<td>0.96</td>
<td>1.33</td>
<td>1.68</td>
<td>2.04</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$l_e (m)$</td>
<td>6.13</td>
<td>6.13</td>
<td>6.13</td>
<td>6.40</td>
<td>6.50</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$T_{p1} (kN/m)$</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.57</td>
<td>0.93</td>
<td>1.29</td>
<td>1.65</td>
<td>2.01</td>
</tr>
<tr>
<td>$l_{e1} (m)$</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.00</td>
<td>1.80</td>
<td>3.60</td>
<td>5.10</td>
<td>6.50</td>
</tr>
<tr>
<td>$T_{p2} (kN/m)$</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2.54</td>
<td>2.90</td>
<td>3.26</td>
<td>3.62</td>
<td>3.98</td>
</tr>
<tr>
<td>$l_{e2} (m)$</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3.50</td>
<td>2.30</td>
<td>1.10</td>
<td>0.20</td>
<td>0.00</td>
</tr>
<tr>
<td>$T_n (kN)$</td>
<td>3.66</td>
<td>5.87</td>
<td>8.08</td>
<td>10.73</td>
<td>13.24</td>
<td>8.89</td>
<td>8.34</td>
<td>8.23</td>
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<td>13.06</td>
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</table>

Table 5.17 FOS calculation for case-3 in Problem-2

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>$\alpha$ (degree)</th>
<th>W (kN)</th>
<th>$T_n$ (kN/m)</th>
<th>$N$ (kN)</th>
<th>L (m)</th>
<th>$C + N \tan \phi = cl + N \tan \phi$ (kN/m)</th>
<th>$W \sin \alpha - T_n \cos (\alpha + \lambda)$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>21.80</td>
<td>14.85</td>
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<td>1.62</td>
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</tr>
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<td>16.70</td>
<td>47.03</td>
<td>0</td>
<td>49.10</td>
<td>1.57</td>
<td>30.70</td>
<td>13.51</td>
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<tr>
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<td>53.33</td>
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<td>118.99</td>
<td>1.68</td>
<td>74.38</td>
<td>40.26</td>
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<td>8.23</td>
<td>118.99</td>
<td>1.68</td>
<td>74.38</td>
<td>40.99</td>
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<tr>
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<td>81.68</td>
<td>8.34</td>
<td>91.32</td>
<td>1.68</td>
<td>57.088</td>
<td>29.83</td>
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<td>32.01</td>
<td>45.54</td>
<td>8.89</td>
<td>53.70</td>
<td>1.42</td>
<td>33.58</td>
<td>17.53</td>
</tr>
<tr>
<td>8</td>
<td>45</td>
<td>8.91</td>
<td>13.24</td>
<td>12.60</td>
<td>0.42</td>
<td>7.88</td>
<td>-1.29</td>
</tr>
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<td>9</td>
<td>34.99</td>
<td>51.98</td>
<td>10.73</td>
<td>63.44</td>
<td>1.83</td>
<td>39.67</td>
<td>22.21</td>
</tr>
<tr>
<td>10</td>
<td>38.66</td>
<td>64.35</td>
<td>8.08</td>
<td>82.41</td>
<td>1.92</td>
<td>51.53</td>
<td>34.87</td>
</tr>
<tr>
<td>11</td>
<td>38.66</td>
<td>74.25</td>
<td>5.87</td>
<td>95.09</td>
<td>1.92</td>
<td>59.45</td>
<td>42.51</td>
</tr>
<tr>
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<td>45</td>
<td>54.45</td>
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<td>77.00</td>
<td>2.12</td>
<td>48.15</td>
<td>36.40</td>
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<td>12.60</td>
<td>1.27</td>
<td>7.89</td>
<td>6.30</td>
</tr>
<tr>
<td>$\Sigma$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>548.06</td>
<td>306.94</td>
</tr>
</tbody>
</table>

Hence, the minimum factor of safety for the designed nailed slope against slope stability in given three cases is $1.432$.
(b) Stability against sliding

Fig. 5.15 Stability analysis of nailed slope in Problem-2 against sliding

Fig. 5.15 presents the reinforced soil mass considered for sliding stability analysis along with the siding surface $QR$.

$L_x = l \times \cos \lambda - t \times \tan \alpha$

$l =$ Length of nail = 8m

$t =$ Depth of first nail = 0.5m

$\alpha =$ Angle of slope with horizontal = 45$^0$

$\lambda =$ Nail inclination with horizontal = 10$^0$

$L_x = 8 \cos 10^0 - 0.5 \tan 45^0 = 7.38m$

Weight of sliding wedge (W) = $\gamma \times$ area of wedge

$W = 22 \times 165.66 = 3644.59kN$

$B_L =$ width of failing wedge = $(6 + 4 + 5 + 7.38)m = 22.38m$

Total resisting force ($\sum R$) = $W \times \tan \phi + c \times B_L = 2277.76kN$
Total destabilising force \((\sum D) = P_a = \frac{\gamma H^2}{2} \times K_a = 408.96 \text{kN}\)

Where,

\[ H = \text{Height of wedge} = 11 \text{m} \]

\[ K_a = \tan^2 \left(45^\circ - \frac{\phi}{2}\right) = 0.307 \]

\[ \phi = 32^\circ, \; c = 0.05 \text{kPa} \; \text{and} \; \gamma = 22 \text{kN/m}^3 \]

Factor of safety against sliding (FOS) = \(\frac{\sum R}{\sum D} = 5.57 > 3.0\)

Hence, the slope is stable against sliding failure.

(b) Check for bearing capacity

The factor of safety against bearing capacity failure \((FS_B)\) is calculated from Eq. 3.45

\[ FS_B = \frac{cN_e + 0.5\gamma B_e N_f}{H_{eq}\gamma} \]

For the given case,

\[ \phi = 32^\circ, \; c = 0.05 \text{kPa} \; \text{and} \; \gamma = 22 \text{kN/m}^3 \]

\[ H_{eq} = H = \text{Height of the wall} = 11 \text{m} \]

\[ B_e = \text{Width of the excavation} = H \; \text{(assumed)} = 11 \text{m} \]

\[ N_e, N_f = \text{Bearing capacity factor (mostly dependent on } \phi) \]

From standard chart for \(\phi=32^\circ\)

\[ N_e = 44.04 \; \text{and} \; N_f = 26.87 \]

\[ FS_B = \frac{0.05 \times 44.04 + 0.5 \times 22 \times 11 \times 26.87}{22 \times 11} = 13.44 > 3.0 \]

Hence, the slope is stable against bearing capacity failure.
5.4 Problem-3

5.4.1 Slope stability without nail

Stability analysis of the slope illustrated in Fig. 5.16

![Fig. 5.16 Geometry of the unreinforced slope of Problem-3](image)

Stability analysis of the unreinforced slope has been performed by Bishop’s method (Fig. 5.17). The actual figure was drawn on the graph paper to estimate the values and the detailed calculation is presented in Table 5.18

Radius of slip surface \( R = 62.41 \) m

Co-ordinate of the centre of slip surface \((x, y) = (3.80, 66.65)\)

Factor of safety for the given case:

\[
F = \frac{\sum_{i=1}^{n}[C + N \tan \phi]}{\sum_{i=1}^{n} W \sin \alpha} = \frac{292.06}{125.58} = 2.326
\]
Fig. 5.17 Stability analysis of unreinforced slope of Problem-3

### Table 5.18 FOS calculation for unreinforced slope of Problem-3

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>Base slope of slice (°)</th>
<th>Weight (W) (kN)</th>
<th>Wsinα (kN)</th>
<th>L (m)</th>
<th>N (kN)</th>
<th>$C + N \tan \phi = cL + N \tan \phi$ (kN)</th>
</tr>
</thead>
<tbody>
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<td>1</td>
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<td>5.28</td>
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<tr>
<td>2</td>
<td>11.31</td>
<td>26.40</td>
<td>5.18</td>
<td>0.51</td>
<td>26.92</td>
<td>17.51</td>
</tr>
<tr>
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<td>11.31</td>
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<td>8.63</td>
<td>0.51</td>
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<td>29.17</td>
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<td>16.44</td>
<td>0.52</td>
<td>59.72</td>
<td>38.81</td>
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<td>16.70</td>
<td>66</td>
<td>18.96</td>
<td>0.52</td>
<td>68.91</td>
<td>44.77</td>
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<td>16.70</td>
<td>74.8</td>
<td>21.49</td>
<td>0.52</td>
<td>78.09</td>
<td>50.74</td>
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<tr>
<td>7</td>
<td>16.70</td>
<td>79.2</td>
<td>22.76</td>
<td>0.52</td>
<td>82.69</td>
<td>53.72</td>
</tr>
<tr>
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<td>26.57</td>
<td>57.2</td>
<td>25.58</td>
<td>0.56</td>
<td>63.95</td>
<td>41.56</td>
</tr>
<tr>
<td>9</td>
<td>21.80</td>
<td>17.6</td>
<td>6.54</td>
<td>0.54</td>
<td>18.96</td>
<td>12.34</td>
</tr>
<tr>
<td>∑</td>
<td></td>
<td>125.58</td>
<td></td>
<td></td>
<td></td>
<td>292.06</td>
</tr>
</tbody>
</table>

$c = 0.05kPa \quad \gamma = 22kN/m^3 \quad \phi = 33°$
5.4.2 Stability of nailed slope

Geometry of nailed slope is presented in Fig. 5.18.

Soil properties given: \( c = 0.05 \text{kPa}, \phi = 33^\circ, \gamma = 22 \text{kN/m}^3 \)

Let, nail length \((l) = 8 \text{m}, \text{diameter} \,(d) = 20 \text{mm}, \text{inclination} \,(\lambda) = 10^\circ \)

Vertical spacing of the nails \((s_v) = 1 \text{m} \)

Horizontal spacing of the nails \((s_h) = 1 \text{m} \)

![Fig. 5.18 Geometry of the reinforced slope in Problem-3](image)

5.4.2.1 Internal stability

(a) Nail tensile capacity \((R_T)\)

From Eq. 3.58

\[
R_T = \frac{\pi d^2}{4} \frac{f_y}{FOS_{RT}} = \frac{415 \times \pi \times 20^2}{4 \times 1.8} = 72.394 \text{kN}
\]

Where,

\( f_y = \text{Yield strength of steel} = 415 \text{MPa} \)

\( d = \text{Diameter of the nail} = 20 \text{mm} \)

(b) Facing design

Facing details are same as the previous problem.
(c) Nail pull out resistance for the full nail length ($T_{n,\text{max}}$)

Nail pull out resistance for the full length is calculated similar to the previous problems and presented in Table 5.19.

Table 5.19 $T_p$ and $T_{n,\text{max}}$ values for different nails in Problem-3

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{p1}$ ($kN/m$)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.57</td>
<td>0.93</td>
<td>1.29</td>
<td>1.65</td>
<td>2.01</td>
<td>2.37</td>
</tr>
<tr>
<td>$T_{p2}$ ($kN/m$)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2.54</td>
<td>2.90</td>
<td>3.26</td>
<td>3.62</td>
<td>3.98</td>
<td>4.34</td>
</tr>
<tr>
<td>$T_p$ ($kN/m$)</td>
<td>0.60</td>
<td>0.96</td>
<td>1.33</td>
<td>1.68</td>
<td>2.04</td>
<td>1.55</td>
<td>1.91</td>
<td>2.27</td>
<td>2.63</td>
<td>2.99</td>
<td>3.35</td>
</tr>
<tr>
<td>$T_{n,\text{max}}$ ($kN$)</td>
<td>4.78</td>
<td>7.66</td>
<td>10.54</td>
<td>13.42</td>
<td>16.30</td>
<td>12.44</td>
<td>15.32</td>
<td>18.20</td>
<td>21.08</td>
<td>23.96</td>
<td>26.83</td>
</tr>
</tbody>
</table>

As for all nails $T_{n,\text{max}} < R_f$ and $T_{n,\text{max}} < R_f$, nail length and diameters are adequately selected.

5.4.2.2 External stability

(a) Slope stability

Slope stability of the nailed structure is checked using Bishop’s Method and the values were estimated from the figures drawn on the graph paper.
Case-1: Slope stability for upper part of nailed structure

The graphical representation of the stability analysis of upper slope is given in Fig. 5.19. Nail tensile force and factor of safety calculation details are given in Table 5.20 and Table 5.21 respectively.

Radius of slip surface (R) = 9.34m
Co-ordinate of the centre of slip surface (x, y) = (8.4, 15.98)

Factor of safety for the given case = \[ F = \frac{\sum_{i=1}^{n=12} [C + N \tan \phi]}{\sum_{i=1}^{n=12} [W \sin \alpha - T \cos (\alpha + \lambda) / 54.75]} = \frac{125.46}{2.237} = 2.237 \]

\[ c = 0.05 \text{kPa} \]
\[ \gamma = 22 \text{kN/m}^3 \]
\[ \phi = 33^\circ \]
\[ \lambda = 10^\circ \]
Table 5.20 Calculation of nail tension for case-1 in Problem-3

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_p$ ($kN/m$)</td>
<td>0.60</td>
<td>0.96</td>
<td>1.33</td>
<td>1.68</td>
<td>2.04</td>
</tr>
<tr>
<td>$l_e$ ($m$)</td>
<td>7.10</td>
<td>6.80</td>
<td>6.70</td>
<td>6.90</td>
<td>7.50</td>
</tr>
<tr>
<td>$T_n$ ($kN$)</td>
<td>4.24</td>
<td>6.51</td>
<td>8.83</td>
<td>11.57</td>
<td>15.28</td>
</tr>
</tbody>
</table>

Table 5.21 FOS calculation for case-1 in Problem-3

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>$\alpha$ (degree)</th>
<th>$W$ (kN)</th>
<th>$T_n$ (kN)</th>
<th>$N$ (kN)</th>
<th>$L$ (m)</th>
<th>$C + N \tan \phi$</th>
<th>$W \sin \alpha - \frac{C + N \tan \phi}{T_n \cos(\alpha + \lambda)}$ (kN)</th>
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<td>1</td>
<td>21.80</td>
<td>1.65</td>
<td>0.00</td>
<td>1.78</td>
<td>0.54</td>
<td>1.18</td>
<td>0.61</td>
</tr>
<tr>
<td>2</td>
<td>16.70</td>
<td>5.23</td>
<td>15.28</td>
<td>5.46</td>
<td>0.52</td>
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<td>-12.15</td>
</tr>
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<td>26.57</td>
<td>8.53</td>
<td>0.00</td>
<td>9.53</td>
<td>0.56</td>
<td>6.22</td>
<td>3.81</td>
</tr>
<tr>
<td>4</td>
<td>26.57</td>
<td>11.28</td>
<td>0.00</td>
<td>12.61</td>
<td>0.56</td>
<td>8.21</td>
<td>5.04</td>
</tr>
<tr>
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<td>0.00</td>
<td>16.04</td>
<td>0.58</td>
<td>10.44</td>
<td>7.07</td>
</tr>
<tr>
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<td>15.68</td>
<td>11.57</td>
<td>19.13</td>
<td>0.61</td>
<td>12.46</td>
<td>0.80</td>
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<tr>
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<td>41.99</td>
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<td>0.00</td>
<td>22.57</td>
<td>0.67</td>
<td>14.69</td>
<td>11.22</td>
</tr>
<tr>
<td>8</td>
<td>41.99</td>
<td>17.33</td>
<td>8.83</td>
<td>23.31</td>
<td>0.67</td>
<td>15.17</td>
<td>6.15</td>
</tr>
<tr>
<td>9</td>
<td>47.73</td>
<td>17.33</td>
<td>0.00</td>
<td>25.76</td>
<td>0.74</td>
<td>16.76</td>
<td>12.82</td>
</tr>
<tr>
<td>10</td>
<td>52.43</td>
<td>16.23</td>
<td>6.51</td>
<td>26.61</td>
<td>0.82</td>
<td>17.32</td>
<td>9.85</td>
</tr>
<tr>
<td>11</td>
<td>57.99</td>
<td>11.00</td>
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<tr>
<td>12</td>
<td>63.43</td>
<td>1.98</td>
<td>0.00</td>
<td>4.43</td>
<td>0.67</td>
<td>2.91</td>
<td>1.77</td>
</tr>
<tr>
<td>$\Sigma$</td>
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<td></td>
<td></td>
<td>122.46</td>
<td>54.75</td>
</tr>
</tbody>
</table>

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Case-2: Slope stability for lower part of nailed structure

Fig. 5.20 Stability analysis of nailed slope for case-2 in Problem-3

The graphical representation of the stability analysis of lower slope is given in Fig. 5.20. Nail tensile force and factor of safety calculation details are given in Table 5.22 and Table 5.23 respectively.

Radius of slip surface (R) = 13.39m
Co-ordinate of the centre of slip surface (x, y) = (-4.41, 12.74)

Factor of safety for the given case = $F = \frac{\sum_{i=1}^{n} [C + N \tan \phi]}{\sum_{i=1}^{n} [W \sin \alpha - T_{n} \cos (\alpha + \lambda)]} = \frac{210.53}{105.55} = 1.995$

Table 5.22 Calculation of nail tension for case-2 in Problem-3

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
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</thead>
<tbody>
<tr>
<td>$T_{P1} (kN/m)$</td>
<td>0.57</td>
<td>0.93</td>
<td>1.29</td>
<td>1.65</td>
<td>2.01</td>
<td>2.37</td>
</tr>
<tr>
<td>$l_{e1} (m)$</td>
<td>3.60</td>
<td>4.20</td>
<td>5.00</td>
<td>6.00</td>
<td>6.60</td>
<td>6.90</td>
</tr>
<tr>
<td>$T_{P2} (kN/m)$</td>
<td>2.54</td>
<td>2.90</td>
<td>3.26</td>
<td>3.62</td>
<td>3.98</td>
<td>4.34</td>
</tr>
<tr>
<td>$l_{e2} (m)$</td>
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<td>0.40</td>
<td>0.00</td>
<td>0.00</td>
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<td>0.00</td>
</tr>
<tr>
<td>$T_{n} (kN)$</td>
<td>10.69</td>
<td>11.15</td>
<td>11.01</td>
<td>11.34</td>
<td>13.26</td>
<td>16.35</td>
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</table>
Table 5.23 FOS calculation for case-2 in Problem-3

<table>
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<th>No. of slice</th>
<th>$\alpha$ (degree)</th>
<th>$W$ (kN)</th>
<th>$T_n$ (kN/m)</th>
<th>$N$ (kN/m)</th>
<th>$L$ (m)</th>
<th>$C + N \tan \phi = cl + N \tan \phi$ (kN)</th>
<th>$W \sin \alpha - T_n \cos (\alpha + \lambda)$ (kN)</th>
</tr>
</thead>
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<td>21.80</td>
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<td>0.00</td>
<td>1.78</td>
<td>0.54</td>
<td>1.18</td>
<td>0.61</td>
</tr>
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<td>15.39</td>
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<td>0.61</td>
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</tr>
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<td>18.15</td>
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<td>0.00</td>
<td>23.87</td>
<td>0.94</td>
<td>17.39</td>
<td>10.73</td>
</tr>
<tr>
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<td>7.01</td>
<td>0.85</td>
<td>5.33</td>
<td>-0.40</td>
</tr>
<tr>
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<td></td>
<td></td>
<td></td>
<td>210.53</td>
<td>105.55</td>
</tr>
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</table>
Case-3: Global slope stability of nailed structure

The graphical representation of the global stability analysis of the nailed slope is given Fig. 5.21. Nail tensile force and factor of safety calculation details are given in Table 5.24 and Table 5.25 respectively.

Radius of slip surface (R) = 41.40m
Co-ordinate of the centre of slip surface (x, y) = (-13.5, 39.16)

Factor of safety for the given case

\[ F = \frac{\sum_{i=1}^{n=12} [C + N \tan \phi]}{\sum_{i=1}^{n=12} [W \sin \alpha - T_s \cos (\alpha + \lambda)]} \]

\[ = \frac{457.79}{223.24} = 2.051 \]

\[ c = 0.05 \text{kPa} \]
\[ \gamma = 22 \text{kN/m}^3 \]
\[ \phi = 33^\circ \]
\[ \lambda = 10^\circ \]
### Table 5.24 Calculation of nail tension for case-3 in Problem-3

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_p$ (kN/m)</td>
<td>0.60</td>
<td>0.96</td>
<td>1.33</td>
<td>1.68</td>
<td>2.04</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$l_e$ (m)</td>
<td>7.10</td>
<td>7.10</td>
<td>7.10</td>
<td>7.30</td>
<td>7.40</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$T_{p1}$ (kN/m)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.57</td>
<td>0.93</td>
<td>1.29</td>
<td>1.65</td>
<td>2.01</td>
<td>2.37</td>
</tr>
<tr>
<td>$l_{e1}$ (m)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.20</td>
<td>2.10</td>
<td>3.90</td>
<td>5.70</td>
<td>6.50</td>
<td>7.40</td>
</tr>
<tr>
<td>$T_{p2}$ (kN/m)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2.54</td>
<td>2.90</td>
<td>3.26</td>
<td>3.62</td>
<td>3.98</td>
<td>4.34</td>
</tr>
<tr>
<td>$l_{e2}$ (m)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3.40</td>
<td>2.80</td>
<td>1.40</td>
<td>0.20</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>$T_n$ (kN)</td>
<td>4.24</td>
<td>6.80</td>
<td>9.35</td>
<td>12.24</td>
<td>15.08</td>
<td>9.32</td>
<td>10.07</td>
<td>9.59</td>
<td>10.13</td>
<td>13.06</td>
<td>17.53</td>
</tr>
</tbody>
</table>

### Table 5.25 FOS calculation for case-3 in Problem-3

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>$\alpha$ (degree)</th>
<th>$W$ (kN)</th>
<th>$T_n$ (kN/m)</th>
<th>$N$ (kN)</th>
<th>$L$ (m)</th>
<th>$C + N \tan \phi = cl + N \tan \phi$ (kN/m)</th>
<th>$W \sin \alpha - T_n \cos(\alpha + \lambda)$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>26.57</td>
<td>1.98</td>
<td>0.00</td>
<td>2.21</td>
<td>0.67</td>
<td>1.45</td>
<td>0.89</td>
</tr>
<tr>
<td>2</td>
<td>13.50</td>
<td>28.71</td>
<td>17.53</td>
<td>29.53</td>
<td>1.54</td>
<td>19.20</td>
<td>-9.38</td>
</tr>
<tr>
<td>3</td>
<td>24.70</td>
<td>60.89</td>
<td>13.06</td>
<td>67.02</td>
<td>1.65</td>
<td>43.55</td>
<td>14.71</td>
</tr>
<tr>
<td>4</td>
<td>26.57</td>
<td>86.63</td>
<td>10.13</td>
<td>96.85</td>
<td>1.68</td>
<td>62.92</td>
<td>30.61</td>
</tr>
<tr>
<td>5</td>
<td>32.15</td>
<td>73.80</td>
<td>9.59</td>
<td>87.17</td>
<td>1.24</td>
<td>56.63</td>
<td>32.17</td>
</tr>
<tr>
<td>6</td>
<td>30.31</td>
<td>120.98</td>
<td>10.07</td>
<td>140.13</td>
<td>2.26</td>
<td>91.04</td>
<td>53.38</td>
</tr>
<tr>
<td>7</td>
<td>38.66</td>
<td>54.45</td>
<td>9.32</td>
<td>69.73</td>
<td>1.92</td>
<td>45.32</td>
<td>27.86</td>
</tr>
<tr>
<td>8</td>
<td>26.57</td>
<td>11.88</td>
<td>0.00</td>
<td>13.28</td>
<td>0.67</td>
<td>8.64</td>
<td>5.31</td>
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<tr>
<td>9</td>
<td>39.81</td>
<td>14.85</td>
<td>15.07</td>
<td>19.33</td>
<td>1.17</td>
<td>12.57</td>
<td>-0.22</td>
</tr>
<tr>
<td>10</td>
<td>38.66</td>
<td>29.70</td>
<td>12.24</td>
<td>38.03</td>
<td>1.92</td>
<td>24.73</td>
<td>10.47</td>
</tr>
<tr>
<td>11</td>
<td>41.99</td>
<td>37.13</td>
<td>9.35</td>
<td>49.95</td>
<td>2.02</td>
<td>32.47</td>
<td>19.08</td>
</tr>
<tr>
<td>12</td>
<td>45.00</td>
<td>29.45</td>
<td>6.80</td>
<td>41.65</td>
<td>1.48</td>
<td>27.07</td>
<td>16.93</td>
</tr>
</tbody>
</table>
\[
\begin{array}{ccccccc}
13 & 48.37 & 31.68 & 4.24 & 47.68 & 2.41 & 32.19 & 21.45 \\
\hline
\sum & \quad & \quad & \quad & \quad & \quad & 457.79 & 223.24
\end{array}
\]

Hence, the minimum factor of safety for the designed nailed slope against slope stability in given three cases is 1.995.

(b) Stability against sliding

![Fig. 5.22 Stability analysis of nailed slope in Problem-3 against sliding](image)

Fig. 5.22 presents the reinforced soil mass considered for sliding stability analysis along with the siding surface \(QR\).

\[L_x = l \times \cos \lambda - t \times \tan \alpha\]

\(l = \) Length of nail = 8m
\(t = \) Depth of first nail = 0.5m
\(\alpha = \) Angle of slope with horizontal = 45°
\(\lambda = \) Nail inclination with horizontal = 10°

\[L_x = 8 \cos 10^\circ - 0.5 \tan 45^\circ = 7.38m\]
Weight of sliding wedge (W) = $\gamma \times \text{area of wedge}$
$W = 22 \times 188.54 = 4147.91 kN$

$B_L = \text{width of failing wedge} = (7 + 4 + 5 + 7.38) m = 23.38 m$

Total resisting force ($\sum R$) = $W \times \tan \phi + c \times B_L = 2694.06 kN$

Total destabilising force ($\sum D$) = $P_A = \frac{\gamma H^2}{2} \times K_A = 466.97 kN$

Where,
$H = \text{Height of wedge} = 12 m$

$K_A = \tan^2 (45 - \frac{\phi}{2}) = 0.295$

$\phi = 33^0$, $c = 0.05 kPa$ and $\gamma = 22 kN / m^3$

Factor of safety against sliding (FOS) = $\frac{\sum R}{\sum D} = 5.77 > 3.0$

Hence, the slope is stable against sliding failure.

(b) Check for bearing capacity

The factor of safety against bearing capacity failure ($FS_H$) is calculated from Eq. 3.45

$$FS_H = \frac{c N_c + 0.5 \gamma B_e N_r}{H_{eq} \gamma}$$

For the given case,
$\phi = 33^0$, $c = 0.05 kPa$ and $\gamma = 22 kN / m^3$

$H_{eq} = H = \text{Height of the wall} = 12 m$

$B_e = \text{Width of the excavation} = H \text{ (assumed)} = 12 m$

$N_c, N_r = \text{Bearing capacity factor (mostly dependent on } \phi)$

From standard chart for $\phi=33^0$, $N_c = 48.09$ and $N_r = 31.94$

$$FS_H = \frac{0.05 \times 48.09 + 0.5 \times 22 \times 12 \times 31.94}{22 \times 12} = 15.94 > 3.0$$

Hence, the slope is stable against bearing capacity failure.
5.5 Problem-4

5.5.1 Slope stability without nail

Stability analysis of the slope illustrated in Fig. 5.23

Stability analysis of the unreinforced slope has been performed by Bishop’s method (Fig. 5.24). The actual figures were drawn on the graph paper to estimate the values and the detailed calculation is presented in Table 5.26.
Radius of slip surface (R) = 125.35m

Co-ordinate of the centre of slip surface (x, y) = (-19.67, 123.81)

Factor of safety for the given case = \[ F = \frac{\sum_{i=1}^{n} [C + N \tan \phi]}{\sum_{i=1}^{n} W \sin \alpha} \]

\[ c = 3.1 \text{kPa} \]
\[ \gamma = 21.89 \text{kN/m}^3 \]
\[ \phi = 30.81^\circ \]

\[ c = 4.1 \text{kPa} \]
\[ \gamma = 22.26 \text{kN/m}^3 \]
\[ \phi = 32.41^\circ \]

\[ c = 0 \text{kPa} \]
\[ \gamma = 22.45 \text{kN/m}^3 \]
\[ \phi = 35.26^\circ \]
Table 5.26 FOS calculation for unreinforced slope of Problem-4

| No. of slice | Base slope of slice (α) (degree) | Weight (W) (kN) | Wsinα (kN) | L (m) | N (kN) | \(C + N \tan \phi = cL + N \tan \phi\) (kN) |
|--------------|---------------------------------|-----------------|------------|-------|-------|--------------------------------|---|
| 1            | 9.93                            | 58.37           | 10.06      | 4.06  | 59.26 | 41.89                           |   |
| 2            | 11.87                           | 168.82          | 34.70      | 4.09  | 172.51 | 121.96                          |   |
| 3            | 13.64                           | 267.16          | 62.96      | 4.12  | 274.90 | 194.35                          |   |
| 4            | 15.65                           | 263.11          | 70.94      | 4.15  | 273.23 | 193.17                          |   |
| 5            | 17.49                           | 243.97          | 73.30      | 4.19  | 255.79 | 179.58                          |   |
| 6            | 19.43                           | 303.18          | 100.79     | 4.24  | 321.47 | 221.48                          |   |
| 7            | 21.32                           | 349.04          | 126.82     | 4.29  | 374.64 | 255.45                          |   |
| 8            | 23.40                           | 291.61          | 115.76     | 4.36  | 317.71 | 219.57                          |   |
| 9            | 25.42                           | 215.40          | 92.42      | 4.43  | 238.46 | 155.94                          |   |
| 10           | 27.38                           | 216.71          | 99.60      | 4.50  | 244.01 | 159.48                          |   |
| 11           | 29.48                           | 202.26          | 99.50      | 4.59  | 232.32 | 152.78                          |   |
| 12           | 31.57                           | 84.70           | 44.32      | 4.17  | 99.40  | 72.19                           |   |
| \(\sum\)    |                                 | 931.17          |            |       |       | 1967.85                         |   |

5.4.2 Stability of nailed slope

Geometry of nailed slope and the soil properties are presented in Fig. 5.25.

Let, nail length \((l) = 8\text{m}\), diameter \((d) = 25\text{mm}\), inclination \((\lambda) = 10^\circ\)

Vertical spacing of the nails \((s_v) = 1\text{m}\)

Horizontal spacing of the nails \((s_h) = 1\text{m}\)
5.4.2.3 Internal stability

(a) Nail tensile capacity \((R_f)\)

From Eq. 3.58

\[
R_f = \frac{\pi d^2}{4} \frac{f_y}{FOS_{RT}} = \frac{415 \times \pi \times 25^2}{4 \times 1.8} = 113.17 \text{kN}
\]

Where,

\(f_y\) = Yield strength of steel = 415MPa

\(d\) = Diameter of the nail = 25mm

(b) Facing design

Facing details are same as mentioned in Problem-1.

(c) Nail pull out resistance for the full nail length \((T_{n,max})\)

Nail pull out resistance for the full length is calculated similar to the previous problems and presented in Table 5.27.
Table 5.27 $T_p$ and $T_{n,\text{max}}$ values for different nails in Problem-4

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{p1}$ ($kN/m$)</td>
<td></td>
<td></td>
<td></td>
<td>1.273</td>
<td>1.966</td>
<td>2.658</td>
<td>1.382</td>
<td>2.149</td>
<td>2.916</td>
</tr>
<tr>
<td>$T_{p2}$ ($kN/m$)</td>
<td></td>
<td></td>
<td></td>
<td>4.015</td>
<td>4.707</td>
<td>5.4</td>
<td>4.444</td>
<td>5.211</td>
<td>5.977</td>
</tr>
<tr>
<td>$T_p$ ($kN/m$)</td>
<td>1.11</td>
<td>1.53</td>
<td>1.96</td>
<td>2.64</td>
<td>3.34</td>
<td>4.03</td>
<td>2.91</td>
<td>3.68</td>
<td>4.45</td>
</tr>
<tr>
<td>$T_{n,\text{max}}$ ($kN$)</td>
<td>8.84</td>
<td>12.27</td>
<td>15.71</td>
<td>21.16</td>
<td>26.69</td>
<td>32.23</td>
<td>23.30</td>
<td>29.44</td>
<td>35.57</td>
</tr>
</tbody>
</table>

As for all nails $T_{n,\text{max}} < R_T$ and $T_{n,\text{max}} < R_f$, nail length and diameters are adequately selected.

5.4.2.4 External stability

(a) Slope stability

Slope stability of the nailed structure is checked using Bishop’s Method and the values are estimated from the figures drawn on the graph paper.
Case-1: Slope stability for upper part of nailed structure

The graphical representation of the stability analysis of upper slope is given in Fig. 5.26. Nail tensile force and factor of safety calculation details are given in Table 5.28 and Table 5.29 respectively.

Radius of slip surface (R) = 12.02m

Co-ordinate of the centre of slip surface (x, y) = (19.09, 23.38)

Factor of safety for the given case = $F = \frac{\sum_{i=1}^{n} [C + N \tan \phi]}{\sum_{i=1}^{n} [W \sin \alpha - T_n \cos(\alpha + \lambda)]} = \frac{158.91}{94.41} = 1.683$
Table 5.28 Calculation of nail tension for case-1 in Problem-4

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_p (kN/m)$</td>
<td>1.11</td>
<td>1.53</td>
<td>1.96</td>
</tr>
<tr>
<td>$l_e (m)$</td>
<td>6.80</td>
<td>6.60</td>
<td>6.80</td>
</tr>
<tr>
<td>$T_n (kN)$</td>
<td>7.51</td>
<td>10.13</td>
<td>13.35</td>
</tr>
</tbody>
</table>

Table 5.29 FOS calculation for case-1 in Problem-4

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>$\alpha$ (degree)</th>
<th>W (kN)</th>
<th>$T_n$ (kN)</th>
<th>N (kN)</th>
<th>L (m)</th>
<th>$C + N \tan \phi = cl + N \tan \phi$ (kN)</th>
<th>$W \sin \alpha - T_n \cos(\alpha + \lambda)$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>21.80</td>
<td>6.57</td>
<td>0.00</td>
<td>7.07</td>
<td>2.12</td>
<td>7.10</td>
<td>2.44</td>
</tr>
<tr>
<td>2</td>
<td>21.80</td>
<td>19.70</td>
<td>0.00</td>
<td>21.22</td>
<td>6.36</td>
<td>14.62</td>
<td>7.32</td>
</tr>
<tr>
<td>3</td>
<td>34.99</td>
<td>29.55</td>
<td>13.35</td>
<td>36.07</td>
<td>9.53</td>
<td>22.96</td>
<td>7.50</td>
</tr>
<tr>
<td>4</td>
<td>38.66</td>
<td>35.02</td>
<td>0.00</td>
<td>44.85</td>
<td>11.30</td>
<td>27.82</td>
<td>21.88</td>
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<td>5</td>
<td>45.00</td>
<td>37.21</td>
<td>10.13</td>
<td>52.63</td>
<td>12.00</td>
<td>32.37</td>
<td>20.51</td>
</tr>
<tr>
<td>6</td>
<td>50.19</td>
<td>35.02</td>
<td>7.51</td>
<td>54.71</td>
<td>11.30</td>
<td>33.93</td>
<td>23.17</td>
</tr>
<tr>
<td>7</td>
<td>61.93</td>
<td>13.13</td>
<td>0.00</td>
<td>27.91</td>
<td>4.24</td>
<td>20.11</td>
<td>11.59</td>
</tr>
<tr>
<td>$\Sigma$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

|               | 158.91            | 94.41  |
Case-2: Slope stability for middle part of nailed structure

The graphical representation of the stability analysis of middle slope is given in Fig. 5.27. Nail tensile force and factor of safety calculation details are given in Table 5.30 and Table 5.31 respectively.

Radius of slip surface (R) = 11.05m
Co-ordinate of the centre of slip surface (x, y) = (9.91, 16.62)

Factor of safety for the given case = \[ F = \frac{\sum_{i=1}^{n} [C + N \tan \phi]}{\sum_{i=1}^{n} [W \sin \alpha - T_n \cos(\alpha + \lambda)]} = \frac{180.49}{92.80} = 1.945 \]
Table 5.30 Calculation of nail tension for case-2 in Problem-4

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{p1} (kN/m)$</td>
<td>1.27</td>
<td>1.97</td>
<td>2.66</td>
</tr>
<tr>
<td>$l_{e1} (m)$</td>
<td>4.20</td>
<td>5.40</td>
<td>6.70</td>
</tr>
<tr>
<td>$T_{p2} (kN/m)$</td>
<td>4.02</td>
<td>4.71</td>
<td>5.40</td>
</tr>
<tr>
<td>$l_{e2} (m)$</td>
<td>2.40</td>
<td>1.00</td>
<td>0.00</td>
</tr>
<tr>
<td>$T_n (kN)$</td>
<td>14.98</td>
<td>15.32</td>
<td>17.81</td>
</tr>
</tbody>
</table>

Table 5.31 FOS calculation for case-2 in Problem-4

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>$\alpha$ (degree)</th>
<th>W (kN)</th>
<th>$T_n$ (kN/m)</th>
<th>$N$ (kN/m)</th>
<th>L (m)</th>
<th>$C + N \tan \phi = cl + N \tan \phi$ (kN)</th>
<th>$W \sin \alpha - T_n \cos (\alpha + \lambda)$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>16.70</td>
<td>7.79</td>
<td>0.00</td>
<td>8.13</td>
<td>1.04</td>
<td>8.61</td>
<td>2.24</td>
</tr>
<tr>
<td>2</td>
<td>26.57</td>
<td>21.15</td>
<td>0.00</td>
<td>23.64</td>
<td>1.12</td>
<td>17.16</td>
<td>9.46</td>
</tr>
<tr>
<td>3</td>
<td>30.96</td>
<td>31.16</td>
<td>17.81</td>
<td>36.34</td>
<td>1.17</td>
<td>24.11</td>
<td>2.58</td>
</tr>
<tr>
<td>4</td>
<td>41.99</td>
<td>36.73</td>
<td>0.00</td>
<td>49.41</td>
<td>1.35</td>
<td>31.79</td>
<td>24.57</td>
</tr>
<tr>
<td>5</td>
<td>38.66</td>
<td>40.07</td>
<td>15.32</td>
<td>51.31</td>
<td>1.28</td>
<td>32.53</td>
<td>14.91</td>
</tr>
<tr>
<td>6</td>
<td>52.43</td>
<td>38.96</td>
<td>14.98</td>
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<td>1.64</td>
<td>40.70</td>
<td>23.94</td>
</tr>
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<td>15.10</td>
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<td></td>
<td></td>
<td></td>
<td>$180.49$</td>
<td>$92.80$</td>
</tr>
</tbody>
</table>
Case-3: Slope stability for middle part of nailed structure

The graphical representation of the stability analysis of lower slope is given in Fig. 5.28. Nail tensile force and factor of safety calculation details are given in Table 5.32 and Table 5.33 respectively.

Radius of slip surface (R) = 14.19m

Co-ordinate of the centre of slip surface (x, y) = (-2.05, 14.06)

Factor of safety for the given case:

\[ F = \frac{\sum_{i=1}^{n} [C + N \tan \phi]}{\sum_{i=1}^{n} [W \sin \alpha - T_\alpha \cos(\alpha + \lambda)]} = \frac{144.32}{83.15} = 1.736 \]
Table 5.32 Calculation of nail tension for case-3 in Problem-4

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{p1}$ (kN/m)</td>
<td>1.38</td>
<td>2.15</td>
<td>2.92</td>
</tr>
<tr>
<td>$l_{c1}$ (m)</td>
<td>5.00</td>
<td>6.20</td>
<td>6.40</td>
</tr>
<tr>
<td>$T_{p2}$ (kN/m)</td>
<td>4.44</td>
<td>5.21</td>
<td>5.98</td>
</tr>
<tr>
<td>$l_{c2}$ (m)</td>
<td>1.80</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>$T_n$ (kN)</td>
<td>14.91</td>
<td>13.32</td>
<td>18.66</td>
</tr>
</tbody>
</table>

Table 5.33 FOS calculation for case-3 in Problem-4

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>$\alpha$ (degree)</th>
<th>W (kN)</th>
<th>$T_n$ (kN/m)</th>
<th>$N$ (kN/m)</th>
<th>L (m)</th>
<th>$C + N \tan \phi = cl + N \tan \phi$ (kN)</th>
<th>$W \sin \alpha - T_n \cos(\alpha + \lambda)$ (kN)</th>
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</thead>
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<td>12.59</td>
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<td>28.74</td>
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<td>-</td>
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<td>16.21</td>
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<td>5.08</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>144.33</td>
<td>83.15</td>
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</tbody>
</table>
Case 4: Global slope stability of nailed structure

Fig. 5.29 Stability analysis of nailed slope for case-4 in Problem-4

The graphical representation of the global stability analysis of the nailed slope is given Fig. 5.29. Nail tensile force and factor of safety calculation details are given in Table 5.34 and Table 5.35 respectively.

Radius of slip surface (R) = 34.33 m

Co-ordinate of the centre of slip surface (x, y) = (2.99, 33.47)

Factor of safety for the given case =

\[
F = \frac{\sum_{i=1}^{n \times 15} [C + N \tan \phi]}{\sum_{i=1}^{n \times 15} [W \sin \alpha - T_n \cos(\alpha + \lambda)]} = \frac{2475.79}{1610.5} = 1.537
\]
Table 5.34 Calculation of nail tension for case-4 in Problem-4

<table>
<thead>
<tr>
<th>Nail No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
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<tbody>
<tr>
<td>$T_p$ ($kN/m$)</td>
<td>1.11</td>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$l_e$ (m)</td>
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<td>3.50</td>
<td>3.20</td>
<td>-</td>
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<td>$T_{p1}$ ($kN/m$)</td>
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<td>-</td>
<td>-</td>
<td>1.27</td>
<td>1.97</td>
<td>2.66</td>
<td>1.38</td>
<td>2.15</td>
<td>2.92</td>
</tr>
<tr>
<td>$l_{e1}$ (m)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<td>-</td>
<td>2.00</td>
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<tr>
<td>$T_{p2}$ ($kN/m$)</td>
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<td>-</td>
<td>-</td>
<td>4.02</td>
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<td>4.44</td>
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<tr>
<td>$l_{e2}$ (m)</td>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>$T_n$ ($kN$)</td>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>5.83</td>
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Table 5.35 FOS calculation for case-4 in Problem-4

<table>
<thead>
<tr>
<th>No. of slice</th>
<th>$\alpha$ (degree)</th>
<th>$W$ (kN)</th>
<th>$T_n$ (kN/m)</th>
<th>$N$ (kN)</th>
<th>$L$ (m)</th>
<th>$C + N \tan \phi = cl + N \tan \phi$ (kN/m)</th>
<th>$W \sin \alpha - T_n \cos (\alpha + \lambda)$ (kN)</th>
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<tbody>
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<td>1</td>
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<td>0.00</td>
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<td>-</td>
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<td>-</td>
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<td>171.46</td>
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<td>280.54</td>
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</tr>
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<td>107.58</td>
</tr>
<tr>
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<td>243.76</td>
<td>5.12</td>
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<td>107.54</td>
</tr>
<tr>
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<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>∑</td>
<td></td>
<td>2475.79</td>
<td></td>
<td>1610.50</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Hence, the minimum factor of safety for the designed nailed slope against slope stability in given four cases is 1.537.

(b) Stability against sliding

Fig. 5.30 Stability analysis of nailed slope in Problem-3 against sliding
Fig. 5.30 presents the reinforced soil mass considered for sliding stability analysis along with the siding surface $QR$.

\[ L_x = l \times \cos \lambda - t \times \tan \alpha \]

- $l =$ Length of nail = 8m
- $t =$ Depth of first nail = 1.5m
- $\alpha =$ Angle of slope with horizontal = 45°
- $\lambda =$ Nail inclination with horizontal = 10°

\[ L_x = 8 \cos 45^\circ - 1.5 \tan 45^\circ = 6.38 \text{m} \]

Let the average unit weight ($\gamma'$), cohesion ($c'$) and friction angle ($\phi'$) be:

\[ \phi' = 33^\circ, c' = 2.4kPa \text{ and } \gamma = 22kN/m^3 \]

Weight of sliding wedge ($W$) = $\gamma' \times$ area of wedge = 7907.558 kN

\[ B_L = \text{width of failing wedge} = (9 + 4 + 6 + 4 + 6 + 6.38)m = 35.38m \]

Total resisting force ($\sum R$) = $W \times \tan \phi + c \times B_L = 5220.14kN$

Total destabilising force ($\sum D$) = $P_A = \frac{\gamma H^2}{2} \times K_A = 1031.77kN$

Where,

\[ H = \text{Height of wedge} = 18m \]

\[ K_A = \tan^2 (45 - \frac{\phi}{2}) = 0.295 \]

Factor of safety against sliding (FOS) = $\frac{\sum R}{\sum D} = 5.1 > 3.0$

Hence, the slope is stable against sliding failure.

\( (b) \text{ Check for bearing capacity} \)

The factor of safety against bearing capacity failure ($FS_H$) is calculated from Eq. 3.45

\[ FS_H = \frac{c N_c + 0.5 \gamma B_e N_y}{H_{eq} \gamma} \]

For the given case,

\[ \phi = 33^\circ, c = 2.4kPa \text{ and } \gamma = 22kN/m^3 \]

\[ H_{eq} = H = \text{Height of the wall} = 18m \]

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\[ B = \text{Width of the excavation} = H \text{ (assumed)} = 18 \text{m} \]

\[ N_c, N_γ\] = Bearing capacity factor (mostly dependent on \( \phi \))

From standard chart for \( \phi = 33^\circ \), \( N_c = 48.09 \) and \( N_γ = 31.94 \)

\[
FS_H = \frac{2.4 \times 48.09 + 0.5 \times 22 \times 18 \times 31.94}{22 \times 18} = 16.26 > 3.0
\]

Hence, the slope is stable against bearing capacity failure.
CHAPTER 6
CONSTRUCTION PROCEDURE OF NAILED SLOPE AND CONSTRUCTION EQUIPMENTS

6.1 Construction procedure of nailed structure

6.1.1 Excavation

Prior to any excavation, surface water controls should be constructed to prevent surface water from flowing into the excavation, as this condition will adversely affect construction and potentially cause instability of the excavated face. Collector trenches behind the limits of the excavation usually intercept and divert surface water.

Initial excavation is carried out to a depth for which the face of the excavation may remain unsupported for a short period of time, e.g. 24 to 48 hours. The depth for each excavation reaches slightly below the elevation where nails will be installed. The width of the excavated platform or bench is such that it can provide sufficient access to the installation equipment. The initial lift is typically taken as 1 to 1.2 m (3 to 4 feet) high. The excavated face profile should be reasonably smooth and not too irregular to minimize excessive shotcrete quantities. Soil profiles containing cobbles and/or boulders may require hand excavation. A level working bench on the order of 10-m (30-ft) wide is required to accommodate the conventional drilling equipment used for nail installation. Track drills smaller than the conventional drilling equipment can work on benches as narrow as 5 m (15 ft.) and with headroom clearance as low as 3 m (9 ft.).

Soil-nailed excavation is usually carried out in stages (Fig. 6.1). The height of the exposed slope face is determined on the basis of its temporary stability. After installation of a row of soil nails, subsequent excavation should progress only when the temporary stability of the excavation is adequate. Soil-nail heads and facing should be constructed before the next stage of excavation, unless the temporary stability of the soil-nailed excavation in the absence of soil-nail head is adequate. The sequence and timing of installing soil nails, constructing soil-
nail heads and facing, and excavation should be monitored and controlled to fulfil these requirements.

Soil excavation is performed using conventional earth-moving equipment from a platform, and final trimming of the excavation face is typically carried out using a backhoe or excavator from a platform. Some of the equipments are named below.

- Backhoe
- Bucket wheel excavator
- Cable excavator
- Hydraulic excavator
- Motor scraper
- Trencher
- Wheel loader

Detail of these equipments is discussed in subsequent sections of this chapter.

If the temporary excavation involves use of structural lateral support, soil nails can serve as tie-backs. Soil nails may be modelled as structural elements providing external forces to the stem wall of the lateral support system. Because the experience of using soil nails in temporary excavation in cohesive soils is limited, special care should be exercised about the effect of creeping on the stability and serviceability of the excavation, in particular if the soil nails are designed to carry sustained loads. If the excavation face becomes unstable at any point of time, soil nail wall construction is suspended and the face is temporarily stabilized by immediately placing an earth berm against the unstable face.

Dump trucks or production trucks are used for transporting loose material such as sand, dirt, and gravel for construction. The typical dump truck is equipped with a hydraulically operated open box with bed hinged at the rear, with the front being able to be lifted up to allow the contents to fall.
6.1.2 Drilling Nail Holes

There are two types of processes which can be carried out after excavation for putting the nails. The nail can be directly pushed into the soil using suitable equipment (Fig. 6.2), in which the nail itself makes its way forward. Alternatively, a hole can be drilled prior to putting the nail by using some drilling equipment. The latter process is discussed here in detail, as this is the one which is commonly used. Some of the drilling equipments used for this method are listed below.

- Drill bit machine
- Rope core drill
- Air leg rock drill
- Horizontal drill machine

Detail of these equipments is discussed in subsequent sections of this chapter.

Drilling can be done by either air-flushed percussion drilling, auguring, or rotary wash boring depending on the ground conditions. The size of drilled hole shall be as per the designed dimension. Use of drilling mud such as Bentonite slurry to assist in drill cutting removal is not allowed, but air may be used. Flushing with air or water before nail insertion is necessary.
in order to remove any possible collapsed materials, which can potentially reduce the grout-ground interface resistance.

The used drill bit must allow cutting through different type of soil conditions. Drill bits shall be provided with venturi holes to allow for proper tremie grouting. Centralizers are not used with self-drilling hollow core bar. It is advisable to use drilling rigs capable of drilling through whatever materials are encountered to the dimensions and orientations required for the soil nail wall design. Drill hole locations and inclinations are required to be within 6” (150 mm) and 2 degrees precision, respectively. The drill holes may be stabilized with temporary casings if they are unstable. Caving or sloughing material is anticipated or encountered when drill holes become unstable. If caving ground is encountered, use of cased drilling methods is suggested to support the sides of drillholes. Where hard drilling conditions such as rock, cobbles or boulders are encountered, percussion or other suitable drilling equipment capable of drilling and maintaining stable drillholes through such materials may be used.

The correctness of the alignment of drill holes is important in preventing clash of soil nails, in particular for closely-spaced or long soil nails, or soil nails with different inclinations and bearings. It is imperative to control and check the initial inclination and bearing of drill holes. If accurate measurements of the inclination and bearing of the drillhole along its length are needed, special equipment such as an Eastman camera may be employed.

The drill hole diameter is selected such that it can develop the specified pullout resistance and also allow encapsulation of encapsulated nails. Typically, the hole size can range from 100 mm to 150 mm. In order to contain the grout, the typical inclination of the drill hole is kept at 15° downward from horizontal. The water, dust, fumes and noise generated during drilling operation should be sufficiently diverted, controlled, suppressed and muffled. One must also ensure that

- The drilling equipment (type, diameter of drill bit, total length of drill rods, flushing medium, etc.) are checked.
- There is no freshly grouted soil nails near the drillhole to be drilled.
- The drill hole diameter, length, inclination and bearing are in accordance with the contract requirements.
For drilling of long soil nails, the drill rate should be suitably controlled to minimize the eccentricity produced by the dip of the drill rods, which may otherwise cause misalignment of the drill hole or may unduly enlarge the diameter of the drill hole and cause hole collapse. Drill holes in soil should be kept open only for short periods of time. The longer the hole is left open, the greater is the risk of collapse. Before the drilling works in a reinforced concrete wall is carried out, safety precautions should be implemented to avoid damage to steel bars in the reinforced concrete wall, such as using metal detector to determine locations of steel bars. Drilling and preparation of cement grout is performed simultaneously allowing soil nail installation and grouting in a single operation.

![Fig. 6.2 Drilling nail holes](image)

6.1.3 Nail Installation and Grouting

Nail bars are placed in the pre-drilled holes. Centralizers are placed around the nails prior the insertion of nails to maintain proper alignment within the hole and also to allow sufficient protective grout coverage over the nail bar. Grout pipe is also inserted in the drill hole at this stage. A grouting pipe is normally attached with the nail reinforcement while inserting the nail into the drilled hole. Sometimes additional correctional protection is used by introducing corrugated plastic sheathing. In additional to this, galvanization and pre-grouted nail encapsulated with corrugated pipe can be considered for durability. The drill hole is then filled with cement grout throughout the pipe. The normal range of water/cement ratio of the
typical grout mix is from 0.45 to 0.5. The grout is commonly placed undergravity or low pressure. The grouting is from bottom up until fresh grout return is observed from the hole. If hollow self-drilling bars are used, the drilling and grouting takes place in one operation. Geo-composite drainage strips are installed on the excavation face approximately midway between each set of adjacent nails prior to the placement of facing (Fig. 6.3). The drainage strips are then unrolled to the next wall lift. The drainage strips extend to the bottom of the excavation where collected water is conveyed via a toe drain away from the soil nail wall.

In case of self-drilling hollow core bars being used, grouting shall be done continuously during the drilling operation through a rotary injection adapter attached to the end of the anchor. Grout will flow through the hollow core hole exiting through the drill bit holes. When self-drilling hollow core bar is used, ground cuts can be mixed with cement grout. Soil nails shall be grouted full length. In case of the solid reinforcing steel, grout shall be injected at the low end of the drilled hole. The grout shall fill the entire drilled hole with a dense grout free of voids or inclusion of foreign material.

It is imperative that oil, rust inhibitors, residual drilling fluids and similar foreign materials are removed from holding tanks/hoppers, stirring devices, pumps, lines, tremie pipes and all other equipment in contact with grout before use. The grout is injected at the lowest point of drill holes through a tremie pipe, e.g., grout tube, casing, hollow-stem auger or drill rod, in one continuous operation. The drill holes are filled progressively from the bottom to top and the tremie pipe is withdrawn at a slow even rate as the hole is filled to prevent voids in the grout. The tremie pipe is extended into grout by a minimum of 5 ft (1.5 m) at all times except when grout is initially placed in a drill hole.

The grout is expected to be free of segregation, intrusions, contamination, structural damage or inadequate consolidation (honeycombing). Cold joints in grout are not allowed except for soil nails that are tested. The temporary casings are pulled out progressively as the grout is placed in. The grout volumes must be monitored and recorded during placement. When using threaded reinforcing steel, the length of drilled hole shall be monitored before and during grouting.
6.1.4 Construction of Temporary Shotcrete Facing

The temporary shotcrete facing is placed to temporarily restrain the exposed soil in cut face (Fig. 6.4). It consists of 3-4 inches of shotcrete reinforced with a single layer of welded wire mesh. The temporary shotcrete facing is placed concurrently with each excavation lift. The reinforcement typically consists of welded wiremesh (WWM), which is placed at approximately the middle of the facing thickness. The mesh is placed in such a manner that the at least one full mesh cell overlaps with subsequent WWM panels. After proper curing of temporary facing (for at least 24 hours), steel bearing plate is placed over the nail head and the bar is lightly pressed into the first layer of fresh shotcrete. Hex nut and washers are subsequently installed to hold the nail head against the bearing plate. Before proceeding with subsequent excavation lifts, the shotcrete must be cured for at least 72 hours or it should have attained at least the specified 3-day compressive strength.

Shotcrete Application

For shotcrete mixtures, there are two opposing requirements: “shootability” and “pumpability”. Shootability is the ability of a mix to stick to a surface, build up thickness, and resist sloughing. Pumpability is the ability of a mix to flow like a viscous fluid. For
shooting, a high flow resistance and high viscosity are ideal, whereas for pumping, a low flow resistance and low viscosity are ideal. Once it is applied, a shotcrete mix with high flow resistance and high viscosity will tend to “stick” and remain there, as the layers of facing are formed. With the proper mix design, shootability to a thickness of 300 mm (12 in.) can readily be achieved without sloughing or sag cracks below rebar.

Two types of shotcrete methods are commonly used: dry mix and wet mix. In the dry mix method, the aggregate and cement are blended in the dry and fed into the shotcrete gun while the mix water is added at the nozzle. Depending on their features, admixtures can be added at the mix plant or with the water. The addition of water at the nozzle allows the plasticity of the shotcrete to be adjusted at the nozzle, if required. In the wet mix method, the aggregate, cement, water, and admixtures are mixed in a batch plant and conveyed to the nozzle by a hydraulic pump. The plastic mix is applied at higher velocities by compressed air. Both shotcrete methods produce a mix suitable for wall facings. Dry mix and wet mix shotcrete use a water-cement ratio of about 0.4 and produce roughly the same mix quality, although shotcrete obtained with the wet mix process yields a slightly greater flexural strength.

Fig. 6.4 Placing the temporary facing
(Including shotcrete, reinforcement, bearing plate, hex nut and washers installation)
**Shotcrete Reinforcement**

Welded wire mesh is commonly used as reinforcement for temporary facing, but occasionally it is also used in permanent facing. The cross-sectional area and mesh opening of the WWM are selected to satisfy structural requirements (i.e., flexural and punching shear capacities) and constructability constraints. The selected WWM must have a width that is consistent with the excavation lift height (equivalent to the vertical nailspacing), plus an overlap of at least 0.2 m (8 in). For example, if the selected nail vertical spacing were 1.5 m (4.5 ft), the ideal width of the WWM panel would be approximately 1.70 m (5.5 ft). Additional reinforcement (“waler bars”) may be placed around nail heads to provide additional flexural capacity at these locations. The waler bars consist of two vertical (one bar at each side of the nail head) and two horizontal bars.

**6.1.5 Construction of Subsequent Levels**

The steps mentioned in 6.1.1 through 6.1.4 are repeated for the remaining excavation stages (Fig. 6.5). At each excavation stage, the vertical drainage strip is unrolled downward to the subsequent stage. A new panel of WWM is then placed overlapping at least one full mesh cell. The temporary shotcrete is continued with a cold joint with the previous shotcrete lift. At the bottom of the excavation, the drainage strip is tied to a collecting toe drain.

![Fig. 6.5- Construction of subsequent levels](image)
6.1.6 Construction of Permanent Facing

The final facing is constructed after the bottom of the excavation is reached and nails are installed (Fig. 6.6). Final facing consists of cast-in-place (CIP) reinforced concrete, reinforced shotcrete, or prefabricated panels. Generally, conventional concrete bars or WWM is provided as reinforcement in permanent facing. When CIP concrete and shotcrete are used for the permanent facing, horizontal joints between excavation stages are avoided to the maximum extent possible.

Shotcrete facing and wall drainage work consists of furnishing all materials and labour required for placing and securing geocomposite drainage material, connection pipes, footing drains, weepholes and horizontal drains (if required), drainage gutter, reinforcing steel and shotcrete for the permanent shotcrete facing and nail head bearing plates and nuts for the soil nail walls. The work shall include any preparatory trimming and cleaning of soil/rock surfaces and shotcrete cold joints to receive new shotcrete. Shotcrete shall consist of an application of one or more layers of concrete conveyed through a hose and pneumatically projected at a high velocity against a prepared surface.

Proportioning and use of admixtures

The proportion of shotcrete is done in such a way that it is pumpable with the concrete pump furnished for the work. The cementing material content should be at least 390 kilograms per cubic meter (660 lbs. per cubic yard) and water/cement ratio should not be greater than 0.45. The admixtures are thoroughly mixed into the shotcrete at the rate specified by the manufacturer. Accelerators (if used) shall be compatible with the cement used, be non-corrosive to steel and not promote other detrimental effects such as cracking or excessive shrinkage. The maximum allowable chloride ion content of all the ingredients shall not exceed 0.10%.

Wall Drainage

The drainage network shall consist of installing geocomposite drain strips, PVC connection pipes and wall footing drains. Exclusive of the wall footing drains, all elements of the drainage network shall be installed prior to shotcreting. Unanticipated subsurface drainage features exposed in the excavation cut face shall be captured independent of the wall drainage network and shall be mitigated prior to shotcrete application.
6.2 Construction Materials

6.2.1 Steel Reinforcements

For corrosion protection, all steel components shall be galvanised. If machine threading after galvanisation is unavoidable, then proper zinc based coating shall be applied onto the thread. For double corrosion protection, the PVC corrugated pipe is often used which shall be of good quality and adequate thickness. Preferably, galvanized corrugated steel pipe shall be used.

6.2.2 Grout Mix

For conventional soil nail, the water cement ratio of the grout mix ranges from 0.4 to 0.5. As the cementitious grout will experience some shrinkage, non-shrink additive can be used to reduce breeding and grout shrinkage. The resistance at grout-soil interface of nail will significantly reduce when the grout shrinks.
6.2.3 Shotcrete

Shotcrete can be continuous flow of mortal or concrete mixes projected at high speed perpendicularly onto the exposed ground surface by means of pneumatic air blowing for dry mix or spraying for wet mix. The high speed shooting mortal or concrete can produce self-compacted cementitious mortar as the facing. The water cement ratio of shotcrete mix is normally range from 0.35 to 0.5. Chemical curing compound or wet gunny sack can be normally used for curing of shotcrete. Sometimes, admixture can be used to speed up the setting time of the shotcrete. The ground surface shall be conditioned before receiving the shotcrete. In general, the surface shall be trimmed to reasonably smooth surface without loose materials and seepage. The ground surface shall be maintained at moisture equilibrium between the soil and the shotcrete.

6.2.4 Centralizers and Bearing Plate

Centralizers shall be fabricated from plastic, stainless steel, or other materials which are non-detrimental to the nail. The bearing plate shall be fabricated from mild steel and not smaller than 200 mm x 200 mm x 20 mm thick. All metal components shall be hot-dipped galvanized to produce a minimum coating thickness of 50 µm. Threads of the nails and nuts shall be cleaned by centrifuging, brushing or similar process after galvanizing. Care shall be taken during transportation handling storage and installation of the nails to prevent damage to the galvanizing.

6.2.5 Grout

Grout should comply with the requirements specified in Table 4.2.
6.3 Construction Equipments
The following equipments are necessary for soil nailing work.

6.3.1 Drilling Equipment
There are few common types of drilling equipment used to install the nails, namely rotary air-flushed and water-flushed, down-the-hole hammer, tri-cone bit. It is important to procure drilling equipment with sufficient power and rigid drill rods.

Fig. 6.7 Drilling Equipment

6.3.2 Grout Mixing Equipment
In order to produce uniform grout mix, high speed shear colloidal mixer should be considered. Powerful grout pump is essential for uninterrupted delivery of grout mix. If fine aggregate is used as filler for economy, special grout pump shall be used.

Fig. 6.8 Grout Mixing Equipment
6.3.3 Shotcreting Equipment
Dry mix method will require a valve at the nozzle outlet to control the amount of water injecting into the high pressurised flow of sand/cement mix. For controlling the thickness of shotcrete, measuring pin shall be installed at fixed vertical and horizontal intervals to guide the nozzle man.

![Fig.6.9 Shotcreting Equipment](image)

6.3.4 Compressor
The compressor shall have minimum capacity to delivered shotcrete at the minimum rate of 9 m³/min. Sometimes, the noise of compressor can be an issue if the work is at close proximity to residential area, hospital and school.

![Fig.6.10 Compressor](image)
6.4 Summary
A detail discussion is presented on the different construction stages of nailed slope. It includes excavation, drilling of nail holes, nail installation and grouting, construction of temporary shotcrete facing, construction of subsequent levels and permanent facing. Specifications are also given for different nailing construction materials e.g. steel reinforcement, grout mix, shotcrete, centralizers and bearing plate. The equipments involved in construction of the nailed slope are also discussed.
CHAPTER 7
FIELD INSPECTION AND MONITORING

7.1 Introduction

Field inspection and monitoring is an important part in construction of soil nailing system. This includes minute inspection for quality control of the construction materials and checking of their required specifications, continuous monitoring of construction work. After construction, the performance (deformation, load carrying capacity, etc.) of nailed slope is also monitored for improvement of future construction and design of such structures. This chapter presents a detailed discussion on the field inspection of various construction materials and stages in soil nailing and monitoring process of nailed structure.

7.2 Field quality control of construction materials

Field quality control of construction materials is necessary to maintain the material specifications as per the design. The quality of material is controlled by following methods:

- Visual examination of materials for defects in workmanship, contamination or damage from handling.
- Checking for the certification of the supplier or manufacturer whether the materials comply with the specification requirements.
- Laboratory testing of representative samples from the supplied sample

The details of quality control of construction material are given here along with a check list in Table 7.1.

- **Checking for specification** - Steel components (soil nail tendons, bearing plates, nuts, washers, and facing reinforcement steel), centralizers, and drainage materials are generally accepted based on satisfactory Mill Test Certificates. Sometimes they are checked by testing for compliance with the specifications. Nail tendon centralizers are also checked to confirm whether they are fabricated with the specified material and of correct diameter. Visual checking is carried out for all soil nail tendons and
reinforcing steel for damage and defects upon delivery and prior to use. A special attention is given in checking of epoxy coated or encapsulated tendons (corrosion protected tendons) for voids in the grout fill placed in the annular portion between the tendon and corrugated tube. This check is accomplished by lightly tapping the encapsulation with a steel rod and listening for hollow sounds that indicate the presence of void. The exterior of epoxy coatings or corrugated encapsulation is visually examined for damage, both upon delivery and prior to insertion into the soil nail drill hole. Mix design for grouting and shotcreting is checked for design strength and specification.

- **Storage and handling**- Cement to be used in grouting are stored in a dry place for avoiding lump formation. Steel reinforcements are handled carefully and kept away from the ground to protect those from corrosion or rusting. Geocomposite drainage materials are protected from dirt and sunlight.

Table 7.1. Check-list for quality control of construction material in soil nailing

<table>
<thead>
<tr>
<th>Sl No.</th>
<th>Specification to be verified</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Steel components (soil nail tendons, bearing plates, nuts, washers, and facing reinforcement steel), centralizers, and drainage materials should be accepted based on satisfactory Mill Test Certificates and obtain samples for testing (when specified) for compliance with the specifications.</td>
</tr>
<tr>
<td>2</td>
<td>Visual check for all soil nails tendons and reinforcing steel for damage and defects upon delivery and prior to use.</td>
</tr>
<tr>
<td>3</td>
<td>Visual check of epoxy coated or encapsulated tendons for compliance with the specifications and for any damage to the corrosion protection.</td>
</tr>
<tr>
<td>4</td>
<td>Mix design of soil nail grout and facing shotcrete should be according to the specification. When specified, grout (cubes) and/or shotcrete samples (test panels and cores) are collected for testing.</td>
</tr>
<tr>
<td>5</td>
<td>Verification for specification of geocomposite drainage materials with the contract one.</td>
</tr>
<tr>
<td>6</td>
<td>Proper field storage of construction materials (cement, reinforcement steel and drainage material) to prevent damage or degradation.</td>
</tr>
</tbody>
</table>
7.3 Construction monitoring

During the construction of soil nailing, it is important to monitor whether it is carried out as per the design specification and some general guideline. These are mentioned in the check list given in Table 7.2.

<table>
<thead>
<tr>
<th>Sl No.</th>
<th>Construction work to be monitored</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td><strong>Excavation</strong></td>
</tr>
<tr>
<td></td>
<td>• At the starting, check is required for any variance between the actual ground surface elevations along the slope face and those shown on the plans.</td>
</tr>
<tr>
<td></td>
<td>• Frequent checking should be carried out to assure that stable excavation conditions are maintained, both for general mass excavation and slope face excavation. Daily inspection is required for the ground next to the slope.</td>
</tr>
<tr>
<td></td>
<td>• Verification of excavations whether they are constructed within specification tolerances of the design outline.</td>
</tr>
<tr>
<td></td>
<td>• Over excavation should be avoided at each excavation step.</td>
</tr>
<tr>
<td>2</td>
<td><strong>Soil nail hole drilling</strong></td>
</tr>
<tr>
<td></td>
<td>• Soil nail hole should be drilled within acceptable tolerances of the specified alignment, length, and minimum diameter.</td>
</tr>
<tr>
<td></td>
<td>• Check for loss of ground or any drill hole interconnection.</td>
</tr>
<tr>
<td>3</td>
<td><strong>Tendon installation</strong></td>
</tr>
<tr>
<td></td>
<td>• Tendons should be inserted to the minimum specified length.</td>
</tr>
<tr>
<td></td>
<td>• Verification for the specified intervals during installation of the centralizers. Centralizers should provide clearance for the minimum specified grout cover and that openings through the centralizer support arms should not obstructed.</td>
</tr>
<tr>
<td></td>
<td>• During handling and installation of tendon, a careful monitoring is required to prevent damage to the corrosion protection.</td>
</tr>
<tr>
<td>4</td>
<td><strong>Grouting</strong></td>
</tr>
<tr>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>• Grout should be injected by the tremie pipe at bottom of the hole, and the end of tremie pipe should always remain below the level of grout as it is extracted.</td>
<td></td>
</tr>
<tr>
<td>• Grout should continue to be pumped as the grout tube, auger, or casing, is removed.</td>
<td></td>
</tr>
<tr>
<td>• Measurement and record of the volume of grout placed in the hole</td>
<td></td>
</tr>
<tr>
<td>• Auger rotation should not be reversed while grouting by auger-cast methods, except as necessary to initially release the tendon.</td>
<td></td>
</tr>
<tr>
<td>• “Auger-cast” or “cased” nails have been installed with the specified tendon length grouted.</td>
<td></td>
</tr>
<tr>
<td>• Checking should be done whether grout is batched in accordance with approved mix designs and the required grout strength test samples have been obtained in accordance with the specifications.</td>
<td></td>
</tr>
<tr>
<td>• Verification for the bonded and unbonded lengths of test nails.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>5</th>
<th><strong>Slope facing and drainage</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>• Geocomposite drain strips and weep hole outlet pipes should be installed as specified and checking is required for interconnection and continuous drainage paths of drain elements.</td>
<td></td>
</tr>
<tr>
<td>• Reinforcing steel should be installed at appropriate locations as per the dimensions specified and care should be given to ensure that it is tied securely and is clean.</td>
<td></td>
</tr>
<tr>
<td>• Slope face finish line and grade should be in accordance with the plans and specifications.</td>
<td></td>
</tr>
<tr>
<td>• Shotcrete should be batched in accordance with the approved mix design and applied as per the specification</td>
<td></td>
</tr>
<tr>
<td>• Construction joints should be clean and acceptable for shotcrete placement.</td>
<td></td>
</tr>
</tbody>
</table>
7.4 Load testing of nails

During construction, soil nails are load tested in the field to verify whether nail design loads can be sustained without excessive movement and with required factor of safety. Such testing is also required for ensuring adequacy of the drilling, installation, and grouting operations during construction of the soil nail wall. In practice, field testing of each row of nails are completed prior to excavation and installation of the underlying row. The load tests carried out for soil nails in the field are described in subsequent sections.

7.4.1 Verification or ultimate load test

This test is conducted to compare the pullout capacity and bond strengths used in design and as observed from the installation. Verification load tests are performed on non-production, “sacrificial” nails prior to the construction. The tests are continued up to failure or, as a minimum, to a test load that includes the design bond strength and pullout factor of safety. The number of verification test to be performed depends on size of the project and ground variability. Two verification tests are recommended for each soil strata encountered. The field test set-up for verification or ultimate load test of soil nail is presented in Fig. 7.1.

Fig. 7.1 Field test set-up for verification or ultimate load test of soil nail

(Zhu et al. 2007)
The criterion for acceptance in verification test is that-

a) No pullout failure should occur at 200 percent of the design load where pullout failure is defined as the load at which attempts to further increase the test load increments results in continued pullout movement of the tested nail.

b) The total measured movement (\( \Delta L \)) at the test load of 200 percent of design load must exceed 80 percent of the theoretical elastic movement of the unbonded length (from back of the reference plate to the top of the grout length). This is expressed as 
\[ \Delta L \geq \Delta L_{\text{min}} \]
where \( \Delta L_{\text{min}} \) is the minimum acceptable movement defined as:

\[
\Delta L_{\text{min}} = 0.8 \frac{P L_u}{E A}
\]  
(7.1)

Where 
\( P = \) Maximum applied test load 
\( A = \) Cross-sectional area of the nail bar 
\( E = \) Young’s modulus of steel 
\( L_u = \) Unbonded length

### 7.4.2 Proof tests

Proof tests are conducted during construction on a specified percentage (5%) of the total production nails installed. Such tests are performed to confirm the uniformity of construction procedure and to assure that the nails have not been drilled and grouted in a soil zone not tested by the verification stage testing. Soil nails are proof tested to a load typically equal to 150 percent of the design load. The acceptance criterion requires no pull-out failure during the loading and the displacement (\( \Delta L \)) under 150 percent of the design load should exceed the minimum acceptable movement (\( \Delta L_{\text{min}} \)) as calculated from Eq. 7.1.

### 7.4.3 Creep test

Creep tests are performed as part of ultimate, verification, and proof testing to ensure that the nail design loads can be safely carried throughout the structure’s service life. In such test, soil nail displacement is measured at a constant load over a specified period of time. The deflection-versus-log-time results are plotted on a semi-log graph, and they are compared with the acceptance criteria presented in the construction specification. Acceptance criteria typically requires that creep movement between the 1- and 10-minute readings, at maximum...
test load, must be less than 1 mm or that the creep movement between the 6-and 60-minute readings must be less than 2 mm at maximum test load.

7.5 Performance monitoring

Nailed slopes are often instrumented to monitor several parameters for improvement of future construction and design of such structures. The parameters monitored are as follows

- Horizontal and vertical movement of slope face, surface and overall structure
- Performance of any structure supported by the reinforced ground
- Deterioration of facing and other soil nailing elements
- Nail force magnitude, maximum nail force and change of force distribution with time
- Drainage behavior of ground

Slope inclinometer, electronic distance measuring equipments are installed at various survey positions on the nailed structure and load cells are installed at nail head for such monitoring purpose. Table 7.3 presents different monitored parameters along with their monitoring plan.
Table 7.3 Monitoring plan for various soil nail parameters

<table>
<thead>
<tr>
<th>Soil nail parameters</th>
<th>Monitoring plan</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Monitoring during construction:</strong></td>
<td></td>
</tr>
<tr>
<td>Face horizontal movements</td>
<td>Using surface markers on the facing and surveying methods, and inclinometer casings installed a short distance (typically 1m) behind the facing</td>
</tr>
<tr>
<td>Vertical and horizontal movements of the top of slope facing and surface of the retained ground</td>
<td>Using optical surveying</td>
</tr>
<tr>
<td>Ground cracks and other signs of disturbance in the ground surface behind the top of slope</td>
<td>Daily visual inspection during construction and if required installation of crack gauges across the cracks</td>
</tr>
<tr>
<td>local movements and or deterioration of the facing</td>
<td>Using visual inspections and instruments like crack gauges or electronic distance measuring (EDM)</td>
</tr>
<tr>
<td>Drainage behavior of the structure</td>
<td>Monitored visually by observing outflow points or through standpipe piezometers installed behind the facing</td>
</tr>
<tr>
<td><strong>Long-term Monitoring</strong></td>
<td></td>
</tr>
<tr>
<td>Magnitude and location of the maximum nail load</td>
<td>Monitored through installation of strain gauge along the length of nail (Fig. 7.2). Strain gauges are generally attached to the nail bar in pairs, and are mounted at 1.5-m spacing in diametrically opposite position to address bending effects.</td>
</tr>
<tr>
<td>Loads at the head of the nail</td>
<td>Load cells are attached at the head of nail</td>
</tr>
<tr>
<td>Horizontal movement of the structure</td>
<td>Inclinometers are installed on the ground surface behind the slope facing at various horizontal distance upto one time the slope height</td>
</tr>
</tbody>
</table>
A typical instrumentation layout for performance monitoring of a vertical nailed slope cut is depicted in Fig. 7.2.

![Diagram of instrumentation layout](image)

Fig. 7.2 A typical instrumentation layout for performance monitoring of a vertical nailed slope (Modified From: FHWA-SA-96-069R)

### 7.6 Summary

Various field inspection and monitoring methods for nailed slopes have been described. It includes quality control techniques for the construction materials and the process itself; and different nail load testing methods along with their acceptance criterion. Performance monitoring of nailed slopes are also discussed with various parameters to be monitored and proper instrumentation process.
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FHWA0-IF-03-017, 2003, Soil Nail Walls, Federal Highway Administration, Washington, D.C.


