CERTIFICATION OF APPROVAL

COMPILATION OF DESIGN METHODOLOGY FOR SOIL NAILING

by

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CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

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TABLE OF CONTENTS

LIST OF TABLES

Tables

LIST OF FIGURES

Figures

CHAPTER 1
INTRODUCTION

1.1 Background Study

Soil nailing is an in-situ technique for reinforcing, stabilizing and retaining excavation and slope. The basic concept of soil nailing is reinforces the existing ground by inserting a passive inclusion into the soil in a closely spaced pattern to increase the overall shear strength of the in-situ soil and restraint its displacement. The nails used in soil-nailing retaining structures are generally steel bars or other metallic elements that can resist tensile, shear stresses and bending moment. They are generally either placed in drilled boreholes and grouted along their total length or driven into the ground. The facing of the soil-nailed structure is to ensure the local stability of the soil between reinforcement layers and protects the ground from the surface erosions and weathering effects.

In soil nailing, similarly to ground anchors, the load transfer mechanism and the ultimate pull-out resistance of the nails depend primarily upon soil type and strength characteristics, installation technique, drilling method, size and shape of the drilled hole, as well as grouting method and pressure used.

The basic design concept of soil-nailed retaining structures relies upon the transfer of resisting tensile forces generated in the inclusions into the ground through friction at the interfaces. The design of any soil nail must consider internal, external and global stability.

1.2 Problem Statement

Soil nailing has gained popularity in Malaysia as slope stabilization as it is known as an effective slope stability method, ease of construction, cost effective and relatively maintenance free. The increasing use of soil nails as permanent structure is a key parameter in current technological developments. Durability of inclusions, long-term performance in fine-grained, and environmental/ architectural requirement for soil-nailed facing has become the major design considerations. It should be emphasized that systematic procedure of soil nail design is necessary to ensure soil nailing perform satisfactorily during its service life.

The design methods were proposed in Germany, the United States and Britain between late 1670's and 1980's. As of 2004, no universally design standard to be used by civil engineer or geotechnical engineer. Currently, in 2005 the United States has established design standard for designing soil nail structure. However, in Malaysia currently there is no design standard or procedure that has been agreed or accepts for design soil nail structure. All the design based on the suggestion from manufacturer or supplier of soil nailing (Tan, 2005).

1.3 Objectives and Scope of Project

The main objective of this project is to compile a manual of practice design, construction, quality control and monitoring of soil-nailed structures. Various design methods are presented and subsequently, recommendations are made for design method for soil nail to be adopted for Malaysian practice to ensure safe and economical design of soil nail in line with international practice. The deliverables of this project includes a study that requires the understanding of the available design methods of soil nailing.

CHAPTER 2
LITERATURE REVIEW

2.1 Description of soil nailing

The technique of soil nailing was first used in France to build a permanent retaining wall cut in soft rock in 1961. Since then, this technology has gained popularity in Europe, particularly France and Germany and continues to lead the world in soil nail technology (FHWA, 1998). It has been successfully utilized worldwide for excavation support, slope stabilization and highway project as shown in Figure 1 and its use continue to grow rapidly.

Use of soil nail construction is increasing in popularity in the United States, where it is used primarily for temporary and permanent support of building excavation and for highway projects. The Federal Highway Administration (FHWA) has implemented this technology on highway projects, such as road widening, since 1980s (FHWA, 1998).

Soil nailing is a method of construction that reinforces the existing ground. Passive inclusion (the nails) are inserted into the soil in a closely spaced, to create in-situ coherent gravity and thereby to increase overall shear strength of the in-situ soil and restrain its displacement.

Soil nailing technique to reinforce slope was introduced to Malaysia in early 1980s and of the early slopes reinforced by soil nailing was Bukit Jugra Army Camp slope in Banting in 1983. While Pos Betau-Ringlet Highway. A new JKR R3 hilly road of about 85 km is estimated to have about 55 000 soil nails to stabilize steep and high hilly cut slopes (Neoh, 2000).

The system consist of reinforced shotcrete facing constructed incrementally from the top down and array of inclusions grouted or driven into the soil mass. These inclusions resist tensile stresses, shear stresses and bending moments. Prefabricated panels or cast-in-place concrete can be subsequently constructed in front, or on the shotcrete facing, if aesthetic or durability considerations warrants the additional expenses.

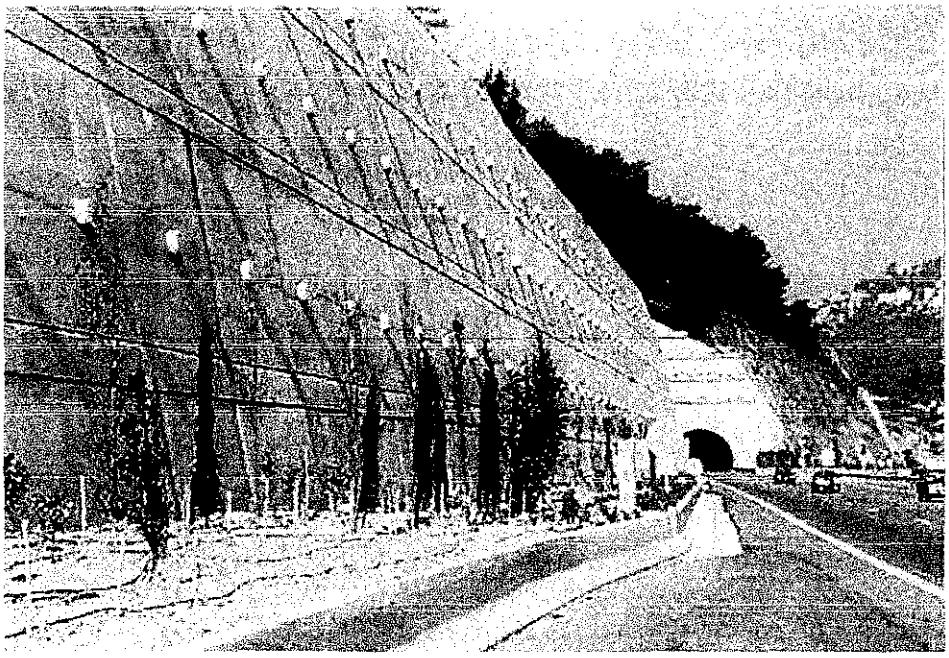

Figure 2.1: Soil nailing as slope stabilization for construction of highway

2.2 Soil Nail Application

Soil nail walls have been found to be an economical solution to many soil reinforcement and excavation support problems. The following section lists some of the typical applications for soil nail walls and some of their benefits (Soil Screw Manual, 2003).

- Alternative to Tieback Wall for Temporary or Permanent Excavation Support

  a. Eliminates the time and expenses of placing H-piles.

  b. Eliminates the labor associated with placing timber lagging or sheet pile

  c. Eliminates the need for expensive structural facing system.

- Alternative to Cast in Place Walls (CIP) in Cuts

  Cast-in-place walls in cuts will require temporary shoring and over excavation to be able to install wall footings. A soil nail wall requires no shoring and can use a smaller footing

- Repair and reconstruction of existing retaining wall

  Replacement and reconstruction of a failed timber or concrete crib wall, MSE wall, gabion wall, or CIP wall is very expensive. An alternative is to reinforce the failed wall with soil nails and replace or repair the facing. This eliminates a very expensive construction step of excavating the failed wall, especially if the wall is supporting another structure

- Roadway Widening under Existing Bridges

  Soil nail walls can eliminate construction steps associated with temporary and permanent walls needed for widening roadways adjacent to existing highway bridges. Soil nail walls can be combined with permanent facings, thus providing a permanent wall for support of bridge fills without the need for temporary shoring by using top down construction sequence

- Land Remedian

  Soil nail walls can be used to reinforce failed slopes and walls in-situ. Soil nails must be drilled beyond the failure surface to a depth great enough to mobilize the nail tensile strength. This analysis is similar to the design of a reinforced fill slope, however, soil nails enable this remediation to be performed in-situ without removal and replacement

2.3 Advantages and Disadvantages

Hereafter, the advantages and disadvantages of soil nailing are briefly discussed:

2.3.1 Advantages

Soil nailing appears to have unique technical and economic advantages over more conventional cut retention technique. These include:

- Reported lower cost due to relatively rapid installation of the unstressed inclusion (nails) which are considerably shorter than earth anchors and relatively thin shotcrete or concrete facing.
- Only light construction equipment is required to install nails as well as simple grouting. Grouting of the borehole is generally accomplished by gravity. This feature may be of particular importance for sites with difficult access.
- Since there are a large number of nails, failure of any one may not detrimentally affect the stability of the system, as would be the case for a conventional tieback system.
- In heterogeneous soils with cobbles, boulders and weathered zones or hard rock zones, it offers the advantages of the small diameter shorter drill holes for nails installation and eliminates the need for the soldier pile installation which is disproportionately costly to install under these condition.
- Soil nailed structure is more flexible than conventional rigid structure. Consequently this structure can conform to the surrounding ground and withstand greater total and differential ground movement in all directions
- Surface deflection can be controlled by the installation of additional nails or stressing in the upper level of nails to a small percentage of their working loads.
- Allow in-situ strengthening on existing slope surface with minimum excavation and backfilling, particularly very suitable for uphill widening, thus environmental friendly.
- The long-term performance of shotcrete facing has not been fully demonstrated particularly in areas subject to freeze-thaw cycles.

2.3.2 Disadvantages

Soil nailing shares with other cut retaining techniques the following disadvantages:

- Groundwater drainage system may be difficult to construct and their long term effectiveness is difficult to ensure.
- In urban areas, the closely spaced array of reinforcements may be interfering with nearby utilities. In addition, horizontal displacement may e somewhat greater than with prestressed tiebacks which may cause distortions to immediately adjoining structure.
- Nail capacity may not be economically develop in cohesive soils subject to creep, even at relatively low load level.
- Generally larger lateral soil strain during removal of lateral support and ground surface cracking may be appearing.
- Less suitable for course grained soil and soft clayey soil which have short self support time, and soil prone creeping.

2.4 Behaviour of Soil Nailing

The basic design concept of soil nailing is to reinforce and strengthen the slopes insitu by installing grouted steel bars or driven pipes, called "nails", into progressively excavated slope/wall by the "top down" process. This process can create a reinforced mass that is internally stable and able to retain the ground mass against active pressure, sliding, bearing and overturning forces (Neoh, 2000). The reinforcements are passive and can develop their reinforcing action through the nail-soil interaction as the slopes deform during and subsequent to construction. Soil nails works predominantly in tension but may develop some bending or shear in certain circumstances when internal strain or deformation is too large (FHWA, 1998).

The tensile forces are developed in the soil nails primarily through the frictional interaction between the soil nails and the ground, and secondarily through the interaction between the soil-nail heads/facing and the ground. The later phenomenon facilitates the development of tension in soil nailing. They also prevent the local failures near the slopes and promote an integral action of the reinforced mass through redistribution of forces among soil nails (GEO, 2006).

All potential failure modes must be considered in evaluating the available nail force to stabilize the active block defined by any particular slip surface.

The failure modes of soil nails can be categorized into the following (Tan & Chow, 2004b):

a) Pullout failure
b) Nail tendon failure
c) Face failure
d) Overall failure (slope instability)

2.4.1 Pullout Failure

This failure results from insufficient embedded length into the resistant zone to resist the destabilizing force. The pullout capacity of the soil nails is governed by the following factors (Tan & Chow, 2004b):

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

2.4.2 Nail Tendon Failure

This failure results from inadequate tensile strength of the nails to provide the resistant force to stabilize the slope. It is primarily governed by the grade of steel used and the diameter of the steel (FHWA, 1998).

2.4.3 Face Failure

This aspect of failure mode for soil nailing is sometimes overlooked as it is generally wrongly "assumed" that the face does not resist any earth pressure (Tan & Chow, 2004b). These failures tend to be fail in either facing failure or the front if nails zone sliding off (FHWA, 1998).

2.4.4 Overall Failure

This aspect of failure mode is commonly analyzed based on limit equilibrium methods. The analyses are carried out iteratively until the nail resistant force corresponds to the critical slip plane from the limit equilibrium analysis. To carry out such iterative analysis, it is important that the nail load diagram (Figure 2.2) is established (Tan & Chow, 2004b).

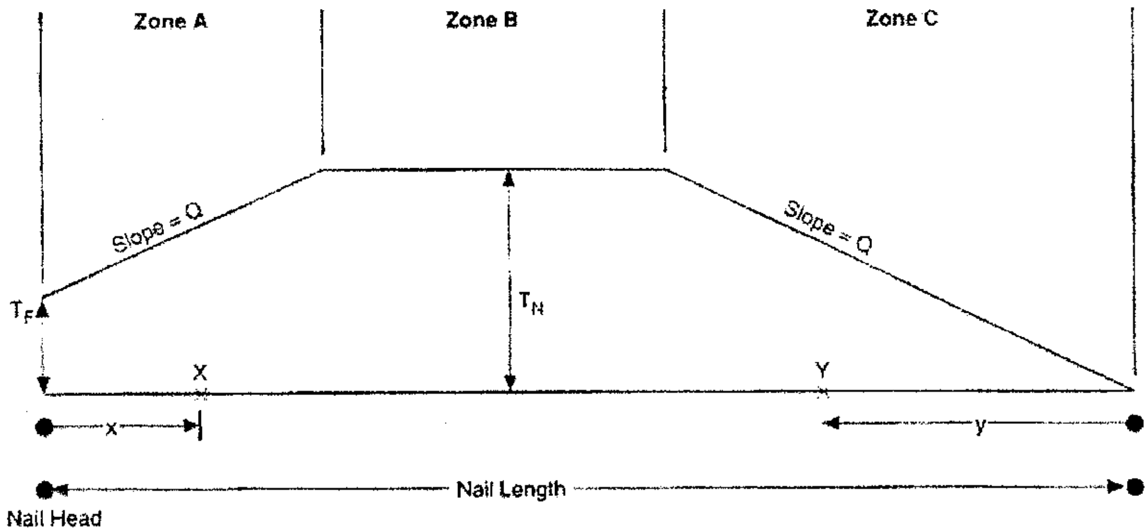

Figure 2.2: Nail load diagram (from FHWA,1998)

2.5 Site Investigation

The feasibility of constructing a soil nailed wall on a project depends on the existing topography, subsurface conditions, soil/rock properties, and the location and condition of adjacent structures (Soil Screw Manual, 2003). It is, therefore, necessary to perform a comprehensive site investigation to evaluate site stability, adjacent structure settlement potential, drainage requirements, anchor capacities, underground utilities and groundwater, before designing a soil nailed earth retention system.

Subsurface investigations must explore not only the location of the face of the soil nailed structure, but the region of the anticipated bond length of the nail (Soil Screw Manual, 2003). Each project must be treated separately, as both the soil conditions and risks may vary widely. A well-planned site investigation should include a review of the regional geology, a field reconnaissance, a subsurface exploration and laboratory testing. The site investigation should provide adequate information to design a stable soil nailed system.

2.5.1 Regional Geology

A review of the regional geology should be performed prior to conducting a field reconnaissance or subsurface exploration to better understand the geology and groundwater conditions of the region. The information acquired in this first phase of the site evaluation will be used to further develop the field reconnaissance and subsurface exploration (FHWA, 1998). Information concerning the regional geology may be obtained from geologic maps, air photographs, surveys and soils reports for adjacent or nearby sites

2.5.2 Field Reconnaissance

Field reconnaissance should be conducted by a geotechnical engineer or by an engineering geologist. A well planned and conducted field reconnaissance should consist of collecting any existing data relating to the subsurface conditions and making a field visit to (FHWA, 1991):

Select limits and intervals for topographic cross-sections.

- Observe surface drainage patterns, seepage and vegetative characteristics to estimate drainage requirements. Corrosion of existing drainage structures should be noted to identify if a corrosive environment may exist for shotcrete and/or steel materials.

- Study surface geologic features including rock outcroppings and landforms. Existing cuts or excavations should be used to identify subsurface stratification.
- Determine the extent, nature, and situation of any above or below ground utilities, basements and/or substructures of adjacent structures which may impact explorations or construction.
- Assess available right-of-way.
- Determine areas of potential instability, such as deep deposits of weak cohesive and organic soils, slide debris, high groundwater table, bedrock outcrops, etc.

2.5.3 Subsurface Exploration

Subsurface exploration should be sufficiently broad to fully evaluate the soil stratigraphy in the zones affected by nailed wall construction, develop sufficient stability analyses, estimate the pullout capacity of the nails and develop sufficient information to design an efficient internal drainage system (FHWA, 1991).

2.6 Preliminary Feasibility Assessment

Based on the results of the site investigation, a preliminary feasibility evaluation can be made to determine if a successful soil nail design can be implemented with a relatively high degree of confidence. The ground conditions for which soil nailing is well suited and the ground conditions that are problematic are presented in the following sections.

Soil types suitable for soil nail (FHWA, 1991):

- Most residual soils and weathered rock mass without adverse geological settings exposed during staged excavation
- Talus slope deposit
- Silts
- Clay with low plasticity that are not prone to creep

- Naturally cemented sands and gravel
- Heterogeneous and stratified soils
- Stiff/cohesive soils
- Well graded granular soil with sufficient apparent cohesion of minimum 5kPa as maintained by capillary suction with appropriate moisture content
- Ground profile above groundwater level

Soil not conductive to soil nail;

- soft plastic clay
- peat/organics soils
- loose, low density and/or saturated soils
- coarse sands and gravel that are uncemented or lack capillary cohesion

2.7 Data required in Soil Nail Design

To perform a soil nail wall design, knowledge of the soil behind the wall face and the foundation soils supporting the wall (Figure 2.2) is required. It also requires knowledge of the project geometry, loading and surcharge conditions, groundwater conditions, and the properties of the soil nails.

2.7.1 Soil parameter

Since a soil nail wall is comprised of over 98% soil, the characteristics of that soil (shear strength, consolidation, permeability, corrosion potential) will greatly influence the soil nail design and the wall performance.

i) Soil Shear Strength

The shear strength of the retained soil must also be determined since this will determine what load will be applied to the back of the soil nail wall. The shear strength of the foundation soil will determine what length the soil nails will need to be to resist bearing and sliding failure modes for a wall of a given height (Soil Screw Manual, 2003)

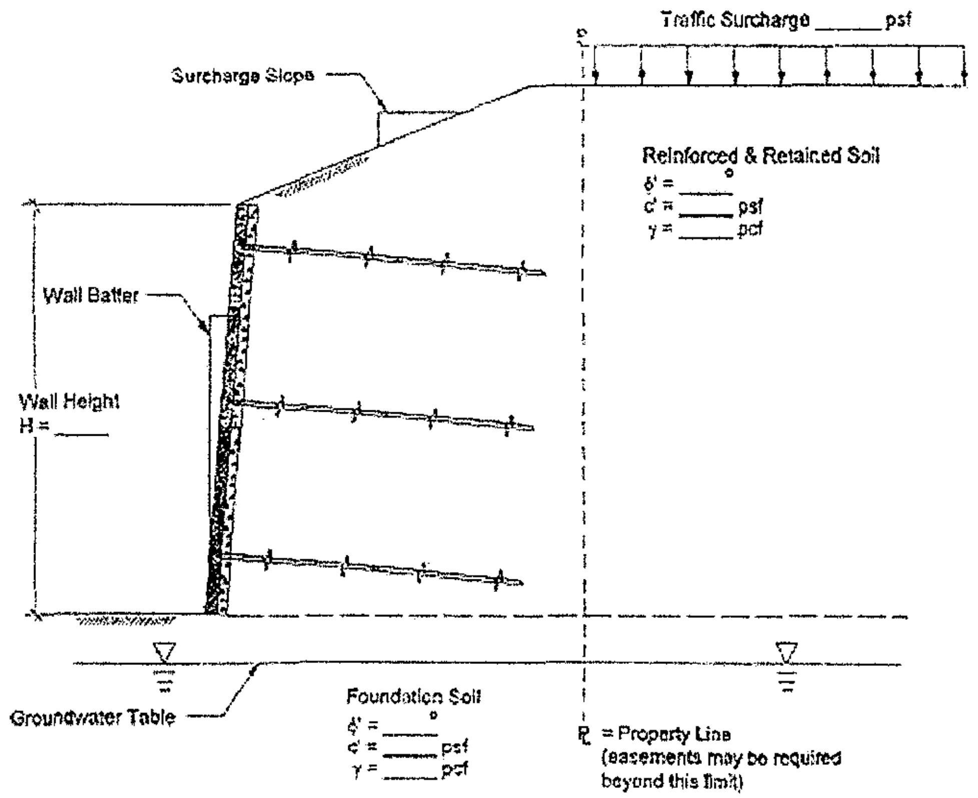

Figure 2.3: Input data required for design of soil nail (from: www.abchance.com)

i) Soil Shear Strength

According to Soil Screw Manual, 2003, the shear strength of the retained soil must also be determined since this will determine what load will be applied to the back of the soil nail wall. The shear strength of the foundation soil will determine what length the soil nails will need to be to resist bearing and sliding failure modes for a wall of a given height

The two components that make up the effective shear strength, s', of a soil are the internal friction angle (Ø') and cohesion, c', of the soil as represented in the equation:

$s' = c' + \sigma' \tan Ø'$

where : σ' = effective normal stress on plane of shearing

Equation is referred to as *Mohr – Coulomb failure criterion.* The value for c' for sands and normally consolidated clays equal to zero. For overconsolidated clays, $c' > 0$ (Das, 2006).

It is important to accurately determine the friction angle of the reinforced soil, retained soil and foundation soils. The friction angle of the soil is best determined from consolidated undrained triaxial compression tests which measure pore water pressures and drained direct shear tests performed at rates slow enough to ensure that pore water pressure does not occur during the test. The friction angle of a soil can also be estimated from direct shear, grain size analyses, standard penetration testing and cone penetration testing for preliminary designs, but is best determined from actual laboratory or field testing for final designs.

ii) Consolidation / Creep

When stress on saturated clay layer is increased, pore water pressure in the clay increase. Gradual increase in the effective stress in the clay layer will cause settlement over a period of time. (Das, 2006)

The tendency of a soil nail to creep in soil will be a function of the consolidation characteristics of the soil being reinforced. In general, if the soil is fine grained, the potential for soil nail movements in the long term is greater than that for granular soils [3]. For permanent soil nail applications, soil nailing should not be performed in soils with moderate to high plasticity, such as soils classified as MH or CH, and caution should be used for temporary applications (Soil Screw Manual, 2003).

Das (2006) point out that ASTM Test Designation D-2435 is a test to determine the consolidation settlement caused by various incremental loading.

iii) Soil Corrosion Potential

Durability considerations require an evaluation of the aggressiveness of the ground and pore water, particularly when field observation indicates corrosion of existing structures. The soil tests most commonly used to evaluate ground aggressiveness are electrical resistivity, pH, and sulfates nad chlorides concentration. The critical values for ground aggressiveness commonly associated with ASTM standards are summarized in Table 1.

Table 2.1 Recommended Electrochemical Properties for Soils when using soil nail (from www.abhance.com)

Test	**ASTM Standard**	**Critical values**
Resistivity	G-57-78 (ASTM)	Below 2000 ohm/cm
pH	G-51-77 (ASTM)	Below 4.5
Sulfates	California DOT test 407	Above 500 ppm
Chlorides	California DOT test 422	Above 100 ppm

2.7.2. Surcharges and Loading Conditions

To accurately perform stability analyses for a soil nail wall, the geometry of the wall cross section is required. This includes the slope at the toe of the wall, the top of the wall and the wall batter (if any). Other surcharge loads can include dead and live loads such as:

- Traffic Surcharges
- Railroad Surcharges

- Buildings
- Tiered Walls
- Construction Equipment during and after construction
- Earthquake Loading
- Rapid Drawdown Conditions
- Traffic Barriers, Sound Walls, Bridge Loadings, Lateral Load from Piles
- Blasting

2.7.3 Drainage and Groundwater Condition

The location of the permanent groundwater table is critical to a successful design. Soil nailing is best suited to applications above the water table (Juran & Elias, 1990). Excess seepage that cannot be controlled by strip drains during construction can deteriorate the excavated face, prevent shotcrete from bonding with the soil and provide excess pressure on the wall face. Therefore, soil nailing may not be feasible in areas where a permanent phreatic surface exists in the proposed wall volume.

Seepage from surface infiltration can be controlled with well-designed drains (Figure 2.3), such as a lined interceptor ditch placed at the top of the wall and a subsurface drain placed inside the wall face

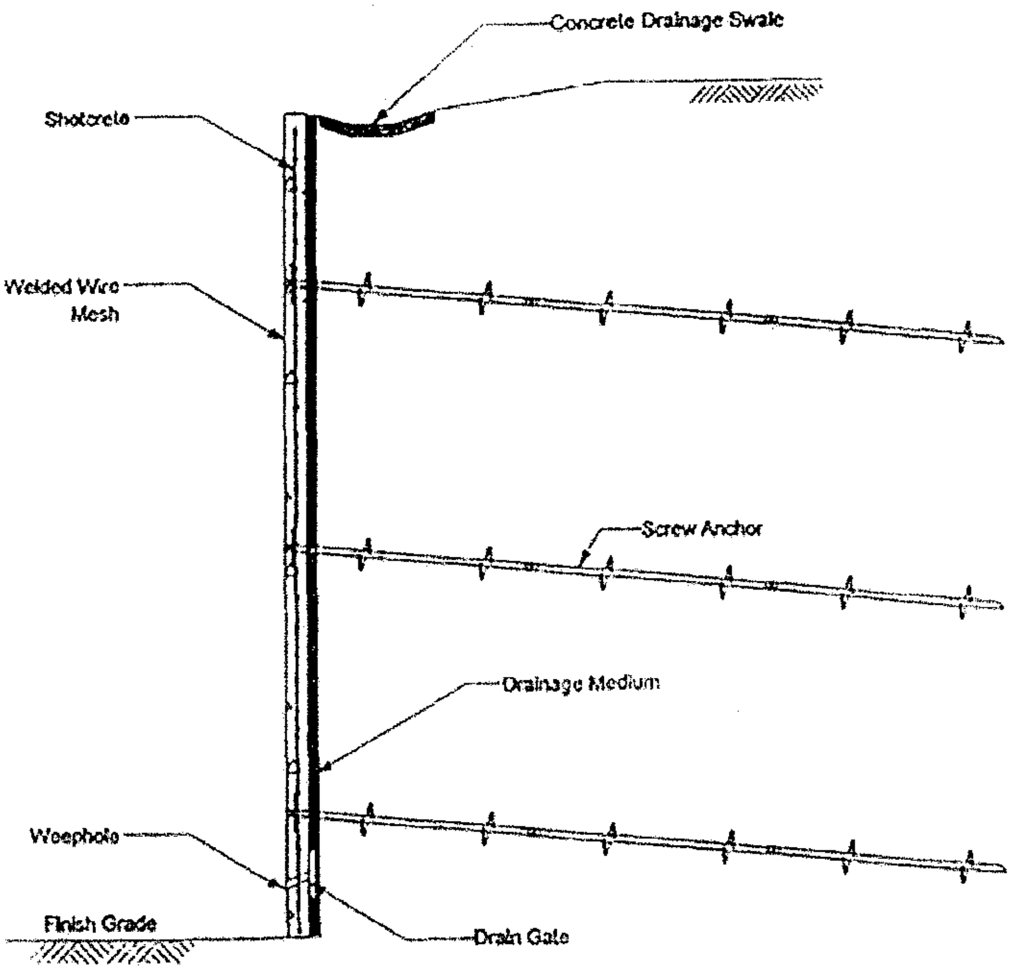

Figure 2.4: Concrete Drainage Swale (*from: www.abchance.com*)

2.7.4 Facing consideration

Prior to design, the type of facing for temporary and permanent walls needs to be identified (Soil Screw Manual, 2003). While shotcrete facing is most commonly used, depending upon the site conditions and the ultimate wall batter or slope, there are other options that may be desirable

i) Temporary Facing

Temporary facing systems that can be used include shotcrete and welded wire mesh; welded wire mesh, steel channels and geotextiles; and timber shoring.

The most effective is shotcrete, since it creates a bond with the soil and fills in voids which may develop due to sloughing of soil at the wall face (Juran & Elias, 1990).

ii) Permanent Facing

Permanent facing systems that can be used with the soil nail system including reinforced shotcrete, cast-in-place and precast concrete panels, concrete masonry segmental wall units, and gabions.

These facings must be designed to structurally support the soil loading applied between soil nails and be attached with a connector that is strong enough to resist punching failure of the nail at the wall face. The design of the permanent shotcrete or concrete facing for flexural stiffness and punching is adequately covered in FHWA-SA-96-069.

For soil nailed slopes where the slope facing is stable without reinforcements, i.e., the soil nails are being used to increase the deep seated slope stability, a facing consisting of an erosion mat and vegetation consistent with the area can be utilized.

2.8 Designs Method in Designing Soil Nailing Structure

Various international codes of practice and design manuals such as listed below are available for design of soil nail (Tan & Chow, 2006):

a) British Standard BS8006: 1995, Code of Practice for Strengthened/Reinforced Soils and Other Fills.
b) HA 68/94, Design Methods for the Reinforcement of Highway Slopes by Reinforced Soil and Soil Nailing Techniques.
c) U.S. Department of Transportation, Federal Highway Administration (FHWA, 1998), Manual for Design and Construction Monitoring of Soil Nail Walls.

2.8.1 British Standard BS8006: 1995, Code of Practice for Strengthened/Reinforced Soils and Other Fills.

The design of soil nail is covered in Section 7.5: Reinforcement of existing ground in BS8006: 1995. In BS8006, the two-part wedge method and the log-spiral method is recommended for analyzing the stability of soil nailed slopes. The use of two-part wedge and log-spiral analysis for soil nailing is illustrated in Figures 2.3 and 2.4. While either two-part wedge and log-spiral method can be used to analyze soil nailed slopes, it is highlighted in BS8006 that there is evidence from full-scale observations indicating that log-spiral approach has produced reasonable agreement with actual structures and the use of log-spiral method provides a convenient platform for calculation when shear as well as tension in the nails are to be determined (Tan & Chow, 2006).

The method outline in BS8006: 1995 is based on the limit state principles with the use of partial factors of safety. The design of soil nailing requires that the risk of attaining ultimate limit and serviceability limit states are minimized with the appropriate use of partial factors of safety on loads, materials and economic ramification of failure.

The ultimate limit states which should be considered are (BS 8006):

a) External stability
   - Bearing and tilt failure, see Figure 2.5a
   - Forward sliding, Figure 2.5b
   - Slip failure around the reinforced soil block, Figure 2.5c

b) Internal stability
   - Tensile failure of the individual reinforcement elements, Figure 2.6a
   - Bond failure of the individual reinforcement elements, Figure 2.6b

c) Compound stability

- Tensile failure of the individual reinforcement elements, Figure 2.7a
- Bond failure of the individual reinforcement elements, Figure 2.7b

The serviceability limit states which should be considered are (BS 8006):

a) External stability

- Settlement of the slope foundation, see Figure 2.8a

b) Internal stability

Post-construction strain in the reinforcement, see Figure 2.8b. It is to be noted however, that in soil nailing, some movement of the nailed mass of earth is expected in order to generate the tensile and shear stresses needed for stability.

Other checks required by BS8006 include face stability to prevent erosion and to ensure load transfer in the active zone (Tan & Chow, 2006).

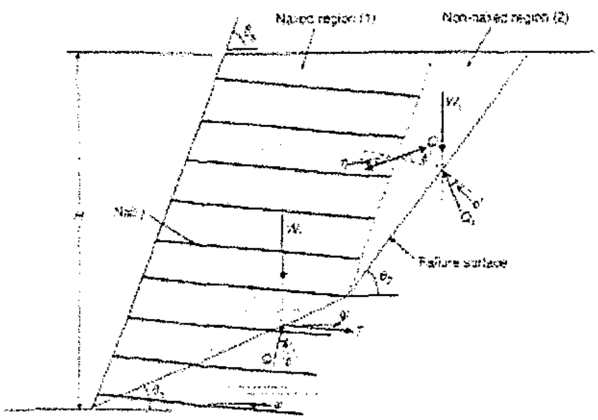

Figure 2.5: Use of two-part wedge analysis for soil nailing (from BS8006: 1995).

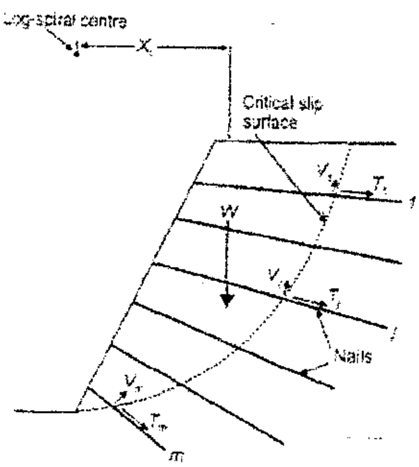

Figure 2.6: Use of log-spiral analysis for soil nailing (from BS8006: 1995).

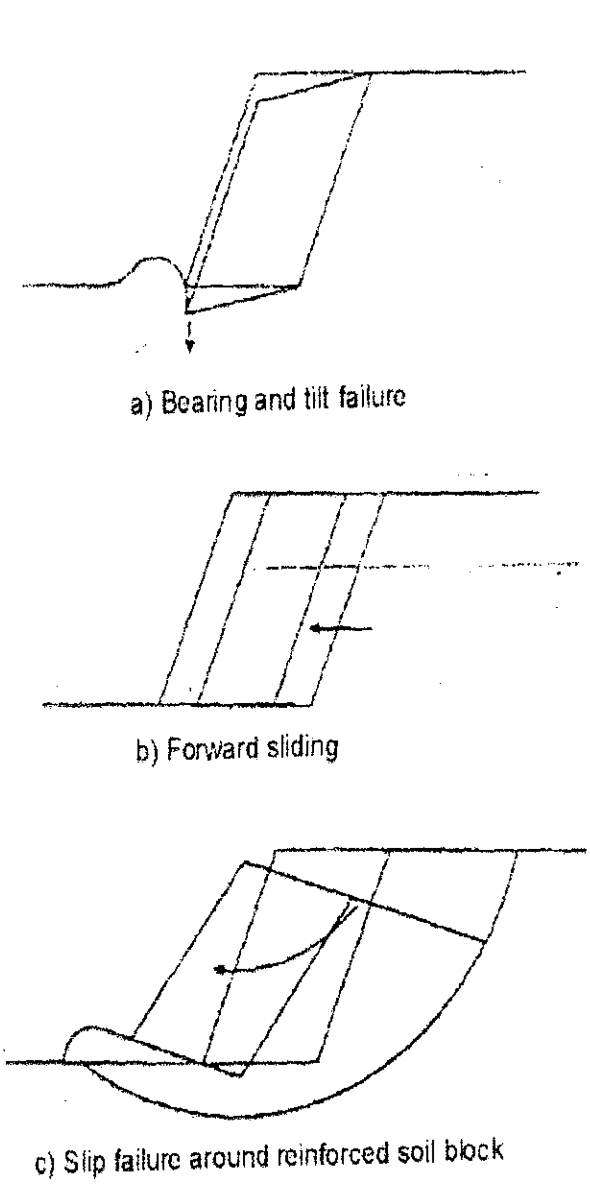

Figure 2.7: Ultimate limit states – external stability (from BS8006: 1995).

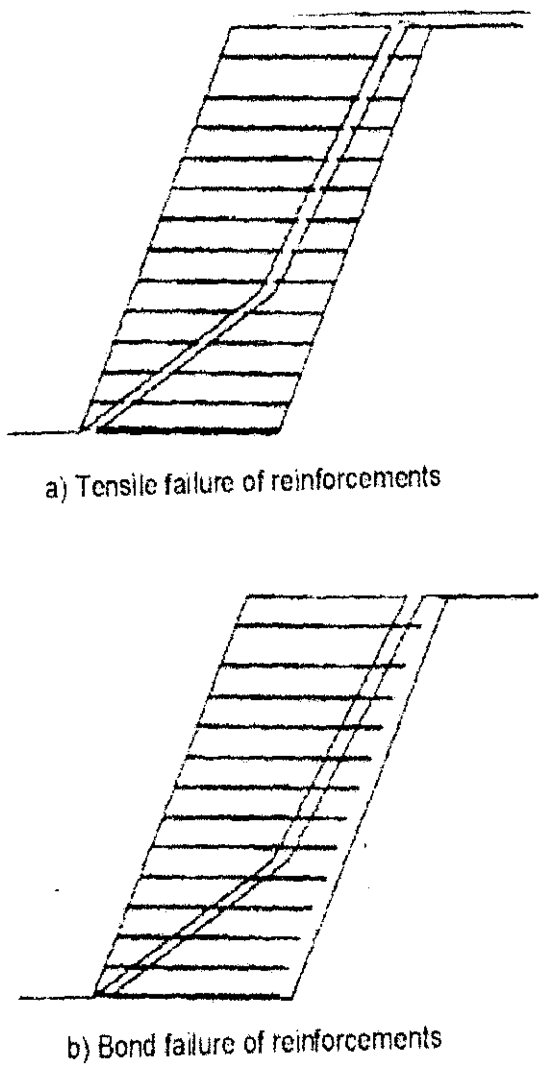

Figure 2.8: Ultimate limit states – internal stability (from BS 8006: 1995).

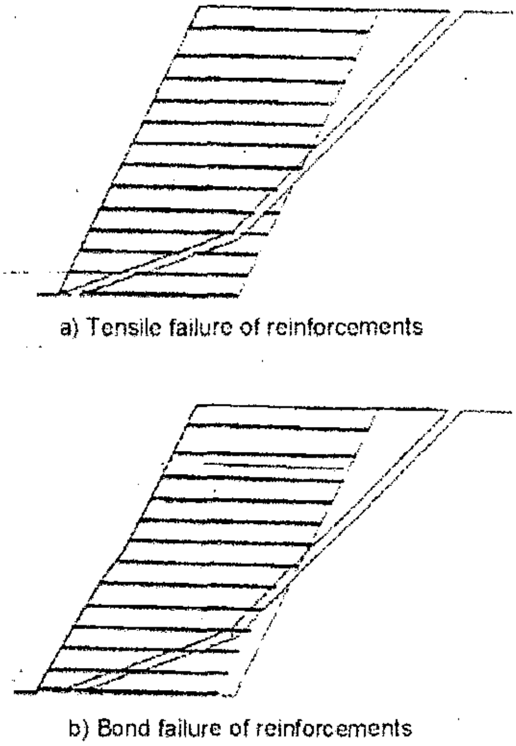

Figure 2.9: Ultimate limit states – compound stability (from BS8006: 1995).

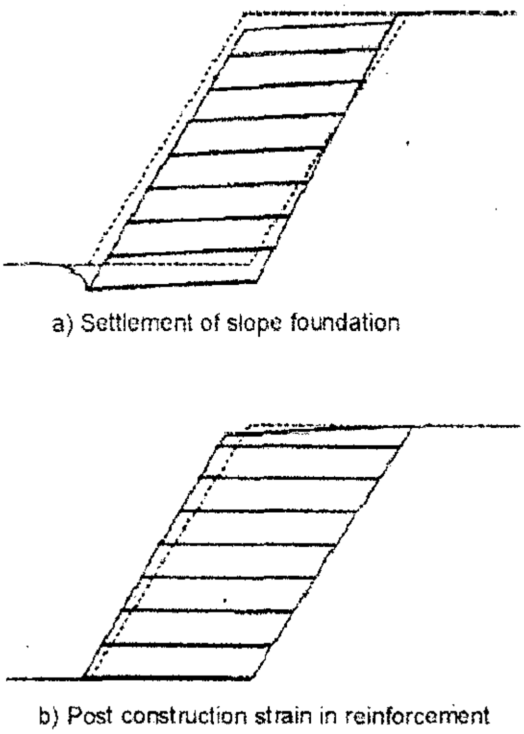

Figure 2.10: Serviceability limit states (from BS8006:1995).

2.8.2 HA 68/94, Design Methods for the Reinforcement of Highway Slopes by Reinforced Soil and Soil Nailing Techniques

The design method outlined in HA 68/94 is based on the two-part wedge mechanism which is similar to Figure 1. In HA 68/94, the two-part wedge method is preferred over the log-spiral method due to its simplicity even though it acknowledges that log-spiral is kinematically superior to the two-part wedge. The design procedures outlined in HA 68/94 is more specific compared to BS8006: 1995 such that it provides a step-by-step guidance for the design of soil nailed slope. In HA68/94, the design approach is categorized into two approaches for different applications of soil nail:

a) Type 1: Design of cuttings into horizontal ground (Figure 2.9).

2.8.3 FHWA, Manual for Design and Construction Monitoring of Soil Nail Walls

The FHWA soil nail design method provides a complete and rational approach towards soil nail design, incorporating the following elements (FHWA, 1998):

a) Based on slip surface limiting equilibrium concepts.
b) Incorporates the reinforcing effect of the nails, including consideration of the strength of the nail head connection to the facing, the strength of the nail tendon itself, and the pullout resistance of the nail-ground interface.
c) Provides a rational approach for determining the nominal strength of the facing and nail/facing connection system, for both temporary shotcrete facings and permanent shotcrete or concrete facings. These strength recommendations are based on the results of both full-scale laboratory destructive tests to failure and detailed structural analysis.
d) Recommends design earth pressures for the facing and nail head system, based on soil-structure interaction considerations and monitoring of in-service structures.
e) Addresses both Service Load Design (SLD) and Load and Resistance Factor Design (LRFD) approaches.
f) For SLD, provides recommended allowable loads for the nail tendon, the nail head system and the pullout resistance, together with recommended factors of safety to be applied to the soil strength.

Recommendations are separately provided for regular service loading, for seismic loading, for critical structures, and for temporary construction conditions.

g) For LRFD, provides recommended load factors and design strengths (i.e., resistance factors to be applied to the nominal or ultimate strengths) for the nail tendon, the nail head system, the nail pullout resistance, and the soil strength. Recommendations are separately provided for regular service and extreme event (seismic) loading, for critical structures, and for temporary construction conditions.
h) Recommends procedures for ensuring a proper distribution of nail steel within the reinforced block of ground to enhance stability and limit wall deformation.

b) Type 2: Cuttings into the toe of existing slopes (Figure 2.10).

The design procedures generally require the determination of nail length in order to satisfy two mechanisms, $T_{max\delta}$ mechanism and $T_{0\delta}$ mechanism as illustrated in Figure 2.11

The $T_{max\delta}$ mechanism is the critical two-part wedge mechanism which requires the greatest total horizontal max reinforcement force. This critical mechanism is unique and will determine the total reinforcement force required and hence the number of reinforcement layers. $T_{max\delta}$ mechanism also governs the length of the reinforcement zone, L at the tope of the slope (Figure 2.11 b).

The $T_{max\delta}$ mechanism defines the length L required for the reinforcement at the base (Figure 2.11c). The key mechanism for the purposes of fixing L_B is forward sliding on the basal layer of reinforcement.

Once the number of reinforcement layers, N, length L_T and length L_B are determined, the optimum vertical spacing of the soil nail is determined to complete the design. The optimum vertical spacing of the soil nail is governed by the need to preserve geometrical similarity at all points up the slope, in order to satisfy reduced-scale $T_{max\delta}$ mechanism which outcrop on the front face (Figure 2.12).

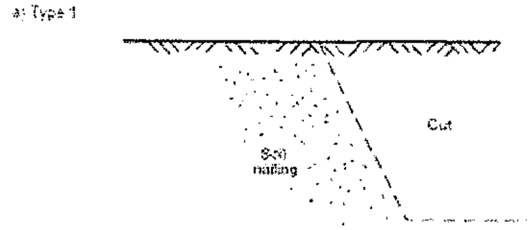

Figure 2.11: Cutting Horizontal Ground (From HA 68/94)

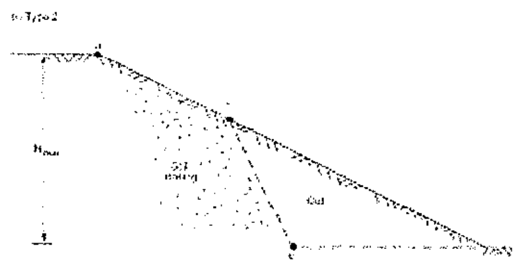

Figure2.12: Cutting into Toe of Existing Slopes (From HA 68/94)

The design process is completed once the following checks are carried out:

a) Check construction condition, missing out the lowest nail, but using short term soil strength parameters, (or using effective stress parameters with the value of pore water pressure parameter, r_u relevant during construction) and T mechanisms (Figure 2.13).

b) Check intermediate mechanisms between $T_{max\delta}$ and $T_{o\delta}$ mechanis.

c) Check that L_B, allows sufficient pull-out length on the bottom row of nails behind the $T_{max\delta}$ mechanism, and if not, extend L_B accordingly. (This is only likely to be critical for small values of drilled hole diameter d_{hole} or large values of horizontal spacing, S_h.

d) The assumption of a competent bearing material beneath the embankment slope should be reviewed and, if necessary, underlying slip mechanisms checked (Figure 2.14).

e) For grouted nails the bond stress between the grouted annulus and the bar should be checked for adequacy.

f) If no structural facing is provided then the capacity of waling plates should be checked (Figure 2.15). It is also likely that increased values of L_T and L_B will be required in this instance.

g) Check that drainage measures are compatible with the pore water pressures assumed. Consider also the potential effects of water filled tension cracks.

h) Check the adequacy of any front face protection provided, such as shotcrete or netting.

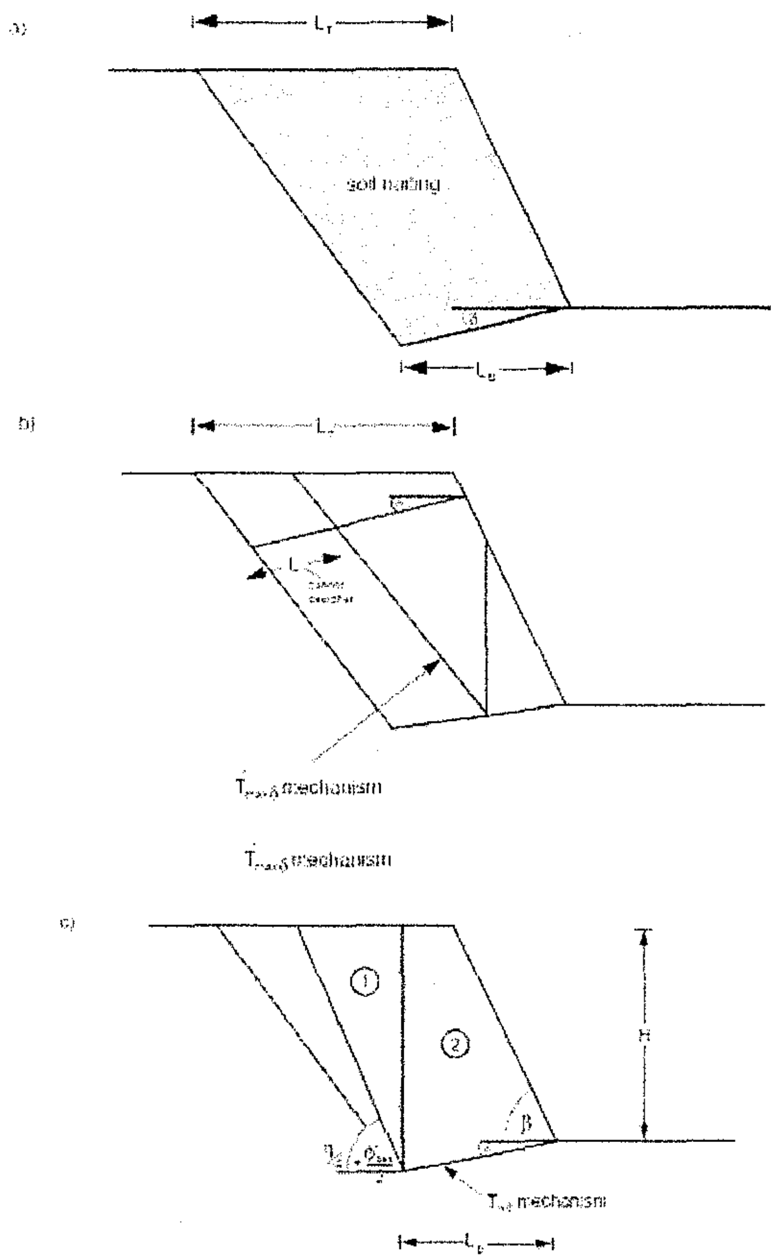

Figure 2.13: General concepts of design method for soil nail (from HA 68/94).

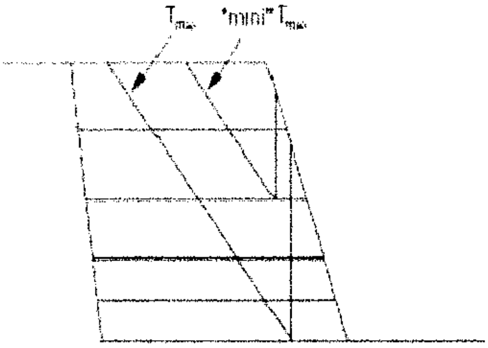

Figure 2.14 Reduced-scale T mechanisms which max outcror on the front face (from HA 68/94).

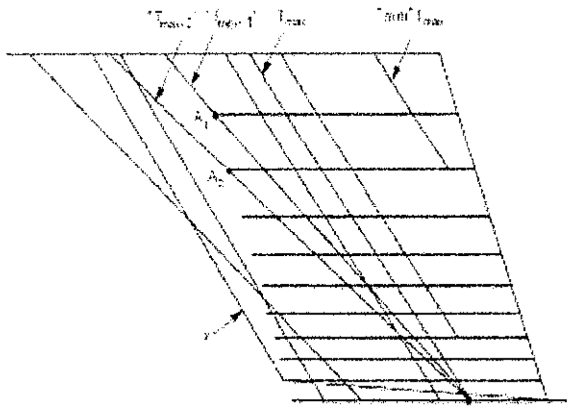

Figure 2.15 Intermediate two-part wedge mechanisms (from HA 68/94). max outcrop on the front face (from HA 68/94).

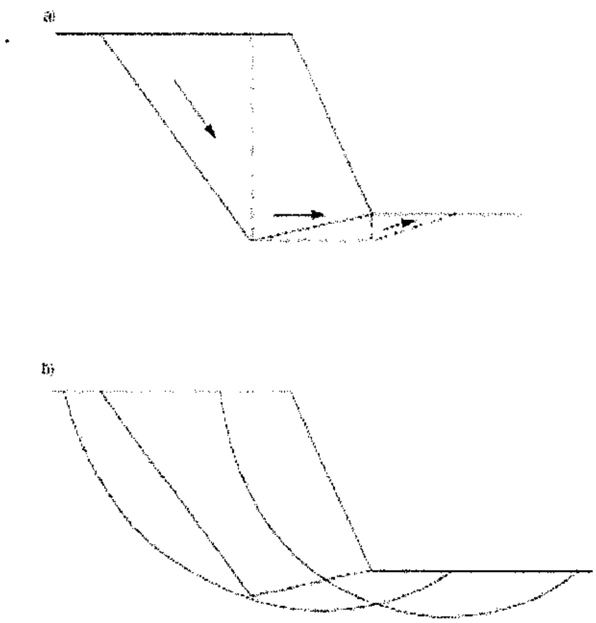

Figure 2.16 Underlying failure mechanisms (from HA68/94).

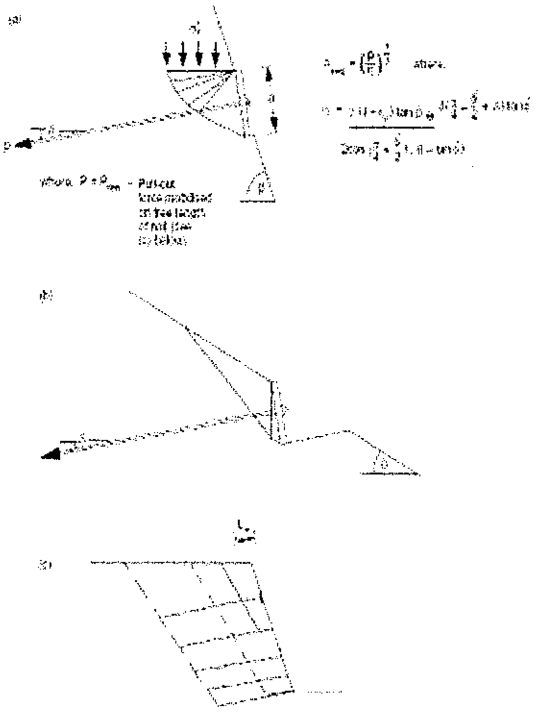

Figure 2.17 Nail plate bearing capacity (from HA 68/94).

i) Identifies the facing reinforcement details to be considered, together with the facing and overall soil nail serviceability checks to be performed.

j) Designs the soil nails and wall facing as a combined integrated soil-nail-wall "system".

The design approach recommended by FHWA is similar to both BS8006 and HA 68/94 in addressing the required ultimate limit and serviceability limit states requirements. The major difference between the FHWA's method and the methods of BS8006 and HA 68/94 is on the failure mechanisms assumed. As discussed earlier, both BS8006 and HA 68/94 recommends the use of two-part wedge and log-spiral failure mechanisms in the design of soil nail while FHWA recommends the "slip surface" method (Tan & Chow, 2006).

Slip surface limiting equilibrium design methods consider the global stability of zones of ground defined by potential failure surfaces. These methods have been widely used in conventional slope stability analyses of unreinforced soil and have been demonstrated to provide good correlations with actual performance in such applications. As with the corresponding slope stability models, a critical slip surface is identified as that yielding the lowest calculated factor of safety, taking into account the support provided by the installed reinforcing. The chose slip surface may be contained entirely or partially within the reinforced zone or entirely outside the reinforced zone. The most significant benefits of the slip surface limiting equilibrium approach to soil nail design are (FHWA, 1998):

a) The method considers all internal, external, and mixed potential slip surfaces for the wall and evaluates global stability for each

b) The method is more convenient and accurate for heterogeneous geometries, soil types, and surcharge loadings than other methods such as the simplified earth pressure method

2.9 Construction Sequence

Typical construction sequence of soil nails can be divided in the following stages (Liew & Khoo, 2005):

a. Initial excavation

This initial excavation will be carried out by trimming the original ground profile to the working platform level where the first row of soil nails can be practically installed. The pre-requisite of this temporary excavation shall be in such a way that the trimmed surface must be able to self support till completion of nail installation. Sometimes, sectional excavation can be carried out for soil with short self support time. If shotcrete/gunite is designed as facing element, the condition of the trimed surface shall be of the satisfactory quality to receive the shotcrete.

b. Drilling of holes

Drilling can be done by either air-flushed percussion drilling, augering or rotary wash boring drilling depending on ground condition. The size of drilled hole shall be as per the designed dimension. Typically, the hole size can range from 100mm to 150mm. In order to contain the grout, the typical inclination of the drill hole is normally tilted at 15° downward from horizontal. 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.

c. Insertion of nail reinforcement and grouting

The nail shall be prepared with adequate centralisers at appropriate spacing and for proper grout cover for first defense of corrosion protection. In additional to this, galvanization and pre-grouted nail encapsulated with corrugated pipe can be considered for durability. A grouting pipe is normally attached with the nail reinforcement during inserting the nail into the drilled hole. The grouting is from bottom up until fresh grout return is observed from the hole. The normal range of water/cement ratio of the typical grout mix is from 0.45 to 0.5.

2.10 Soil Nail Wall Performance

Monitoring is generally not required for a permanent slope or retaining wall reinforced by soil nails that carry transient loads. For soil nails that carry sustained loads, monitoring of the ground movement and loads mobilised along representative soil nails should be carried out during construction and for a considerable period, e.g. at least two wet seasons after construction. An inclinometer may be used to obtain the full vertical profile of the horizontal ground movement. Monitoring of piezometric pressures should also be carried out to aid the interpretation of deformation data. Where the soil nails carrying sustained loads are used in temporary structures, movement monitoring should be carried out until the service of the soil nails is no longer required. Monitoring of the load in these soil nails is generally not warranted (GEO, 2007)

CHAPTER 3
METHODOLOGY

There are some procedure are develop in order to carry out this project. This is to ensure that the project flow is smooth and accomplish in the given period. For this project, the works were progressed based on the methodology.

3.1 Research.

The research involve in this study scope are the research on most of the information about soil nailing. A comprehensive research has been done in order to get as much information as possible regarding this topic. Research had been conducted by reading the journal about soil nailing from various established authors. Besides, the information also gathered via internet or World Wide Web.

3.2 Literature Review

All the information and data collected based on other people works related to the topic has been reviewed. The information had been sorted into respective categories for easier understanding and references such as type of soil nail, advantages and disadvantages of soil nailing, design parameter, soil nail behavior and so on.

3.3 Compile Available Design Methods

The various design method in designing soil nail structure have been compiled and further study has been made in understanding of different approach in designing the soil nail system.

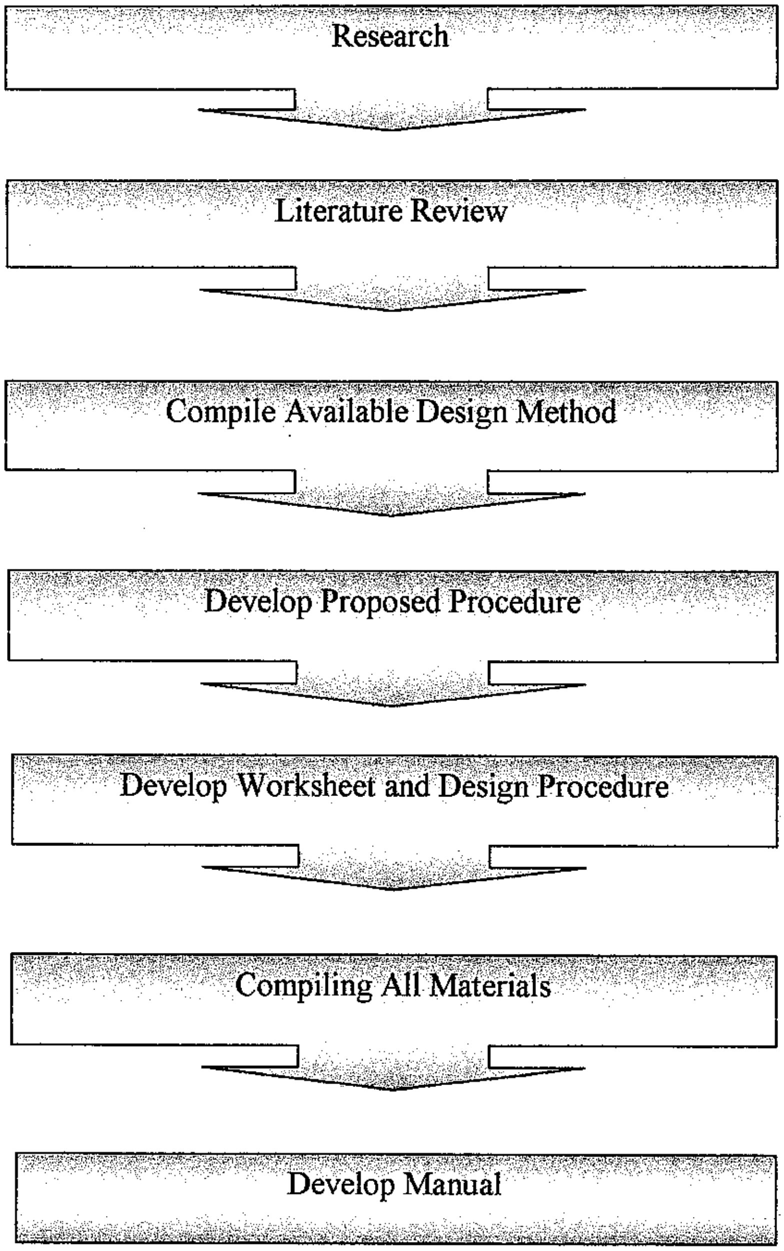

Figure 3.1: Methodology of the Project

3.4 Develop Proposed Procedure

With various design method available in designing soil nail, the procedure has been develop for Malaysian practice based on the recommend design methods.

3.5 Develop Worksheet and Design Procedure

A simple spreadsheet has been developed using Microsoft Excel for manual calculation in design soil nail system. A design example has been done using the proposed procedure for an easy understanding.

3.6 Compiling All the Materials

All the useful information available in establishing a manual has been compiled according to respective topic.

3.7. Develop Manual

The proposed design procedure, installation method and soil nail wall performance was compiling in a simple manual of practice.

CHAPTER 4
RESULTS AND DISCUSSION

4.1 Generals

Soil nailing has gained popularity as slope stabilization method since it has distinct advantages of strengthening the lopes without causing further disturbance. It also known as cost effective, with savings realized mainly from the ease of construction. Compared to tie bask wall, the advantages of soil nail include:

- Elimination of the need for a high-capacity structural facing (H-Piles, walers or thick *CIP* facings). In many cases, this lowers cost and construction time.
- Smaller reinforcing elements can be installed with smaller equipment. There is no need for large equipment to drill or drive H-piles, thus allowing more flexibility, even in areas with overhead obstructions.
- Reduced right-of-way requirements, since soil nails are shorter than tiebacks.
- Reduced construction time, since H-piles are not required, and soil nails do not require post-tensioning.

4.2 Behaviour of Soil Nail

The fundamental mechanism of soil nailing structure the development of tensile forces in the "passive" reinforcement as a result of the restraint that the reinforcement and the attached facing offer to lateral deformation of the structure. The maximum tensile load develop within each nails occurs within the body of reinforced soil at distance from the facing depends on the vertical location within the wall. The line of maximum tension load within each nail often considered dividing the soil mass into two separate zone, active zone and restraint zone.

Active zone is the region close to facing. Shear stress exerted by the soil on the reinforcement is directed outward and tend to pull the reinforcement out of the ground. While restraint zone is the region where shear stress are directed inward and tend to restraint the reinforcement from pullout. Reinforcement act to tie the active zone to the restraint zone.

For stability to be achieved (FHWA, 1998):

a. the nail tensile strength must be adequate to provide the support force to stabilize the active block
b. the nails must be embedded a sufficient length into the resistant zone to prevent the a pullout failure
c. combined effect of the nail head strength (as determined by the strength of the facing connection system) and the pullout resistance of the length of the nail between the face and the slip surface must be adequate to provide required nail tension at the slip surface (interface between active and resistance zones)

4.3 Potential Behaviour of the Soil Nail Wall System

The failure modes of soil nailing can be categorized in the following;

a) Pullout Failure
b) Nail Tendon Failure
c) Face Failure

4.3.1 Pullout Failure

This failure results from the insufficient embedded length into the resistance zone to resist destabilizing force. Therefore, Tan & Chow (2006) point out that in designing soil nail structure, it is necessary to determine a appropriate ground-grout bond stress and pull-out capacity based on critical slip plane. While during the construction, it is necessary to ensure diameter of grouted hole as specified by the designer is achieved at site and the hole is properly grouted throughout the nail length. (Grouting using tremie method filling from bottom up and non-shrink grout shall be used).

4.3.2 Nail Tendon Failure

Nail tendon failure is resulted from inadequate tensile strength of the nail to provide resistance force to stabilize the slope. According to Tan & Chow (2006), this failure primarily governs primarily governed by the grade of steel used and the diameter of the steel. Besides specifying the appropriate nail size corresponding to the required resistant force, it is important that proper detailing with regards to corrosion protection of the nails are specified and properly executed at site. Thus, to avoid the failure, the designer responsibility is to determine of required diameter, spacing of spacers/centralizers and corrosion protection requirements while contractor must ensure spacers/ centralizers are rigidly secured to the nail and corrosion protection carried out as per requirements. Special care shall also be exercised during insertion of the pre-grouted corrugated soil nails to prevent bending and accidental knocking that could cause cracks to the grout and thus, loss of bonding between the grout and the steel bar (potential pullout failure).

4.3.3 Face Failure

The designer and contractor each have important roles to play to prevent face failure. The designer responsible in provide adequate shotcrete thickness and reinforcement provided with proper detailing. While the contractor responsible Constructor: To ensure shotcrete thickness and reinforcement as per requirements. A proper shooting technique by experience nozzleman and correct shotcrete mix are important to ensure shotcrete of good quality..

4.4 Construction sequence

Soil nailing works usually carried out "top down" construction. Construction sequence and associated temporary works are also important to ensure the stability of the slope. Thus, it must be highlighted that soil nailing works which involve cutting of slopes should be carried out in stages where the next stages of works (cutting to final level) can only be carried out when the preceding level of soil nail has been installed and shotcreted. Therefore, the stability of the slopes prior to installation of soil nail walls shall be assessed to determine the allowable height of slopes that can be cut at every stage of the works (Tan & Chow, 2006).

4.5 Available Design Methods

There are three (3) common documents have been refer in designing soil nail structure, namely:

4.5.1 BS8006:1995, Code of Practice for Strengthen/Reinforced Soils and Other Fills

In BS8006, the two-part wedge methods and the log-spiral methods are recommended in analyzing the stability of soil nailed structure However, according to Chow & Tan (2006), there is highlighted in BS8006 that there is evidence from full-scale observation indicating that log-spiral approach has produced reasonable agreement with actual structure and the use of log-spiral method a convenient platform for calculation when shear as well as tension in the nails are to be determined. This method is based on the limit state principles with the use of partial factors of safety. In design of soil nailing requires that the risk of attaining limit and serviceability limit states are minimize with the use of appropriate factor of safety on loads, materials and economic consequence of failure.

External stability checks for reinforced soil wall can be carried out using conventional analysis methods used for a gravity retaining wall. BS8006 recommendations on external loads and partial safety factors should be taken into consideration when carrying out the external stability checks (Tan & Chow, 2004a). BS8006 provides internal stability checks using two methods:

a) Coherent gravity method

b) Tie back wedge method

The tie back wedge method is based on the principles currently employed for classical or anchored retaining walls. Meanwhile, the coherent gravity method is based on the monitored behavior of structures using inextensible reinforcements and has evolved over a number of years from observations on a large number of structures, supported by theoretical analysis. Coherent Gravity method should only be used for inextensible reinforcements and for simple wall geometry. For complex wall geometry, curved walls or multi-tiered wall, comparison should also be made using the Tie Back Wedge method and the design which gives longer reinforcement length or closer reinforcement spacing is to be adopted (i.e. whichever is more conservative) (Tan & Chow, 2004a). BS8006 also required the face stability in preventing erosion and to ensure the load transfer in the active zone.

4.5.2 HA 68/94, Design Methods for the Reinforcement of Highway Slopes Reinforced Soil and Soil Nailing Technique.

The design methods outlined in HA 68/94 is based on two-part wedge mechanism. The two-part is preferred than log-spiral methods since its simplicity and more specific compared to BS8006:1995 (Chow & Tan, 2006). Designing soil nailing using this method required the determination of nail length in order to satisfy two mechanisms, total horizontal reinforcement force and the length required for the reinforcement at the base.

4.5.3 FHWA, Manual for Design and Construction Monitoring of Soil Nail Wall

The FHWA soil nail design method provides a complete and rational approach towards soil nail design, incorporating the following elements (FHWA, 1998):

a) Based on slip surface limiting equilibrium concepts.
b) Incorporates the reinforcing effect of the nails, including consideration of the strength of the nail head connection to the facing, the strength of the nail tendon itself, and the pullout resistance of the nail-ground interface.
c) Provides a rational approach for determining the nominal strength of the facing and nail/facing connection system, for both temporary shotcrete facings and permanent shotcrete or concrete facings. These strength recommendations are based on the results of both full-scale laboratory destructive tests to failure and detailed structural analysis.
d) Recommends design earth pressures for the facing and nail head system, based on soil-structure interaction considerations and monitoring of in-service structures.
e) Addresses both Service Load Design (*SLD*) and Load and Resistance Factor Design (*LRFD*) approaches.
f) For *SLD*, provides recommended allowable loads for the nail tendon, the nail head system and the pullout resistance, together with recommended factors of safety to be applied to the soil strength. Recommendations are separately provided for regular service loading, for seismic loading, for critical structures, and for temporary construction conditions.
g) For *LRFD*, provides recommended load factors and design strengths (i.e., resistance factors to be applied to the nominal or ultimate strengths) for the nail tendon, the nail head system, the nail pullout resistance, and the soil strength. Recommendations are separately provided for regular service and

extreme event (seismic) loading, for critical structures, and for temporary construction conditions.

h) Recommends procedures for ensuring a proper distribution of nail steel within the reinforced block of ground to enhance stability and limit wall deformation.

i) Identifies the facing reinforcement details to be considered, together with the facing and overall soil nail serviceability checks to be performed.

j) Designs the soil nails and wall facing as a combined integrated soil-nail-wall "system".

Comparison with BS8006 and HA 68/94, FHWA proposed the similar design approach which required ultimate limit and serviceability limit state. The only major different is FHWA recommend 'slip surface' methods while the other two proposed the use of two-part wedge and log-spiral methods.

Slip surface limiting equilibrium design methods consider the global stability of zones of ground along potential failure surface. Chow & Tan (2006) point out that slip surface method have been demonstrated to provide good correlations with actual performance in such applications and identified as yielding the lowest calculated factor of safety in slope stability models.

4.6 Recommended Design Approach for Malaysian Practice

Based on the finding on the researches that have been conducted, the recommended design method to be adopted for Malaysian practice is FHWA method with some modifications. The design procedure (Figure 4.1) are predominantly based on the methods proposed in FHWA's manual and must comply with the requirement of BS8006 and incorporated with some good practiced from HA 68/94 in order to improves its applicability for Malaysian practice. This is because the method is complete and it provides a rational approach towards soil nail design inclusive of design aspects for shotcrete, soil nail head, etc. The other factor that this method favorable for Malaysian practiced is the assumption of slip surface limiting equilibrium failure mechanism where it can be easily adopted in practical applications. As it has been known that various commercial slope stability analysis software are available to carry out such analysis and generally, practicing engineers are more familiar with slip surface limiting equilibrium failure mechanism as compared to two-part wedge and log-spiral failure mechanisms (Tan & Chow, 2006). Comparison of design requirements between 3 methods are presented in Table C-1, Appendix C.

According to FHWA (1998), the most significant benefits of the slip surface limiting equilibrium approach to the soil nail wall design are:

1. The methods considers all internal, external and mixed potential slip surface for the wall (bearing capacity of the nailed mass and overall stability of any slope on which wall in constructed are typically evaluated separately) and evaluates global stability for each
2. The method does not required specification of a maximum tension line
3. The method is more convenient and accurate for hetegeneous geometries, soil types and surcharges loading than the simplified earth pressure.

A major step involve (as shown in Manual in Appendix A) in designing design soil nail structures are as follows:

Step 1: Set up critical design cross-section(s) and a select a trial design

This step involves selecting a trial design for the design geometry and loading conditions. The ultimate soil strength properties for the various subsurface layers and design water table location should also be determined. Table 4.1 provides some guidance on the required input such as the design geometry and relevant soil parameters. Subsequently, a proposed trial design nail pattern, including nail lengths, tendon sizes, and trial vertical and horizontal nail spacing, should be determined.

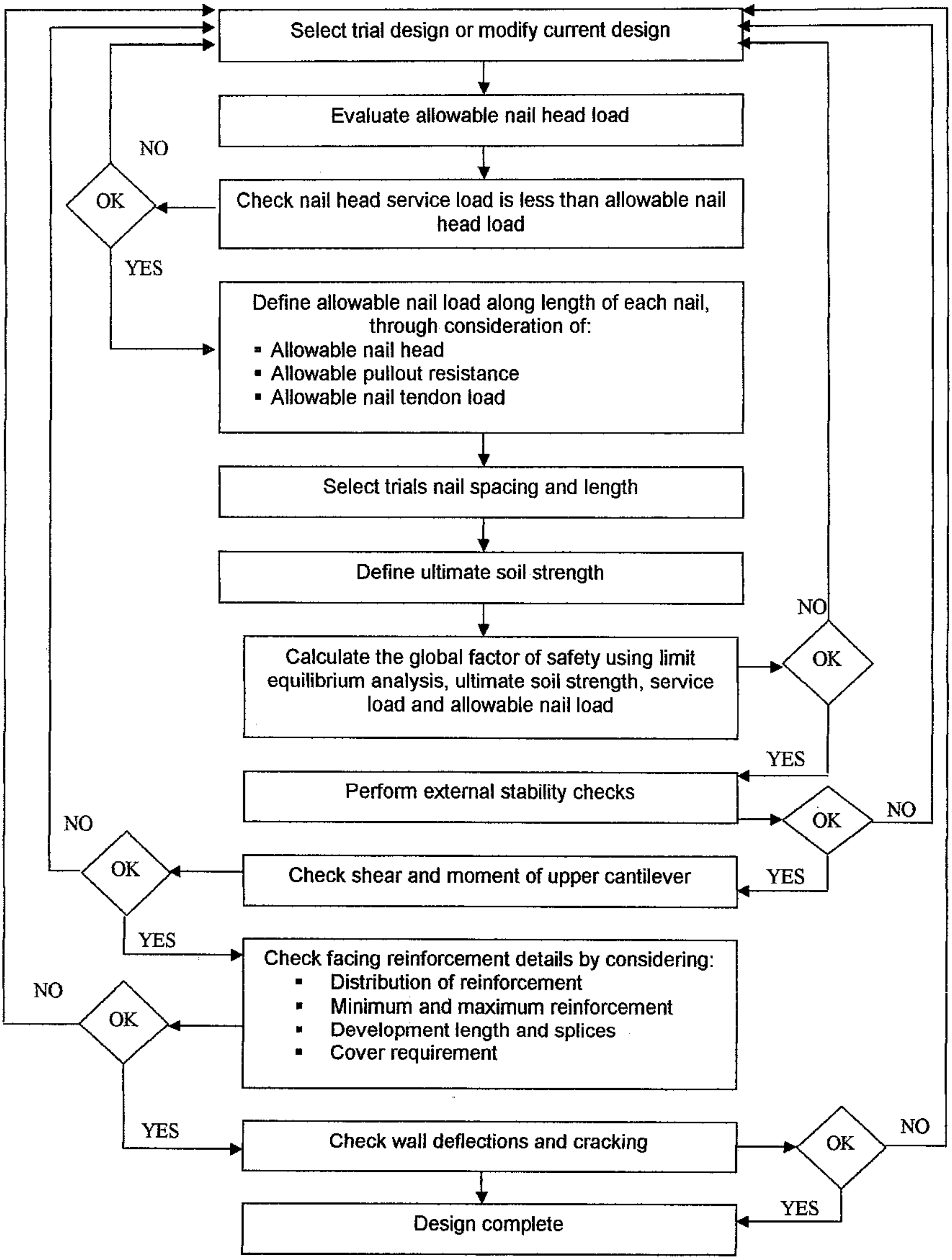

Figure 4.1 Recommended Design Procedures

Table 4.1: Input required for Soil Nail Design

		Remarks
Soil Properties	Bulk density, γ	-
	Ultimate friction angle, Φ_{ult}	-
Wall geometry	Ultimate soil cohesion, C_{ult}	-
	Wall height, H	-
	Wall inclination, α	-
	Height of upper cantilever, C	-
	Height of lower cantilever, B	-
	Backslope angle, β	$\beta < \Phi_{ult}$
	Soil-to wall interface friction angle, δ	Typically 2/3 Φ_u
	Nail inclination, η	Typically 15°
	Vertical spacing of nail, *SV*	Typically 1.5 m to 2.0 m
Nail and shotcrete properties	Horizontal spacing of nail, *SH*	Typically 1.5 m to 2.0 m
	Characteristic strength of nail, Fy	Typically 460 N/mm^2
	Nail size/diameter	Minimum Φ20 mm
	Ultimate bond stress, *Qu* (kN/m) Values given in Tables 2 & 3 in kN/m^2 Multiply with perimeter of grout column (*p x DG C*) to obtain value in kN/m	Table A-1
	Shotcrete strength	-
	Thickness of shotcrete	-
	Depth / Width of steel plate	Minimum plate width 200 mm
	Thickness of steel plate	Minimum plate thickness 19mm
	Reinforcement for shotcrete	Use BRC reinforcement
	Waler bars	Typicaly 2T12
	Concrete cover	Typically 50-75 mm
	Diamet er of grout column	Typically 125 mm

Factor of safety	Soil strength	Table A-3
	Nail tendon tensile strength	Table A-3
	Ground-grout pullout resistance	Table A-3
	Facing flexure pressure	Table A-2
	Facing shear pressure, C_s	Table A-2
	Nail head strength facing flexure / punching shear,	Table A-2
	Nail head service load,	Typically 0.5
	Nail head service load,	Typically 2.5

Step 2: Compute the allowable nail head load

The allowable nail head load for the trial construction facing and connector design is evaluated based on the nominal nail head strength for each potential failure mode of the facing and connection system, i.e. flexural and punching shear failure. The flexural and punching strength of the facing is evaluated as follows in accordance to the recommendations of FHWA (1998):

Flexural Strength of the facing

Critical nominal nail head strength, T_{FN}

$$T_{FN} = C_F(m_{v,NEG} + m_{v,POS})(8S_h / S_v)$$

Where

T_{FN}	= Critical nail head strength
C_F	= Flexure pressure factor
$M_{v,NEG}$ & $m_{v,POS}$	= Vertical nominal unit moment resistance at the nail head and mid-span
S_h & S_v	= Horizontal/vertical nail spacings

Vertical nominal unit moment

$$m_{v(NEG,POS)} = \frac{A_s F_y \gamma}{b}\left(d - \frac{A_s F_y}{1.7 f'_c b}\right)$$

Where:

A_s = area of tension reinforcement in facing panel width 'b'

b = width of unit facing panel (equal to S_v)

d = distance from extreme compressive fiber to centroid of tension reinforcement

f'_c = compressive strength of the concrete

Punching Shear Strength of the facing

Nominal internal punching shear strength of the facing, V_N

$$V_N = 0.33(f'_c(MPa))^{1/2}(\pi)(D'_c)(h_c)$$

$$D'_c = b_{PL} + h_c$$

Nominal nail head strength, T_{FN}

$$T_{FN} = V_N[1/1 - C_s(A_c - A_{GC})/(S_V S_H - A_{GC}]$$

Cs = pressure factor for punching shear

The allowable nail head load is then the lowest calculated value for the two different failure modes.

Step 3: Minimum Allowable Nail Head Service Load Check

This empirical check is performed to ensure that the computed allowable nail head load exceeds the estimated nail head service load that may actually be developed as a result of soil-structure interaction. The nail head service load actually developed can be estimated by using the following empirical equation:

$$t_f = F_f K_A \gamma H S_V S_H$$

Ff = empirical factor (= 0.5)

KA = coefficient of active earth pressure

γ = bulk density of soil

H = height of soil nail wall

S_H = horizontal spacing of soil nails

S_V = vertical spacing of soil nails

Step 4: Define the Allowable Nail Load Support Diagrams

This step involves the determination of the allowable nail load support diagrams. The allowable nail load support diagrams are useful for subsequent limit equilibrium analysis. The allowable nail load support diagrams are governed by:

a) Allowable Pullout Resistance, Q_u

$Q = \alpha_Q$ x Ultimate Pullout Resistance, Q_u

b) Allowable Nail Tendon Tensile Load, T_{NN}

$T_N = \alpha_N$ x Tendon Yield Strength, T_{NN}

c) Allowable Nail Head Load, T_{FN}

$T_F = \alpha_F$ x Nominal Nail Head Strength, T_{FN}

Step 5: Select Trial Nail Spacing and Lengths

Performance monitoring results carried out by FHWA have indicated that satisfaction of the strength limit state requirements will not of itself ensure an appropriate design. Additional constraints are required to provide for an appropriate nail layout. The following empirical constraints on the design analysis nail pattern are therefore recommended for use when performing the limiting equilibrium analysis:

a) Nails with heads located in the upper half of the wall height should be of uniform length

b) Nails with heads located in the lower half of the wall height shall be considered to have a shorter length in design even though the actual soil nails installed are longer due to incompatibility of strain mobilised compared to the nails at the upper half. However, further refinement in the nail lengths can also be carried out if more detailed analyses are being carried out, e.g. using finite element method (FEM) to verify the actual distribution of loads within the nails.

The above provision ensures that adequate nail reinforcement (length and strength) is installed in the upper part of the wall. This is due to the fact that the top-down methods of construction of soil nail walls generally results in the nails in the upper part of the wall being more significant than the nails in the lower part of the wall in developing resisting loads and controlling displacements. If the strength limit state calculation overstates the contribution from the lower nails, then this can have the effect of indicating shorter nails and/or smaller tendon sizes in the upper part of the wall, which is undesirable since this could result in less satisfactory in-service performance. The above step is essential where movement sensitive structures are situated close to the soil nail wall. However, for stabilization works in which movement is not an important criterion, e.g. slopes where there is no nearby building or facilities, the above steps may be ignored (Tan & Chow, 2004b).

Step 6: Define the Ultimate Soil Strengths

The representative soil strengths shall be obtained using conventional laboratory tests, empirical correlations, etc. The limit equilibrium analysis shall be carried out using the representative soil strengths (NOT factored strengths).

Step 7: Calculate the Factor of Safety

The Factor of Safety (FOS) for the soil nail wall shall be determined using the "slip surface" method (e.g. Simplified Bishop method, Morgenstern-Price method, etc.). This can be carried out using commercially available software to perform the analysis. The stability analysis shall be carried out iteratively until convergence, i.e. the nail loads corresponding to the slip surface are obtained. The required factor of safety (FOS) for the soil nail wall shall be based on recommended values for conventional retaining wall or slope stability analyses (e.g. 1.4 for slopes in the high risk-to- life and economic risk as recommended by GEO, 2000).

Step 8: External Stability Check

The potential failure modes that require consideration with the slip surface method include:

a) Overall slope failure external to the nailed mass (both "circular" and "sliding block" analysis are to be carried out outside the nailed mass). This is especially important for residual soil slopes which often exhibit specific slip surfaces, defined by relict structure, with shear strength characteristics that are significantly lower than those apply to the ground mass in general. Therefore, for residual soil slopes, the analyses must consider either general or non-structurally controlled slip surfaces in association with the strength of the ground mass, together with specific structurally controlled slip surfaces in association with the strength characteristics of the relict joint surfaces themselves. The soil nail reinforcement must then be configured to support the most critical condition of these two conditions.

b) Foundation bearing capacity failure beneath the laterally loaded soil nail "gravity" wall. As bearing capacity seldom controls the design, therefore, a rough bearing capacity check is adequate to ensure global stability.

Step 9: Check the Upper Cantilever

The upper cantilever section of a soil nail wall facing, above the top row of nails, will be subjected to earth pressures that arise from the self-weight of the adjacent soil and any surface loadings acting upon the adjacent soil. Because the upper cantilever is not able to redistribute load by soil arching to adjacent spans, as can the remainder of the wall facing below the top nail row, the strength limit state of the cantilever must be checked for moment and shear at its base, as described in Figure 4.2.

For the cantilever at the bottom of the wall, the method of construction (top-down) tends to result in minimal to zero loads on this cantilever section during construction. There is also the potential for any long-term loading at this location to arch across this portion of the facing to the base of the excavation. It is therefore recommended by FHWA, 1998 that no formal design of the facing be required for the bottom cantilever. It is also recommended, however, that the distance between the base of the wall and the bottom row of nails not exceed two-thirds of the average vertical nail spacing.

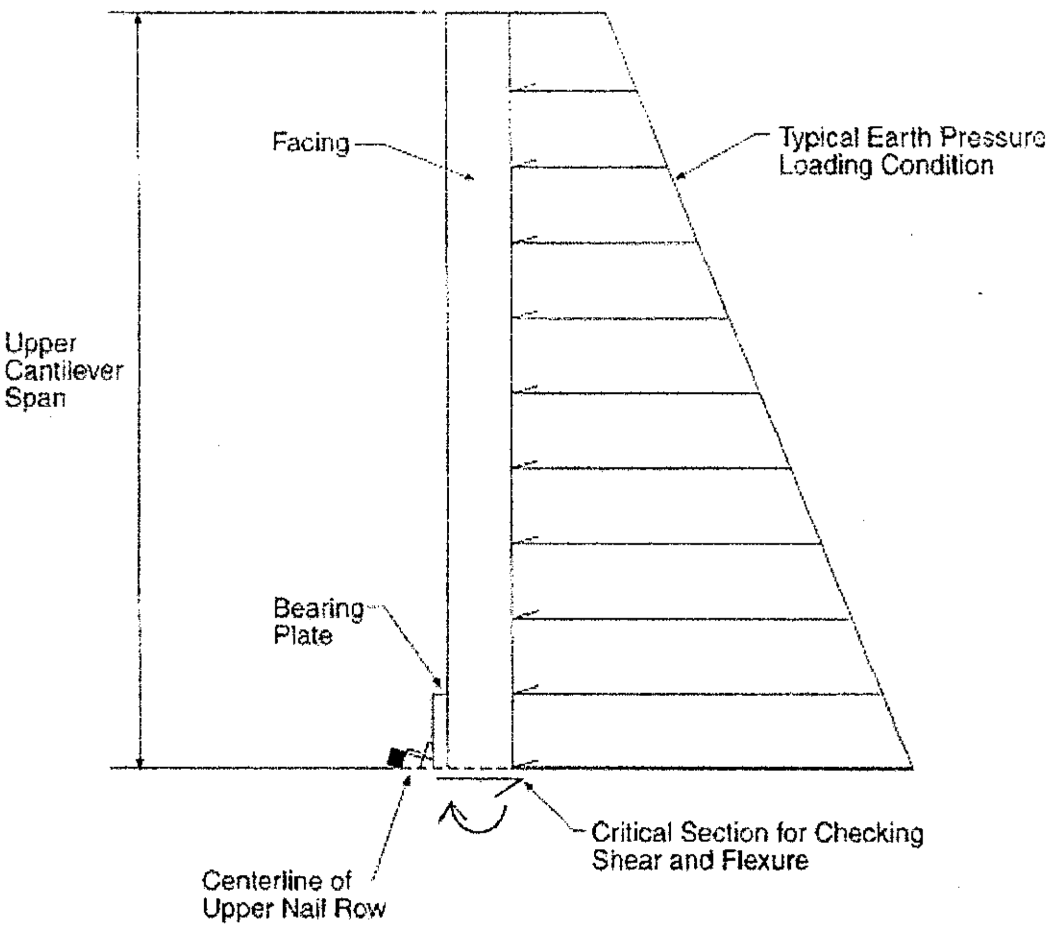

Figure 4.2: Upper Cantilever Design Check (from FHWA 1998)

Step 10: Check the Facing Reinforcement Details

Check waler reinforcement requirements, minimum reinforcement ratios, minimum cover requirements, and reinforcement anchorage and lap length as per normal recommended procedures for structural concrete design.

It is recommended that waler reinforcement (usually 2T12) to be placed continuously along each nail row and located behind the face bearing plate at each nail head (i.e.

between the face bearing plate and the back of the shotcrete facing). The main purpose of the waler reinforcement is to provide additional ductility in the event of a punching shear failure, through dowel action of the waler bars contained within the punching cone.

Step 11: Serviceability Checks

Check the wall function as related to excess deformation and cracking (i.e. check the serviceability limit states). The following issues should be considered:

a) Service deflections and crack widths of the facing
b) Overall displacements associated with wall construction
c) Facing vertical expansion and contraction joints

4.7 Soil Nailing Wall Performance and Monitoring

According to FHWA (1998) manual's an observation and monitoring should typically include:

a) face horizontal movement using surface markers on the face and surveying methods and inclinometer casings installed at short distance (typically 1m) behind the facing
b) Vertical and horizontal movement of the top of the wall facing and the ground surface behind the shotcrete facing using optical surveying methods
c) Ground cracks and other signs of disturbance in the ground surface behind the top of wall, by daily visual inspection during the construction and if, necessary crack cages.
d) Local movement and or deterioration of the facing using visual inspection and instrument such as crack cages
e) Drainage behavior of the structure, especially if groundwater was observed during construction. Drainage can be monitored visually by observing outflow points or through standpipe piezometer installed behind the facing.

The important parameter must be identified in monitoring soil nail wall performance. Kutsche and Tarquino (2006) stated that the important parameter is the overall resistance offered by installed soil nail as compared to the design resistance required by soil nail load diagram developed for a particular design. FHWA (1998) manual point out that the most significant measurement of the overall performance of the soil nails wall system is the deformation of the wall or slope during and after construction.

It has been known that two basic for quality control/ quality assurance for soil nail wall project practiced in United States namely (Kutsche and Tarquino, 2006):

- The soil nail elements, specifically unconfined compressive strength testing of soil nail grout and proof/verification testing of the soil nails
- The shotcrete facing, specifically the unconfined compressive strength and boiled absorption testing of shotcrete.

An understanding on the roles played by the designer and contractor is important to ensure design intention are communicated to the site and similarly, site constraint are made know to the designer. Construction sequence on soil nailing works also influences the degree of success of the works especially for slope remedial works. It is therefore recommended that the designer clearly indicate the required stages of works in relevant drawings and work specifications. Finally, proper supervision of soil nailing works to ensure conformance to design requirements and specifications is important (Tan & Chow 2004b). A sample of checklist which is enclosed in the Appendix B.

CHAPTER 5

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The main objective of this project is to develop a manual of practice design, construction, quality control and monitoring of soil-nailed structures has been achieved. The manual discussed 3 methods commonly referred in designing soil nail structure namely as BS8006:1995, Codė of Practice for Strengthen/Reinforced Soils and Other Fills, HA 68/94, Design Methods for the Reinforcement of Highway Slopes Reinforced Soil and Soil Nailing Technique and FHWA, Manual for Design and Construction Monitoring of Soil Nail Wall. The design procedure are predominantly based on the methods proposed in FHWA's manual and must comply with the requirement of BS8006 and incorporated with some good practiced from HA 68/94 in order to improves its applicability for Malaysian practice.

This is because the method is complete and it provides a rational approach towards soil nail design inclusive of other design aspect such as shotcrete, soil nail head, etc. which important to ensure satisfactory performance of soil nailed slope. The design procedure presented also satisfies the ultimate limit and serviceability limit stases requirement of BS 8006:1995. Some good practices highlighted in HA 68/94 are also incorporated in the proposed in order to improve its applicability for Malaysian practice.

5.2 Recommendation

a. The analysis of slope stability should be done with commercial software such as SLOPEW and STED for consistent and reliable results.

b. Further experimental research and model testing on establishing the statically significant data base for the seismic performance assessment

c. Development and experimental evaluation of reliable seismic method for engineering use of soil nailing in earthquake zones.

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APPENDIX A
Input Required for Soil Nailing Design

Table A-1: Ultimate Bond Stress – Rock (from Table 3.4, FHWA, 1998)

Construction Method	Soil Type	Unit Ultimate Bond Stress kN/m^2
Open Hole	Marl / Limestone	300 – 400
	Phillite	100 – 300
	Chalk	500 – 600
	Soft Dolomite	400 – 600
	Fissured Dolomite	600 – 1000
	Weathered Sandstone	200 – 300
	Weathered Shale	100 – 150
	Weathered Schist	100 – 175
	Basalt	500 – 600

Table A-2: Nail Head Strength Factor (from Table 4.4, FHWA, 1998)

Failure Mode	Nail Head Strength Factor (Group I)	Nail Head Strength Factor (Group IV)	Nail Head Strength Factor (Group VII) (Seismic)
Facing Flexure	0.67	1.25(0.67)=0.83	1.33(0.67)=0.89
Facing Punching Shear	0.67	1.25(0.67)=0.83	1.33(0.67)=0.89

Table A-3: Strength Factor and Factor of Safety (form Table 4.5, FHWA, 1998)

Element	Strength Factor (Group I), α	Strength Factor (Group IV),	Strength Factor (Group VII), (Seismic)
Nail Head Strength	α_F = Table A-2	See Table A-2	See Table A-2
Nail Tendon Tensile Failure	$\alpha_N = 0.55$	1.25(0.55)=0.69	1.33(0.55)=0.73
Ground-Grout Pullout Resistance	$\alpha_Q = 0.50$	1.25(0.50)=0.63	1.33(0.50)=0.67
Soil	F= 1.35(1.50*)	1.08(1.20*)	1.01(1.13)*
Soil-Temporary Construction Condition †	F= 1.20(1.35*)	NA	NA

Note:
Group I: General loading conditions
Group IV: Rib shortening, shrinkage and temperature effects taken into consideration
Group VII: Earthquake (seismic) effects (Not applicable in Malaysia)
* Soil Factors of Safety for Critical Structures
† Refers to temporary condition existing following cut excavation but before nail installation. Does not refer to "temporary" versus "permanent" wall.

APPENDIX B
Sample Checklist for Construction Supervision of Soil Nailing Works

No.	Checklist Items	Acknowledge by	Checked by	
		Contractor	Client	
1.0	**EARTHWORK FOR SOIL NAIL SLOPE**	**SIGNATURE**	**YES**	**NO**
1.1	The construction sequences (stages of construction) shall be referred to the construction drawing			
1.2	The soil excavation shall not exceed 3m height per stage before soil nails, horizontal drains and shotcrete surface are completed.			
1.3	The next stage of excavation (after Item 1.2) shall only be allowed after the soil nails, horizontal drains and shotcrete surface are completed.			
1.4	The 4V:1H slope surface shall be covered with shotcrete after the installation of soil nails. No portion of the slope should be left exposed at 4V:1H gradient for more than 3 **days.**			
1.5	Temporary slope protection using canvas shall be carried out to prevent slope erosion			
1.6	Contractor that refuse to follow or not following the above construction sequences shall be WARNED and BLACKLISTED			
2.0	**SOIL NAIL**	**SIGNATURE**	**YES**	**NO**
2.1	Soil Nailing Material • Steel Nail reinforcement shall comply with BS 4449 or equivalent standard. (Only nails greater than 12m in length can be spliced using mechanical splicer approved by Engineer.) • Galvanizing: galvanize steel bar/ steel plate/ washer/ hexagon nut (All threading			

	process on the steel elements shall be completed before galvanized or else the epoxy paint shall be applied on the threaded portion) • Centralizer: Provide only plastic centralizer or equivalent of a minimum diameter 25mm smaller than the nominal diameter of the drilled hole.			
2.2	Steel Welded Wire fabric • Shall comply to BS 4483 or equivalent • Lap mesh shall be at least 200mm or one mesh grid standard in both directions which ever is larger. • Tie wires shall be bent flat in the plane of the mesh and not forming large knot. • Spacer: Provide sufficient spacer (eg: at least 1m interval) and ensure the spacer is solid.			
2.3	Horizontal Drain • Provide as required and shown on drawings (slotted and unslotted PVC) with end cap • Provision shall be made to ensure that the hole does not collapse prior to the insertion of the slotted drain			
2.4	Grout for Nails • Provide non-shrink neat cement or non-shrink sand cement grout with pumpable mixture capable of reaching minimum 28 days cube strength of 30 MPa in accordance with BS 1881. • To achieve non-shrink effect, additives shall be added (e.g. Intraplast Z). • Please record name and percentage of the additives that have been used as follows: ‹ _____________ (name) ‹ _____________ (percentage) • Have the additives been approved by the			

	Engineer? ‹ Yes / No • Cube test to be carried out after every batching of grout.			
2.5	Permanent Structural Shotcrete Facing • Materials - Cement: Ordinary Portland Cement complying with BS12 or MS 522 and Portland Pulverized Fuel Ash Cement complying with MS 1227. - Aggregate: shall comply with BS 882 - Accelerating additives shall be compatible with the cement used, be non-corrosive to steel and not promote other detrimental effects (cracking and excessive shrinkage) and shall not contain calcium chloride. - Water used in the shotcrete mix shall be potable, clean and free from substances or element, which may be injurious to concrete and steel or cause staining • Quality Shall be produced by dry or wet mix process achieving a minimum compressive strength of 18MPa in 7 days and 30MPa in 28 days. • Construction Testing Shall carry out a test panel and send cores for testing in accordance to BS 1881			
3.0	**NAIL INSTALLATION**	**SIGNATURE**	**YES**	**NO**
3.1	General procedures: Check the size (diameter) of drill bit and compare with the required diameter of soil nail as specified in the drawings. Any anomalies shall be reported immediately to the Engineer. ‹ ____________mm (diameter of drill bit) ‹ ____________mm (required soil nail diameter)			

	• Check the diameter of hole being formed. ‹ ____________mm • Mark clearly and accurately the point of the soil nail location. The drilled hole shall be located within 150mm of the location shown on drawing. • Supervisor and driller to ensure the drilling methods is suitable for maintaining open drill holes and do not promote mining and loosening of the soil at the perimeter of the drill hole or fracture soils with weak stratification planes by control the flush volumes and pressure. Provide nail length and nail diameter necessarily as required but not less than lengths and diameter as shown in the construction drawing. • At the point entry, the nail angle shall be within ± 3 degrees of the inclination as shown in the construction drawing. • Centralizers shall be provided at 2m intervals for the whole length of nail with the last centralizer located at 300mm from the end of each nail and ensure that not less than 30mm of grout cover is achieved along the nail. • Record the depth where the seepage of groundwater was observed (if any). • Inject grout at the lowest point of the drill hole. (Pump grout through tubes, casing, hollow stem auger or drill rods such that the hole is filled from the bottom to the top to prevent air voids until clean grout is seen to run from the top of the hole). Remark: Grout pipe must be used or else the particular soil nail will be rejected. Grouting equipment shall have capability of continuous mixing and producing grout free of lumps.			

4.0	SHOTCRETING	SIGNATURE	YES	NO
4.1	General procedures: • Slope surface to receive shotcrete shall be cleaned with air blast to remove loose material, mud, rebound from previously placed shotcrete and other foreign matter that will prevent bonding of shotcrete. • Dampen the surface before shotcreting. • During placement of shotcrete, the horizontal drains and weep holes shall be protected against contamination or clogging of shotcrete to ensure proper functioning. • Thickness measuring pins (non-corrosive) shall be installed on 1.5m grids in each direction. • Check the thickness of measuring pins using normal ruler or measuring tape. ‹ ____________mm • Thickness, method of support, air pressure and water content of the shotcrete shall be controlled in such a manner as to preclude sagging of sloughing off. • The shotcrete shall be applied from the bottom up to prevent accumulation of rebound shotcrete on the surface, which is to be covered. • Horizontal and vertical corners and hollow areas shall be filled first. • Checking for hollow areas on the completed shotcrete surface shall be carried out with a hammer. • All shotcrete which lacks uniformity, exhibits segregation, honeycombing or lamination, or which contains any dry patches, slugs, voids or sand pockets shall			

	be removed and replace with fresh shotcrete. • In situ core test shall be carried out for verification. • Immediately after the completion of shotcreting works, keep shotcrete surface continuously moist for at least 24 hours for curing purpose. • The opened cut area shall be protected with canvas or suitable material to avoid erosion. As built drawing showing the location, dimensions, photos and details of the soil nail wall shall be produced by the contractor.			
5.0	**PULL OUT TEST**	**SIGNATURE**	**YES**	**NO**
5.1	List of equipment • A single acting hollow hydraulic jack connected to hydraulic pump and pressure gauge with minimum capacity of 20MT • A pull out steel fabricated cage • A steel bracket • At least 4 displacement gauges • A pressure meter • Nut and washers • Stopwatch to measure the period of observation.			
5.2	General Procedures • Pull out test should be carried out in ground types and in environmental conditions similar to those existing at the proposed site. • The stressing equipment, pressure gauge and load cells should be calibrated by the			

	manufacturer and in accordance with clause 10.6 BS 8081:1989. • The load cycle, load increments and minimum periods of observation shall be as instructed by the Engineer. • As built drawing showing the location of pull out test, dimensions, photos and details of the test shall be produced by the contractor.			

APPENDIX C
Comparison of Design Requirement of Available Design Methods

Table C-1: Comparison of Design requirement of available design methods

No	Design Requirement	Design Methods								
		FHWA 1998			BS 8006:1995			HA 68/94		
		1	2	3	1	2	3	1	2	3
1.	Stability Analysis									
	External Stability			/			/			/
	Internal Stability			/			/			/
	Face Stability			/			/			/
2.	Reinforcing Effect			/		/			/	
3.	Construction Check			/		/				/
4.	Serviceability Check			/			/			/
5.	Facing Reinforcement			/			/			/
6.	Drainage			/		/				/
7.	Facing Protection			/		/				/
8.	Bond Stress Estimation		/			/				/
9.	Reinforcement Details		/				/		/	

Note:
1: Low emphasis
2: Medium emphasis
3: High emphasis

APPENDIX D
Guideline for Design & Construction of Soil Nail Wall

TABLE OF CONTENT

CHAPTER 1: INTRODUCTION AND APPLICATION CRITERIA

LIST OF FIGURES

Figures

LIST OF TABLES

Tables

CHAPTER 1: INTRODUCTION AND APLLICATION CRITERIA

1.1 Purpose and Scope of Guideline

The specific purpose of this guideline is to introduce the concept of soil nailing into Malaysian practice and provide the guidance on designing and specifying soil nailing for those application which is technically suited and economically effective.

The scopes of this guideline include:

Chapter 1	: An overview of soil nailing technology and a discussion of the advantages, limitation and recommendations application of the soil nailing.
Chapter 2	: A brief description of the use soil nails in US and Malaysia, of the method of construction, and the behavior of soil nail.
Chapter 3	: Recommended of method for site investigation and testing.
Chapter 4	: Recommended design procedure.
Chapter 5	: Work design example.
Chapter 6	: Wall performance and monitoring.

1.2 Soil Nail Description

Soil nail is a structural element which provides load-transfer to the excavation support and slope stabilization applications. The 'nail' consists of steel bars or other metallic element that can resist tensile stresses, shear stresses and bending moments which commonly encapsulated with grout cover for corrosion protection and improved load transfer to the ground as shown in Figure 2.1.

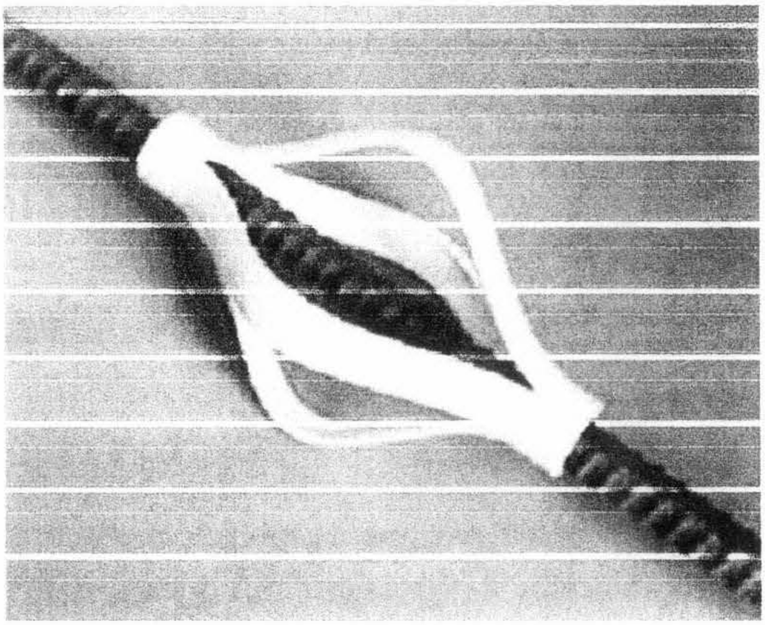


Figure 1.1: Steel tendons used in soil nail

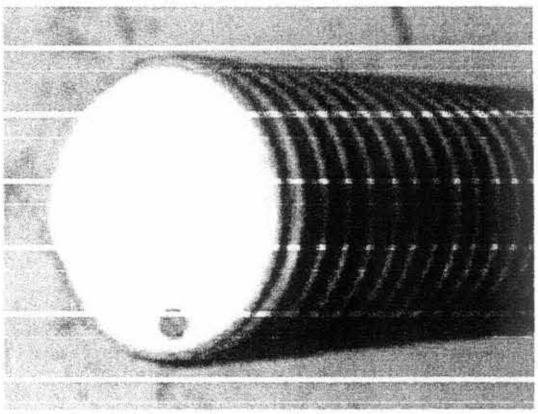


Figure 1.2: A cross section of the steel tendon

1.3 The Soil Nail Concept

The basic design of soil nailing is to reinforce and strengthen the slopes in-situ by installing closely spaced bars, called 'nails' into excavated slope as 'top down' construction. This process can create a reinforced mass that internally stable and able to retain the passive ground against active pressure, sliding, bearing and overturning forces. The reinforcements are passive and can develop their reinforcing action through the nail-soil interaction as the slopes deform during and subsequent to construction. Soil nails works predominantly in tension but may develop some bending or shear in certain circumstances when internal strain or deformation is too large.

The resisting tensile forces mobilized in the grouted rebar can induce an apparent increase of normal stresses along the potential slip surface to increase the overall shearing resistance of in-situ soil. The effect of the rebars is thus improving stability by:

a) Increasing the normal force and hence, the soil shear resistance along the potential slip surface in frictional soil
b) Reducing the driving force along the potential slip surfaces in both frictional and cohesive soils.

In soil nailing, the reinforcement is installed horizontally or sub-horizontally (approximately parallel to the direction of the major tensile straining in the soil) so that it can contribute to the support of the soil partially by directly resisting destabilizing forces and partially by increasing the normal loads (and hence the shear strength) of the potential slip surfaces as shown in Figure 1.3.

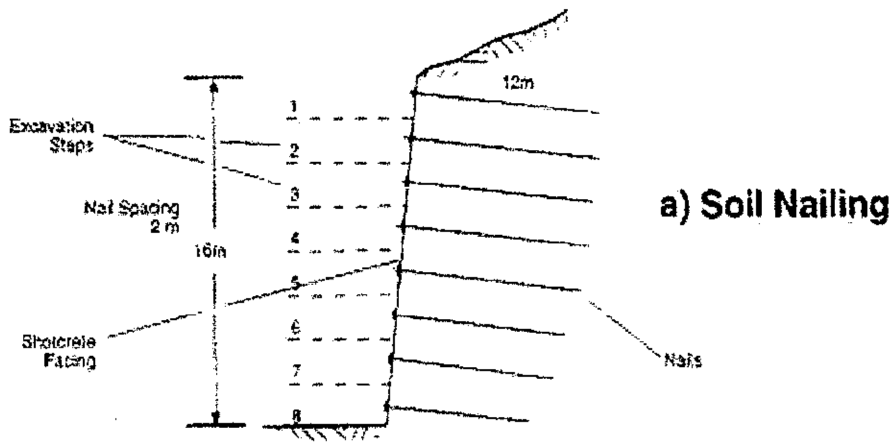


Figure 1.3 Soil Nail Reinforcement Techniques

1.4 Advantages of Soil Nailing

Soil nailing walls have been found to have many advantages as tie back. The 'top down' construction of the soil nailing offers the benefits:

- Improved economy and lessen environmental impact through the elimination of the need for cut excavation and backfilling

- Improved economy and material saving through incorporation of the temporary excavation support system into permanent support system
- Improved safety by eliminating cramped excavation cluttered with internal bracing

Compared to Tieback Wall the advantages of soil nailing include:

- Elimination of the need for a high capacity structural facing (H-Piles, walers or thick CIP facing) since the maximum earth pressure loads are not transferred to the excavation faced. In many cases, this lowers cost and construction time.
- Improved construction flexibility in heterogeneous soil with cobbles, boulders, or other hard inclusion as this obstruction offer fewer problems for relatively small diameter nail drilled holes than they do for large diameter soldier pile installation through the bridge deck or in hand dug pit.
- The vertical component of the nail reaction are smaller than those for in tie back also distributed more evenly over the entire excavation face. This eliminates the needs for significant wall embedment below grade.
- Reduced right-of-way requirement as the nail are typically shorter than the tieback anchors

1.5 Limitation of the Soil Nailing

- Permanent underground easements may be required.
- Reinforcements may interfere with existing or future utilities.
- Use of soil nails in soft, cohesive soils subject to creep may not be economical, even at low load levels
- Horizontal displacements may be greater than those associated with tieback construction, and therefore, may limit use adjacent to critical structures.
- Shotcrete facings on permanent walls require special drainage considerations to eliminate the potential for freeze-thaw damage, particularly in frost heave susceptible soils such as silts and fine sands.

Limitations specific to soil nailing construction are:

- For near vertical walls, the soil being nailed must be able to stand unsupported to a height of 3 to 6 feet while it is being nailed and covered with shoring or shotcrete. Alternatively, a construction sequence using slotted cuts, nailing and berming may work, but will add to the cost. Soil without a short-term cohesion, such as loose to medium clean sands and gravels, may not be well suited for soil nailing.

- The groundwater table should be lowered below the bottom of the wall during and after construction. Seepage through the face will soften soils, resulting in local instability or slumping during construction, and reduce the bond between the soil and the shotcrete face. In the long term, the build-up of pore pressure behind the wall and the potential for frost heaving need to be controlled through the placement of permanent drains behind and below the wall face.
- Soil nailing in very low shear strength soil may require a very high soil nail density, and thus be uneconomical.
- Soil nailing in sensitive soils and expansive soils for permanent long-term applications is not recommended. For temporary wall applications in these soils, the potential for loss of shear strength or swelling and heave due to moisture or loadings must be considered.

1.6 Ground Condition Best Suited for Soil Nailing

In general, the economical use of soil nailing require that the ground able to stand unsupported vertical or steeply slope cut of 1 to 2 meter for a 1 to 2 days. In addition it is highly desirable that an open drill holes can maintain its stability for at least several hours. In context with those conditions, the following ground types are suitable for soil nailing:

- Most residual soils and weathered rock mass without adverse geological settings exposed during staged excavation
- Talus slope deposit
- Naturally cemented sands and gravel with some cohesion
- Heterogeneous and stratified soils
- Stiff/cohesive soils such as clayey silts and clay with low plasticity that are not prone to creep
- Well graded granular soil with sufficient apparent cohesion of minimum 5kPa as maintained by capillary suction with appropriate moisture content
- Ground profile above groundwater level

1.7 Ground Condition not Well-suited for Soil Nailing

The following ground types or condition are not considered well suited to soil nailing or limit its application:

- Loose clean granular soils with field standard penetration N values lower than about 10 or relatives densities of less than about 30 percent. These type of soils will not generally exhibit stand-up time and are also sensitive to vibrations induced by construction equipment
- Granular cohesionless soil of uniforms size (poorly graded) with a uniformity coefficient less than 2, unless in a very dense condition. During the construction, these soil type will tend to ravel when exposed due to lack of apparent cohesion
- Soil containing excessive moisture or wet pockets such that they tend to slough and create the face stability problem when exposed i.e., the apparent cohesion is destroyed. For most ground types, the water table is not appropriate as such condition usually creates very difficult construction.
- Organic soils or clay with Liquidity Index greater than 0.2 and undrained shear strength less than 50 kN/m^2.
- Rock or decomposed rock with weak structural discontinuous that are inclined steeply towards and daylight into excavation face

CHAPTER 2: DESCRIPTION OF SOIL NAILING AND BASIC MECHANIC

2.1 Background of Soil Nailing

Retaining walls using anchored bars date back to the 1960's and earlier. Soil nailing technology can be traced back to the use of the "New Austrian Tunneling Method" (NATM), in which grouted rock bolts and shotcrete were used for supporting tunnels. This technology was reportedly first applied for the permanent support of retaining walls in a cut in soft rock in France in 1961. The use of grouted "soil nails" and driven soil nails, which consist of solid steel bars and steel angle iron, continued to grow in the 1970's, in France and Germany. The first wall built in France using current soil nail techniques was reported to have been built by Soletanche, in Versailles in 1972, using a high density of grouted soil nails in sand. The wall was on a 21-degree batter, was 60 feet tall, had a reinforced concrete facing and supported an excavation for a railroad track.

In North America, soil nails were first introduced for temporary excavation support in Vancouver, B.C., in the late 1960's and early 1970's. The first documented project in the U.S. was in Portland, Oregon for excavation support of a hospital foundation. The maximum excavation depth was 45 feet. The soils consisted of medium dense to dense silty fine sands. The work was reported to have been completed in 50 to 70 percent of the time required for conventional tieback construction and at a 15 percent cost saving.

Soil nailing technique to reinforce slope was introduced to Malaysia in early 1980s and of the early slopes reinforced by soil nailing was Bukit Jugra Army Camp slope in Banting in 1983. While Pos Betau-Ringlet Highway , a new JKR R3 hilly road of about 85km, is estimated to have about 55 000 soil nails to stabilize steep and high hilly cut slopes.

2.2 Construction Sequence

The following is the typical sequence to construct a soil nail wall using the drill and grout methods of nail installation (FHWA, 1998).

a. Excavate Initial Cut

   It is necessary to ensure that all the surface water will be controlled during the construction process. This is usually done by the use of collector trenches to intercept and divert the surface water before it can impact the construction. The initial cut is excavated typically about 1 to 2 meters depending on the stability of soil to stand the unsupported for a minimum period of 24 to 48 hours. Where face stability is problematic for these periods of times, a stabilizing berm can be left in place until the nails has been installation. For the case that faces stability problems to be most severe, placing of a flash coat of shotcrete is another option.

Final trimming of the excavation face is typically done with a backhole or hydraulic excavator. Usually, the exposed length of the cut is indicated by the area of face that can be stabilized and shotcreted in the course of working shift. Ground disturbance during the excavation should be minimized and loosed areas of the face removed before facing support is applied. The excavated face profile should be reasonably smooth and regularly in order to minimize subsequent shotcrete properties.

b. Drill Hole for Nail

Nails hole are drilled at predetermined locations to a specific length and inclination using drilling method appropriate for the ground. Drilling methds include both uncased methods for more competent material (rotary or rotary percussive methods using air flush and dry auger methods) and cased method for less stable ground (single tube and duplex rotary methods with air or water flush, and hollow stem auger methods).

c. Install and Grout Nail

Plastic centralizer is commonly used to center the nail in the drillhole. However where the nails are installed through a hollow stem auger, centralizer are generally ineffective and a stiffer (200mm or lower slump) grout mix is used to maintain the position of the nail and prevent it from sinking to the bottom of the hole. The nails which are commonly from 19 to 35 mm bars are inserted into the hole and the drillhole is filled with cement grout to bond the nail bar to the surrounding soil.

For peranent nails, the steel bar is typically protected against corrosion damage with a heavy epoxy coating or by encapsulated in a grout-filled corrugated plastic sheathing.

d. Place Drainage System

A prefabricated synthetic drainage mat, placed in vertical strips between the nails head on horizontal spacing equal to that of the nails, is commonly installed against the excavation face before shotcreting occurs, to provide frainage behind the shotcrete face. The drainage strips are extended down to the base of the wall with each excavation lift and connected either directly to a footing drain or to weep holes that penetrate the final wall facing.

e. Place Construction Facing and Installation Bearing Plates

The construction facing typically consist of a mesh-reinforced wet mix shotcrete layer on the order of 100 mm thick, although the thickness and reinforcing details will depend on the specific design. Following placement of the shotcere, a steel bearing plate and securing nuts are placed at each nail head and the nut is hand wrench tightened sufficiently to embed the plate a small distance into the still plastic shotcrete.

f. Repeat Process to Final Grade

The sequence of excavate, install nail and drainage system, and place construction facing is repeated until the final wall grade is achieved. The shotcrete facing may be placed at each lift prior to nail hole drilling and nail installtin, particularly in situation where face stability is a concern.

g. Place Final Facing

For architectural and long term structural durability reason, a CIP concrete facing is the most common final facing being used for transportation application of permanents nail walls. Under the appropriate circumstances, the final facing may also consist of a second layer of structural shotcrete applied following completion of the final excavation. Pre-cast concrte panels may also be used as the final facing for soil nail walls.

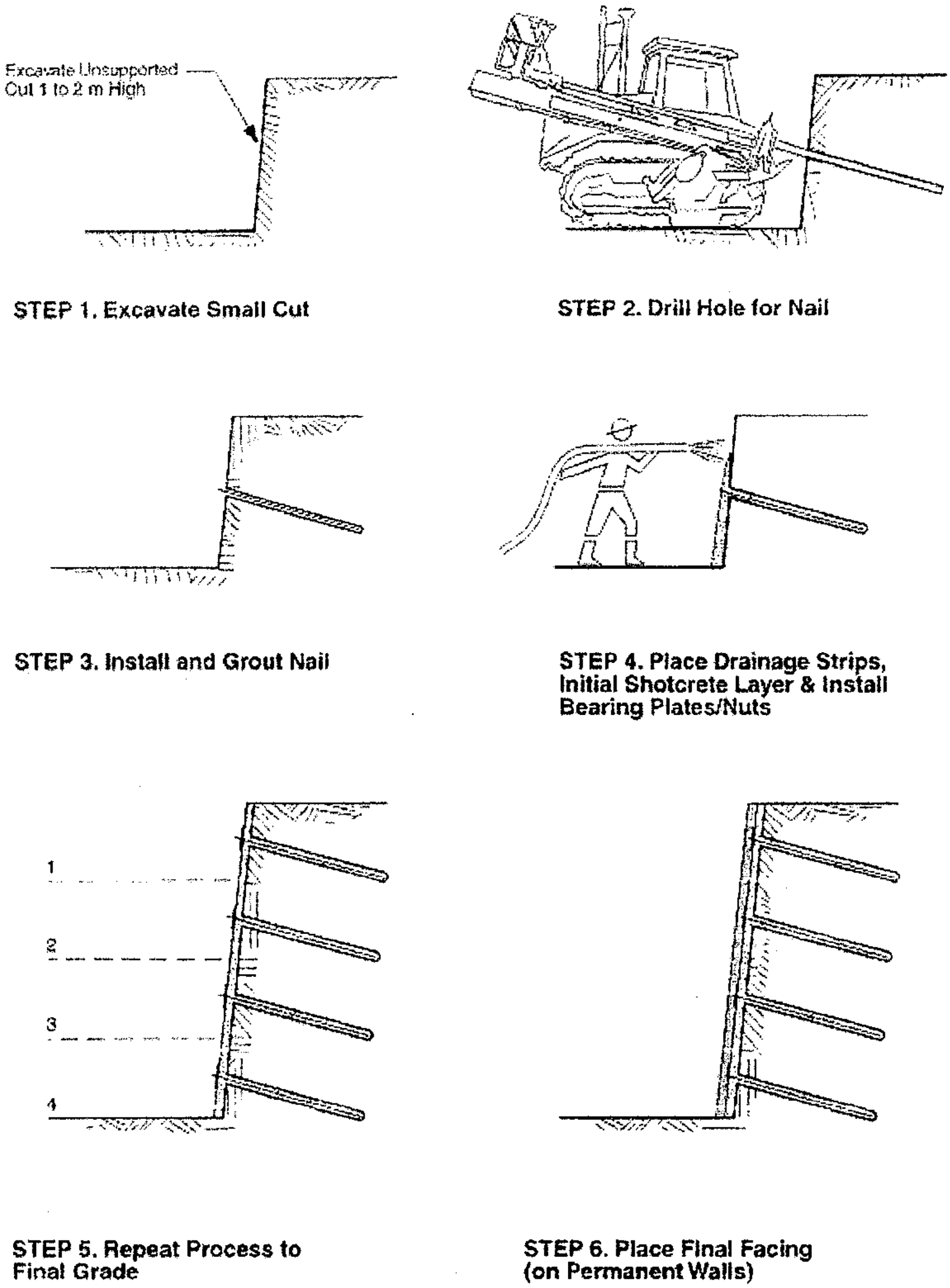


Figure 2.1: Typical Nail Wall Construction Sequence (from FHWA 1998)

2.2 Behavior of soil nail

The fundamental mechanism of soil nailing structure is the development of tensile forces in the "passive" reinforcement as a result of the restraint that the reinforcement and the attached facing offer to lateral deformation of the structure. The reinforcements interact with the ground to support the stressed and strains that would otherwise cause the unreinforced ground to fail. These reinforcements are oriented to correspond in general with the direction of max tensile straining within the soil in order for the generation of tensile loads is dominant.

The tensile forces are developed in the soil nails primarily through the frictional interaction between the soil nails and the ground, and secondarily through the interaction between the soil-nail heads/facing and the ground. The later phenomenon facilitates the development of tension in soil nailing. They also prevent the local failures near the slopes and promote an integral action of the reinforced mass through redistribution of forces among soil nails (GEO, 2006).

The tensile forces in the soil nails reinforce the ground by directly supporting some of the applied shear loadings and increasing the normal stresses in the soil on the potential failure surface, thereby allowing higher frictional shearing resistance to be mobilised. Apart from tension, the shear and bending moment developed in the soil nails may provide secondary resistance to the applied shear loadings. However, due to relatively slender dimensions of the soil nails, these reinforcing contributions are limited by the small flexural strength, and they are usually negligible (FHWA, 1998).

The internal stability of a soil-nailed system is usually considered in respect of two zones, namely the active zone and the passive zone (or resistant zone), which are separated by a potential failure surface (Figure 2.2). Active zone is the region in front of the potential failure surface, where it has a tendency to detach from the slope or retaining wall. Passive zone is the region behind the potential failure surface, where it remains more or less intact.

The soil nails act to tie the active zone to the passive zone. For stability to be achieved:

a. the nail tensile strength must be adequate to provide the support force to stabilize the active block
b. the nails must be embedded a sufficient length into the resistant zone to prevent the a pullout failure

c. combined effect of the nail head strength (as determined by the strength of the facing connection system) and the pullout resistance of the length of the nail between the face and the slip surface must be adequate to provide required nail tension at the slip surface (interface between active and resistance zones)

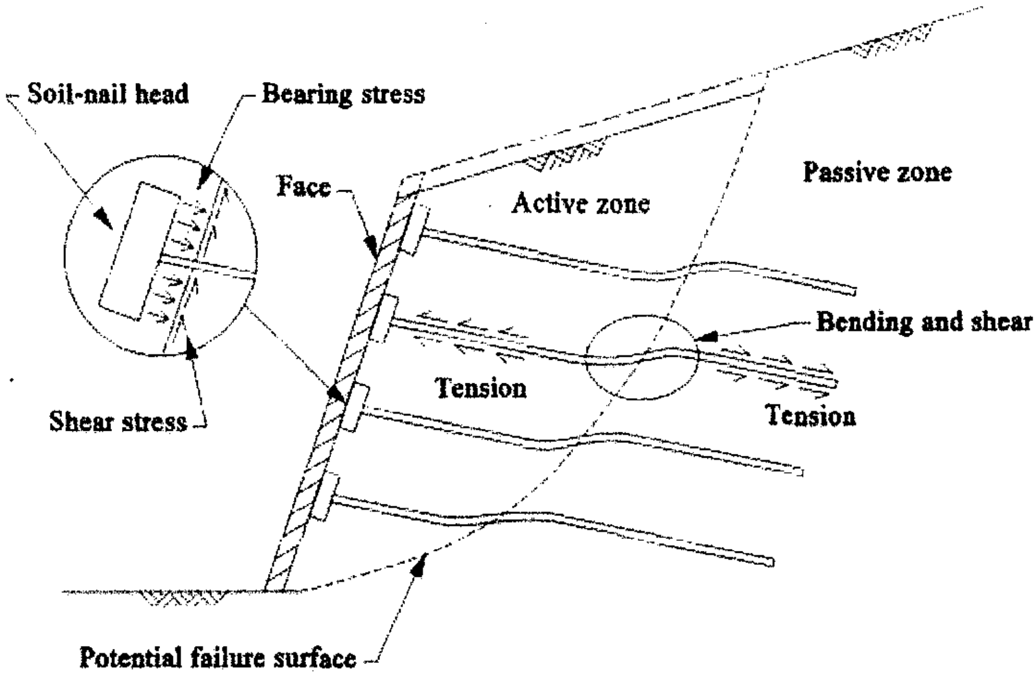


Figure 2.2: Two zones model of soil nailed system (GEO,2006)

2.3 Potential Failure Mechanism of Soil Nailing System

All potential failure modes must be considered in evaluating the available nail force to stabilize the active block defined by any particular slip surface.

The failure modes of soil nails can be categorized into the following:

a) Pullout failure
b) Nail tendon failure
c) Face failure
d) Overall failure (slope instability)

2.2.1 Pullout failure

Pullout failure as illustrated in Figure 2.3 results from insufficient embedded length into the resistant zone to resist the destabilizing force. The pullout capacity of the soil nails is governed by the following factors:

a) The location of the critical slip plane of the slope.

b) The size (diameter) of the grouted hole for soil nail.

c) The ground-grout bond stress (soil skin friction).

2.2.2 Face failure

For a modest strength facing systems, the most likely failure modes of the wall is for the facing or connection to fail as illustrated in Figure 2.4.This aspect of failure mode for soil nailing is sometimes overlooked as it is generally wrongly "assumed" that the face does not resist any earth pressure. For soil nailing works which involve slopes of relatively low height and gentle gradient, the earth pressure acting on the shotcrete face is relatively small and nominal shotcrete thickness and reinforcement is adequate.

2.2.3 Nail Tendon Failure

Nail tendon failure as illustrated in Figure 2.5 results from inadequate tensile strength of the nails to provide the resistant force to stabilize the slope. It is primarily governed by the grade of steel used and the diameter of the steel. Typically a minimum nail size of 25mm is used as nail sizes smaller than 25mm may cause installation problems for moderate to long nail lengths due to their low stiffness. Besides specifying the appropriate nail size corresponding to the required resistant force, it is important that proper detailing with regards to corrosion protection of the nails are specified and properly executed at site.

2.2.4 Overall failure (slope instability)

This aspect of failure mode is commonly analyzed based on limit equilibrium methods. The analyses are carried out iteratively until the nail resistant force corresponds to the critical slip plane from the limit equilibrium analysis. To carry out such iterative analysis, it is important that the nail load diagram (Figure 2.6) is established. From Figure 2.5, it can be seen that the nail load diagram consists of three zones, A, B and C. Zone A is governed by the strength of the facing, T and also the ground-grout bond stress, Q. If the facing of soil nails is designed to take full tensile capacity of the nail, then the full tensile capacity of the nail can be mobilized even if the critical slip circle passes through Zone A. However, to design the facing with full tensile capacity of nails instead of lower T is not economical for high slope (e.g. more than 15m). Zone B is governed by the nail tendon tensile strength and Zone C is governed by the ground-grout bond stress, Q.

From the diagram, it is clear that the mobilized nail resistance should not exceed the nail load envelope developed from the three failure criteria discussed earlier. Therefore, the nail resistance to be input into slope stability analysis should refer to the nail load diagram (Figure 2.6) corresponding to the available bond length for the critical slip plane (Figure 2.7).

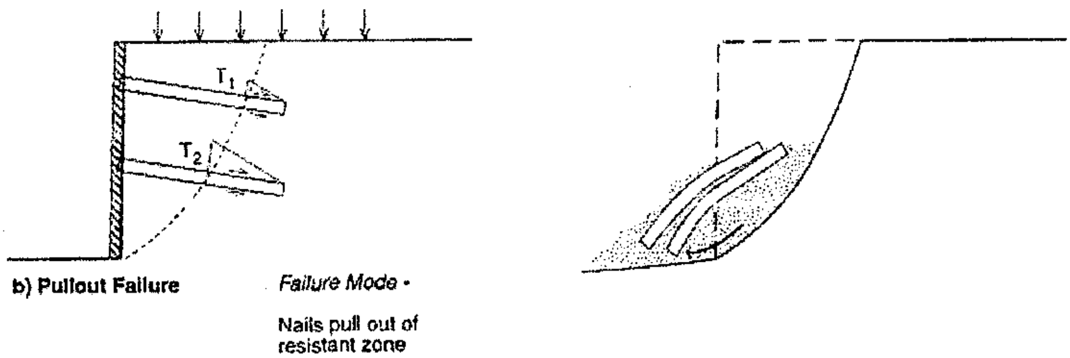


Figure 2.3: Pullout failure mode (from FHWA,1998)

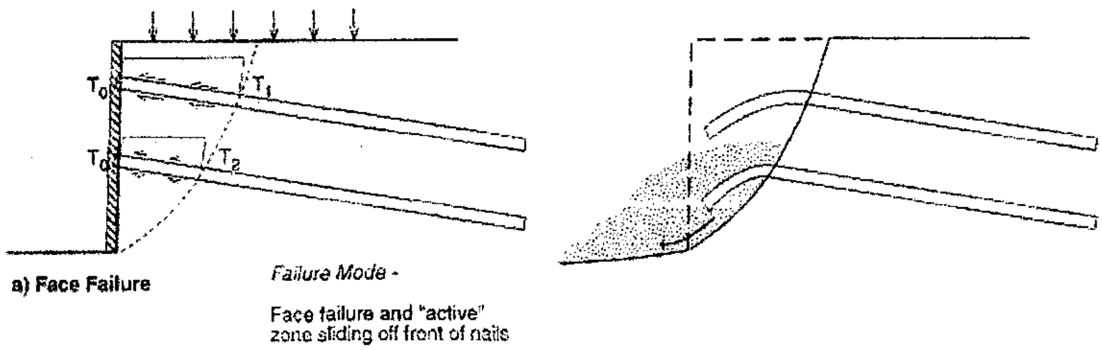


Figure 2.4: Face failure mode (from FHWA,1998)

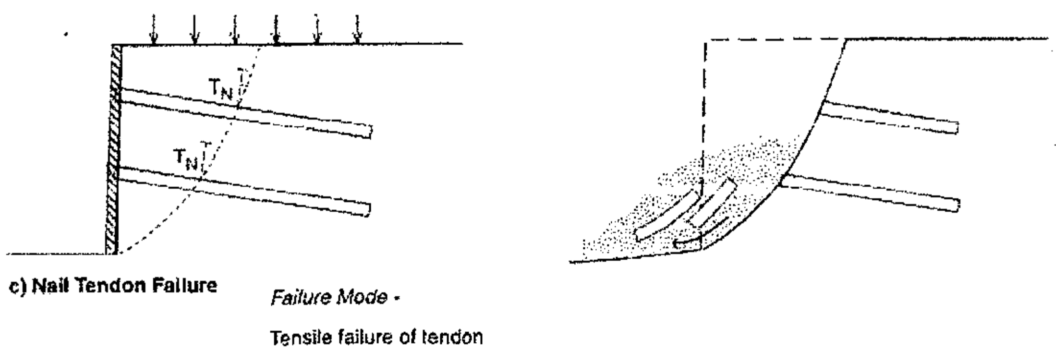


Figure 2.5: Nail tendon failure mode (from FHWA,1998)

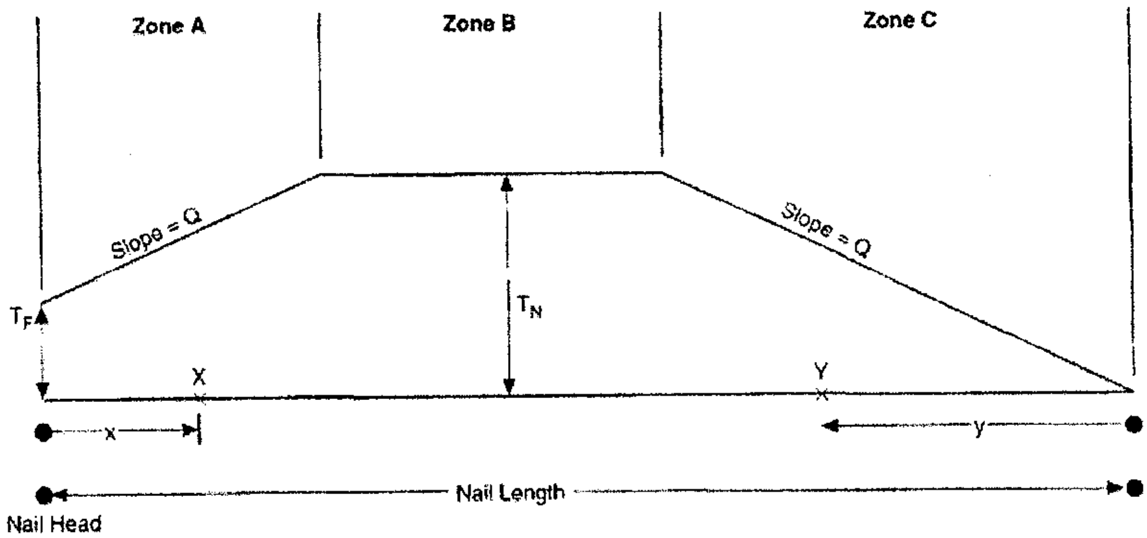


Figure 2.6: Nail load diagram (from FHWA 1998)

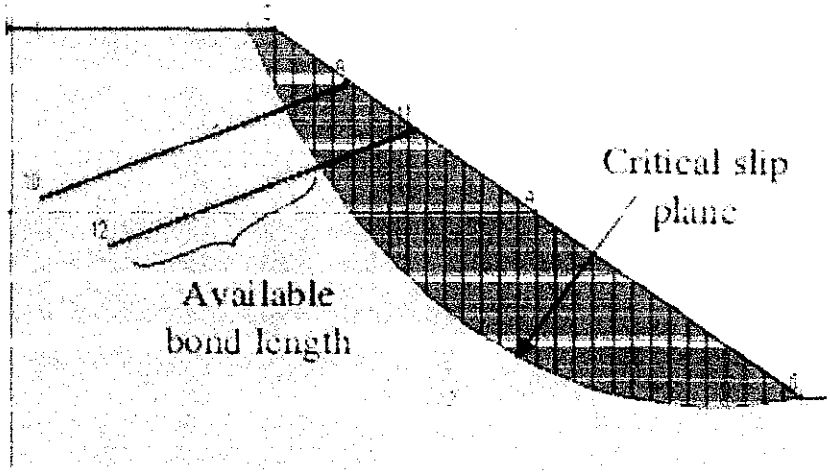


Figure 2.7: Available bond length from slope stability analysis. (Tan & Chow 2004a)

2.4 Nail – Ground Interaction

In the active zone, forces are developed in soil nails through interaction among the ground, the soil nails, the soil-nail heads and the slope facing (Figure 2.2). The reinforcing action of the soil nails is provided primarily through two fundamental mechanisms of nail-ground interaction, namely (i) the nail-ground friction that leads to the development of axial tension or compression in the soil nails, and (ii) the soil bearing stress on the soil nails and the nail-ground friction on the sides of soil nails that lead to the development of shear and bending moments in the soil nails.

If the soil nails are aligned close to the direction of the maximum tensile strain of the soil, the reinforcing action is provided primarily by the tension in the soil nails developed through the mechanism of nail-ground friction. Some secondary reinforcing action is also provided by the shear stresses and bending moments in the soil nails developed through the mechanism of soil bearing stresses as well as the nail-ground friction at the sides of soil nails. Many studies have, however, demonstrated that the contributions of shear stresses and bending moments of soil nails are negligible under service load conditions (Jewell & Pedley, 1992). In contrast, if the soil nails are aligned in the direction of compressive strain in the soil, compressive forces will be developed in the soil nails. This can lead to a decrease in normal effective stress at the soil in the potential shear surface, which reduces the shearing resistance of the reinforced ground mass.

In general, the tensile efficiency of a soil nail decreases as the inclination of soil nail to horizontal, as indicated in Figure 3.2, increases. For most soils, where the soil nails are sub-horizontally inclined, the minimum deformation required to mobilise the full bending and shear resistance of a soil nail is about one order of magnitude greater than that required to mobilise the full tensile strength, and hence the primary action of the soil nails is in tension (Clouterre, 1991; FHWA, 1998). However, if the soil nails are deeply inclined, the efficiency of the soil nails will be reduced significantly as some of the soil nails may be in compression. Therefore, steeply inclined soil nails should be used with caution. Figure 3.3 shows the effect of reinforcement orientation on the shear strength of the reinforced soil.

CHAPTER 3: SITE INVESTIGATION

3.1 Ground Characterization

The feasibility of an economical and reliable design for soil nailing depends on the existing topography, subsurface conditions, soil/rock properties, and the location and condition of adjacent structures. Subsurface investigation must valuate site stability, adjacent structure settlement potential, drainage requirements, anchor capacities, underground utilities and groundwater, before designing a soil nailed earth retention system.

Subsurface investigations must explore not only the location of the face of the soil nailed structure, but the region of the anticipated bond length of the nail. Each project must be treated separately, as both the soil conditions and risks may vary widely. Basic ingredients for a rational subsurface investigation program include review of the regional geology, a field reconnaissance, a subsurface exploration and laboratory testing. The aim of the investigation is to determine the most economical means, adequate information about the block of ground in which nails will be installed to permit the safe, economical design and construction. This includes the information on groundwater and an assessment of excavation face stability.

The primary design considerations for soil nail walls are adequate stability, durability and limited wall deflections. The most critical component in the design and construction of a soil nail wall is an adequate design phase site investigation.

The recommended phase of site investigation for soil nail walls are:

1. Regional Geology
2. Field Reconnaissance
3. Subsurface Exploration
4. Laboratory Testing

3.2 Regional Geology

A review of the regional geology should be performed prior to conducting a field reconnaissance or subsurface exploration to better understand the geology and groundwater conditions of the region. The information acquired in this first phase of the site evaluation will be used to further develop the field reconnaissance and subsurface exploration. Information concerning the regional geology may be obtained from geologic maps, air photographs, surveys and soils reports for adjacent or nearby sites.

3.3 Field Reconnaissance

Field reconnaissance should be conducted by a geotechnical engineer or by an engineering geologist. A well planned and conducted field reconnaissance should consist of collecting any existing data relating to the subsurface conditions and making a field visit to:

- Select limits and intervals for topographic cross-sections.
- Recording site access condition for works forces and equipments
- Observe surface drainage patterns, seepage and vegetative characteristics to estimate drainage requirements. Corrosion of existing drainage structures should be noted to identify if a corrosive environment may exist for shotcrete and/or steel materials.
- Determine the extent, nature, and situation of any above or below ground utilities, basements and/or substructures of adjacent structures which may impact explorations or construction.
- Assess available right-of-way.
- Determine areas of potential instability, such as deep deposits of weak cohesive and organic soils, slide debris, high groundwater table, bedrock outcrops, etc.
- Study surface geologic features including rock outcroppings and landforms. Existing cuts or excavations should be used to identify subsurface stratification.

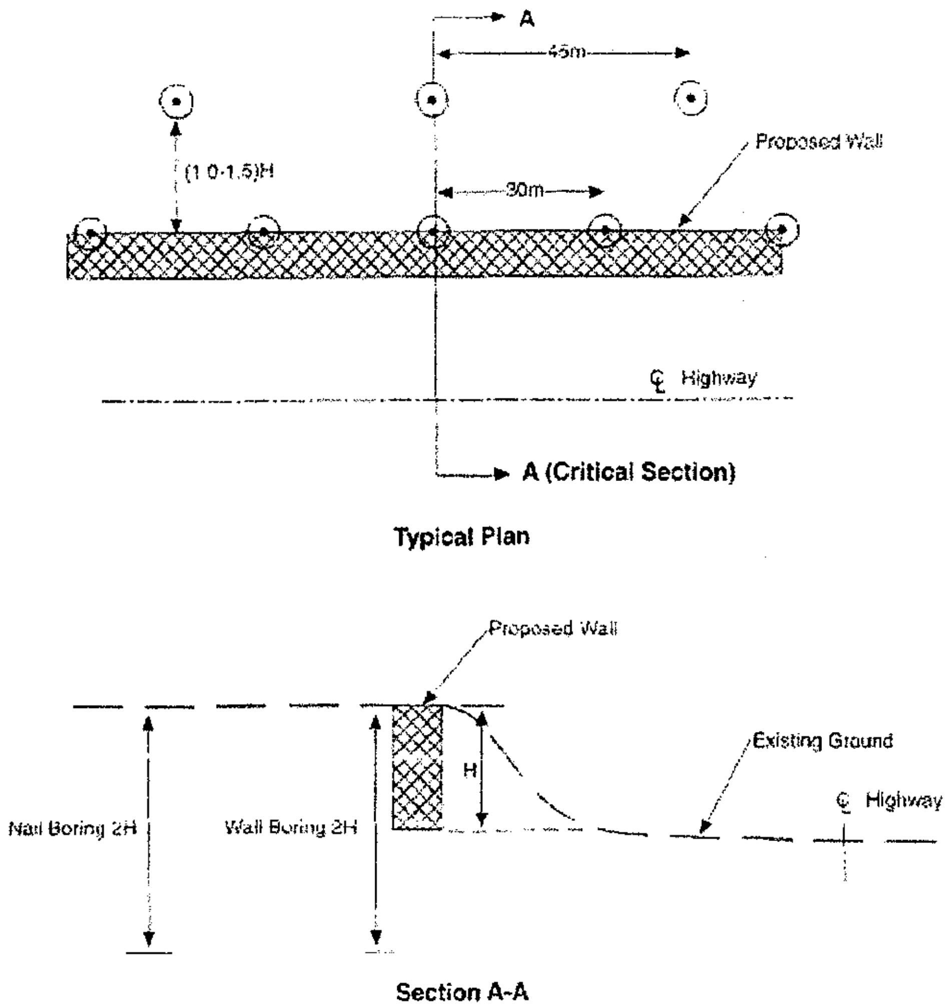
3.4 Subsurface Exploration

Subsurface exploration should be sufficiently detailed to determine soil/rock stratigraphy in the zones affected by the proposed soil nail wall construction, develop subsurface cross-section adequate for stability analyses, allow an estimate of the pull out capacity of the nails and develop the sufficient information to design an efficient internal drainage system. The subsurface exploration program may consist of soil borings, test pits, cone penetration tests, soil soundings, etc.

The number, type, and location of the subsurface explorations are usually determined by the geotechnical engineer, based on the results of the field reconnaissance and available existing subsurface data. The exploration must be sufficient to evaluate the geologic and subsurface profile in the area of construction.

Engineering must be used on a project-by-project basis to determine the final subsurface program. The following are recommended general guideline:

1. Wall boring spaced at approximately 30 m intervals along the structure alignment (figure 3.1). In flat or gentle sloping ground, the nail borings are recommended to lie on approximately 45 m centers at distance behind the wall equal to approximate 1.5 times height. For sloping ground condition, the distance behind the wall of the nail borings may be increased up to approximately 1.5 to 2.0 times the wall height, depending on the backslope. Static cone penetrometer tests may be substituted for up to half of the boring in a line. At critical sections, boring may be added in front of the proposed wall line to better define the soil/rock stratigraphy.
2. Boring depths will be a function of the encountered subsurface conditions, but where bedrock is within a reasonable depths, extract a minimum rock core length of 3m. The core is used to distinguish between the boulders and bedrocks and to identify the rock type. Wall borings and nail borings are usually extended to a minimum depth equal to at lest the proposed wall height below the wall base, or 3 m into if rock is encountered at lesser depth.
3. Standard penetration tests (SPT) should be performed at 1.5 m intervals and the soil samples sent to the soils laboratory for visual identification, classification and testing. In ground that may contain thin weak soil layers, continuous SPT sampling is recommended. Undisturbed tube samples or in-situ strength testing should be taken in cohesive soil deposits at 1.5 m to 3 m depths intervals in sufficient borings to determine the characteristic and variations of the soils deposits. Careful static water level determination must be made on completion of the boring. A notation should be made on removal of tools and/or casing to whether the hole stayed open or of the depth f collapse. At least one nail and one wall boring should be converted to a water observation well for long term water level readings.
4. Test cuts or pits are recommended to be approximately 6 to 8 m long and 2 to 2.5 m deep and left open for 3 to 4 days. A daily inspection is recommended, with "stand-up" condition documented and photographic record prepared. The long axis of the cut or pit should be parallel to, and located in front of the proposed wall face. In residual soils, a joint survey is made to determine the major joint system and heir orientation and joint surface characteristic.



Notes:

1. Nail boring located (1.5-2.0)H behind facing for sloping ground conditions
2. Test cut or test pit recommended, when feasible, to evaluate excavation “stand-up” time.

Figure 3.1: Site Exploration Guideline for Soil Nail Walls (from FHWA 1991)

3.5. Laboratory Testing

Soil samples should be visually examined and appropriate tests performed for classification according to the Unified Soil Classification System (ASTM D 2488-69). The focus on testing is to obtain reliable estimates of the unit weight and strength of the soil or rock. These tests will permit the engineer to decide what further tests will best describe the engineering behavior of the soil at a given project site. Index testing includes determining the moisture content, Atterberg limits, compressive strength and gradation. Soils test to determine the corrosion potential of the soil should also be conducted.

Soil

Shear strength determination from unconfined compression tests, direct shear tests, or triaxial compression tests will be needed for the stability analysis. Both undrained and drained (effective stress) strength parameters will be needed for cohesive soils to permit evaluation of both long-term and short-term conditions.

Creep Potential

The Atterberg limits can be used to identify clays soil that should be considered as either non-application for soil nailing or as potentially problematically with respect to long term creep. Nails should be located in organic soils or cohesive soils with Liquidity Index greater than 0.2 and undrained shear strength less than 50kN/m2 without evaluating the long term creep behavior of the soil nails by performing tests. The Liquidity Index is define as:

$$LI = \frac{W - WP}{Wl - WP}$$

WL = Liquid Limit Water Content

WP = Plastic Limit Water Content

W = Natural Water Content

Corrosion Potential

Properties to indicate the potential aggressiveness of the in-situ soil within the reinforced zone should be measured. The tests include: pH, electrical resistivity, and salt content (sulfate, sulfides, and chlorides). These test results will provide necessary information for planning degradation potential and protection. The critical values for ground aggressiveness commonly associated with ASTM standards are summarized in Table 3.1.

Table 3.1: Recommended Electrochemical Properties for Soils when using soil nail

Test	ASTM Standard	Critical values
Resistivity	G-57-78 (ASTM)	Below 2000 ohm/cm
pH	G-51-77 (ASTM)	Below 4.5
Sulfates	California DOT test 407	Above 500 ppm
Chlorides	California DOT test 422	Above 100 ppm

Rock

Analysis of rock properties is more field oriented as the presence and location of fissures, joints or other discontinuous will control the overall strength of the rock mass. Determination of rock properties (mass strength) is based on information form both laboratory and field testing:

a. from the rock mass and the depth of overburden
b. rock type
c. rock quality designation (RQD)
d. Joint spacing and orientation
e. Startification
f. Rock materials
g. Water pressure in joints.

3.6 Final Evaluation

Based on the results of the site investigation, a preliminary feasibility evaluation can be made to determine if a successful soil nail design can be implemented with a relatively high degree of confidence. This requires an understanding of ground conditions for which soil nailing is and is not well suited. These conditions are discussed in Chapter 1.

Based on the subsurface investigation results, it is important to show boreholes profiles in the cross section of slopes to obtain a representatives subsoil profile. Generally, the subsoil can be divided into three (3) layers:

a. SPT < 50 – layer of soft to hard overburden materials. CIU or shear box test result can be used as they usually carried out on samples recovered from this layer.
b. SP > 50 – layer of very hard overburden materials. Higher strength can be used through samples are usually not obtained from this layer.
c. Bedrock layer. Usually a very high strength as assigned to the bedrock in the stability analysis as the slip plane could not penetrate the bedrock

The selection of soil parameters shall be based on the following criteria:

a. For the design of new slopes, peak strength obtained form CIU test or shear box test can be used. It is recommended that for Conventional Approach (CIRIA & Common Practice), the moderately conservative soils parameter shall be adopted. It is important to note that the peak strength from the CIU test shall be determined from the relevant stress range and the peak strength should never be extrapolated form the tested stress range.
b. For back analysis of collapsed slopes, residual strength obtained form multiple reversal shear box test or ring shear test may be used as reference.
c. For fill embankment to be seated on soft ground, undrained shear strength shall be used for the ground in the stability analysis.

3.7 Estimating Pullout Resistance

Verification of the ultimate soil-nail pullout resistance, Q_u assumed in design is essential to ensure structure safety. It should be considered an extension of design. Further, the actual pullout resistance achieved can be affected by:

- Soil or rock type and shear strength
- Roughness of drillhole wall (will vary with drilling method used)
- Final drillhole diameter
- Loose drill cutting left along the bottom of the drillhole (can occur particularly with auger drilling or when air is used to remove drill cutting if air compressor capacity is not large)

- Contractor drilling and grouting techniques, expertise and workmanship
- Amount of time hole left open before grouting

Nail pullout resistance should be based on experience with open hole methods of construction if soil condition allow. If inadequate exist to provide a conservative design value, and then a pre-contract test nail program should be considered to determine the appropriate design values, particularly on large project. It is imperative that field pull out testing be done during construction to verify the estimated pullout resistance used in the design.

A. Cohesionless (Granular) Soils

For tremie or low pressure grouted nails in dry cohesionless soil, the ranges of ultimate pullout resistance are indicated in Table 3.2.

B. Cohesive Soil

For tremie grouted nails, the ultimate pullout resistance can be estimated as 0.25 to 0.75 times the average undrained shear strength with the lower factors associated with the stiffer and harder clays. For augered holes, a lower factor may be warranted because it is influenced by the care teaken in cleaning the drillhole. For sandy and silty clays, the factor is slightly higher than the range above. Typical values of the ultimate pullout resistance for cohesive soil are indicated in Table 3.3.

Table 3.2: Ultimate Bond Stress for Cohesionless Soil

Construction Method	Soil Type	Unit Ultimate Bond Stress (kN/m^2)
Open Hole	Non-plastic silts	20-30
	Medium dense sand and silty sand/sandy silk	50-75
	Dense silty sand and gravel	80-100
	Very dense silty sand and gravel	120-240
	Loess	25-75

Table 3.3: ultimate bond Stress for Cohesive Soil

Construction Method	Soil Type	Unit Ultimate Bond Stress (kN/m^2)
Open Hole	Stiff Clay	40-60
	Stiff Clay Silt	40-100
	Stiff Sandy Clay	100-200

C. Rock

The ultimate pullout resistance for tremie grouted nails in component massive rock may be taken as 10 percent of the uniaxial compressive strength of the rock up to a maximum value of 4000 kN/m^3. Estimated pullout resistance for different rock types are given below.

Table 3.4: Ultimate Bond Stress for Rock

Construction Method	Soil Type	Unit Ultimate Bond Stress (kN/m2)
Rotary Drilled	Marl/Limestone	300-400
	Phillite	100-300
	Chalk	500-600
	Soft Dolomite	400-600
	Fissured Dolomite	600-1000
	Weathered Sandstone	200-300
	Weathered Shale	100-150
	Weathered Schist	100-175
	Basalt	500-600

CHAPTER 4: DESIGN OF SOIL NAIL WALL

4.1 Introduction

The design procedure presented in this manual draws heavily on a FHWA documents "Manual for Design & Construction Monitoring of Soil Nail Walls" (FHWA-SA-96-069). The design is based on a slip surface limit equilibrium design approach that combines conventional reinforced slope design requirements with reinforced soil wall design methods. It incorporates the reinforcing effect of the nails, including consideration of the strength of the nail head connection to the facing, the strength of the nail tendon itself, and the pullout resistance of the nail-ground interface.

4.1.1 Limit State

The reliability of a soil-nailed system depends not only on the calculated factor of safety, but also on the methods of analysis, uncertainties in the ground and groundwater level and also loss of function. Thus, to provide for an acceptable level of safety, the design procedure for soil nailing retaining wall addresses the following important limit states:

Strength Limit State

The strength limit state is the limit that addresses potential failure mechanisms or collapse states of the soil nail wall system. Strength limit states address the stability under expected forces. Extreme limit states address the survival under extreme loads, e.g., seismic loading.

Service Limit State

The service limit state is the limit that addresses loss of service function that resulting from excessive wall deformation and is defined by restriction on stress, deformation and facing crack width under regular service conditions.

4.1.2 Design Approach

The design approach presented on this manual is Service Load Design (*SLD*). This design is defined in the Standard Specification for Highway Bridges, 15^{th} Edition (AASHTO, 1992). *SLD* of soil nailing retaining wall required the allowable nails loads and the factored soil strength exceed the applied loads. The allowable nails loads are determined by both structural (i.e., allowable tendon stress or loads) and geotechnical (i.e. , allowable pullout resistance) elements. The factored soil strength is determined by applying a factor of safety to the ultimate soil strength. In order to define the maximum demand on the resisting elements, several combinations of loading are applied to capture the maximum potential destabilizing effect of the loads.

The service limit state is investigated by addressing the overall displacement of the walls and the reinforced and retained ground and by applying limitation on the cracks widths (steel stress) in the wall facing in certain cases.

4.2 Soil Nail Wall Stability Consideration

All potential failure modes of soil nail must be consider in order to address the strength limit state condition for soil nail wall. These failure modes including external modes of failure that do not specifically intersect the reinforcements themselves, internal modes that involve failure either the reinforcing tendon or the facing or both and mixed failures modes that involve internal failure of the reinforced zone and which extend beyond the physical limits of the reinforced block of ground (Figure 4.1). Both internal and mixed failure modes involve considering of yield and rupture of the nails, pullout of the nails and failure of the wall facing or the facing's connection to the nails.

Local stability of the facing during excavation is one of the most important considerations in soil nail wall construction. This failure of mode is not amendable to conventional stability analysis and is typically addressed during design by field test cut to demonstrate that the face can stand unsupported for sufficient time to allow nail and construction facing installation. Local sloughing of the face, possibly extending through to the surface, can be relatively sudden and is most prevalent at shallow depths where loose fill/highly weathered materials is more likely to be encountered.

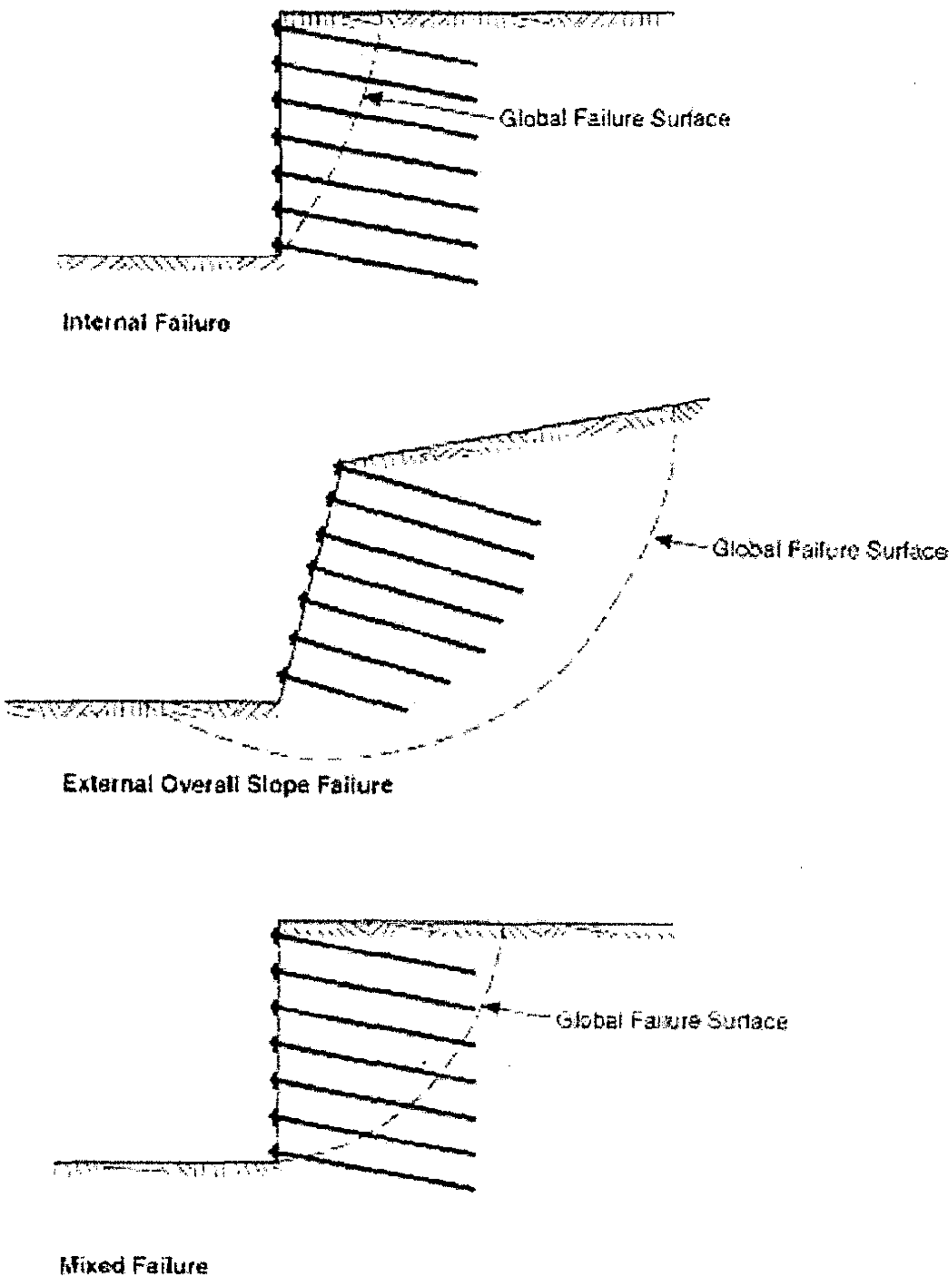


Figure 4.1 Potential Failure Modes of Soil Nail Wall (from FHWA)

4.2.1 Basic Concept

The limiting equilibrium approach to soil nail wall strength limit state design is summarized on Figure 4.2. The method is demonstrated for potential planar slip surface in which a global factor of safety is defined as the ratio of the resisting to driving forces along the potential slip surface.

The equilibrium of an unreinforced block of the ground is initially addressed in Figure 4.2. Figure 4.2 (a) show a free body diagram on the left, acted upon by the self weight of the block of soil located above the slip surface. Considering force equilibrium of the block enables calculation of the normal stress and shear forces on the potential sliding plane. The factor of safety can then be defined as the ratio of the resisting forces to the driving forces, as shown. The expression for the global factor of safety, F is a Conventional factor of safety for an unreinforced slope. Shown next to the free body diagram is a conventional force polygon in which factor of safety F is that, when applied to both cohesive and frictional components of the soil shear strength, will close the force polygon and satisfy limiting equilibrium. For the force planar slip surface considered, the same expression for the global factor of safety F, is derived from considering equilibrium of the free body diagram, can be derived from the force polygon.

A single reinforcing element is introduced to examine the manner in which the reinforcement improves the factor of safety or the stability of the sliding block of ground (Figure 4.2 (b)). The global factor of safety F can be derived from a consideration of either the free body diagram or the force polygon. The effect *pf* the reinforcement is to improve stability by both

a) Increasing the normal force and hence the shear resistance along the slip surface in frictional soil

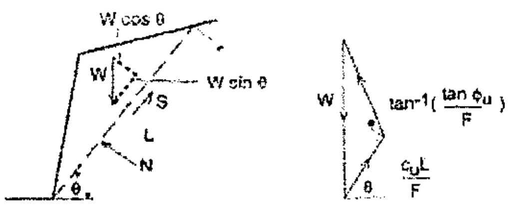
b) Reducing the driving force along the slip surface in both frictional and cohesive soils.

More importance is the shape of the nail strength diagram indicated in Figure 4.2 (b) and further presented on Figure 4.3 for clarity. Figure 4.3 shows that, for any particular sliding wedge, the reinforcing contribution of the nail are a function of the location at which the associated slip surface intersect the nail. The nail reinforcing strength may be limited by tensile failure of the nail tendon, pullout of the nail or structural failure of the facing/nail head connection system. The contribution of any nail to the stability of a particular sliding block will be the least

a) The tensile strength of the nail
b) The pullout resistance of the length of nail beyond the slip surface
c) The nail head strength stud plus the pullout resistance of the length of nail between the slip surface and face of the wall.

Multiple nails are considered (Figure 4.2 (c)) as a simple extension of the single nail problem and shows the available design support form any particular sliding block of ground depends on where the nail intersects the sliding surfaces. Examining Figure 4.2 (c), it can be seen that for the identified slip surface, the upper nail does not intersect the slip surface and therefore does not contribute to its stability. However, the upper nail does not contribute to the stability of the shallower slip surface (closer to the excavation) that intersects the nail. The middle nail provides support T2 that is equal to the pullout resistance of the length of nail beyond the slip surface. The bottom nail provides support T3 and that is equal to the strength of the nail head together with the pullout resistance of the length of the nail between the slip surface and the facing, at that location.

(a) Unreinforced Slope

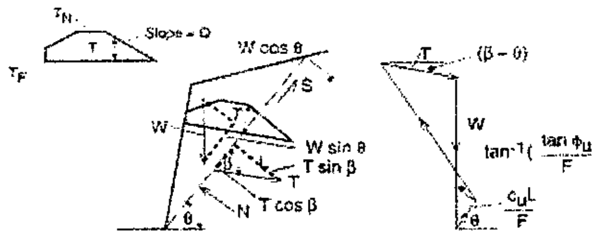


$$F = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{S}{W \sin\theta}$$

$$= \frac{c_uL + N \tan\phi_u}{W \sin\theta}$$

$$= \frac{c_uL + W \cos\theta \tan\phi_u}{W \sin\theta}$$

(b) Reinforced Slope - Single Nail

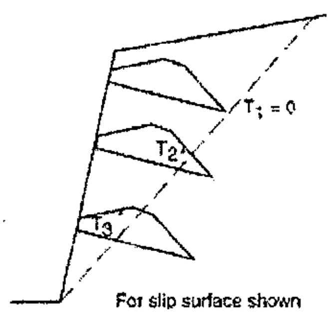


$$F = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{S}{(W \sin\theta - T\cos\beta)}$$

$$= \frac{c_uL + N \tan\phi_u}{(W \sin\theta - T\cos\beta)}$$

$$= \frac{c_uL + (W\cos\theta + T\sin\beta)\tan\phi_u}{(W \sin\theta - T\cos\beta)}$$

(c) Reinforced Slope - Multiple Nails



For slip surface shown

$T = T_2 + T_3$

Expressions for Factor of Safety as given in (b).

(d) Demonstration Example (SLD)

Unreinforced Wall [see (a)]

$$F = \frac{c_uL + W\cos\theta \tan\phi_u}{W \sin\theta}$$

for

$c_u = 5\ \text{kN/m}^2$
$L = 5\ \text{m}$
$\phi_u = 30°$
$W = 115$ kN/m of wall length
$\theta = 60°$

$$F = \frac{5 \times 5 + 115 \cos 60° \tan 30°}{115 \sin 60°}$$

$= 0.58$

Reinforced Wall [see (b)]

$$F = \frac{c_uL + (W\cos\theta + T\sin\beta)\tan\phi_u}{(W\sin\theta - T\cos\beta)}$$

for

$\beta = 75°$
$T = 80$ kN/m of wall length

$$F = \frac{5 \times 5 + (115 \cos 60° + 80 \sin 75°) \tan 30°}{(115 \sin 60° - 80 \cos 75°)}$$

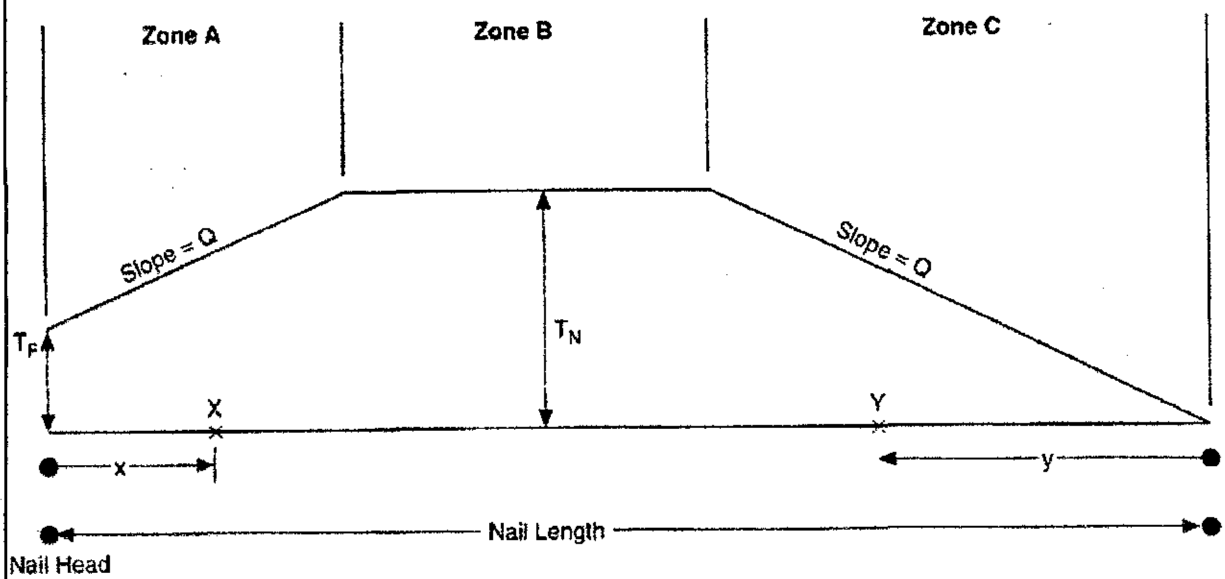
$= 1.3$

TERMINOLOGY

Service Load Design

W = service load (F)
C = ultimate soil cohesion (F/L^2)
φ = ultimate soil friction angle (°)
F = global factor of safety applied to soil shear strengths
T_{FN} = nominal nail head strength (F)
$T_F = \alpha_F T_{FN}$ = allowable nail head load (F)
T_{NN} = nominal nail tendon strength (F)
$T_N = \alpha_N T_{NN}$ = allowable nail tendon load (F)
Q_u = ultimate pullout resistance (F/L)
$Q_d = \alpha_Q Q_u$ = allowable pullout resistance (F/L)
T = allowable nail load (F)
S = resisting shear force (F)
N = normal force (F)

Figure 4.2 Soil Nail Design: Basic Concept and Terminology (from FHWA 1998)



Nail Support to Slip Surfaces intersecting the Nail in Zone A at Point X = $T_F + Qx$

Nail Support to Slip Surfaces intersecting the Nail in Zone B = T_N

Nail Support to Slip Surfaces intersecting the Nail in Zone C at Point Y = Qy

T_F	= strength of nail head-facing connector	= Allowable Nail Head Load (SLD) = Design Nail Head Strength (LRFD)
T_N	= nail tendon tensile strength	= Allowable Nail Tendon Load (SLD) = Design Nail Tendon Strength (LRFD)
Q	= nail-ground pullout resistance	= Allowable Pullout Resistance (SLD) = Design Pullout Resistance (LRFD)

Figure 4.3: Nail Support Diagram (from FHWA 1998)

4.2.2 Nail Elements

The three (3) aspects that controlling the nail resistance are:

a. Grout-ground strength
b. Grout – Tendon bond
c. Structural strength of nail reinforcement

a. Grout-Ground Strength

The grout-ground strength shall be assessed with considerations of material type, soil/rock strength, method of drilling (roughness of drilled hole), hole cleaning, open hole duration, hole diameter, grouting method and the groundwater condition. FHWA has tabulated some recommended ultimate grout-ground resistance as in Table 1. For larger hole size, the ultimate grout-ground resistance would be less than the one with smaller hole size. This is primary due to relatively poor confinement and higher stress relief for larger drilled hole. For fine cohesive soils, the ultimate grout-ground resistance can be 0.25 to 0.75 times of the undrained shear strength.

In Malaysia, the grout-ground interface resistance for residual soils can be assessed based on empirical expression using SPT-N values.

$Fs = 5 \sim 6 \times \text{SPT-N}$ (kPa)

If the drilled hole is wet or saturated, caution shall be taken to downgrade the grout-ground interface resistance with verification of pull-out test.

If unrealistically high grout-ground interface resistance is used in the design, the installed nail will either faces the pull-out failure or experience excessive creep. It is not acceptable for soil nail having creeping movement of more than 2 mm in one log-cycle of holding time (says from 6 minutes to 60 minutes).

Table 4.1: Recommended Ultimate Grout-Ground Resistance (from FHWA)

Construction Method	Material Type	Ultimate Grout-Soil Resistance (kPa)
Open Hole	Non plastic silt	20 ~ 30
Open Hole	Medium dense sand & silty sand/sandy silt	50 ~ 70
Open Hole	Dense silty sand & gravel	80 ~ 100
Open Hole	Very dense silty sand & gravel	120 ~ 240
Open Hole	Loess	25 ~ 75
Open Hole	Stiff clay	40 ~ 60
Open Hole	Stiff clayey silt	40 ~ 100
Open Hole	Stiff sandy caly	100 ~ 200
Rotary Drilled	Marl/ Limestone	300 ~ 400
Rotary Drilled	Phyllite	100 ~ 300
Rotary Drilled	Chalk	500 ~ 600
Rotary Drilled	Soft dolomite	400 ~ 600
Rotary Drilled	Fissured dolomite	600 ~ 1000
Rotary Drilled	Weathered sandstone	200 ~ 300
Rotary Drilled	Weathered shale	100 ~ 150
Rotary Drilled	Weathered schist	100 ~ 175
Rotary Drilled	Basalt	500 ~ 600

b. Grout – Tendon Bond

For deformed reinforcing bars and continuous threadbars used for nail tendons, the bond between the grout and nail tendons is primarily a result of mechanical interlock, in which the grout mobilized its shear strength against the bar deformations and the ultimate strength of the tendon can be developed within a short embedment length in the grout (e.g.12 to 15 bar diameter). The loose powdery rust appearing on bars after short exposures before installation has no significant effect on the grout.

Grout-tendon bond (in term of force per unit length of nails) is typically an order of magnitude or more higher than the ground-grout bond and is therefore not critical for soil nailing applications when proper grout mix and installation techniques are used.

4.2.3 Structural Tensile Strength of Nail Reinforcement

If the applied nail loading is greater than the structural strength of the nail tendon itself, yield and subsequent rupture may occur. The nominal nail tendon strength, T_{NN}, will be used to define the maximum structural tensile strength of the nail tendon as follows:

$$T_{NN} = A_b F_y$$

Where A_b = nominal area of the bar from Table 4.2

Table 4.2: Bar Size

Bar Designation	Nominal Diameter (mm)	Nominal Area (mm^2)
10	9.6	71
13	12.7	129
16	15.9	199
19	19.1	284
22	22.2	387
25	25.4	510
29	28.7	645
32	32.3	819
36	35.8	1006
43	43.0	1452
57	57.3	2581

4.2.4 Internal Stability

The strength of the nail head may be controlled by the flexural strength of the facing, the punching shear strength of the facing and connection system, or the tensile capacity of headed studs that are typically used in a permanents wall facing connection system.The nail head strength defines the available reinforcement strength at the head of the nail, which is one of the elements required to define the overall reinforcing capacity of the nails.

4.2.5 External Stability

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. It may involve the consideration of (Figure 4.4):

a. Horizontal sliding of the retaining structure along its base, under the lateral earth pressure of the ground retained behind the reinforced mass.
b. Foundation bearing failure of the retaining structure associated with overturning, under the combined structure self weight and lateral earth pressure loading
c. Overall slope stability of the ground on which the retaining structure is located.

Excavation in deep deposits soft to medium clays can move excessively if the weight of the retained soils exceeds the bearing capacity if the soil at subgrade or a deep seated failure develops. Retained excavation in granular soils is generally not subjected to basal instability since the walls are free-draining and the shear strength is adequate at the base. The exception for granular soils is the case where substantial hydrostatic forces build up behind the wall due to inadequate drainage

The external stability of the soil nail walls which are constructed in clay soils must consider the reduction with time in the factor of safety, excess pore water pressure and shear strength. For cuts in overconsolidated clays, the long term reduction in shear can be appreciable.

Designer should be use general bearing capacity theory to check the foundation stability of soil nail walls. The reinforced gravity wall created by soil nailing will be acted on by self-weight together with earth pressure loads from the retained soil. Standard bearing capacity reductions for both inclined and eccentric loading should therefore be considered.

- The geometry of the general bearing capacity failure surface extends to a depth of about 1.5 times the width of the footing in relatively homogeneous soils. The typical base width of soil nail wall may be greatly exceeding typical foundation widths and this requires the designer should the soil and groundwater condition to greater depths than would be common for conventional footings. Changes in soil type or strength and the presence of groundwater in the failure depth can substantially affect the results.
- For fine-grained soils, both drained and undrained loading condition should be evaluated. Construction of a soil nail wall involves unloading of the soil in front of the wall and this can results in long term degradation of soil strength in this area of the foundation. For these condition, undrained strength analyses relevant to short term construction conditions may be less critical than long term drained strength analyses
- For depths of clay beneath the wall that are in the order of the width of the nailed block, general bearing capacity methods that accounts for eccentric and inclined loading should be applied. A minimum factor of safety of 2.5 times is required.
- For depths of clay beneath the wall that are significantly less than the width of the nailed block, bearing failure modes may be limited to a portion of the nailed block. Under these conditions, there may be essentially no net lateral loading on the nailed block portion, since the nailed tensile loads may balance the earth pressure loads. In addition, the weight of the block may be partially supported by side shear forces acting along the vertical failure surface that passes through the soil nail block. Under these conditions the following applies:

$$FS = \frac{N_c C_u}{H\,(\gamma - C_u / y)} \leq 2.5$$

Where:

H	= height of exacavtion
Y	= cohesive soil depth below subgrade << width of nailed block
C_u	= ultimate cohesion
γ	= unit weight
N_c	= bearing capacity factor

- Overstresses of thin layer immediately below the assumed footing level are not accounted for in the general bearing capacity approach. Soft soil layers that exist within a depth less than the footing width should be analyzed for overstress. In addition, wedge or other non-circular surfaces through the soft layer should be checked

In general a rigorous analysis of bearing capacity will be required in cohesive soils under the following conditions:

- For cohesive soil depth below subgrade equal to the width of the nailed block

$$FS = \frac{5.14C_u}{H\gamma} \leq 2.5$$

- For cohesive soil depth below subgrade less than the width of the nailed block

$$FS = \frac{5.14C_u}{H(\gamma - C_u / y)} \leq 2.5$$

Where: H = height of exacavtion

Y = cohesive soil depth below subgrade << width of nailed block

C_u = ultimate cohesion

γ = unit weight

4.3 Design Approach

4.3.1 Slip Surface Method

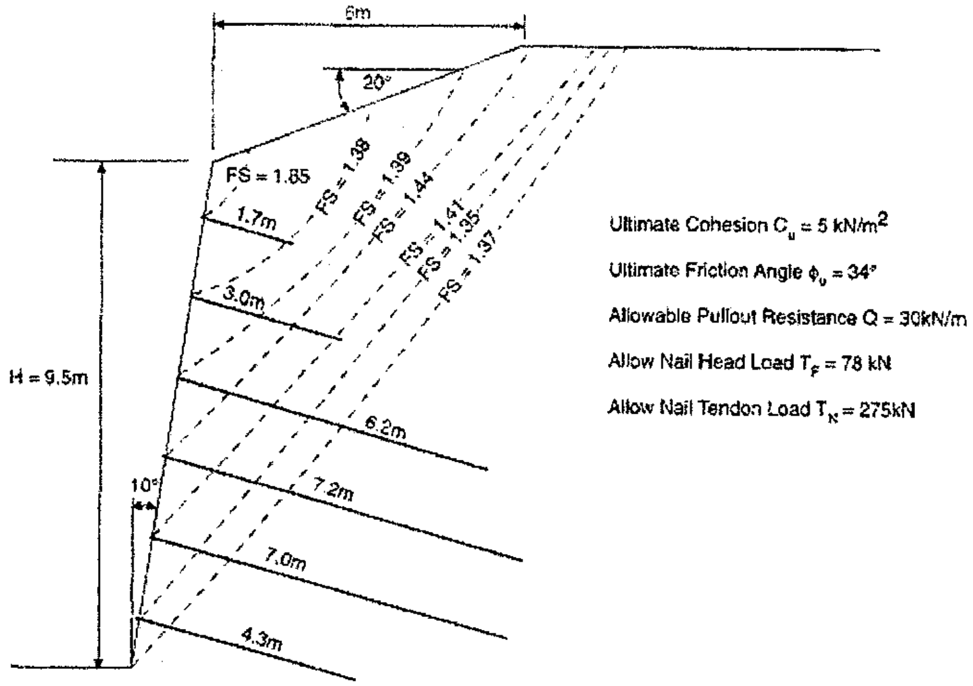
Slip surface limiting equilibrium design methods consider the global stability of zones of ground defined by potential failures surface. These methods have been widely used in conventional slope stability analyses of unreinforced soils and have been demonstrate to provide good correlation with actual performance in such applications. Furthermore, virtually all current practical design methods for soil nail wall are based on the slip surface limiting equilibrium technique. As with the corresponding slope stability models, a critical slip surface is identified as that yielding the lowest calculated factor of safety, taking into account the support provided by the installed reinforcing.

As with the classical slope stability limiting equilibrium models from which the soil nail models have been derived, a variety of slip surface shape can be analyzed. These shapes include planar, bilinear, and piecewise linear surface, together with circles and log spiral.

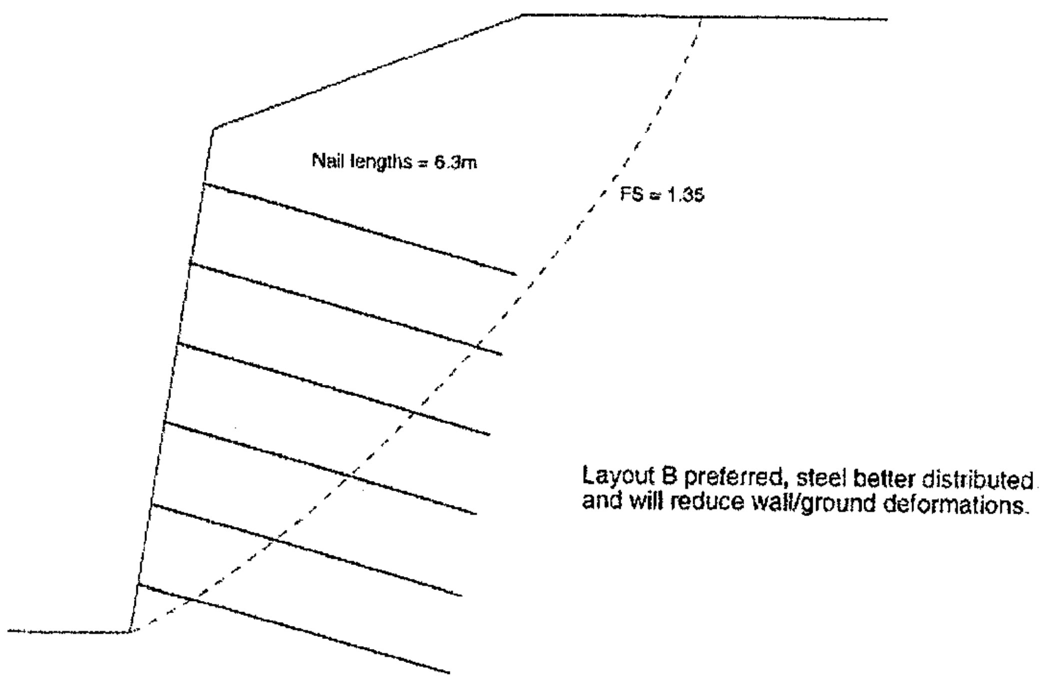
The most significant benefit of the slip surface equilibrium approach to soil nail wall design are:

1. The method consider all internal, external and mixed potential slip surfaces for the wall and evaluates global stability for each
2. The methods does not require specification does not require specification of a maximum tension line
3. The method is more convenient and accurate for heterogonous geometries, soil types and surcharges loading than the simplified earth pressure methods.

A limitation of the slip surface in the design of reinforced soil structure is that it is possible to define a wide variety of reinforcement distribution that satisfy strength limit state requirement but that are not satisfactorily from a serviceability perspective (i.e., result in excessive deformations of the reinforced mass). Figure 4.4 shows that two fundamental different nail layouts that results in calculated factors of safety that meet the requirement for any potential slip surface, would constitute an unsuitable design because of the deformation likely to be associated with such an arrangement of nails.



Layout A



Layout B

Figure 4.4 Different Nail Pattern Yielding Same Calculated Minimum Factor of Safety (from FHWA)

4.4 Layout and Dimensioning

The overall geometry (location, face batter, height) of the wall should be defined before performing detailed design calculations. It is known that the overall geometry and ground material properties will determine the critical design section for analysis. Subsurface restriction that could effects the nail layout must be identified and a preliminary nail pattern established.

4.4.1 Wall Location and Dimensioning

The location of the wall facing will be established by its intended function, related wall height constraint controlled by site geometry, environmental, economic, aesthetic o technical considerations. Vertical (vs. battered) walls and walls on tangent or circular radius provide for easier constructability, particularly for wall line and nail location survey control.

Once the design location of the top of the wall has been established, it is necessary to obtain a detailed topography survey along the wall line so that the grade at top of the cut can be precisely determined before the preparation of detailed plans. This will ensure:

1. The upper nails are not inadvertently specified as being located above the ground surface
2. Local grading requirement can be identified (i.e., for surface water control)
3. The size of any upper cantilever wall sections can be defined
4. The quantities and locations of any required backfill can be determined.

If any buried utilities or other subsurface are present within the reinforced zone, they must be located so that the impact on design and construction can be identified.

The selected dimensions of the reinforcement of shotcrete facing of soil nail wall are important for both structural aspects and wall constructability. Welded wire reinforcement commonly referred to as a fabric or mesh id used for reinforcement of shotcrete facing of soil nail wall. It shall comply with BS 4438 or equivalent. The lap mesh shall be at least 200mm or one mesh grid standard in both directions which is larger. It must be ensure that the tie wires shall be bent in the plane of the mesh and not forming large knot.

4.4.2 Preliminary Nail Layouts

A trial of nail layout pattern including nail length, locations, spacing, strengths and inclination is required for design analysis.

Nail Inclination

In Malaysia, nail inclination typically 20° to 15°.

Nail Spacing

In Malaysia, the vertical and lateral spacing is generally 0.75 m as most common reinforced soil wall specialist contractors use 1.5 m x 1.5 m concrete face panel with two anchor points per panel per level.

Nail Layout Locations

Nail column can be vertical or offset row to row. Vertical column provide for easier field layout and control of nail locations and provide more horizontal space for placement of the vertical geocomposite drain strip. It may preferable with some precast panel facing systems to facilitate facing connection and encapsulation of nail heads for corrosion protection. The offset pattern will improve the excavation face stability during construction, through the enhanced development of soil arching. The offset pattern is especially recommended where it is anticipated that the excavation face may be marginally stable.

Constructability will also generally be easier if nail rows are laid out:

1. Parallel to the base of wall grade for longer relatively uniform height wall on steeper grades
2. Horizontally (for easier field survey and layout)for longer relatively uniform height wall with no or very slight bottom of wall grade, with periodic step-us along the wall if necessary.
3. Top and intermediate nails rows parallel to top of wall profile and bottom row parallel with bottom of wall transitions between the rows where required, for shorter variable height walls.

The upper row of nail should placed to limit the height of the construction facing upper cantilever, above the top row of nails, to less than about 1.0 m. The top row of nails should be approximately centered within the first shotcrete lift of the construction facing to minimize the potential for a topping failure of the facing during the initial construction.

At bridge abutment, it should be verified that the design elevation for the first row of nails allow sufficient head-room for the drilling equipment to access and work beneath the deck. Plus longer relatively uniform height wall is sufficient clearance from all existing foundation should be ensured.

For sites characterized by an upper soil horizon consisting of loose soils or fill, temporary or permanents flatter cut slopes at the top of the soil nail wall shall be used to allow the installation of the first row of nails at greater depths.

Nail Lengths and Strength

Reinforcements usually are high yield bar (BS 4449) though the polymer based reinforcement such as fiberglass or galvanized steel pipe also can also be used in practice. Common rebar are Y16, Y20, Y25, Y32 and their maximum structural capacity are generally 50 kN, 80 kN, 130 kN and 200 kN respectively.

BS 8006 recommends that:

a. The minimum reinforcement length is 0.7H for normal retaining structures where H is the maximum height of the wall or higher than the wall if there is a sloping backfill.
b. For abutments (bridges), the minimum length shall be (whichever is longer):
    1. 0.6H + 2meter
    2. 7.0 meter
c. If the reinforcement length is to be stopped, the maximum difference between the steps shall be less than 0.15H

resistance is more critical. The expression is suitable for the steel reinforcement ratio in the facings less than 0.35%.

$$T_{FN} = C_F(m_{v,NEG} + m_{v,POS})(8S_h / S_v)$$

Where

T_{FN}	= Critical nail head strength
C_F	= Flexure pressure factor (Table 2)
$M_{v,NEG}$ & $m_{v,POS}$	= Vertical nominal unit moment resistance at the nail head and mid-span
S_h & S_v	= Horizontal/vertical nail spacings

For individual reinforced concrete pad facing and grid beam, the same approach by considering development of full development of positive and negative plastic moments can be used to the nail head strength. Figure 4.6 shows the typical pressure behind the facing. The pressure factor for facing flexure, C_F is determined from Table 4.3:

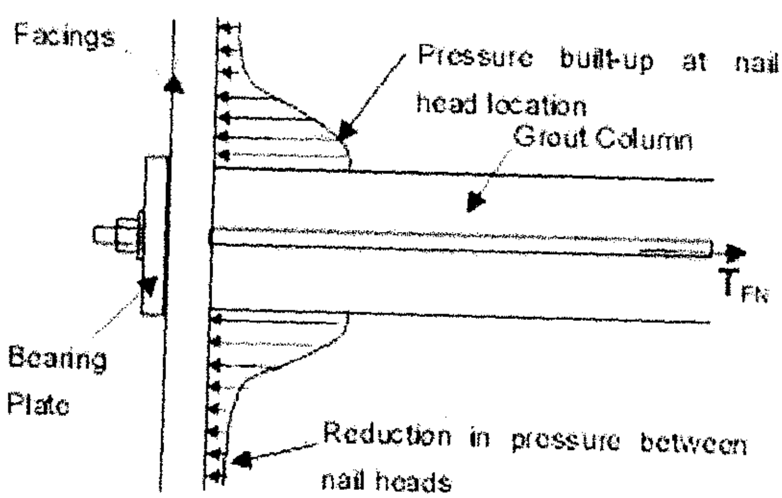


Figure 4.6: Typical Earth Pressure Diagram (form Liew 2005)

Table 4.3: Recommended Pressure Factor for Facing Design

Facing Thickness (mm)	Temporary Facing		Permanent Facing	
	C_F	C_S	C_F	C_S
100	2.0	2.5	1.0	1.0
150	1.5	2.0	1.0	1.0
200	1.0	1.0	1.0	1.0

The vertical nominal unit moment, m_v may be found as follows:

$$m_{v(NEG,POS)} = \frac{A_s F_y \gamma}{b}\left(d - \frac{A_s F_y}{1.7 f'_c b}\right)$$

Where:

A_s = area of tension reinforcement in facing panel width 'b'

b = width of unit facing panel (equal to S_v)

d = distance from extreme compressive fiber to centroid of tension reinforcement

f'_c = compressive strength of the concrete

4.5.2 Punching Shear Strength of the Facing

This failure mechanism consists of punching of a cone-shaped block of concrete facing centered about the nail head as shown in Figure 4.7. Bearing plate connection is popular type of nail head connection in Malaysia soil nailing industry. The design of punching shear for flat slab design can be referred to BS8110. FHWA has also given similar ultimate punching assessment with the following expression.

$$T_{FN} = V_N\left(\frac{1}{1 - C_s(A_c - A_{GC})/(S_V S_H - A_{GC}}\right)$$

Where:

C_s = Punching shear pressure factor (Table 2)

A_c = Soil contact area of cone-shaped block

A_{GC} = Cross sectional area of grout column

V_N = Nominal internal punching shear strength

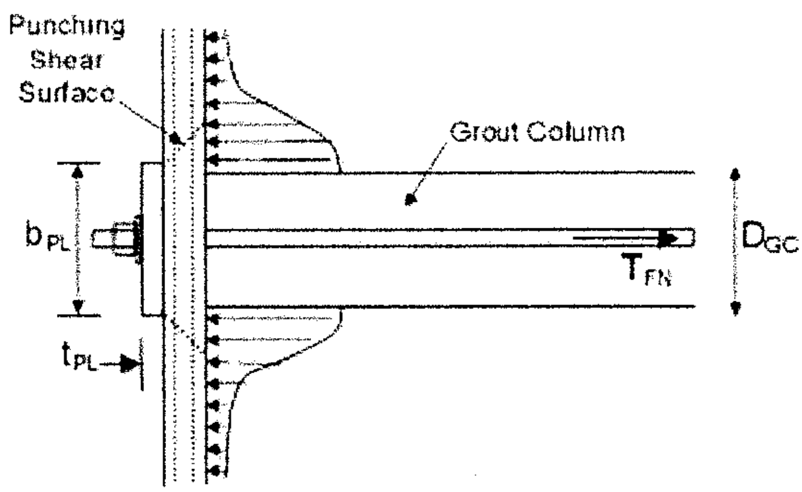


Figure 4.7: Typical Punching Shear of Bearing Plate Connection (from Liew 2005)

The flexural stiffness of the facing increases with thickness and steel reinforcement ratio, and decreases with increasing nail spacing. The relatively low flexural facing stiffness and comparative high nail head support stiffness will encourage effective arching effect resulting in highly non-uniform pressure distribution between the mid-span of facing and nail head as shown in Figures 4.6 and 4.7. Therefore, the nail head strength may possibly be higher than the abovementioned assessment. Nevertheless, it would be conservative to ignore such arching phenomenon.

4.5.4 Selecting Nominal Nail Head Strength

Table 4.4 summarize nominal nail head strength for facing flexure and facing punching shear failure modes, for common temporary and permanent facing designs.

For temporary shotcrete construction facing, the flexural failure mode consider standard 100 mm thick facing of shotcrete compressive strength equal to 28 MPa, with two No.13 continuous waler bars at each rows of nails and various nail head spacing and size of steel mesh reinforcement with and without vertical bearing bars at each nail head connection. For the punching shear failures mode of bearing plate through the shotcrete construction facing, both internal and total nominal nail head strength are given for different sizes of bearing plate (nail spacing and drill hole diameter are fixed at typical values as the results are relatively insensitive to these parameter)

For permanent CIP or shotcrete facing, the flexural failure mode consider a standard fixed pattern of facing reinforcement (No.13 bars at 300 mm spacing each way) and two facing thickness of 200 mm and 150 mm that represent the practical minimum facing thickness that can be constructed for CIP facing and permanents facing respectively.

<u>Temporary Shotcrete Construction Facing</u>

Facing Flexure

Facing thickness:	100 mm
Steel Yield:	420 MPa
Shotcrete Comp. Strength:	28 MPa
Walers:	2 x No.13

Table 4.4 (a): Nominal Nail Head Strength

Nail Spacing (m)	WW Mesh	Vertical Bearing Bars	T_{FN} (kN)
1.25 x 1.25	152x152 MW13xMW13	-	58
		2 X NO. 13	122
	152x152 MW18xMW18	-	81
		2 X NO. 13	145
	152x152 MW25xMW25	-	111
		2 X NO. 13	166
	102x102 MW9x MW 9	-	59
		2 X NO. 13	124
	102x102 MW 13x MW 13	-	86
		2 X NO. 13	149

	102x102 MW 18x MW 18	-	119
		2 X NO. 13	170
1.5 X1.5	152x152 MW13xMW13	-	58
		2 X NO. 13	112
	152x152 MW18xMW18	-	81
		2 X NO. 13	135
	152x152 MW25xMW25	-	111
		2 X NO. 13	163
	102x102 MW9x MW 9	-	59
		2 X NO. 13	113
	102x102 MW 13x MW 13	-	86
		2 X NO. 13	139
	102x102 MW 18x MW 18	-	119
		2 X NO. 13	170
1.75 X1.75	152x152 MW13xMW13	-	58
		2 X NO. 13	105
	152x152 MW18xMW18	-	81
		2 X NO. 13	127
	152x152 MW25xMW25	-	111
		2 X NO. 13	156
	102x102 MW9x MW 9	-	59
		2 X NO. 13	106
	102x102 MW 13x MW 13	-	86
		2 X NO. 13	132
	102x102 MW 18x MW 18	-	119
		2 X NO. 13	164

Facing Punching Shear:

Facing thickness:	100 mm
Shotcrete Comp. Strength:	28 MPa
Drill Hole Diameter:	200 mm
Nail Spacing:	1.5m x 1.5m

Table 4.4(b): Nominal Nail Head Strength

Bearing Plate Width (mm)	V_N (kN)	T_{FN} (kN)
200	165	184
225	178	204
250	192	224

Permanent Facing

Facing Flexure

Steel Yield:	420 MPa
Shotcrete Comp. Strength:	28 MPa
Reinforcement:	No. 13 bars @ 300 mm
Nail Pattern:	Vertical Spacing = Horizontal Spacing

Table 4.4 (c): Nominal Nail Head Strength

Facing Thickness (mm)	T_{FN} (kN)
150	206
200	278

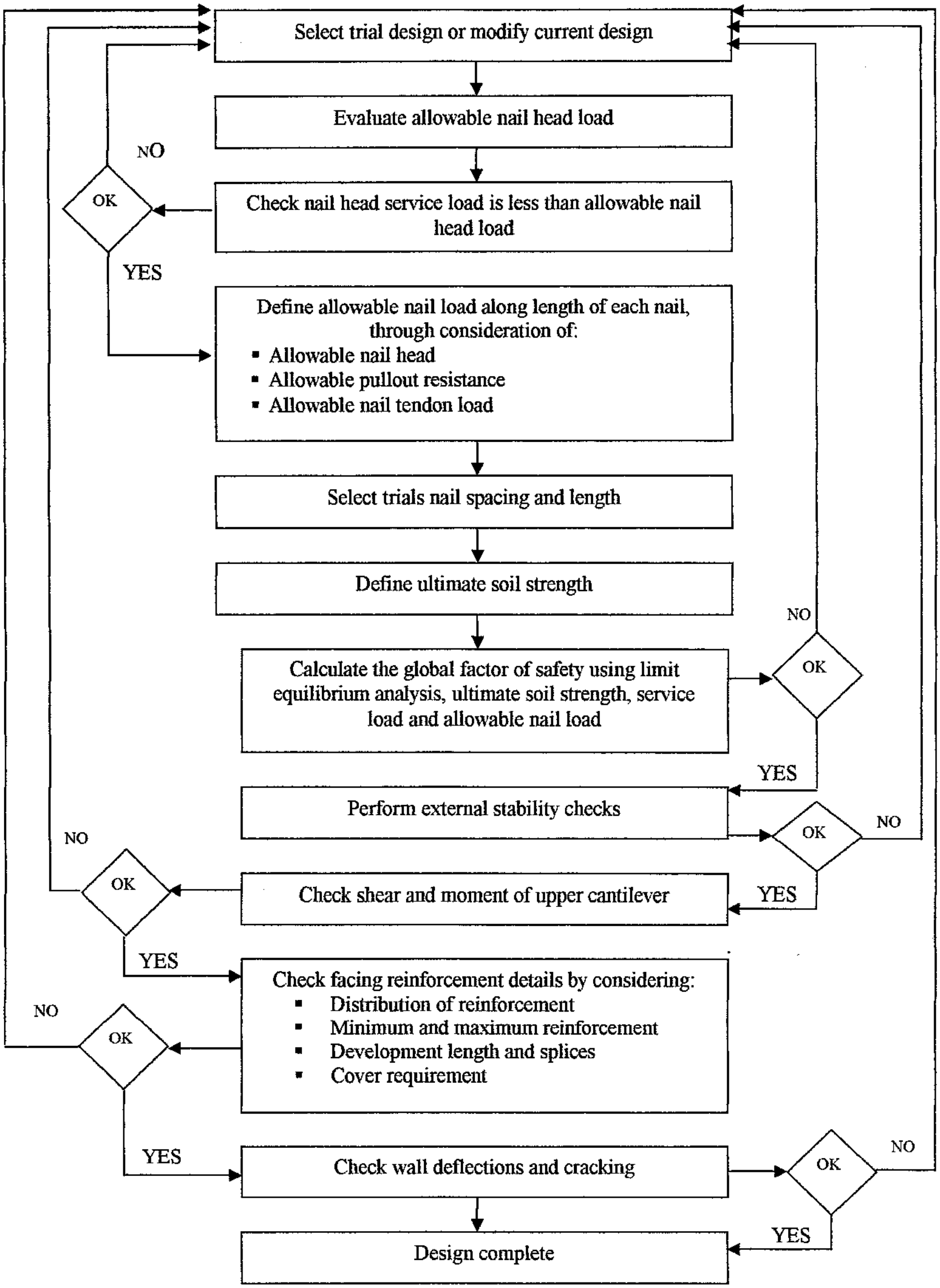


Figure 4.8: Design Procedure

4.6 Soil Nail Wall Design

The recommended design procedures are predominantly based on the methods outlined in FHWA's manual as it is comprehensive, systematic and can be easily adopted for Malaysian practice with some modifications. The design procedures proposed must also comply with the requirements of BS8006 and some good practices from HA 68/94 is also incorporated in order to improve its applicability for Malaysian practice. The major steps involved in the design are summarized as follows:

Step 1: Set Up Critical Design Cross-Section(s) and Select a Trial Design

This step involves selecting a trial design for the design geometry and loading conditions. The ultimate soil strength properties for the various subsurface layers and design water table location (should be below wall base) should also be determined. Table 4.5 provides some guidance on the required input such as the design geometry and relevant soil parameters. Subsequently, a proposed trial design nail pattern, including nail lengths, tendon sizes, and trial vertical and horizontal nail spacing, should be determined.

Table 4.5: Input required for Soil Nail Design

		Remarks
Soil Properties	Bulk density, γ	-
	Ultimate friction angle, Φ_{ult}	-
Wall geometry	Ultimate soil cohesion, C_{ult}	-
	Wall height, H	-
	Wall inclination, α	-
	Height of upper cantilever, C	-
	Height of lower cantilever, B	-
	Backslope angle, β	$\beta < \Phi_{ult}$
	Soil-to wall interface friction angle, δ	Typically 2/3 Φ_u
	Nail inclination, η	Typically 15°
	Vertical spacing of nail, S_v	Typically 1.5 m to 2.0 m
Nail and shotcrete properties	Horizontal spacing of nail, S_H	Typically 1.5 m to 2.0 m
	Characteristic strength of nail, Fy	Typically 460 N/mm^2
	Nail size/diameter	Minimum Φ20 mm
	Ultimate bond stress, Qu (kN/m) Values given in Tables 2 & 3 in kN/m2 Multiply with perimeter of grout column	Table 4.6

	(p x DG C) to obtain value in kN/m	
	Shotcrete strength	-
	Thickness of shotcrete	-
	Depth / Width of steel plate	Minimum plate width 200 mm
	Thickness of steel plate	Minimum plate thickness 19 mm
	Reinforcement for shotcrete	Use BRC reinforcement
	Waler bars	Typicaly 2T12
	Concrete cover	Typically 50-75 mm
	Diamet er of grout column	Typically 125 mm
Factor of safety	Soil strength	Table 4.8
	Nail tendon tensile strength	Table 4.8
	Ground-grout pullout resistance	Table 4.8
	Facing flexure pressure	Table 4.6
	Facing shear pressure, C_s	Table 4.6
	Nail head strength facing flexure / punching shear,	Table 4.7
	Nail head service load,	Typically 0.5
	Nail head service load,	Typically 2.5

Note: In Malaysia, the ultimate bond stress is usually obtained based on correlation with SPT "N" values and typically ranges from 3N to 5N.

The allowable bond stress, Q can be determined using the following equations:

$$Q = \sigma'_n \tan \phi'_{des} + c'_{des} \text{ (kN/m}^2\text{)}$$

Where

σ'_n = average radial effective stress

Φ'_{des}, c'_{des} = design values for the soil shearing resistance

The average radial effective stress, σ'_n acting along the pull-out length of a soil nail may be derived from:

$$\sigma'_n = \tfrac{1}{2}(1+K_L)\sigma'_v$$

Where:

Table 4.8: Strength Factor and Factor of Safety (form Table 4.5, FHWA, 1998)

Element	Strength Factor (Group I), α	Strength Factor (Group IV),	Strength Factor (Group VII), (Seismic)
Nail Head Strength	α_F = Table 4	See Table 4	See Table 4
Nail Tendon Tensile Failure	$\alpha_N = 0.55$	1.25(0.55)=0.69	1.33(0.55)=0.73
Ground-Grout Pullout Resistance	$\alpha_Q = 0.50$	1.25(0.50)=0.63	1.33(0.50)=0.67
Soil	F= 1.35(1.50*)	1.08(1.20*)	1.01(1.13)*
Soil-Temporary Construction Condition †	F= 1.20(1.35*)	NA	NA

Note:
Group I: General loading conditions
Group IV: Rib shortening, shrinkage and temperature effects taken into consideration
Group VII: Earthquake (seismic) effects (Not applicable in Malaysia)
* Soil Factors of Safety for Critical Structures
† Refers to temporary condition existing following cut excavation but before nail installation. Does not refer to "temporary" versus "permanent" wall.

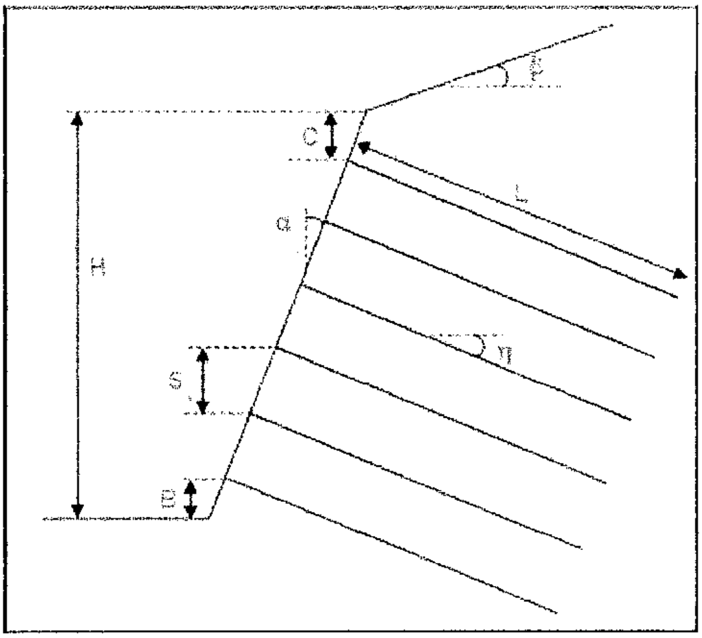


Figure 4.9: Definition of notation used in Table 4.5 (Tan & Chow 2006)

Step 2: Compute the Allowable Nail Head Load

The allowable nail head load for the trial construction facing and connector design is evaluated based on the nominal nail head strength for each potential failure mode of the facing and connection system, i.e. flexural and punching shear failure. The flexural and punching strength of the facing is evaluated as follow in accordance to the recommendations of FHWA, 1998:

Flexural Strength of the facing

Critical nominal nail head strength, T_{FN}

$$T_{FN} = C_F(m_{v,NEG} + m_{v,POS})(8S_h / S_v)$$

Where

T_{FN}	= Citical nail head strength
C_F	= Flexure pressure factor (Table 2)
$M_{v,NEG}$ & $m_{v,POS}$	= Vertical nominal unit moment resistance at the nail head and mid-span
S_h & S_v	= Horizontal/vertical nail spacings

Vertical nominal unit moment

$$m_{v(NEG,POS)} = \frac{A_s F_y \gamma}{b}\left(d - \frac{A_s F_y}{1.7 f'_c b}\right)$$

Where:

A_s	= area of tension reinforcement in facing panel width 'b'
b	= width of unit facing panel (equal to S_v)
d	= distance from extreme compressive fiber to centroid of tension reinforcement
f'_c	= compressive strength of the concrete

Punching Shear Strength of the facing

Nominal internal punching shear strength of the facing, V_N

$$V_N = 0.33(f'_c(MPa))^{1/2}(\pi)(D'_c)(h_c)$$

$$D'_c = b_{PL} + h_c$$

Nominal nail head strength, T_{FN}

$$T_{FN} = V_N\left[1/1 - C_s(A_c - A_{GC})/(S_V S_H - A_{GC}\right]$$

Cs = pressure factor for punching shear (Table 4.6)

The allowable nail head load is then the lowest calculated value for the two different failure modes.

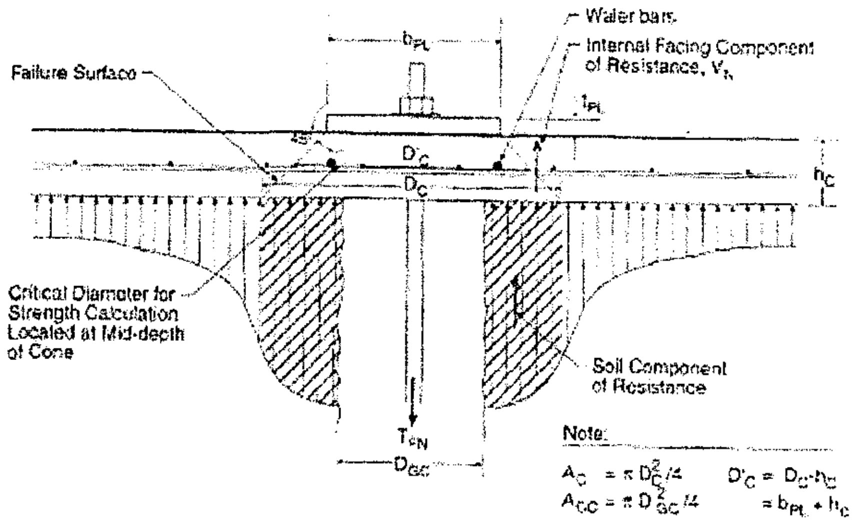


Figure 4.10: Bearing plate connection details (from FHWA. 1998)

Step 3: Minimum Allowable Nail Head Service Load Check

This empirical check is performed to ensure that the computed allowable nail head load exceeds the estimated nail head service load that may actually be developed as a result of soil-structure interaction. The nail head service load actually developed can be estimated by using the following empirical equation:

$$t_f = F_f K_A \gamma H S_V S_H$$

F_f = empirical factor (0.5)

KA = coefficient of active earth pressure

γ = bulk density of soil

H = height of soil nail wall

S_H = horizontal spacing of soil nails

S_V = vertical spacing of soil nails

Step 4: Define the Allowable Nail Load Support Diagrams

This step involves the determination of the allowable nail load support diagrams. The allowable nail load support diagrams are useful for subsequent limit equilibrium analysis. The allowable nail load support diagrams are governed by:

a) Allowable Pullout Resistance, Q_u

$Q = \alpha_Q$ x Ultimate Pullout Resistance, Q_u

b) Allowable Nail Tendon Tensile Load, T_{NN}

$T_N = \alpha_N$ x Tendon Yield Strength, T_{NN}

c) Allowable Nail Head Load, T_{FN}

$T_F = \alpha_F$ x Nominal Nail Head Strength, T_{FN}

Where

α_Q , α_N, α_F = strength factor (Table 4.8)

Next, the allowable nail load support diagrams shall be constructed according to Figure 4.3.

Step 5: Select Trial Nail Spacing and Lengths

Performance monitoring results carried out by FHWA have indicated that satisfaction of the strength limit state requirements will not of itself ensure an appropriate design. Additional constraints are required to provide for an appropriate nail layout. The following empirical constraints on the design analysis nail pattern are therefore recommended for use when performing the limiting equilibrium analysis:

a) Nails with heads located in the upper half of the wall height should be of uniform length
b) Nails with heads located in the lower half of the wall height shall be considered to have a shorter length in design even though the actual soil nails installed are longer due to incompatibility of strain mobilised compared to the nails at the upper half. However, further refinement in the nail lengths can also be carried out if more detailed analyses are being carried out, e.g. using finite element method (FEM) to verify the actual distribution of loads within the nails.

The above provision ensures that adequate nail reinforcement (length and strength) is installed in the upper part of the wall. This is due to the fact that the top-down methods of construction of soil nail walls generally results in the nails in the upper part of the wall being more significant than the nails in the lower part of the wall in developing resisting loads and controlling displacements as shown in Figure 18. If the strength limit state calculation overstates the contribution from the lower nails, then this can have the effect of indicating shorter nails and/or smaller tendon sizes in the upper part of the wall, which is undesirable since this could result in less satisfactory in-service performance. The above step is essential where movement sensitive structures are situated close to the soil nail wall. However, for stabilization works in which movement is not an important criterion, e.g. slopes where there is no nearby buildings or facilities, the above steps may be ignored.

Step 6: Define the Ultimate Soil Strengths

The representative soil strengths shall be obtained using conventional laboratory tests, empirical correlations, etc. The limit equilibrium analysis shall be carried out using the representative soil strengths (NOT factored strengths). For cut slope, effective stress (drained or long-termcondition) is normally more critical than total stress strength parameters, c' and (undrained condition). Therefore, effective stresses, determined from testing of representative samples of matrix materials are used in analysis. The most common approach to measure shear strength of residual soils is through a large number of small scale in situ (field) and laboratory tests. In situ tests include the standard penetration tests (*SPT*), cone penetrometer tests (*CPT* or *CPTU*), vane shear tests and pressuremeter tests. Laboratory tests commonly used are shear box tests consolidated undrained triaxial compression tests with pore water pressure measurements (*CIU*) and consolidated drained triaxial compression tests (*CID*) carried out on undisturbed soils (from Mazier sampler without trimming and without side drains). Shear box tests with the direction of shearing in specified orientation are sometimes carried out to explore the effects of anisotropy and shear strength in structural discontinuities.

Step 7: Calculate the Factor of Safety

The Factor of Safety (*FOS*) for the soil nail wall shall be determined using the "slip surface" method (e.g. Simplified Bishop method, Morgenstern-Price method, etc.). This can be carried out using commercially available software to perform the analysis. The stability analysis shall be carried out iteratively until convergence, i.e. the nail loads corresponding to the slip surface are obtained. The required factor of safety (*FOS*) for the soil nail wall shall be based on recommended values for conventional retaining wall or slope stability analyses (e.g. 1.4 for slopes in the high risk-to- life and economic risk as recommended by GEO, 2000).

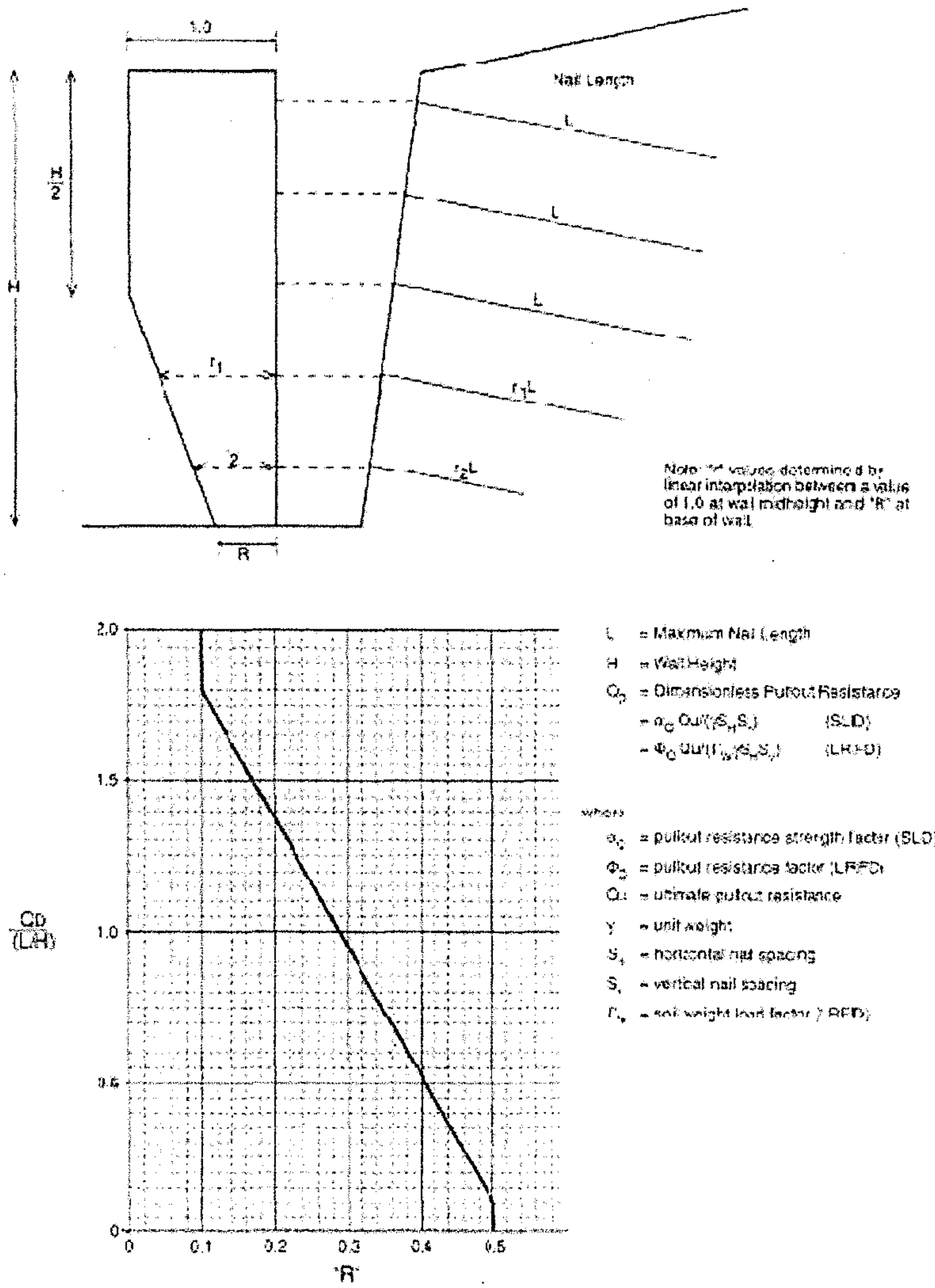


Figure 4.11: Nail length distribution assumed for design (from FHWA, 1998).

Notes: "r" values determined by linear interpolation between a value of 0.1 at wall mid-height and "R" at base of wall. I

Where

L = maximum nail length

H = wall height

Q_D = Dimensionless Pullout Resistance

$= \alpha_Q Q_u/(\gamma S_V S_H)$

Where

α_Q = pullout resistance strength factor

Q_u = ultimate pullout resistance

γ = unit weight

S_H = horizontal nail spacing

S_V = vertical nail spacing

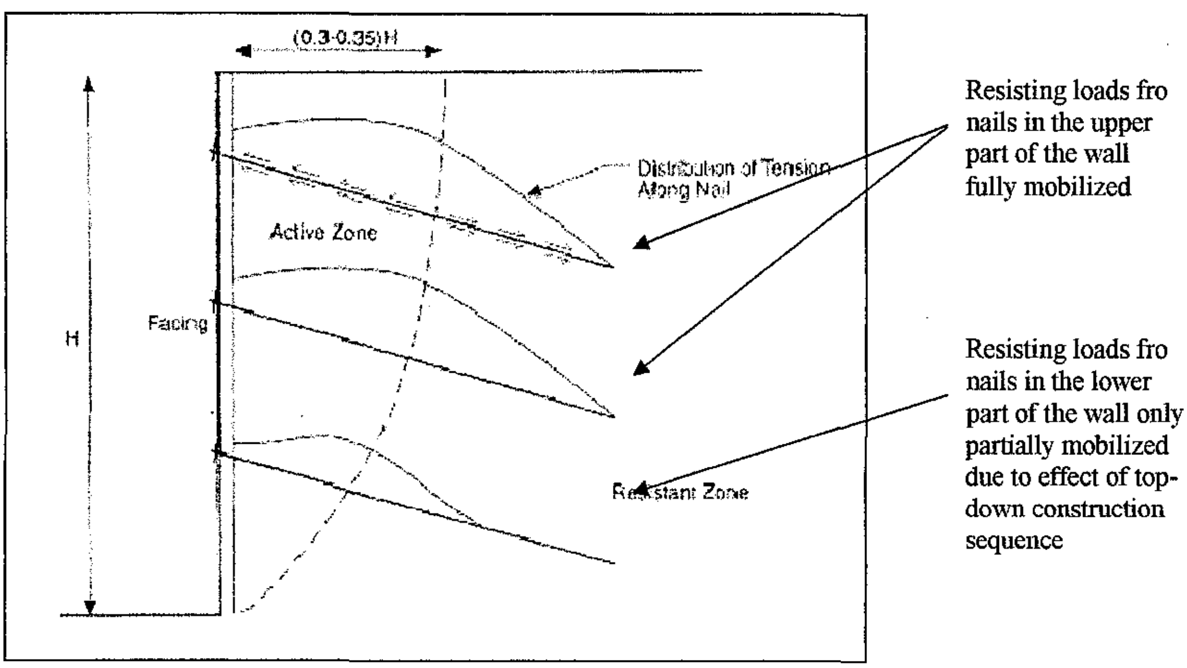


Figure 4.12: Conceptual soil nail behaviors (from FHWA, 1998).

Step 8: External Stability Check

The potential failure modes that require consideration with the slip surface method include:

a) Overall slope failure external to the nailed mass (both "circular" and "sliding block" analysis are to be carried out outside the nailed mass). This is especially important for residual soil slopes which often exhibit specific slip surfaces, defined by relict structure, with shear strength characteristics that are significantly lower than those apply to the ground mass in general. Therefore, for residual soil slopes, the analyses must consider either general or non-structurally controlled slip surfaces in association with the strength of the ground mass, together with specific structurally controlled slip surfaces in association with the strength characteristics of the relict joint surfaces themselves. The soil nail reinforcement must then be configured to support the most critical condition of these two conditions.

b) Foundation bearing capacity failure beneath the laterally loaded soil nail "gravity" wall. As bearing capacity seldom controls the design, therefore, a rough bearing capacity check is adequate to ensure global stability.

Step 9: Check the Upper Cantilever

The upper cantilever section of a soil nail wall facing, above the top row of nails, will be subjected to earth pressures that arise from the self-weight of the adjacent soil and any surface loadings acting upon the adjacent soil. Because the upper cantilever is not able to redistribute load by soil arching to adjacent spans, as can the remainder of the wall facing below the top nail row, the strength limit state of the cantilever must be checked for moment and shear at its base, as described in Figure 4.13.

For the cantilever at the bottom of the wall, the method of construction (top-down) tends to result in minimal to zero loads on this cantilever section during construction. There is also the potential for any long-term loading at this location to arch across this portion of the facing to the base of the excavation. It is therefore recommended by FHWA, 1998 that no formal design of the facing be required for the bottom cantilever. It is also recommended, however, that the distance between the base of the wall and the bottom row of nails not exceed two-thirds of the average vertical nail spacing.

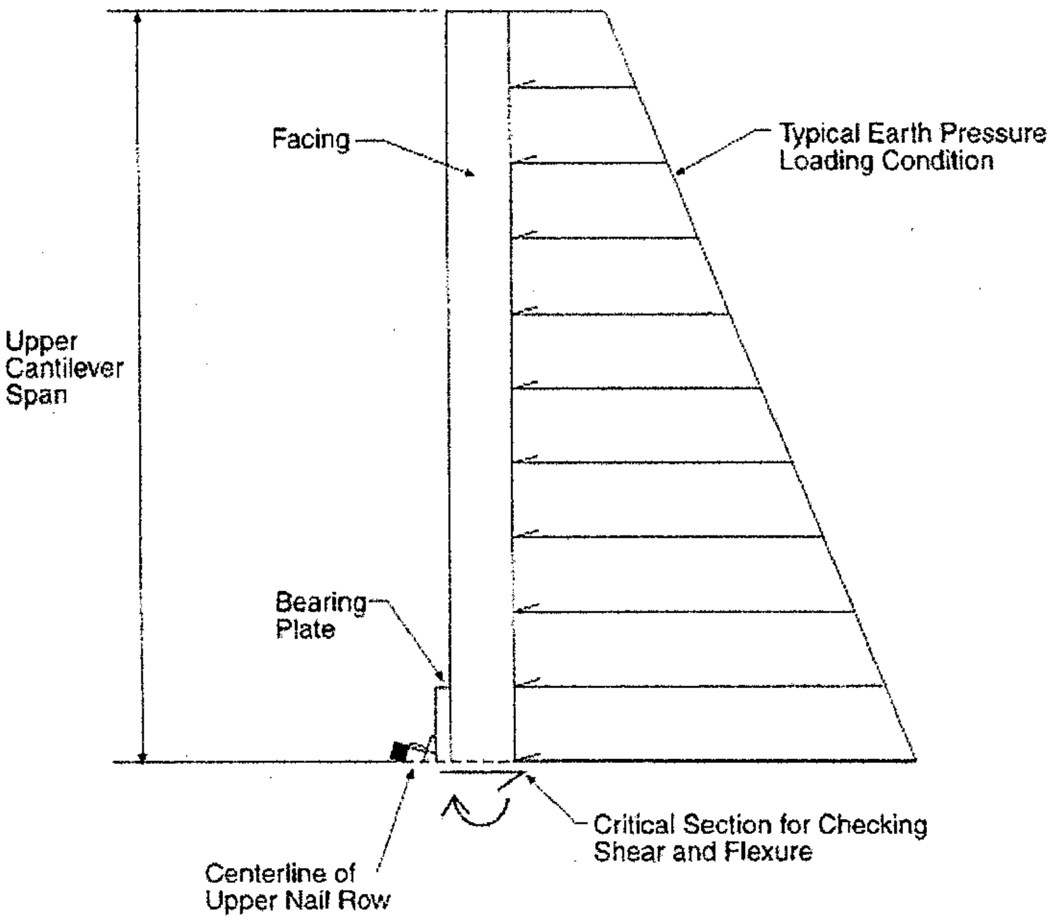


Figure 4.13: Upper Cantilever Design Check

Step 10: Check the Facing Reinforcement Details

Check waler reinforcement requirements, minimum reinforcement ratios, minimum cover requirements, and reinforcement anchorage and lap length as per normal recommended procedures for structural concrete design.

It is recommended that waler reinforcement (usually 2T12) to be placed continuously along each nail row and located behind the face bearing plate at each nail head (i.e. between the face bearing plate and the back of the shotcrete facing). The main purpose of the waler reinforcement is to provide additional ductility in the event of a punching shear failure, through dowel action of the waler bars contained within the punching cone.

Step 11: Serviceability Checks

Check the wall function as related to excess deformation and cracking (i.e. check the serviceability limit states). The following issues should be considered:

a) Service deflections and crack widths of the facing
b) Overall displacements associated with wall construction
c) Facing vertical expansion and contraction joints

Step 12: Construction Checks

For very high and steep slopes, the critical duration may be during the construction phase. Therefore, construction conditions shall be checked as per recommendations of HA 68/94 by missing out the lowest nail, but using short term soil strength parameters, (or using effective stress parameters with the value of r_u relevant during construction).

In addition, it is also recommended that the critical stages of works for soil nailing to be highlighted to the contractor and be included as part of the construction drawings and work specifications to ensure satisfactory performance of the soil nailed slope in the long-term and also during construction.

4.8 Corrosion Protection

The long term performance of permanents soil nailing requires that they be able to withstand corrosive attack from their local environment. Characteristic defining the corrosive potential of the soil environment are summarized in Table below:

Table 4.9: Recommended Electrochemical Properties for Soils when using soil nail

Test	**ASTM Standard**	**Critical values**
Resistivity	G-57-78 (ASTM)	Below 2000 ohm/cm
pH	G-51-77 (ASTM)	Below 4.5
Sulfates	California DOT test 407	Above 500 ppm
Chlorides	California DOT test 422	Above 100 ppm

It is important that proper detailing with regards to corrosion protection of the nails are specified and properly executed at site. Some of the important consideration includes:

- Adequate cover for soil nails is provided by ensuring rigid spacers/centralizer at appropriate spacing. Figure shows examples of typical spacers used.
- Corrosion protection on the nails using galvanized steel bars or by encapsulation inside a corrugated plastic sheath.

However it is not recommended to use pre-grouted corrugated plastic sheath for soil nails in Malaysia due to lack of good quality workmanship and control at site. For soil nails that need to use corrugated plastic sheath, then larger diameter hole with the diameter of the corrugated plastic sheath at least three times the diameter of the steel bar or minimum of 75 mm, whichever is larger should be used. In addition, a minimum grout cover between the sheath and the borehole wall should not be less than 12 mm (FHWA 1998) but commonly 25 mm is recommended for practical purposes. Special care shall also be exercised during insertion of the pre-grouted corrugated soil nails to prevent bending and accidental knocking that could cause cracks to he grout and thus, loss of bonding between the grout and the steel bar (potential pullout failure). Finally, the designer and constructor also have to ensure that the spacers/centralizers are rigidly fixed to the nails and do not deform during insertion and grouting (Figure 4.14).

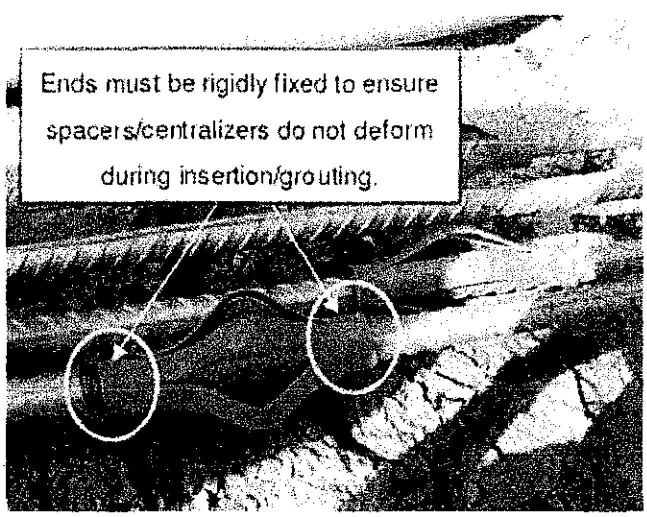


Figure 4.14: Typical spacers/centralizers for soil nails.

4.9 Wall Drainage

Surface water runoff and adverse groundwater conditions should be properly controlled to ensure the satisfactory performance of a soil-nailed system, both during construction and throughout its design life. Concentrated surface water flows may result in erosion, washout failures or shallow landslides. Build-up of high groundwater pressures behind the system may result in reduction of global stability. High groundwater levels may also adversely affect the grout quality as well as accelerate the corrosion rate of steel reinforcement. Suitable surface drainage provisions e.g. crest channels with upstand and stepped channels, and subsurface drainage provisions, e.g. raking drains, should be provided to soil-nailed systems based on the actual site conditions.

With respect to subsurface groundwater control, long term drainage measures may include the following:

- Face Drains: these are typically wide prefabricated geotextile drain strip that are placed in vertical strips down the excavation face, on horizontal spacing corresponding to the nail horizontal spacing and discharging either into a base drain or through the weep holes at the bottom of the wall.
- Shallow Drain: these are typically 300 – 400 mm long, 50-10mm diameter PVC pipes discharging through the face and located where heavier seepage is encountered.
- Horizontal Drain: deep horizontal drain, typically consisting of 50 mm diameter slotted or perforated tubes and inclined upwards at 5 to 10 degrees to the horizontal, may be installed to control the ground water pressure imposed on the retained soil mass.

During construction, sufficient temporary drainage should be provided at all times, especially during the wet season, to avoid any adverse effects of uncontrolled concentrated water ingress or surface water flow. The temporary site drainage should be maintained and cleared of any blockage on a regular basis to ensure that the drains remain functional at times of heavy rainfall. The contractor should be encouraged, or required where appropriate, to construct part of the permanent drainage measures, e.g. crest drain and the associated discharge points, at an early stage of the works to enhance the temporary drainage provisions. During the construction of subsurface drains, due attention should be paid to avoid damaging the installed soil nails adjacent to the drains.

4.10 Simplified Design Charts for Preliminary Design of Cut Slope Walls

Simplified design chart have been develop for a 15° nail inclination, uniform ground condition, and non-critical installation assuming a safety factor of *F* of 1.35 (FHWA 1998).

Geometric Variables of Backslope Angle, β and Face or Batter Angle, δ

Four sets of design chart are presented (three chart per set) with each set of charts corresponding to a single backslope angle of 0, 10, 20 or 34 degrees. For intermediate backslope angle, interpolate between the charts. For each backslope angle, design information is presented for two face batter angles of 0 and 10 degrees from the vertical. For intermediate face batter angles, interpolate between the charts.

Strength Variables – Factored Friction Angle, Φ_D and Dimensionless Cohesion, c_D

Find the dimensionless nail tensile capacity, T_D by entering the vertical axis of the first chart of the appropriate chart set

The dimensionless nail tensile capacity is the factored nominal nail strength normalized with respect to the soil unit weight, γ, the vertical height of slope, H_1; and the nail spacing, S_V, S_H

$$T_D = \alpha_N T_{NN} / (\gamma H S_V S_H)$$

Preliminary Nail Size

Find $A_N = T_{NN}/F_Y$ and enter table 4.2 to find the bar size

The dimensionless pullout resistance Q_D is the factored ultimate pullout resistance, normalized with respect to the soil unit weight and nail spacing:

$$Q_D = \alpha_U Q_U / (\gamma H S_V S_H)$$

Find dimensionless pullout resistance shown as being corporated into the ratio (T_D/Q_D) on the horizontal axis of the second and third charts of each set.

Preliminary Nail Length

Compute Q_D and the ratio T_D. Enter either chart 2 and 3 of the appropriate chart set to find the ratio *L/H*. Since H is known, compute the preliminary length L.

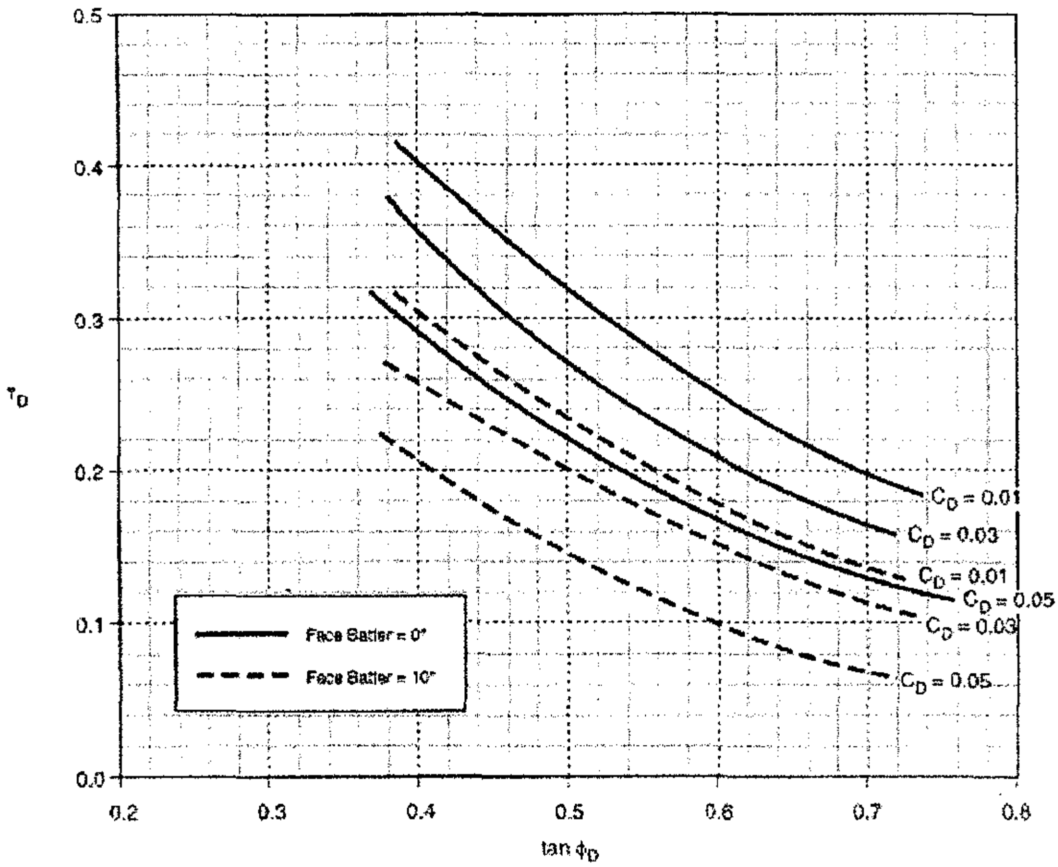
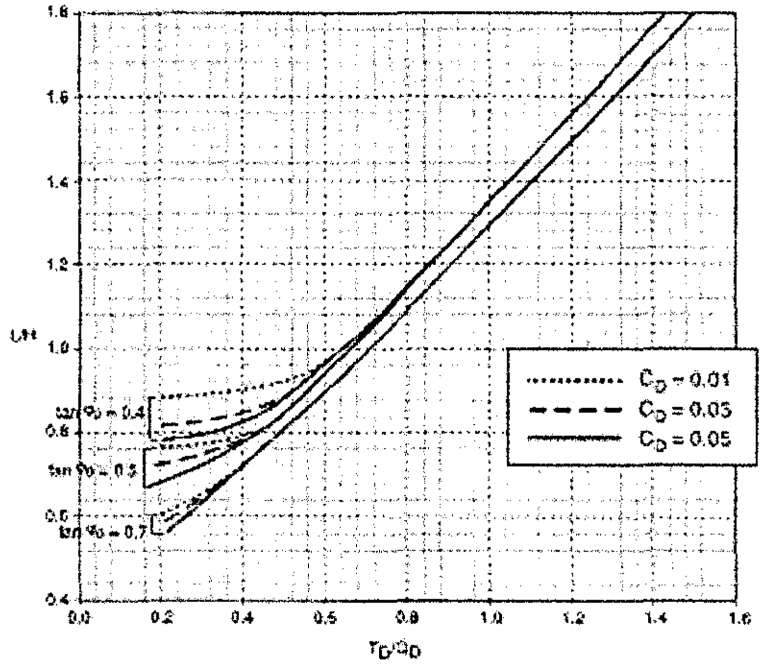
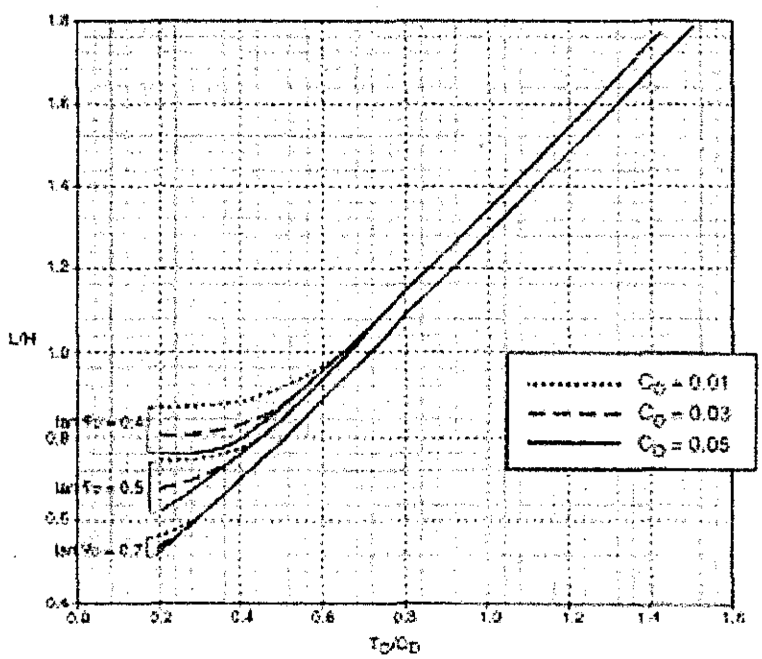


Figure 4.15: Preliminary Design Chart 1A. Backslope = 0° (from FHWA 1998)



Backslope = 0° Face Batter = 0° (Chart 1B)



Backslope = 0° Face Batter = 10° (Chart 1C)

Figure 4.16: Preliminary Design Chart 1B and 1C (from FHWA 1998)

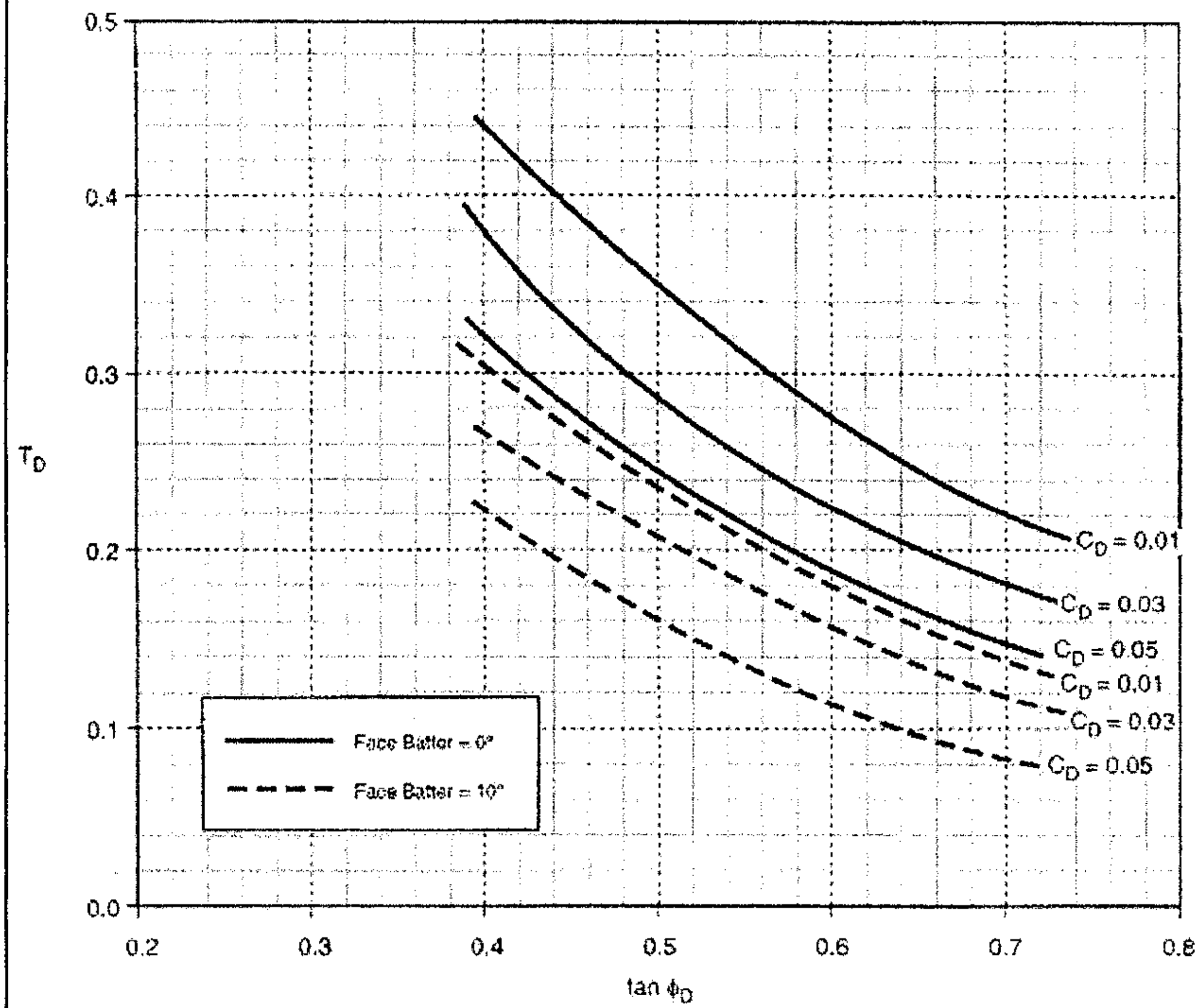
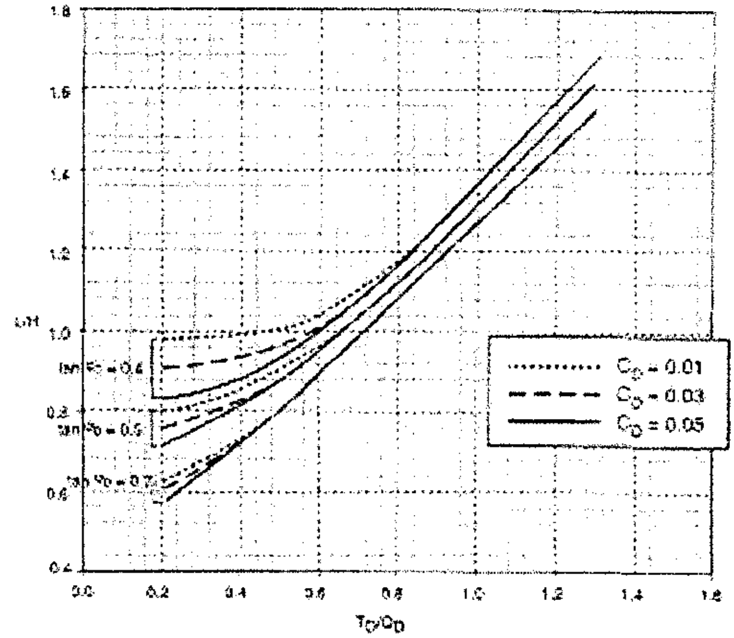
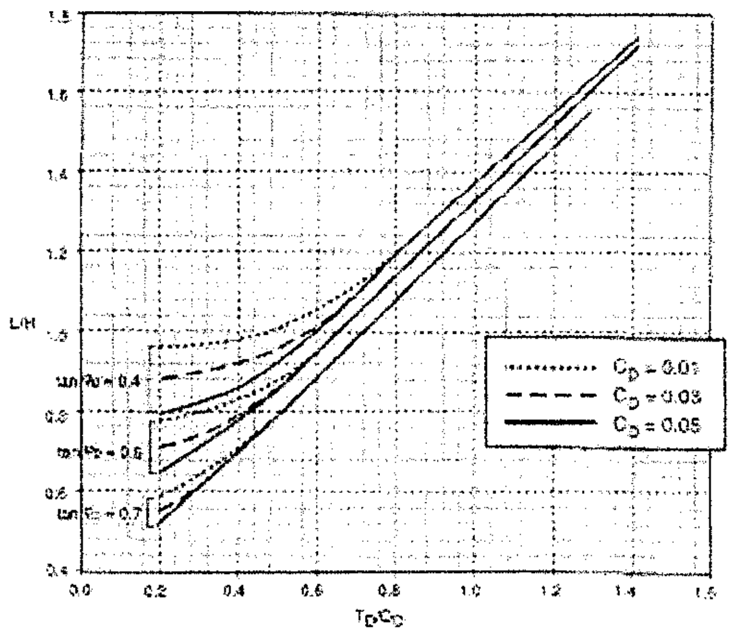


Figure 4.17: Preliminary Design Chat 2A. Backslope = 10°(from FHWA 1998)



Backslope = 10° Face Batter = 0° (Chart 2B)



Backslope = 10° Face Batter = 10° (Chart 2C)

Figure 4.18: Preliminary Design Chart 2B and 2C

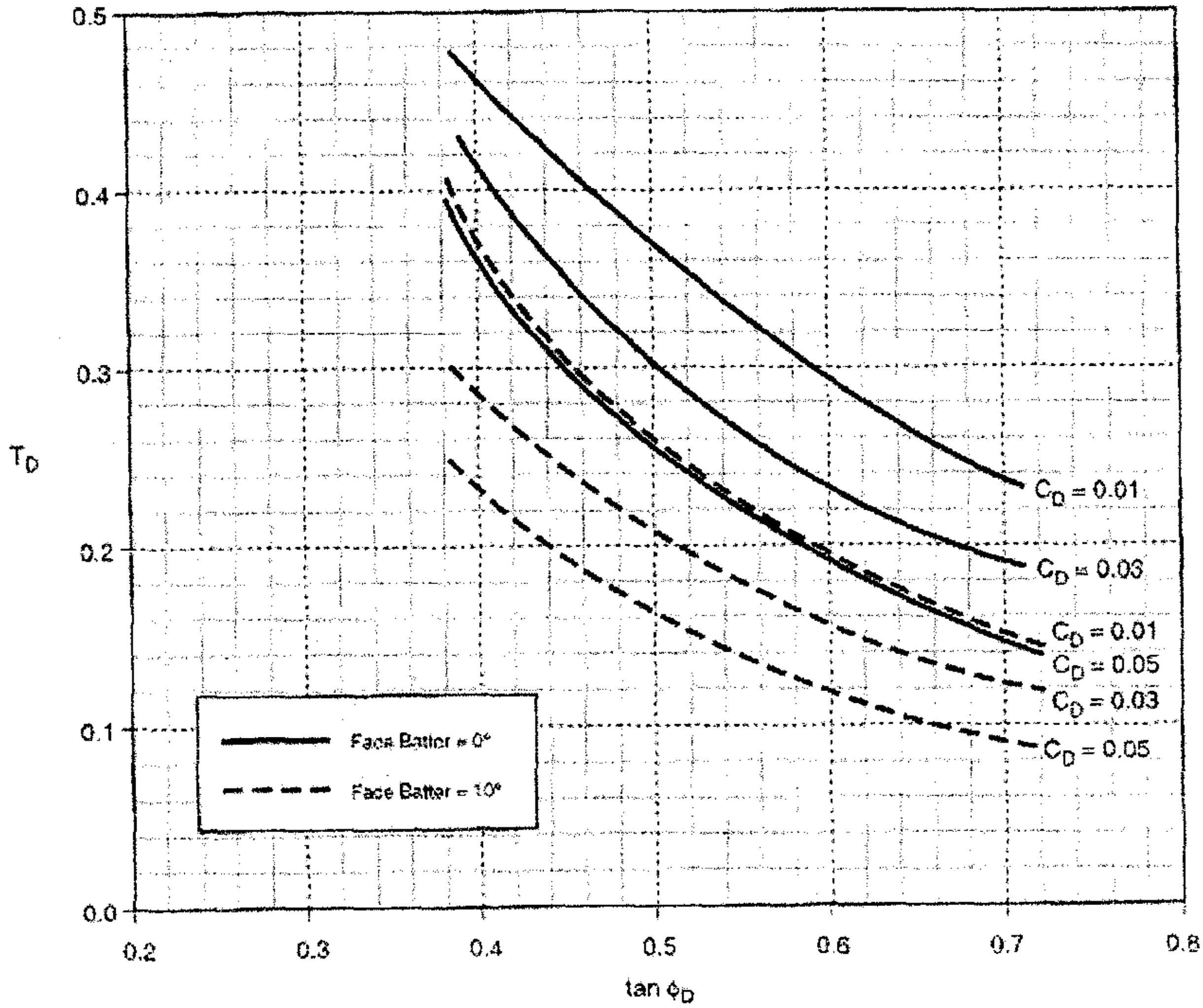
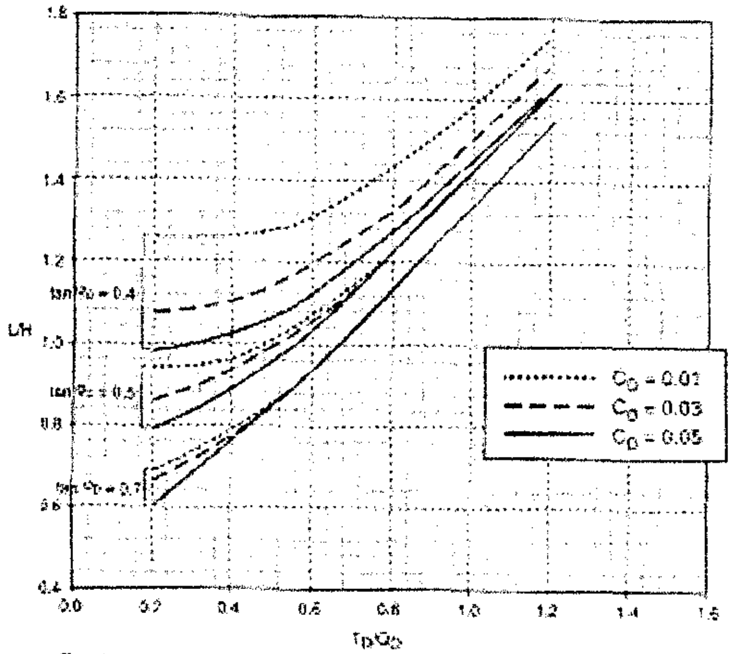
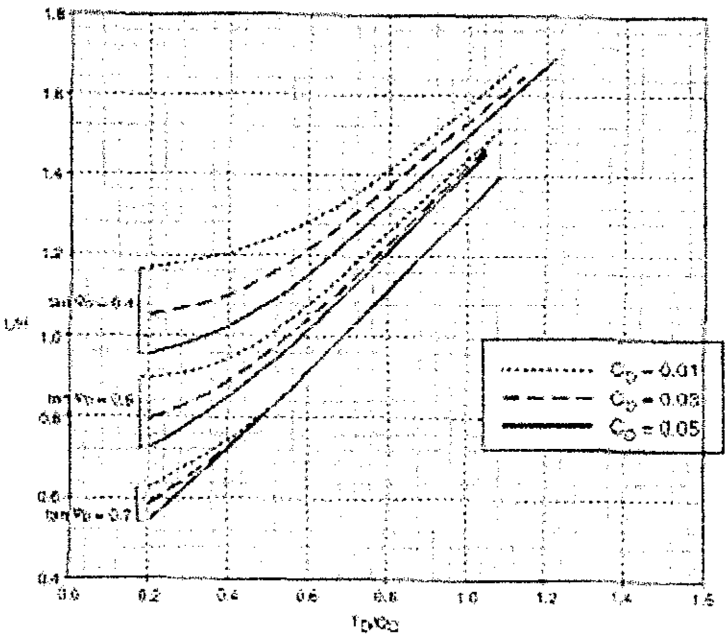


Figure 4.19: Preliminary Design Chart 3A. Backslope = 20° (from FHWA 1998)



Backslope = 20° Face Batter = 0° (Chart 3B)



Backslope = 20° Face Batter = 10° (Chart 3C)

Figure 4.20: Preliminary Design Chart 3B and 3C (form FHWA 1998)

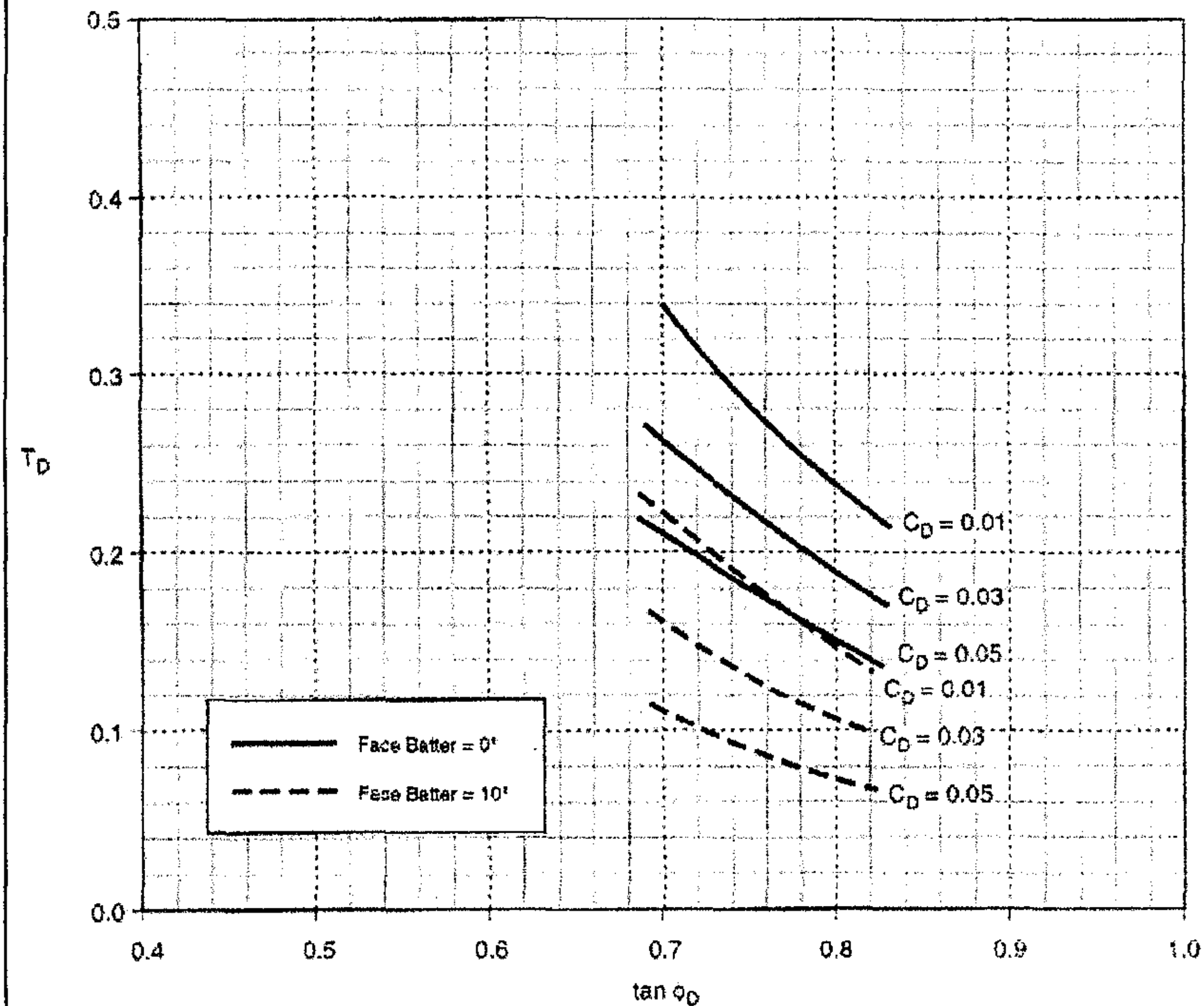
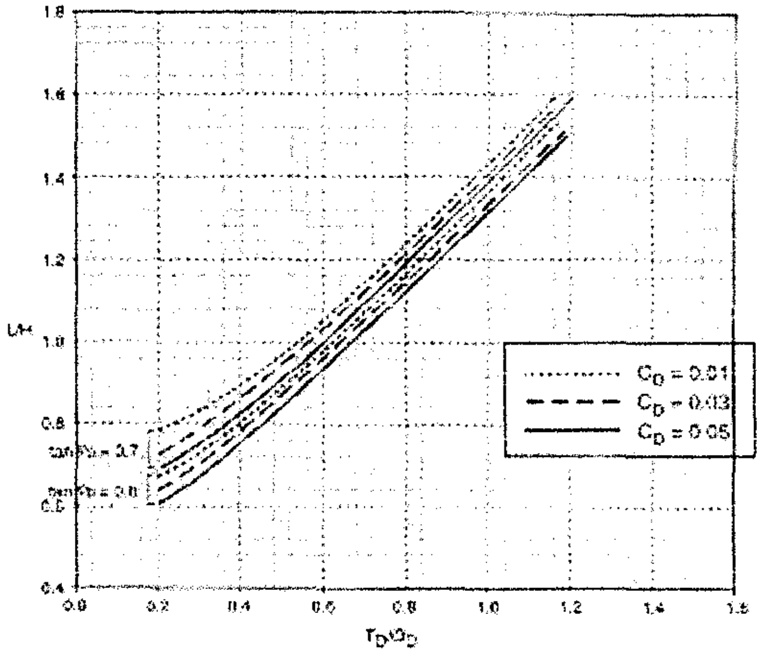
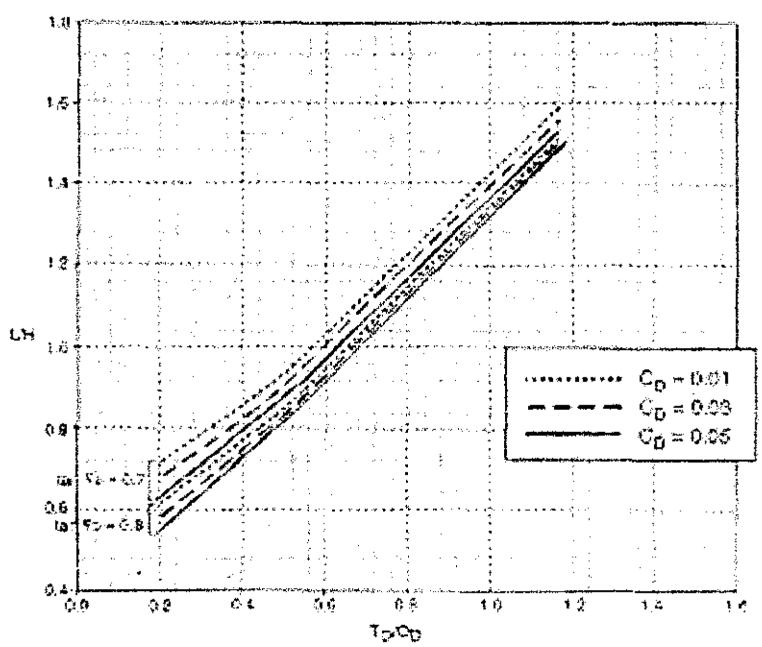


Figure 4.21: Preliminary Design Chart 4A. Backslope = 34° (from FHWA 1998)



Backslope = 34° Face Batter = 0° (Chart 4B)



Backslope = 34° Face Batter = 10° (Chart 4C)

Figure 4.22: Preliminary Design Chart 4B and 4C (from FHWA 1998)

CHAPTER 5: WORKED DESIGN EXAMPLES

The proposed design is summarized and demonstrated by the example of a cutslope wall.

5.1 Design Examples

A soil nail technique been proposed to be used in for a road cut through medium dense slity sands. In accordance with the Standard Specifications for Highway Bridges, 15th Edition, Service Load Group I (Table 4.4) defines the static loading condition for this problem.

Step 1: Set Up Critical Design Cross-Section and Select a Trial Design

The site investigation confirmed the subsurface soil and ground water conditions, established that 2.5 meters high vertical cuts will stand unsupported for minimum of several days. The soil profile has average in-situ densities of 18.0 kN/m and the soil strength parameters are estimated at a friction angle of 34.0° and cohesion of 5.0 kN/m^2. The ultimate pullout resistance recommended on order of 60.0 kN/m.

The encapsulated nail will be used for corrosion protection. The site investigation confirmed that there will be no requirement for horizontal drain as the ground water table is located well below the base of the proposed wall.

The wall will have a vertical height of 9.5 meters, with face batter of 10.0° from the vertical and will have a 20.0° slope at the top of the wall, as shown in Figure 5.1. The trial nail spacing will be at 1.5 meters, vertically and horizontally, and the nailed installed at the 15.0° below horizontal for constructability reasons.

The preliminary design chart used to determine the preliminary value for nail length and bar size. Select the design chart corresponding to the appropriate backslope angle. Figure 5.1 show that the design section has a face batter of 10° and backslope angle of 20°. Therefore use the design chart set presented in Figure 5.1

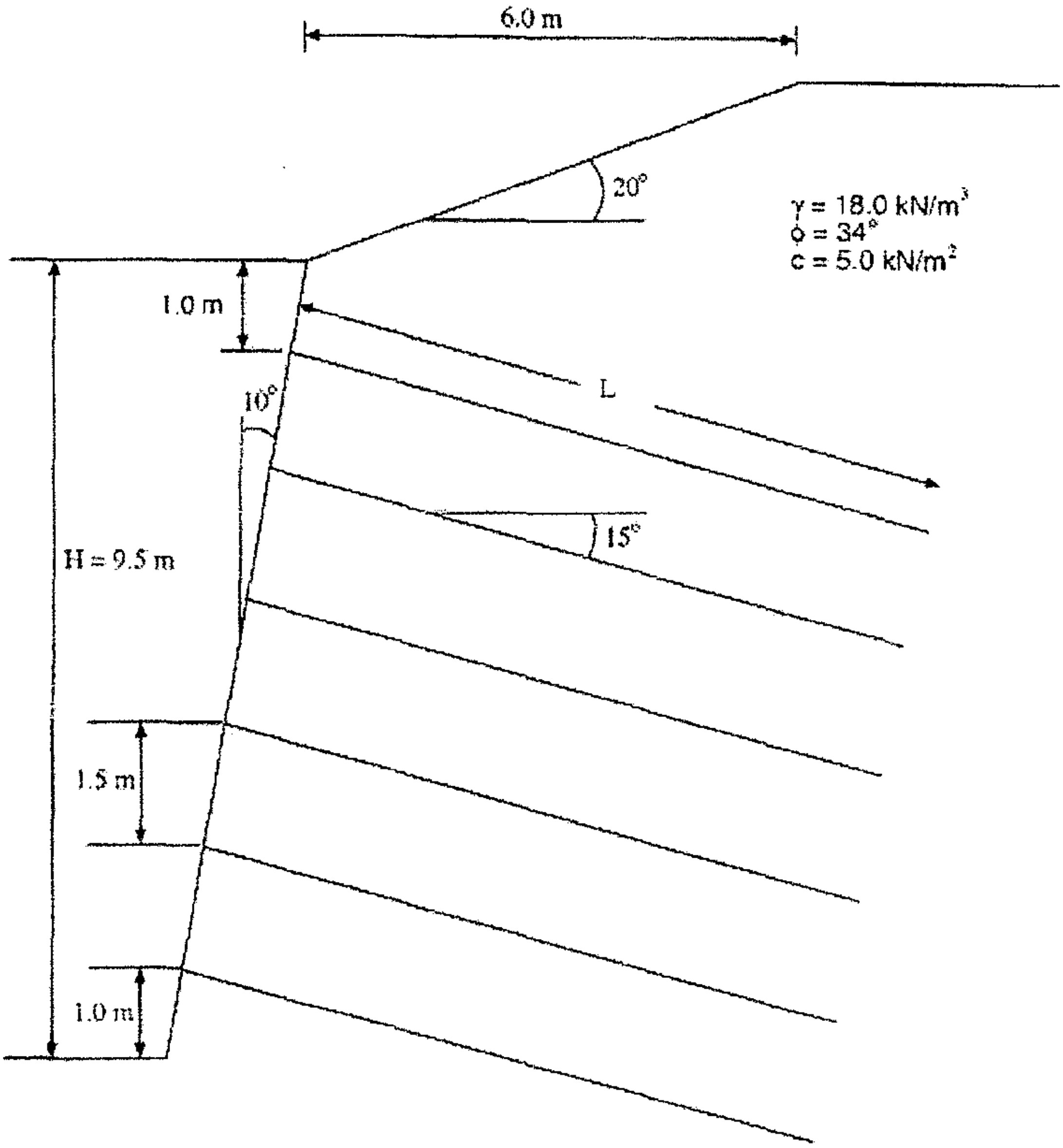


Figure 5.1: Cutslope Design Examples (from FHWA 1998)

The computed factored friction of angle and factored soil cohesion are as follows:

$\Phi_D = \tan^{-1} [\tan (\Phi_U)/F_\Phi]$

$= \tan^{-1} [\tan (34°)/1.35]$

$= 26.5°$

$\tan (\Phi_U) = \tan 26°$

$= 0.5$

$c_D = c_U/(F_C \gamma H)$

$= (5.0 \text{ kN/m2})/[1.35(18.0 \text{ kN/m3})(9.50\text{m})]$

$= 0.22$

From Chart A, (Figure) $T_D = 0.23$

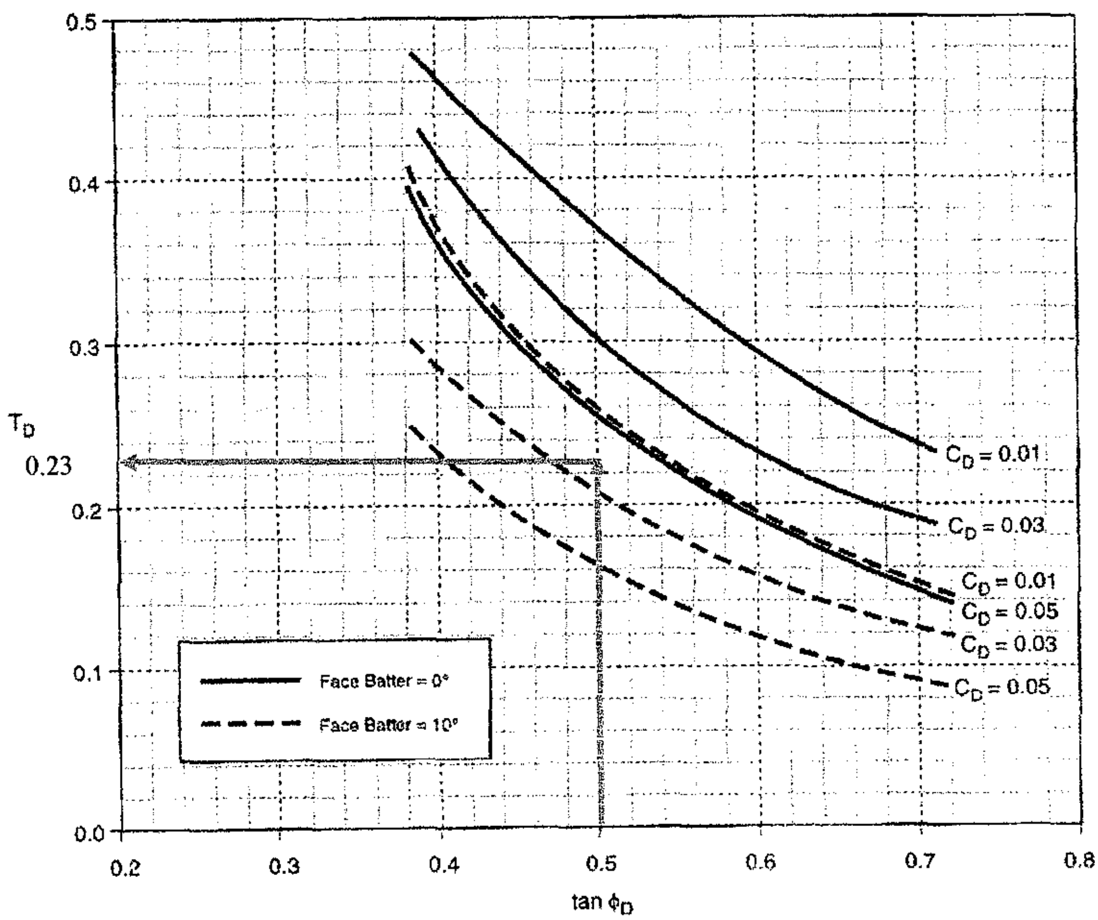


Figure 5.2: Chart A, design Chart for Backslope 20° and face batter 10°

The nominal nail tensile strength T_{NN} can be determined from:

$T_{NN} = \gamma H S_V S_H T_D / \alpha_N$

$= (18.0\ kN/m3)(9.50m)(1.50m)(1.50m)(0.23) / 0.55$

$= 161\ kN$

Area of bar (A_B) $= T_{NN} / F_y$

$= 161 / 0.42$

$= 383\ mm^2$

From Table

No. 22 $\approx 387\ mm^2$

No. 25 $\approx 510\ mm^2$

Select No. 25 bar for ease of handling and installation.

The nail pullout resistance Q_D can be determined by

$Q_D = \alpha_Q Q_U / (\gamma S_V S_H)$

$= (0.50)\ (60.0) / [(18.0)\ (1.50)\ (1.50)]$

$= 0.74$

Divide the calculated nail tensile capacity T_D by the nail pullout resistance Q_D and determine the required nail length form the appropriate Chart.

$T_D / Q_D = 0.23/0.74$

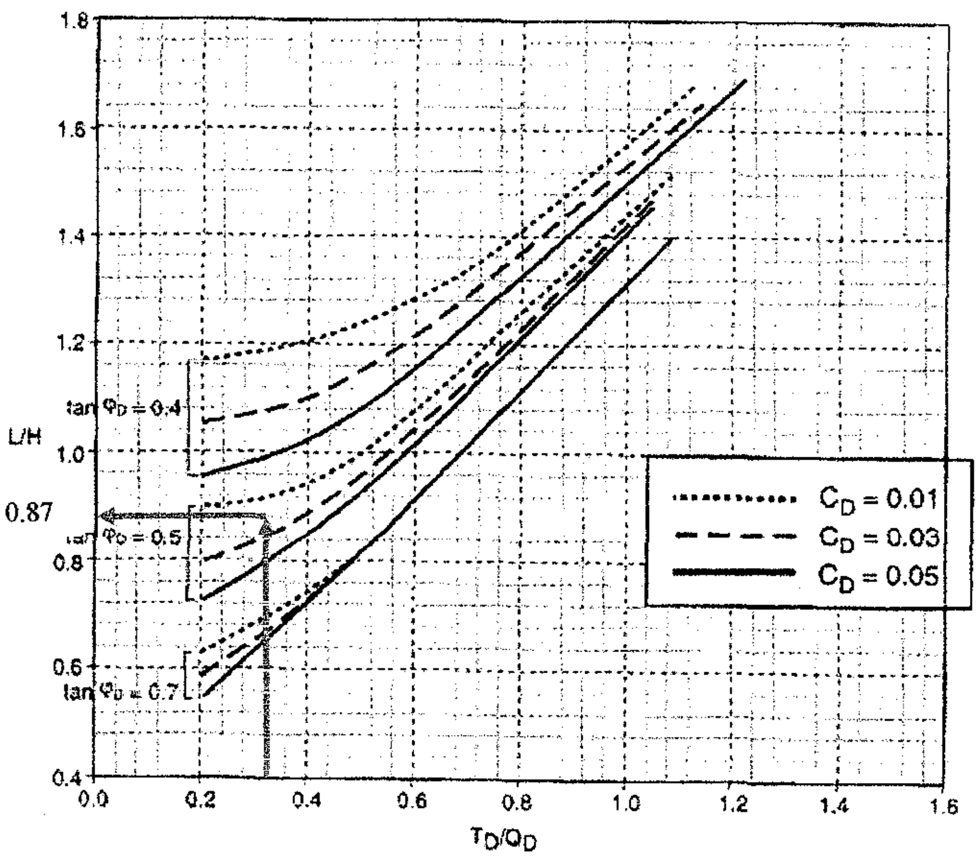
$= 0.31$

From Chart C,

$L/H = 0.87$

$L = 0.87(9.50)$

$= 8.3\ m$



Backslope = 20° Face Batter = 10° (Chart 3C)

Figure 5.3: Chart C, design Chart for Backslope 20° and face batter 10° (from FHWA 1998)

Therefore, the trial designs assumed:

No. 25, Grade 420 steel bars with the length of 8.3 m

Temporary shotcrete construction facing (28 days compressive strength of 28 MPa) with a nominal thickness of 100 mm

Reinforced with single layer of 152x152 MW19xMW19 welded wire mesh, 2 T13 waler bars and 2T13 bearing bars

Nails connected to shotcrete with 225 mm square, 25 mm thick bearing plate.

Step 2: Compute Allowable Nail Head Loads

Temporary Shotcrete Construction Facing

i) Strength Criteria: Facing Flexure

For facing structure, try 152 x 152 MW19xMW19 mesh (steel area = 122.8mm^2), with two No.13 waler bars and two No. 13 bearing bars (steel area = 129 mm^2). The yield stress of the reinforcement is specified as 420 MPa and the specified design concrete compressive strength at 28 days is 28 MPa.

Compute the negative and positive nominal unit moment resistance of the facing in the vertical direction using the equation:

$$m_{v(NEG,POS)} = \frac{A_s F_y \gamma}{b}\left(d - \frac{A_s F_y}{1.7 f'_c b}\right)$$

The areas of the vertical steel over the supports (2 No. 13 vertical bars and mesh vertical wires) and at midspan (mesh vertical wires) for a facing width b equal o 1.5 are computed as

$A_{S,NEG}$ = (122.8 mm^2/m)(1.5 m) + 2(129 mm^2)

= 443 mm^2

$A_{S,POS}$ = (122.8 mm^2/m)(1.5 m)

= 185 mm^2

The average nominal unit moment resistances are computed as below:

$$m_{v(NEG,)} = \frac{(443mm^2)(420MPa)}{1500mm}\left(50.0mm - \frac{(443mm^2)(420MPa)}{1.7(28MPa)(1500mm)}\right)$$

= 5.88 kNm/m

$$m_{v(POS,)} = \frac{(185mm^2)(420MPa)}{1500mm}\left(50.0mm - \frac{(185mm^2)(420MPa)}{1.7(28MPa)(1500mm)}\right)$$

= 2.53 kNm/m

The facing flexure pressure factor C_F for a 100mm thick temporary facing is 2.0. The nominal nail head strength for facing flexure computed as below:

$$T_{FN} = C_F(m_{v,NEG} + m_{v,POS})(8S_h / S_v)$$

$$T_{FN} = 2.0(5.88kNm/m + 2.53kNm/m)(8)(1.50)/(1.50)$$

= 135 kN

ii) Strength Criteria: Facing Punching Shear

The nominal internal punching shear strength of the facing is computed using equation:

$$V_N = 0.33(f'_c(MPa))^{1/2}(\pi)(D'_c)(h_c)$$

Where

h_C = 100mm

D'_c = $b_{PL} + h_C$

= 225mm + 100mm

= 325 mm

The resulting nominal internal punching shear strength of the facing is:

$V_N = 0.33(28MPa))^{1/2}(\pi)(325mm)(100mm)$

V_N = 178 kN

From table 4.6, the pressure factor for punching shear C_S for a 100mm thick temporary construction facing.

The punching cone bottom diameter:

D_C = $D'_C + h_C$

= 325 + 100

= 425 mm

The diameter of the grout column is estimated to be about 125 mm

The corresponding area are as follows:

A_C = $0.25(\pi)(D_C)^2$

= $0.25(\pi)(425)^2$

= 1.42×10^5 mm^2

A_{GC} = $0.25(\pi)(D_{GC})^2$

= $0.25(\pi)(125)^2$

= 1.22×10^5 mm^2

Therefore, the nominal nail head strength for punching shear is:

$T_{FN} = V_N[1/1 - C_s(A_c - A_{GC})/(S_V S_H - A_{GC}]$

$T_{FN} = 178[1/1 - 2.5(1.42x10^5 - 1.22x10^5)/((1500mm)(1500mm)) - 1.22x10^4]$

T_{FN} = 208 kN

Thus, the allowable nail head load is 135 kN

Step 3: Minimum Allowable Nail Head Service Load Check

The active earth pressure coefficient determined by:

K_a $= (1 - \sin \Phi) / (1 + \sin \Phi)$

$= (1 - \sin 34^{\circ}) / (1 + \sin 34^{\circ})$

$= 0.2827$

The empirical value for nail head service load factor is 0.5

The nail head load can be estimated by using the following equation:

$t_f = F_f K_A \gamma H S_V S_H$

$t_f = 0.5(0.2827)(18.0 kN/m3)(9.50)(1.50)^2$

$= 54$ kN

$t_f = 54\text{kN} < 135\text{kN}$

OK, the estimated nail head service load does not exceed the allowable nail head load

Step 4: Define the Allowable Nail Load Support Diagram

Determination of the allowable nail load support diagrams are governed by the allowable pullout resistance, the allowable nail head load and the allowable nail tendon tensile load.

The allowable Pullout Resistance, *Q*

Q $= \alpha_Q Q_U$

α_Q $= 0.50$ (Table)

Q_U $= 60.0$ kN/m

Q $= 0.50(60$ kN/m)

$= 30.0$ kN/m

Allowable Nail Tendon Tensile Load, T_N

T_N $= \alpha_N T_{NN}$

α_N $= 0.55$ (Table)

T_{NN} $= A_B F_Y$

$= (510 \text{ mm}^2)(0.42 \text{ kN/mm}^2)$

$= 214$ kN

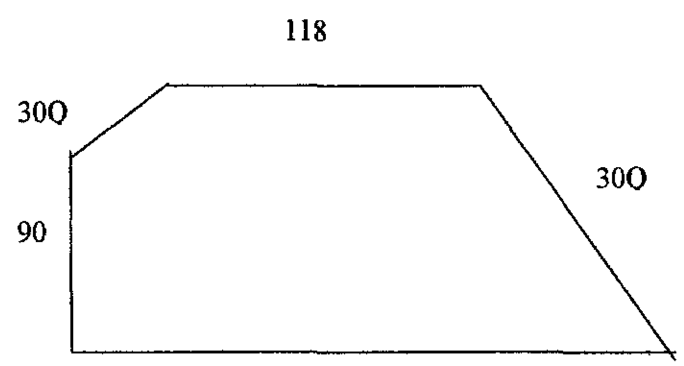
T_N $= (0.55)(214)$

$= 118$ kN

Allowable Nail Head Load

As per step 2, the allowable nail head load is 135 kN

The nail support diagram is constructed by plotting the nail head load (135 kN) vertically, extending the pullout resistance (Q) from the nail head load until the nail tendon (T_N) load is reached. The nail tendon load is extended horizontally until the pullout resistance line (Q) for the end of the box is intersected.



Step 5: Select Trial Nail Spacing and Length

In step 1, a preliminary nail length of 8.3 meters at a horizontal and vertical spacing of 1.5 m was selected. However, this length only represents the nail length in the upper half of the wall. The nail length in the lower half of the wall needs to be artificially shortened prior to performing a limit equilibrium analysis in order that the upper nail lengths are adequate to resist the anticipated loads at small deflections. Figure is used as follows to determine the distribution of nails lengths with depth.

The dimensionless nail pullout resistance, Q_D is calculated:

Q_D $= \alpha_Q Q_U / (\gamma S_V S_H)$

$= (0.50)\ (60.0kN/m3)\ /\ [(18.0kN/m3)\ (1.50m)(1.50m)]$

$= 0.74$

The dimensionless nail length is:

L/H $= (8.3m) / (9.5m)$

$= 0.87$

Q_D / (L/H) = 0.74 / 0.87

= 0.85,

From Chart, the value "R" factor is 0.32

Relative nail length are calculated from figure 4.11 for the nail head elevations shown on figure and "R" values of 0.32

Nail no	Trial length , m	Rx	Trial Nail length distribution
1	8.3	1.0	8.3
2	8.3	1.0	8.3
3	8.3	1.0	8.3
4	8.3	0.89	7.4
5	8.3	0.68	5.6
6	8.3	0.46	3.8

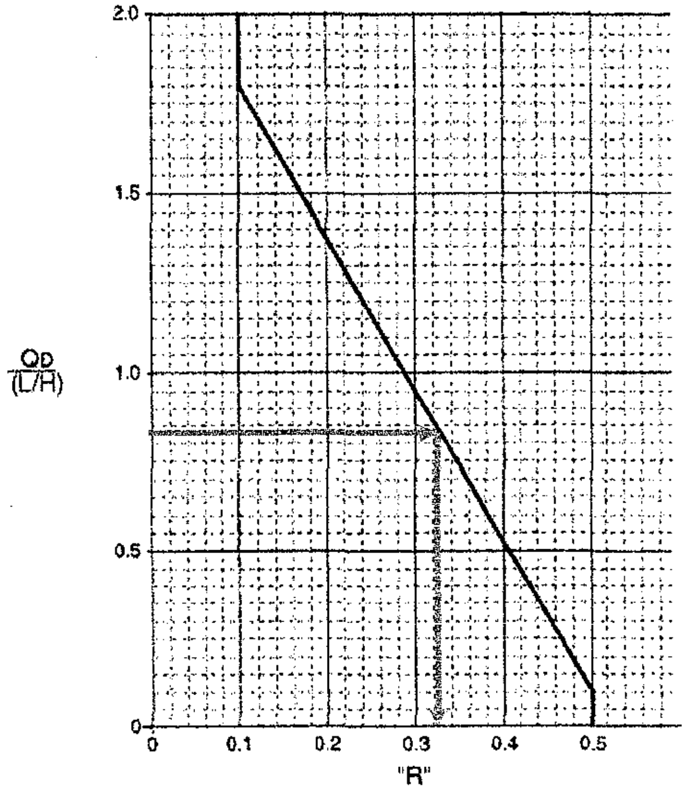


Figure 5.4: Design Chart D (from FHWA 1998)

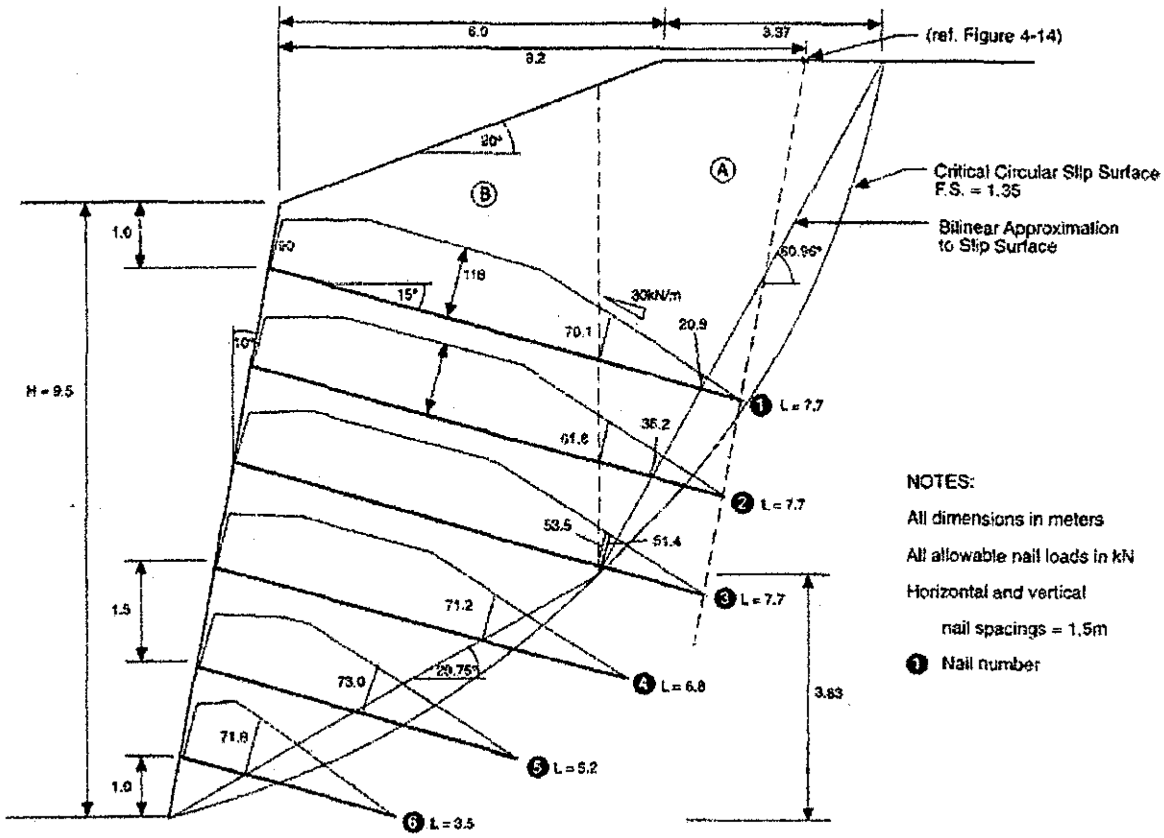


Figure5.5: Cutslope Design Examples Trial design Critical Cross Section Static Loading (from FHWA 1998)

Step 6: Define the Ultimate Soil Strengths

Ultimate Friction Angle, Φ_U = 34.0°

Ultimate cohesion, c_U = 5.0 kN/m^2

Step 7: Calculate the Factor of Safety

An iterative limiting equilibrium analysis is performed using appropriate computer software to determine the actual nail length that are required for a global safety factor of 1.35 (table Group I loading). The maximum nail length has been calculated interactively to be 7.7 meters.

Step 8: External Stability Check

A bearing capacity check is not necessary for the static design.

Step 9: Check Upper Cantilever

The height of the upper cantilever above the top nail is identical (1.0 meters) for temporary shotcrete. Therefore, the static loading is defined in two cases.

For a method of construction, the appropriate earth pressure coefficient for the upper cantilever design is an active earth pressure coefficient. For a soil friction angle of 34°, zero cohesion (ignore it), a soil/wall interface friction angle of (2/3) (34°) = 22°. K_a = 0.247.

The load component normal to the wall has a corresponding earth pressure coefficient

= 0.247 cos (22°)

= 0.229

Shear Check

From force equilibrium, compute the one-way unit service shear force for the facing at the level of the upper row of nails

$$\text{Shear force, } v_1 = 0.5(\text{soil - wall friction angle})\gamma H^2$$

$$= 0.5(0.229)\ (18.0\ \text{kN/m}^3)\ (1.0)$$

$= 2.06$ kN/m

Compute the nominal one way unit shear strength of the facing based on the equation

$V_{NS} = 0.166(f'_c)^{1/2}d$

$V_{NS} = 0.166\,(28)^{1/2}0.05$

$= 43.9$ kN/m

From Table, the facing shear strength factor, α_F equals to 0.667. Therefore the allowable one-way unit shear is computed to be;

$V = \alpha_F V_{NS}$

$= (0.67)(43.9 \text{ kN/m})$

$= 29.4$ kN/m

Since $v_1 < V$, the design for shear is adequate

Flexure Check

From moment equilibrium, compute the one way unit service moment for the facing at the level of the upper row of nails. The point of application is taken as 0.33H above the base of the cantilever

$m_s = (0.33)(H/\cos 10°)(v)$

$= (0.33)(1.0\text{m}/\cos 10°)(2.06 \text{ kN/m})$

$= 0.690$ kNm/m

Compute the nominal unit resistance of the facing. From the step 2, $m_{V,NEG}$, is computed to be 5.88 kNm/m. The strength factor, α_F for facing flexure is 0.67. Therefore, the allowable one way unit moment for the upper cantilever is:

$M = \alpha_F\, m_{V,NEG}$

$= 0.67\,(5.88 \text{ kNm/m})$

$= 3.94$ kNm/m

Since $m_s < M$, the facing for flexure is adequate.

Step 10: Check the Facing Reinforcement

Waler Reinforcement

The wale reinforcement to be place continuously along each nail row and located behind the face bearing plate at each nail head.

Step 11: Serviceability Checks

Shotcrete Construction Facing

Because of the temporary nature of the wall, the serviceability requirements are waived for the construction of the facing.

CHAPTER 6: SOIL NAILING MONITORING AND PERFROMANCE

This Chapter provides specific guidance on the monitoring and maintenance of soil-nailed systems. Proper supervision of soil nailing works to ensure conformance to design requirement and specification is important and checklist a sample of which is enclosed in the Appendix.

6.1 Monitoring

Monitoring is generally not required for a permanent slope or retaining wall reinforcedby soil nails that carry transient loads. For soil nails that carry sustained loads, monitoring of the ground movement and loads mobilised along representative soil nails should be carried out during construction and for a considerable period, e.g. at least two wet seasons after construction.

Good practical construction aspect of the soil nailing with particular reference to the quality control and acceptance criteria are necessary to avoid unsatisfactory performance or failures of the soil nail walls.

6.1.1 Drilling

There are many types of drilling techniques/tools and proper drilling through any and all ground conditions are very important to ensure satisfactory performance of soil nailing.

Basic requirements of proper or efficient drilling for soil nails are deployment of suitable machines (appropriate combination of thrust, torque, rotary speed, percussive force and flushing methods) and skilled operator to ensure:-

- To complete the drilling as soon as possible, typically less than 1 hour for the specified nail geometry.
- Machines shall be capable of permitting continuous and straight penetration in material that may invariably change abruptly from some localized soft to extremely hard or rock strata, etc.
- Capable of providing a constant diameter, stable drilled hole, drilling debris wholly and cleanly removed, etc. Drill rod should be at least *N* size and attached with an alignment control devise.

Rotary percussive drilling method using suitable top hammer or down-the-hole (DTH) hammer with proper drill bits (minimum 100mm diameter) to suit the types of material generally can meet the above requirements. Advantage of rotary percussive drilled grout holes are:-

- High and consistent penetration rate (12 – 20 m/hr) with minimum hole deviation when compared with rotary or auguring methods.
- Relatively small, light and mobile drill rigs can be used. High maneuverability.

To ensure good performance or high pull-out strength of soil nails, the hole has to be drilled and completed soonest possible, cleansed thoroughly and subsequently grouted immediately. To ensure reliable and effective cleaning of the drilled hole just before grouting, an additional drilled length 0.5 m to 1.0 m to the design nail length should be provided so that cleaning of cuttings and debris towards the bottom of the hole by the compressed air through the drill rod can be effectively and eventually carried out.

Drilled hole alignment deviation up to 20 mm in 3m for soil nails up to 30 m long can be considered acceptable. Reported/recorded alignment deviations for top drive percussion hammer and *DTH* hammer are generally < 20 mm and 15 mm in 3 m respectively. Set-up tolerance of drill rod shall be within 75mm from the designed position.

Drilling logs or records shall include not only operator/technician name, the location, date/ time of start/finish of drilling and soil type encountered, but logs shall also include observed exceptions or peculiarities such as marked variations in penetration rate, caving /sloughing of drillholes, flush / cuttings characteristics (wetness and sizes of cutting, etc.), drill response, drill length, deviation, date/time and method of grouting, grout pressure, photos, etc. These information are important and shall be considered when selecting the representative soil nails for pull-out tests.

6.12 Reinforcement

For permanent works, the rebars generally shall be protected against corrosion by hot-dip galvanizing (BS729) with minimum coat thickness of 85 microns or 610gm/m^2. For proven aggressive ground (resistivity < 2000 ohm-cm or pH < 4.5 or sulphate content > 200ppm, or chloride content > 100 ppm), the rebar shall be enclosed in corrugated HDPE sheath (min 1mm thick and the annular space between the rebar and sheath > 10mm). Typical details of nail head construction. Load likely to act on the nail head depends on the steepness of the slope/wall, bond strength mobilized in the active wedge, location of the rupture surface and the bearing capacity of slope surface soil. Typical standard design of soil nail plus the usual QC tests. To reduce deformation of soil nailed wall, it is a common practice to lock-in a load of about 5% to 10% of the soil nail working load, with a torque wrench and lock nuts. For sites where providing green environment is necessary, HDPE geocell with infilled topsoil and turfs or hydroseeding can be adopted as facing with buried nail head.

Quality centralizers at about 2 m spacing shall be securely and firmly fixed to ensure the rebar is not eccentrically grouted. Centralizers shall be made from quality PVC or galvanized steel sized to facilitate easy inserting, sized to allow free flow of grout and sized to allow the tremie grout pipe insertion to the bottom of the drill hole.

Only soil nails of more than 12 m long shall be spliced or coupled. The tensile strength of the mechanical splice or coupler shall be capable to develop the full tensile strength of the rebar as tested and certified by the manufacturer. Inserting of rebar shall be guided manually. Rebar shall be free from dirt and soil. Excessive force shall not be allowed in inserting the nail. In case of insertion refusal, the rebar shall be withdrawn and reinserted after the drill hole is redrilled and air reflushed. It is a good practice to withdraw some of the inserted rebars randomly to check the conditions of the centralizers. It is not uncommon to find many centralizers are damaged or deformed significantly, especially when poor quality centralizers with improper fixing methods are adopted.

6.1.3 Grouting

Quality and performance of insitu grout depend on quality of grout mix formulation, technique of grouting and conditions of drill hole. Water should be added to the mixer before any cement and admixtures. Mixing should be by a high speed colloidal shear mixer (> 1000 rpm) for a few minutes until a homogeneous grout free from undispersed cement, free from slumps, segregation, sedimentation and bleeding of water is obtained. The grout is then transferred through a 5mm sieve to remove lumps into a storage tank attached with a paddle agitator to prevent sedimentation and to avoid entrapment of air bubbles. Grout should be pumped into the drill hole as soon as possible and within the initial setting time (< 30 minutes after mixing). If normal paddle mixer (> 150 rpm) instead of high speed colloidal mixer is used, longer mixing time (> 10 minutes) is required and retarder may also be necessary.

The following important QC tests shall be carried out at least once or twice daily or every 40 cubic metres of grout used:-

- Crushing strength tests of 100mm cubes at 7 days and 28 days (BS 1881) shall be minimum 15kPa and 30 MPa respectively.
- Bleeding test (< 0.5% by volume 3 hour after mixing or 2% when measured at 20°C).

- Flow cone efflux time test (< 15 seconds, ASTM C939-87) to assess fluidity or grout rheology / flowability /penetrability.
- Non-destructive insitu grout strength test (ASTM C1074) to determine the rate of grout strength gain tests to determine the installed nail length may also be specified (optional)

6.2 Parameter to Be Monitored

The most significant parameters should be identified, with care taken to identify secondary parameters that should be measured if they could influence the primary parameters. The most significant measurement of overall performance of the soil nail wall system is the deformation of the wall or slope during and after construction.

The following list provides the important parameters that should be considered during soil nail wall performance using geotechnical instrumentation:

- Vertical and horizontal movement of the wall
- Vertical and horizontal movement of the surface of the overall structure
- Local movement or deterioration of the facing elements
- Drainage behavior of the ground
- Performance of any structure supported by the reinforced ground, such as roadways, etc
- Loads in the nails, with special attention to the magnitude and location of the maximum load
- Load distribution in the nail due to surcharge loads
- Nail loads at the wall face
- Temperature (may cause real changes in other parameters and also affect instrument readings)
- Rainfall (often a cause of real changes in other parameter)

6.3 Soil Nail Wall Performance Monitoring Instruments

The instrument should be selected based on the parameter to be measured, the instrument's reliability and simplicity and the instrument's compatibility with the readout devices specified for the project. Other factor should be considered include the influence of the instrument's installation on construction and skills of the personnel who will read the instrument.

6.3.1 Slope Inclinometer

The most significant measurement of overall performance of the wall is the deformation of the soil nail wall during and after construction. Slope inclinometer, preferably install about 1 meter behind the soil nail wall face, and provide the most comprehensive data on the wall deformation.

The measuring system typically consists of a portable probe that measures its own orientation relative to vertical. The probe is mounted on wheel s and is raised or lowered within a grooved casing installed vertically in the ground. Readings are taken by hand or on data loggers.

6.3.2 Survey Point

Soil nail wall deformation can be measured directly by optical surveying method or with electronic distance measuring (EDM) equipment. While, ground movement behind the soil nail wall can be assessed by monitoring an array or pattern of ground surface points established behind the wall face and extending for a horizontal distance at least equal to the wall height. In addition reflector prisms attached to selected nails permit electronic deformation measurement of discreet points on the soil nail wall face.

Frequent monitoring if the ground during the progress of construction allows the actual performance to be checked against the design assumption, provides a real-time record of performance, thereby allowing modification of the construction procedure in response to changed conditions. This can useful if wall deformations become significant because poorer ground than originally anticipated is encountered.

The survey system should be capable of measuring horizontal and vertical displacement to accuracy of 3mm or better.

6.3.3 Soil Nail Strain Gages

Soil nail instrument with strain gages allow assessment of the soil nail load distribution as the excavation progresses and following completion of the soil nail wall installation. By strain gauging individual nails in the laboratory and during filed tests, the development and distribution of the nail forces may be measured.

6.34 Load Cells at the Nail Head

Load cells installed at the soil nail head are used to provide reliable information on the actual loads that are develop at the facing.

6.4 Pullout Test

The purpose of pull-out tests up to 2.0 times the design load is to verify the designed pull-out resistance or designed bond strength and also to verify the adequacy of drilling, installation and grouting techniques. Usually, at least 1% to 5% of installed nails should be subject to pull-out test. The results of pull-out tests shall be carefully analyzed with the purpose to revise the design accordingly.

The pull-out strength or bond strength of soil nails depends on but not limited to:-

- Insitu soil/rock type, density, permeability and strength
- Reinforcement type and size, length
- drilling technique and procedure
- hole cleanliness and wetness
- Grout characteristics, strength, pressure, etc.

Testing is not everything unless the test results are adequate and representative so that the results can statistically represent the untested soil nails on the safe side. In this respect, the representative weakest nails based on site observation, SI report & installation records shall be selected for pull-out tests.

FHWA (1998) recommends that at least 2 preliminary pull-out tests or verification tests shall be carried out per different soil/rock unit or per different drilling/grouting method for each nailed slope/hill.

The temporary unbonded length of the test nail shall be at least 1 m or preferably 3 m. The loading schedule for verification test is as follows:-

Load	**Hold Time**
(5% DTL)	1 minutes
0.25 DTL	10 minutes
0.50 DTL	10 minutes
0.75 DTL	10 minutes
1.00 DTL	10 minutes
1.25 DTL	10 minutes
1.50 DTL (Creep Test)	60 minutes
1.75 DTL	10 minutes
2.00 DTL (Max test Load)	10 minutes

DTL = Design Test Load (kN)
= Lb x Qd

Lb = As-built bonded test length (min 1 m)
= 0.9fy As/ 2.0Qd,
where fy and As are yield stress and area of rebar respectively.

Qd = Design / allowable pull-out resistance (kN/m)

At least 2 calibrated dial gauges of 0.025 mm accuracy shall be used to measure nail head movement. Each load increment shall be held for at least 10 minutes. Nail movement at creep test (1.50 DTL) shall be taken at 1, 2, 3, 5, 6, 10, 20, 30, 50 and 60 minutes. The load during the creep test shall be maintained within 2% of the intended load by use of a calibrated load cell.

For working pull-out tests or proof tests, the testing procedure including creep test is similar to verification test except that the max test load (MTL)
(MTL) = 1.5 x DTL
and
Lb = 0.9fy A_s/1.5Qd.

A pull-out test is deemed acceptable when:

a) For verification tests, a total creep movement of less than 2mm per log cycle of time between the 6 and 60 minute readings is measured during the creep test and the creep rate is linear or decreasing throughout the creep test load hold period.

b) For proof tests, a total creep movement of less than 1mm is measured between the 1 and 10 minute readings and the creep rate is linear or decreasing throughout the creep test load hold period.

c) The total measured movement at the max test load (MTL) exceeds 80% of the theoretical elastic elongation (le) of the test nail unbonded length

$l_e = 0.8P\ (UL)\ (10^6)$

Where

P = max applied load

UL = length from the back of nail to jack connection to the top of the bond

As = 2 rebar cross-sectional area (mm)

E = rebars's modulus = 200,000 MPa

d) A pull-out failure does not occur at the max test load. Pull-out failure is defined as the load at which attempts to further increase the test load simply result in continued pull-out movement of the test nail.

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APPENDIX A
Sample Checklist for Construction Supervision of Soil Nailing Works

No.	Checklist Items	Acknowledge by	Checked by	
		Contractor	Client	
1.0	**EARTHWORK FOR SOIL NAIL SLOPE**	**SIGNATURE**	**YES**	**NO**
1.1	The construction sequences (stages of construction) shall be referred to the construction drawing			
1.2	The soil excavation shall not exceed 3m height per stage before soil nails, horizontal drains and shotcrete surface are completed.			
1.3	The next stage of excavation (after Item 1.2) shall only be allowed after the soil nails, horizontal drains and shotcrete surface are completed.			
1.4	The 4V:1H slope surface shall be covered with shotcrete after the installation of soil nails. No portion of the slope should be left exposed at 4V:1H gradient for more than 3 **days.**			
1.5	Temporary slope protection using canvas shall be carried out to prevent slope erosion			
1.6	Contractor that refuse to follow or not following the above construction sequences shall be WARNED and BLACKLISTED			
2.0	**SOIL NAIL**	**SIGNATURE**	**YES**	**NO**
2.1	Soil Nailing Material • Steel Nail reinforcement shall comply with BS 4449 or equivalent standard. (Only nails greater than 12m in length can be spliced using mechanical splicer approved by Engineer.) • Galvanizing: galvanize steel bar/ steel plate/ washer/ hexagon nut (All threading process on the steel elements shall be completed before galvanized or else the epoxy paint shall be applied on the threaded portion) • Centralizer: Provide only plastic centralizer or equivalent of a minimum diameter 25mm smaller than the nominal diameter of the drilled hole.			
2.2	Steel Welded Wire fabric • Shall comply to BS 4483 or equivalent • Lap mesh shall be at least 200mm or one mesh grid standard in both directions which ever is larger. • Tie wires shall be bent flat in the plane of the mesh and not forming large knot.			

	• Spacer: Provide sufficient spacer (eg: at least 1m interval) and ensure the spacer is solid.			
2.3	Horizontal Drain • Provide as required and shown on drawings (slotted and unslotted PVC) with end cap • Provision shall be made to ensure that the hole does not collapse prior to the insertion of the slotted drain			
2.4	Grout for Nails • Provide non-shrink neat cement or non-shrink sand cement grout with pumpable mixture capable of reaching minimum 28 days cube strength of 30 MPa in accordance with BS 1881. • To achieve non-shrink effect, additives shall be added (e.g. Intraplast Z). • Please record name and percentage of the additives that have been used as follows: ‹ ____________ (name) ‹ ____________ (percentage) • Have the additives been approved by the Engineer? ‹ Yes / No • Cube test to be carried out after every batching of grout.			
2.5	Permanent Structural Shotcrete Facing • Materials - Cement: Ordinary Portland Cement complying with BS12 or MS 522 and Portland Pulverized Fuel Ash Cement complying with MS 1227. - Aggregate: shall comply with BS 882 - Accelerating additives shall be compatible with the cement used, be non-corrosive to steel and not promote other detrimental effects (cracking and excessive shrinkage) and shall not contain calcium chloride. - Water used in the shotcrete mix shall be potable, clean and free from substances or element, which may be injurious to concrete and steel or cause staining • Quality Shall be produced by dry or wet mix process achieving a minimum compressive strength of 18MPa in 7 days and 30MPa in 28 days. • Construction Testing Shall carry out a test panel and send cores for testing in accordance to BS 1881			
3.0	**NAIL INSTALLATION**	**SIGNATURE**	**YES**	**NO**

3.1	General procedures: Check the size (diameter) of drill bit and compare with the required diameter of soil nail as specified in the drawings. Any anomalies shall be reported immediately to the Engineer. ‹ ____________mm (diameter of drill bit) ‹ ____________mm (required soil nail diameter) • Check the diameter of hole being formed. ‹ ____________mm • Mark clearly and accurately the point of the soil nail location. The drilled hole shall be located within 150mm of the location shown on drawing. • Supervisor and driller to ensure the drilling methods is suitable for maintaining open drill holes and do not promote mining and loosening of the soil at the perimeter of the drill hole or fracture soils with weak stratification planes by control the flush volumes and pressure. Provide nail length and nail diameter necessarily as required but not less than lengths and diameter as shown in the construction drawing. • At the point entry, the nail angle shall be within ± 3 degrees of the inclination as shown in the construction drawing. • Centralizers shall be provided at 2m intervals for the whole length of nail with the last centralizer located at 300mm from the end of each nail and ensure that not less than 30mm of grout cover is achieved along the nail. • Record the depth where the seepage of groundwater was observed (if any). • Inject grout at the lowest point of the drill hole. (Pump grout through tubes, casing, hollow stem auger or drill rods such that the hole is filled from the bottom to the top to prevent air voids until clean grout is seen to run from the top of the hole). Remark: Grout pipe must be used or else the particular soil nail will be rejected. Grouting equipment shall have capability of continuous mixing and producing grout free of lumps.			
4.0	**SHOTCRETING**	**SIGNATURE**	**YES**	**NO**

4.1	General procedures: • Slope surface to receive shotcrete shall be cleaned with air blast to remove loose material, mud, rebound from previously placed shotcrete and other foreign matter that will prevent bonding of shotcrete. • Dampen the surface before shotcreting. • During placement of shotcrete, the horizontal drains and weep holes shall be protected against contamination or clogging of shotcrete to ensure proper functioning. • Thickness measuring pins (non-corrosive) shall be installed on 1.5m grids in each direction. • Check the thickness of measuring pins using normal ruler or measuring tape. ‹ ____________mm • Thickness, method of support, air pressure and water content of the shotcrete shall be controlled in such a manner as to preclude sagging of sloughing off. • The shotcrete shall be applied from the bottom up to prevent accumulation of rebound shotcrete on the surface, which is to be covered. • Horizontal and vertical corners and hollow areas shall be filled first. • Checking for hollow areas on the completed shotcrete surface shall be carried out with a hammer. • All shotcrete which lacks uniformity, exhibits segregation, honeycombing or lamination, or which contains any dry patches, slugs, voids or sand pockets shall be removed and replace with fresh shotcrete. • In situ core test shall be carried out for verification. • Immediately after the completion of shotcreting works, keep shotcrete surface continuously moist for at least 24 hours for curing purpose. • The opened cut area shall be protected with canvas or suitable material to avoid erosion. As built drawing showing the location, dimensions, photos and details of the soil nail wall shall be produced by			

	the contractor.			
5.0	**PULL OUT TEST**	**SIGNATURE**	**YES**	**NO**
5.1	List of equipment • A single acting hollow hydraulic jack connected to hydraulic pump and pressure gauge with minimum capacity of 20MT • A pull out steel fabricated cage • A steel bracket • At least 4 displacement gauges • A pressure meter • Nut and washers • Stopwatch to measure the period of observation.			
5.2	General Procedures • Pull out test should be carried out in ground types and in environmental conditions similar to those existing at the proposed site. • The stressing equipment, pressure gauge and load cells should be calibrated by the manufacturer and in accordance with clause 10.6 BS 8081:1989. • The load cycle, load increments and minimum periods of observation shall be as instructed by the Engineer. • As built drawing showing the location of pull out test, dimensions, photos and details of the test shall be produced by the contractor.			