A STUDY OF APPLICABILITY OF DIFFERENT FACING TYPES IN SOIL NAILING

Buddhika Indramal Kumarage

168965F

Degree of Master of Engineering

Department of Civil Engineering

University of Moratuwa
Sri Lanka

March 2021

A STUDY OF APPLICABILITY OF DIFFERENT FACING TYPES IN SOIL NAILING

B.I Kumarage

168965F

Thesis/Dissertation submitted in partial fulfillment of the requirements for the degree of Master of Engineering in Geotechnical Engineering

Department of Civil Engineering

University of Moratuwa
Sri Lanka

March 2021

Declaration

I declare that this is my own work and this thesis does not incorporate without acknowledgment any material previously submitted for a Degree or Diploma in any other University or institute of higher learning and to the best of our knowledge and belief it does not contain any material previously published or written by another person except where the acknowledgment is made in the text.

Also, I hereby grant to University of Moratuwa the non-exclusive right to reproduce and distribute my thesis, in whole or in part in print electronic or other medium. I retain the right to use this content in whole or part in future works (such as articles or books)

Signature of the candidate: ………………….. Date: …………………..

B.I. Kumarage

The above candidate has carried out research for the Master's thesis under my supervision.

Signature of the supervisor: ………………….. Date: …………………..

Professor S.A.S Kulathilaka,
BSc. Eng. Hons (Moratuwa), PhD (Monash), CEng, MIE(SL)
Department of Civil Engineering,
University of Moratuwa,
Sri Lanka.

Abstract

Soil nailing has been used in Sri Lanka lately, in a wide range of infrastructure projects, as a cost-effective stabilization technique that can be implemented quickly. However, detailed designs with rigor, for nailhead/ facing are not often carried out in the local practice. Full face shotcrete, grid beams connecting nail heads, isolated nail heads (Pillows), and combinations of all these facings types used in the local practice. When the full face shotcreting is not used vegetation is used as a surface protection cover in between the nail heads with the help of a geotextile and nail heads are combined with a wire mesh of specified tensile strength. The mesh is expected to provide stability against any local shallow failures. The versions without full-face shotcrete blend nicely with the natural environment and have gained greater acceptance.

In this research design guidelines for different facing types available in published literature are critically reviewed to assess the suitability under high rainfall intensities in local residual soil formations. A number of sites rectified with soil nailing with different types of facing have now experienced few seasons of rainfall and their performance is assessed. The cost-effectiveness and construction difficulties are also reviewed.

Based on these factors some guidelines are developed to decide on the most appropriate type of facing depending on the prevailing local conditions.

keywords; slope rectification, soil nailing, facing types for soil nailing

B I Kumarage (NBRO)   S A S Kulathilaka (University of Moratuwa)

Acknowledgment

Firstly, I would like to express my sincere thankfulness to my supervisor, Prof. S.A.S. Kulathilaka for his guidance and support as well as the encouragement provided throughout the research work. His profound insight and vast experience in the field of geotechnical engineering made the suggestions and comments he made extremely versatile and contributed immensely to the success of this research. Also, my sincere thanks go to Dr. L.I.N. De Silva as the course coordinator and all the academic staff of Geotechnical Engineering: Prof. U.G.A. Puswewala, Dr. U.P. Nawagamuwa, and Dr. (Mrs.) A.S. Ranathunga for supporting in various means with regards to the academic works with excellent cooperation and guidance.

I am enormously grateful to Dr. Asiri Karunawardena, Director General, National Building Research Organisation (NBRO) for his guidance, support, and continual encouragement that was given throughout the process. I also owe my gratitude to Mr. U.K.N.P. Dharmasena, Senior Engineer (Construction) of the CRIP-Road Project for his support throughout the project period and after. Last but not least, I extend my gratitude for the continued support and insight provided by Eng. P.R.C. Ariyarathne, who was the in-charge of the Design Unit of NBRO, as well as all the other colleagues of NBRO for supporting in every possible way.

Table of Contents

List of figures

List of tables

CHAPTER 1. INTRODUCTION

1.1 Background

Soil Nailing is a technique used to reinforce the existing sloping grounds to enhance the stability. The method uses closely spaced passive structural elements that are installed into the ground. These structural elements, which are generally in the form of deformed steel bars are called Soil Nails. The steel bars could be either installed in bored holes which are subsequently grouted, self-drilled, or driven or rotated (Phear, et al., 2005). In Sri Lanka, the most commonly used technique is the bored and grouted method.

In-situ soil is a very poor structural element when subjected to tension. The function of Soil Nails is to use the tensile capacity of reinforcement bars to enhance the overall shear strength of soils. Soil Nails stabilize the soil structures by the tension developed in them as a result of the deformation of the retained soil mass. The developed tensile loads are then transferred to the surrounding ground through bond shear stresses along the grout-ground interface (Carlos A. Lazarte, et al., 2015, pp. 1-2).

The concepts of Soil Nailing have originated initially for rock-bolting and multi-anchorage systems and parts for the soil reinforcement which had similarities with the current soil nailing methods. In the early 1960s, in dam projects in France, retaining walls for spillways were being built using numerous bars grouted into the bedrock and faced with reinforced concrete. A similar method of tunneling design was developed between 1957 and 1965, referred to as the New Austrian Tunneling Method. The idea was to excavate the tunnel and immediately install steel bars to reinforce the peripheral ground, grout them, and then construct a sprayed concrete lining reinforced with steel wire meshes to provide primary support. Having realized the potential of nailing through the tunneling works, the first reported soil nailed wall was built to support a railway cutting in France in 1972 (Phear, et al., 2005). In this first application, this new nailing method had proved to be cost-effective and the construction had been faster than traditional slope support methods (Carlos A. Lazarte, et al., 2015).

The first reported research program on soil nailing has been carried out in Germany at the University of Karlsruhe from 1975 to 1981. This program involved full-scale testing of experimental walls. Thereafter the French Clouterre research program started in 1986. This research involved large-scale testing, monitoring of completed soil nail walls, field tests, and numerical simulations (French National Project Clouterre, 1991). The findings of this research were later translated into English and adopted in other countries.

1.2 Soil Nail Facings

Soil nails do not stabilize the exposed surface soil when used to stabilize existing slopes or excavations (Phear, et al., 2005). Hence, a system of soil nails generally consists of head plates and/or facings of some form that connects the installed soil nails at the surface of the slope. In addition to the function of stabilizing the surface soil, the reaction forces produced by soil-nail heads/facing during the deformation of the retained soil mass, contribute to developing tensile forces along with soil nails. This tensile force acts in addition to the force developed by frictional interaction between the soil nails and the surrounding soil (Geotechnical Engineering Office, 2008). The amount of these forces transferred to the head/ facing varies depending on the type and stiffness of facing (Phear, et al., 2005).

BS EN 14490:2010 (BSI, 2010) defines 3 main types of facings, as follows.

i. Hard facing:
   Soil nails and facing in combination have to stabilize the slope between the nails. Therefore, the elements shall be dimensioned to sustain the expected maximum destabilizing forces.
ii. Flexible facing:
   Flexible facings are designed to provide the necessary restraint to the areas of slope face between the bearing plates, as well as erosion control. The selection of the type of flexible facing is dependent upon slope angle, soil friction angle value, slope height, and predicted loading. The common flexible facings include geogrids steel fabrics and geosynthetic.

iii. Soft facing:

The primary function of soft facing is erosion control and protection against surface raveling. In many cases, the soft facing has to reinforce the vegetation layer, either in the temporary or the permanent situation.

In the Sri Lankan practice, most of the types stated above are used in combinations. Such main types of facings are as follows:

- Hard Facings; Full-face shotcrete, reinforced concrete grid beams.
- Flexible Facings; High tensile steel wire mesh with metal bearing plates and high tensile steel wire mesh with isolated reinforced concrete nail heads
- Soft facings; Nailheads are not structurally connected. Some connection is provided to facilitate the growth of vegetation to prevent erosion. This may include, Coir meshes, Geosynthetic mesh products.
- Mixed Types; Reinforced concrete grid beams along with high tensile flexible wire mesh with vegetation cover in the slope surface.

1.3 Failures in Soil-Nailed Systems

Occasionally, failures of soil-nailed walls occur even though not many public records are available. Most of the failures are caused by, inadequate understanding of the substratum and groundwater conditions, lack of understanding that soil nailing is a soil reinforcement technique (having one row of nails for smaller slopes, having nails that are too long and too widely spaced), and poor workmanship standards.

Several failures can also be specially attributed to matters related to facings such as (Phear, et al., 2005):

- inferior facing design (mainly for flexible facings)
- lack of attention towards the interrelationship between nail spacing and type of facing and arching mechanisms in the reinforced soil zone
- lack of attention to construction details (plates at nailhead are often too small)
- delays between those critical activities, (excavation, installation of nails, and placement of facing)

Hong Kong GEO Report No. 222 (Ng, Lau, Shum, & Cheung, 2007) presents several reported cases of failures involving soil nailing.

Failure of a 45 m High Soil-nailed Slope in Malaysia reported in this study is related to a slope facing failure. The slope was reinforced with 12m long soil nails. Nail spacing was 1.5m and the facing was a 100mm thick shotcrete (G25) with two layers of reinforcement mesh. The landslide was shallow and most of the nails remained on the slope after failure. Post landslide investigation suggested that the flexural and punching shear capacity of the shotcrete facing was inadequate, and the groundwater or the rainfall was not a contributory factor.

Similarly, other failure cases on soil-nailed slopes in Hong Kong with vegetation cover which involved local and minor erosion or detachment from shallow depths inside the active zone have been reported in this report (Ng, Lau, Shum, & Cheung, 2007). These failures emphasize the need for a well-structured facing design for soil nailed slopes, whether they are of soft, flexible, or hard forms.

1.4 Use of Soil Nail Facings in Sri Lanka

Here in Sri Lanka, from the early stages of adopting the soil nailing technology, different types of facings had been used. In the local practice, extensive analysis for the overall stability of soil-nailed slopes is carried out using numerous limit equilibrium methods. Nailhead designs and details are mostly adopted from the Hong Kong approach described in this thesis. However, detailed designs with rigor for nailhead/ facing are not often carried out. In the current practice, the selection of the head/ facing type is mainly based on the following three major factors.

- Observed nature of the surface slope soil/ material
- Ease of construction
- Aesthetics

It should be noted that some of the technologies such as the extruded crib beams used by the Japanese, as well as the high capacity flexible systems introduced by Europeans were used only in few instances, due to their higher cost and the unavailability of experienced local contractors to conduct such work.

Due to the short history of adopting soil nailing, major failures of soil-nailed slopes are not reported in Sri Lanka. However, the limited number of known failures that relate to facings have occurred during the construction stage. A major failure was recently reported in the Diyagala - Ginigathhena, which could be attributable to front face pullout failure. The construction works were carried out by a foreign contractor. Soil nails installed in the slopes were observed to be intact and protruding out of the slope after the collapse. This was a typical example of soil mass in the front face sliding off in between the soil nails, as described in FHWA 1998 (Byrne, Cotton, Porterfield, Wolshlag, & Ueblacker, 1998). A proper investigation carried out on this failure is yet to be published.

Due to the variable nature of soils in the slopes in Sri Lanka, the presence of boulders, cavities in the existing slopes have made the construction process much difficult. The typical arrangement for grid beams (Grillage structures) requires the beams to be embedded into the nailed slopes. The difficulties of excavation in the slopes lead to delays as well as minor failures during the rainy period. To overcome this issue, the construction team is forced to change the facing type, to isolated nailheads (concrete pads) or full-face shotcrete.

Similarly, when sandy (collapsible) soils are observed at the surface, the facing is most likely changed to full-face shotcrete, due to the construction difficulties. Also, when much harder slope faces are observed, nail heads are limited to the steel plates due to the difficulties of constructing separate nail heads, and the flexible mesh is installed to connect the steel-plated nail heads. Because of these site-specific design changes which are frequently required, a design framework for nail heads and facings focused on the facing types used in Sri Lanka is invaluable.

No reported studies which relate to nailhead/ face design and construction in Sri Lanka are available at this stage. Even though not much attention is paid towards the designing and studying of facings in Sri Lanka, based on the studies carried out over the past 40 years, it is clear that the contribution of facing is significant in terms of overall slope stability, in addition to shallow surface protection. Therefore, it shall be emphasized that the facing design and construction shall have more focus and detail in any major soil nailing work.

1.5 Scope and Objectives of the Research

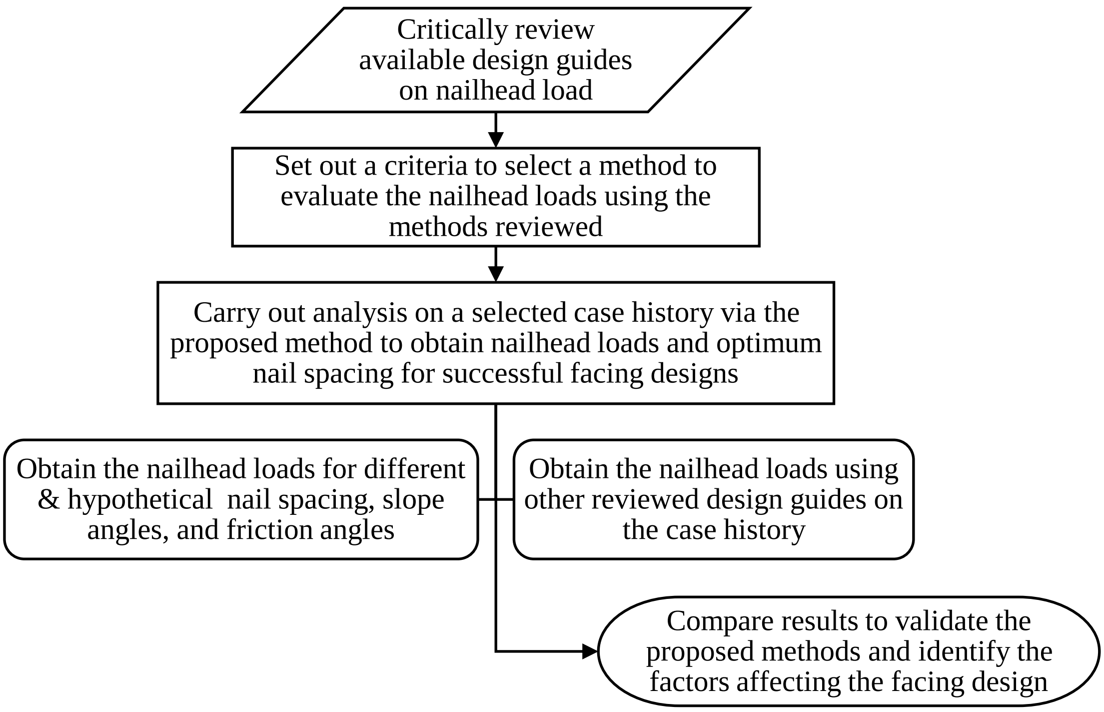
- Understanding the behavior of forces on the soil nails with different types of facings.
- Analyzing the performance of soil nails and the surface slope stability after the application of various facing types.
- Assessment of the real-world performances of the different facing types used in the slope rectification works in Sri Lanka
- Based on the above, developing some guidelines to decide on the most appropriate type of facing, depending on the prevailing local conditions

1.6 Methodology Followed

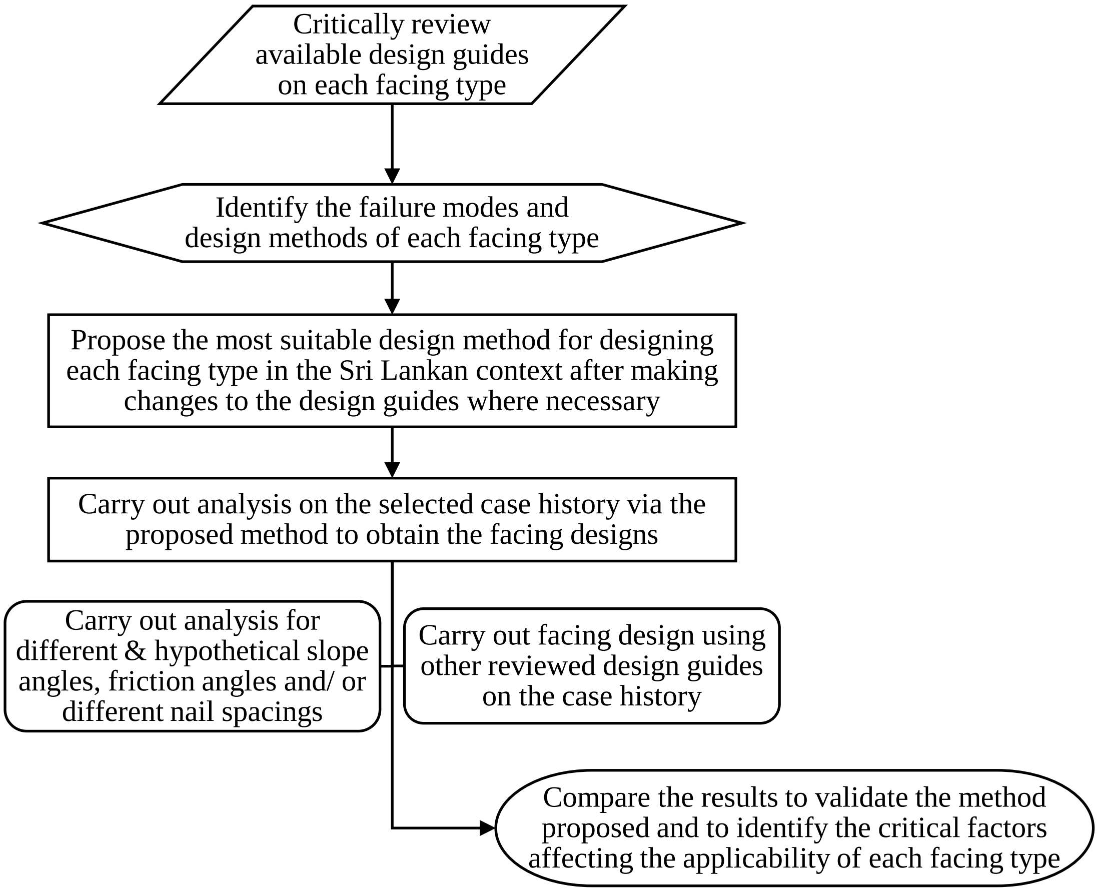
Literature Review

Review available literature and identify the critical factors affecting soil nail facing design for commonly used & applicable soil nailing facing types in Sri Lanka.

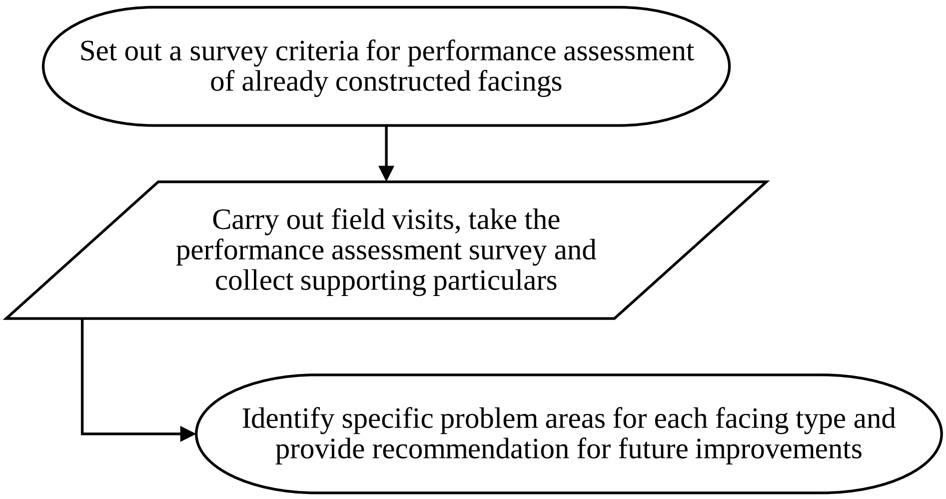
Nailhead load (Nail forces on facing)



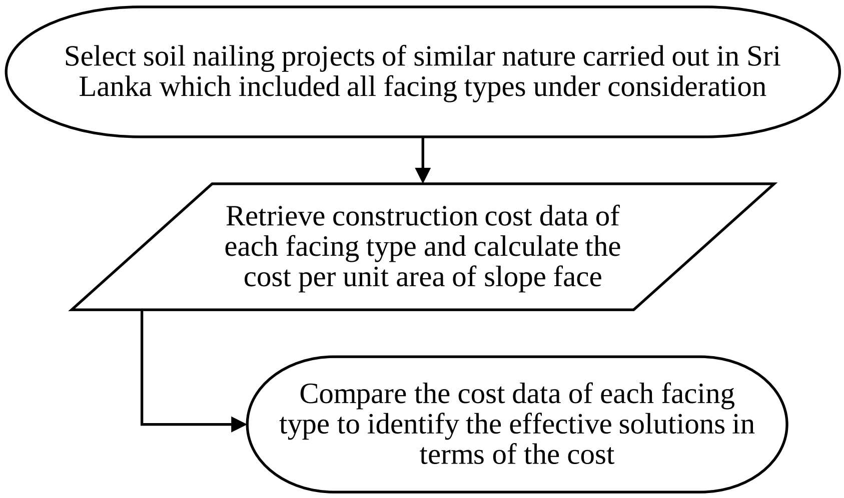
Facing design



Performance assessment



Cost assessment



Conclusion

Present a concluding note on the applicability of each facing type based on,

i. the factors affecting each facing design identified during the design stages,
ii. the problem areas of each facing type identified in the performance assessemt, and
iii. the cost differences between each facing type,

to be used as a guide for the application and design of soil nail facings in the local practice.

1.7 Structure of the Thesis

Chapter 02 presents the literature review of the areas related to the soil nail facing designs. This chapter first discusses the research carried out which included experimental models as well as field studies. Then the design guides available worldwide are compared in relation to their applicability for the methods used in Sri Lankan practice.

Chapter 3 discusses the nailhead loads in soil nailing systems. It briefs the applicable methods discussed in the preceding chapter and recommends the methods that could be used in the local practice. Further, an application of the recommended method for a case history is demonstrated in detail. The effect of variable slope and nailing parameters on the nailhead load is further discussed in this chapter.

Chapters 4 to 7 are dedicated for the design of shotcrete facings, grid beam facings, flexible facing systems, and isolated nailheads respectively. They outline the methods available for designing each type of facings and recommend methods that could be applicable in the local practice. Additionally, the recommended methods are applied for the selected case history to demonstrate the usage of the methods in detail. The effect of variable slope and nailing parameters on the design of facings is further discussed in these chapters. Lastly, a comparison of the results obtained by different methods of facing design is also carried out for each facing type.

Chapter 8 presents a field study carried out to assess the performance of different types of soil nail facings constructed in Sri Lanka. This demonstrates the performance of each facing type during their service life using several observable features such as crack formation, erosion, and local instabilities.

Chapter 9 carries out a comparison of costs of each type of facing which is then used to compare and determine the most appropriate and economical facing type for local practice under different conditions.

Chapter 10 presents the conclusion of the study.

CHAPTER 2. REVIEW OF LITERATURE

2.1 Research Carried out on Facings

Generally, there is a good understanding of the tensile loads developed within the nails. However, the extent and the distribution of the nail head/ facing loading developed in nailed structures are not understood to a similar level. This is mainly because of the poor quality of field monitoring data. The data obtained from strain gauges installed near the facing is difficult to interpret due to the presence of bending effects. Nailhead loads can be most reliably obtained from load cells installed at the facing. (Byrne, Cotton, Porterfield, Wolshlag, & Ueblacker, 1998)

2.1.1 Experimental Full-scale Structures and Laboratory Model Tests

Gässler & Gudehus (1981)

Gässler & Gudehus (1981) carried out full-scale experiments of Soil nails and shotcrete facings. Earth pressures behind the shotcrete facing were measured by load cells and forces along selected soil nails were monitored by strain gauges. Earth pressure diagrams at rest and during different surcharges levels are presented in their publication (Figure 2-1).

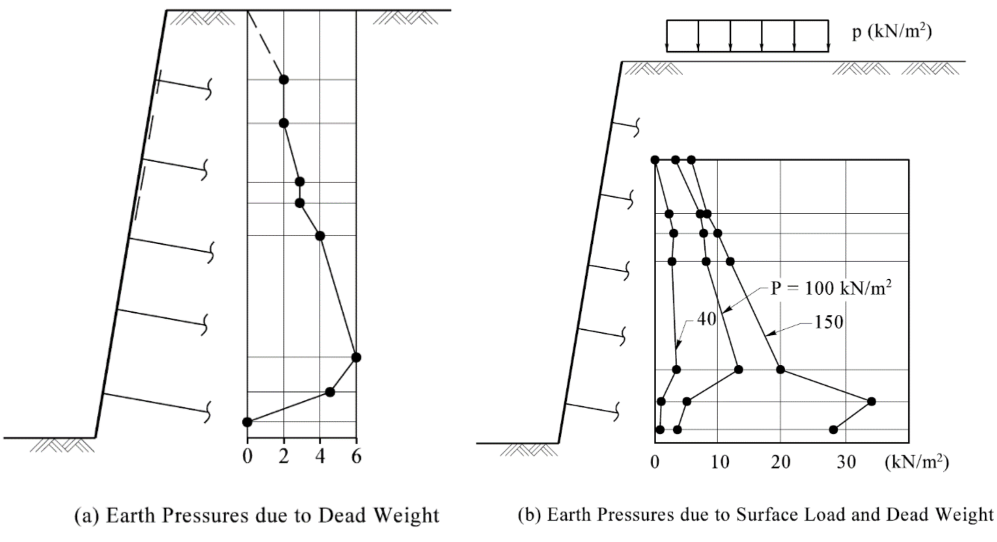


Figure 2-1. Earth Pressure Behind Facing (Gässler & Gudehus (1981))

The facing loads during the added surcharge were 60% to 70% of the Coulomb active pressure. Thus, they recommended that the facings shall be designed to withstand this reduced earth pressure (Shiu & Chang, 2004).

Plumelle & Schlosser (1990)

Plumelle & Schlosser (1990) reported their work which included experimental tests on facings. In the study of this 6m high retaining wall, only the upper portion of the wall was reinforced. The lower portion was supported by 3nos of 1m high panels and the reinforced part was also supported at the face by a separate panel. Forces behind the nail head panels were monitored while removing supported panels one by one during the test (Figure 2-2) (Shiu & Chang, 2004).

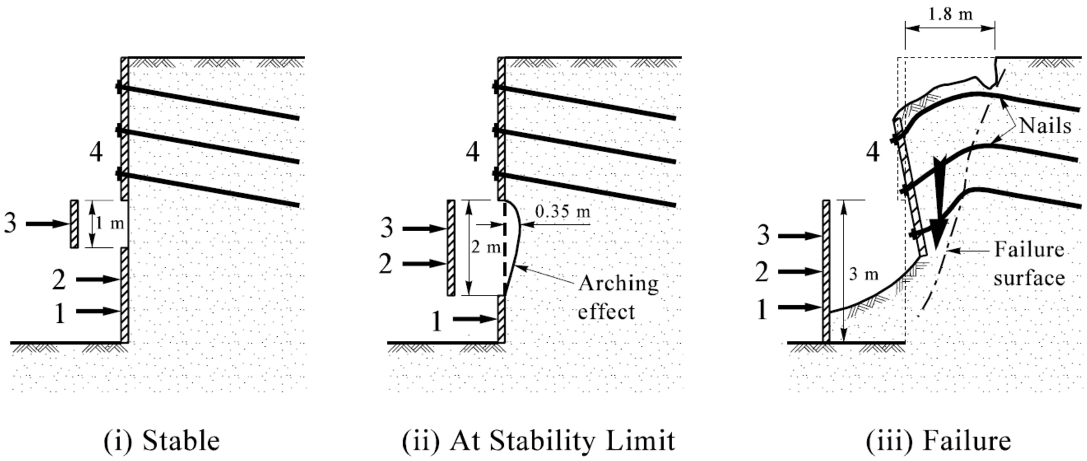


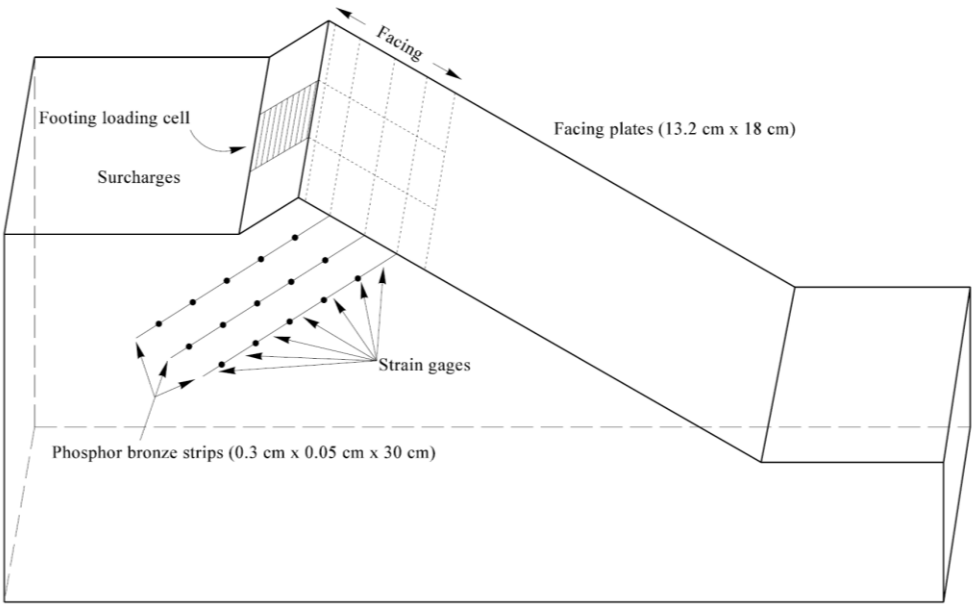
Figure 2-2. Stability of Excavation (after Plumelle and Schlosser, 1990)

The removal of panel 3 did not show any instability. With the removal of both panels 3 and 2, the structure has approached the stage of limiting stability. Arching effects have maintained the stability at this stage. With the removal of panel 1 complete failure has occurred. Although this is not an example of the instability of the facing in nailing, it has illustrated the effect of arching in maintaining stability.

Gutierrez & Tatsuoka (1988)

Gutierrez & Tatsuoka (1988) experimented with 3 different types of sand slope models namely; (i) unreinforced slope, (ii) slope reinforced with metal strips, without a facing, and (iii) slope reinforced with metal strips, with a facing. Surcharge loads were induced to the slopes in the form of loaded footings at the crest (Figure 2-3). Retaining the active zone was not feasible without the provision of a facing. Thus, on the nailed slope with no facing, the failure was closer to the slope face. Whereas, in the nailed slope with facing, the failure surface was observed at a greater depth.

All in all, it was evident from the results that the nailed slope with facing could withstand higher surcharge loads than the slope with no facing.



.

Figure 2-3. Instrumentation of Model Slope Reinforced with Metal Strips and Facing (after Gutierrez and Tatsuoka, 1988)

Additionally, it was observed that the maximum tensile forces generated along the nails of the slope with facing were larger than that of the slope with no facing as shown in Figure 2-4. Also, a considerable tensile force was induced at the nail-face connection in the case of slopes with a proper facing. These results show that in addition to helping to prevent shallow failures, the facing can enhance the overall stability of the slopes (Shiu & Chang, 2004)

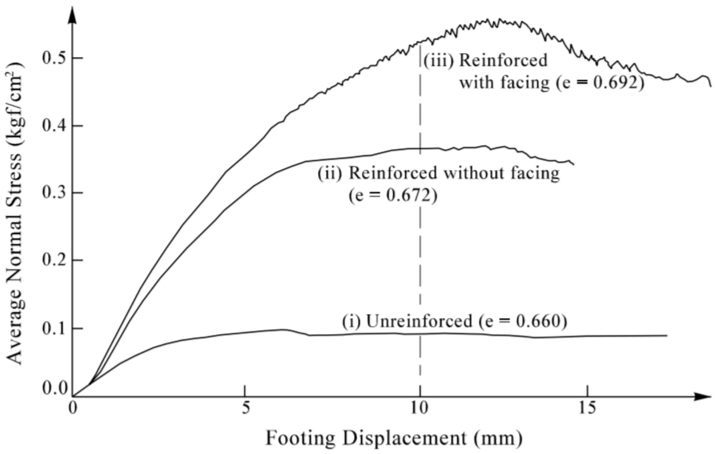
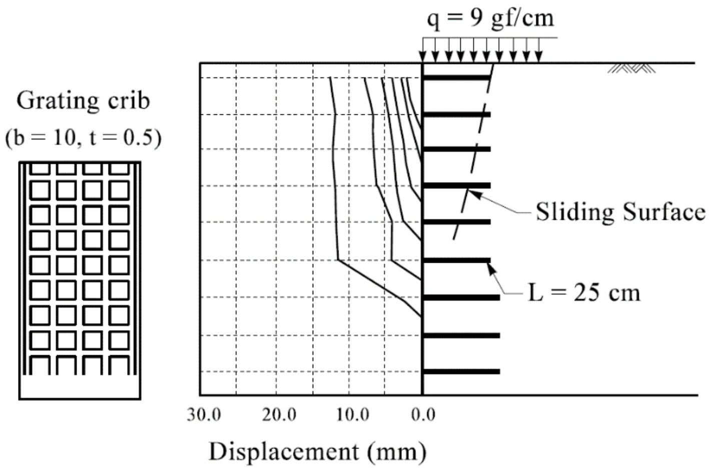


Figure 2-4. Surcharge load and displacement relationship of soil nailed slopes (after Gutierrez and Tatsuoka, 1988)

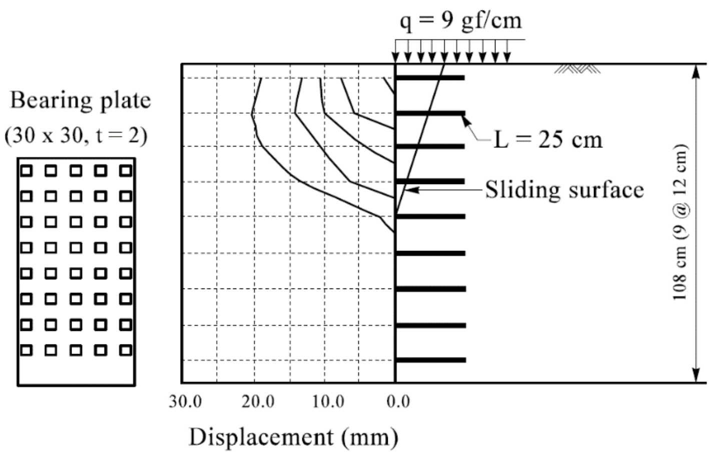
Muramatsu et al (1992)

Muramatsu et al (1992) carried out small-scale model tests as well as full-scale field tests to study the effects of soil nail heads. In these small model tests, two types of facings were used, a continuous “grating crib” and an isolated “bearing plate” type (Figure 2-5). Tests were carried out with varied surface areas of each type of above facings.



b = grating beam facing breadth (mm)
t = facing thickness (mm)
q = surcharge
L = length of soil nails

Type I



Bearing plate size = 30 mm x 30 mm
facing thickness = 2 mm
q = surcharge
L = length of soil nails

Type II

Figure 2-5. Slope Facing Types and Reinforced Slope Deformation Modes (after Muramatsu et al, 1992)

The results depict that the limiting excavation height of the nailed slopes increases with the increasing ratio of facing area to slope area. Also, the strain gauges indicated a relatively higher nailhead displacement in the slope with bearing plates.

Additionally, the results of the strain gauge tests indicated that the distribution of tensile forces along the nail is more or less symmetrical to the maximum tensile force developed at the middle portion of the nails, in the nails with bearing plates. Whereas in the nails with grating crib facing, the maximum tensile force was at the nail-head connection area as shown in Figure 2-6 (Shiu & Chang, 2004).

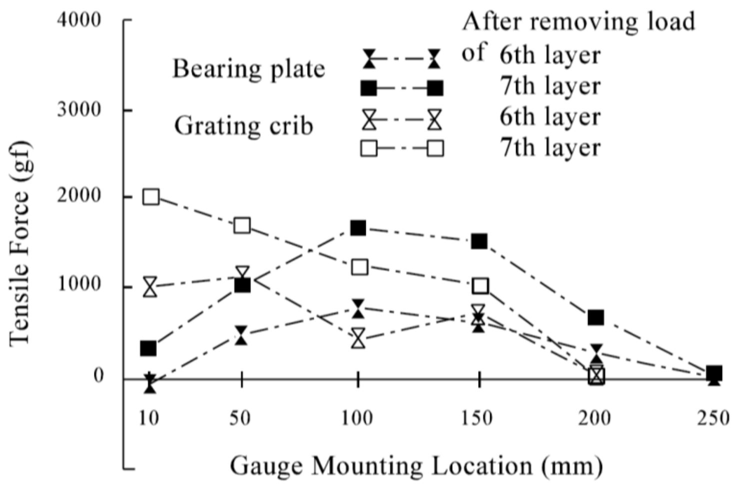


Figure 2-6. Distribution of Tensile Force Along Soil Nails with Different Facing Types (after Muramatsu et al, 1992)

Tei et al (1998)

Tei et al (1998) carried out tests for small-scale centrifuge models. Model slopes were 200mm high and at the initially tested 30g acceleration, this height corresponds to a height of 6m. Four types of facings, flexible (rough and smooth) & rigid (rough & smooth) were used in this experiment to investigate the effect of facing roughness and stiffness on the stability of the nailed slopes. Flexible facings were made of Perspex with a thickness of t=0.6 mm, and the stiff facings of Perspex with a thickness of t=5.2 mm.

The results indicated that the horizontal displacement of the slopes with stiffer facings was significantly lower than those with less stiffness as shown in Figure 2-7 (Tei, Tayor, & Milligan, 1998).

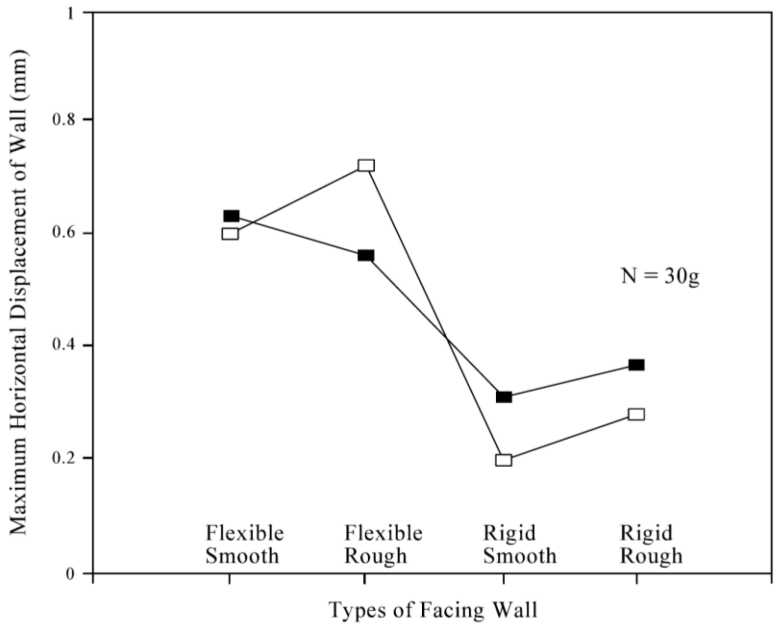


Figure 2-7. Horizontal Displacements Related to Facing Roughness and Flexibility (after Tei et al, 1998)

2.1.2 Field Studies

The behavior of earth pressures and the nail loads for different facings at the service conditions has been studied from the field monitoring tests carried out in Europe and the USA.

Muramatsu et al (1992)

The field study carried out by Muramatsu et al (1992) had a similar setup to their model studies. Slope height was 9.5m, slope angle 80°, nail length 5m at 10°, and the nail spacing was 1.5mx1.5m. Two different types of facings were introduced to the two identical slopes with identical nailing patterns. One was a 100mm thick Full-face shotcrete surface and the other was a 200mmx200mm sprayed concrete crib beam. The tensile forces developed during the tests yielded different distribution patterns for the two facing types (Figure 2-8). For the full-shotcrete facing, the maximum tensile force was observed at the mid-length of the installed soil nails. Whereas, in the crib facing, the maximum tensile force was at the nail-facing connection area. Additionally, the maximum tensile forces developed in the crib-faced nails were higher than those of full-shotcrete facings. This indicates that more rigid (Hard) crib beam facings contribute more to the overall stability of the slope compared to the more flexible full shotcrete facing. In addition to their field models, numerical analysis was also conducted using 2D finite element models, showed similar trends to the above experimental results as presented in the GEO report 175 (Shiu & Chang, 2004).

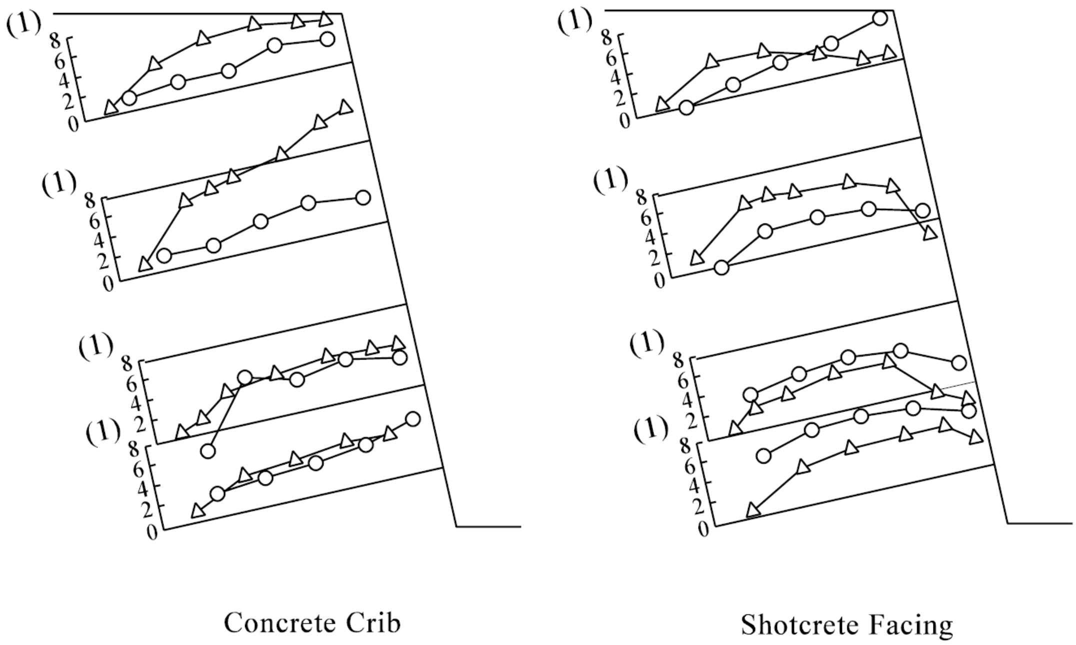


Figure 2-8. Distribution of Tensile Force in Reinforcing Bars (after Muramatsu et al, 1992)

Stocker & Riedinger (1990)

Stocker & Riedinger (1990) presented the results of field monitoring tests of instrumented soil nailed walls up to 15m high. The facing type was full-face shotcrete of 250mm thickness. Monitoring was carried out for over 10 years. At this service stage, the earth pressure measured behind the facing was uniform with the depth, and the measured pressure was 63% of the calculated active earth pressure.

Similarly, in the field monitoring carried out in the French National Research Project, the ratio of nailhead load to the maximum nail load was 0.4 to 0.5. This ratio increased with the rigidity of the facing and the increase of nail spacing (French National Project Clouterre, 1991).

The results of the measured nail head loads published by FHWA (1998) depict similar results. Accordingly, the average “normalized nailhead load” (T_0/ K_a γ_s H S_V S_H) (further discussed in Section 2.4.2) was 0.4 to 0.45 under the service load conditions (Byrne, Cotton, Porterfield, Wolshlag, & Ueblacker, 1998).

Advance facing systems

Other than these studies, more recent studies on advance facing methods such as the use of high tensile flexible meshes for soil nailed slopes have been done. Some of the studies have been published in the Proceedings of the Second World Landslide Forum (Margottini, Canuti, & Sassa, 2013). One such paper, High Capacity Flexible Systems for Slope Stabilization in La Gomera (Spain) (Laguna L. M., et al., 2013), provides a detailed review of the high capacity flexible system used in Spain. The method uses a mesh made of a high-tensile steel wire with a yield strength of 1,770 N/mm^2 combined with steel rope reinforcement, and bar anchorages. This study further illustrates the better use of the high-capacity meshes and components produced by Geobrugg A.G. (Switzerland). Additionally, they emphasize the non-availability of this technology in English literature and they state that they aim to disseminate this technology in other countries via this study (Laguna L. M., et al., 2013).

Another publication in the same forum, Soil Nailing with Flexible Structural Facing: Design and Experiences (Giacchetti, Grimod, & Cheer, 2013), describes a simplified and realistic design approach for the calculation of the flexible structural facing of soil nailing, developed by Officine Maccaferri. This approach describes the importance of selecting the appropriate membrane stiffness of the meshes used in flexible facings.

2.2 Commonly used Facing Design Approaches

Detailed facing design guides are available for soil nail facing design methods in the USA, UK, France, Germany, Japan, and Hong Kong. These design methods have several fundamental similarities such as as well as differences. Particularly, calculation methods of nailhead load take similar forms if not identical in the US, France based literature. Moreover, studies adopted in Japan, the US, Germany, and Hong Kong for the nailhead load calculations show a similar trend of tensile stress distribution along the nail. However, the Hong Kong publications suggest a different approach to this.

Moreover, the methods developed in the UK for designing isolated nailheads are similar to those in the Hong Kong guidelines. Mainly, the guidelines and development originating from these countries have been discussed under separate topics in this thesis although, almost all types of facing designs are practiced in those countries.

The majority of design methods have been mainly based on the results of the research studies presented in Section 2.1. Primarily in the UK and Japan, and Hong Kong, guidelines are available to design isolated nail heads in which the capacity is governed by the bearing capacity of the soil. The designs done in this manner can be used with flexible or soft-facing types. Whereas, other studies mainly describe nail heads that are integrated into the full-face shotcrete and the focus was mainly on flexural and punching shear strength of the nail head connection. Also, design guidelines for Grid Beams (Grillage structures/ Crib) have been originated in Hong Kong and Japan, although not many Japanese guidelines are publicly available in the English language.

All these different design approaches have been briefly discussed under this subsection and they will be addressed in detail from Chapter 4 to Chapter 7 under the design of different types and elements of facings.

2.2.1 British approach

The standard BS code of practice for reinforced soils BS8006: 1999 states that the facing shall be designed to withstand the soil pressure corresponding to the reactions at the connections. However, BS8006: 1999 does not provide direct guidelines for designing facings.

Whereas, HA 68/94 (The Highways Agency, 1994) Design Methods for the Reinforcement of Highway Slopes by Reinforced Soil and Soil Nailing Techniques presents a design approach for an isolated nail head named “waling plate”. Bearing force on the nail head is the maximum tension in the nail minus the frictional forces developed along the nail inside the active zone. This residual force is used to check the bearing capacity of the soil based on the friction angle of the soil (ϕ). The cohesion of soil is not considered in these calculations.

2.2.2 US-FHWA approach

US Federal Highway Design manual (Carlos A. Lazarte, Victor Elias, R. David Espinoza, & Paul J. Sabatini, 2003); (Carlos A. Lazarte, et al., 2015) provides guidelines for designing soil nail facings where the nail heads are connected to the full face facings. This manual provides comprehensive guidance for calculating nail head loads as well as for carrying out the structural design of the nail heads.

The method also considers the facing load as a contributing factor for the magnitude and distribution of the nail load. The nail load diagram is based on the factored nail head resistance (structural strength), factored pullout resistance (Active and Resistive), and the allowable tensile resistance of the nail (Figure 2-9).

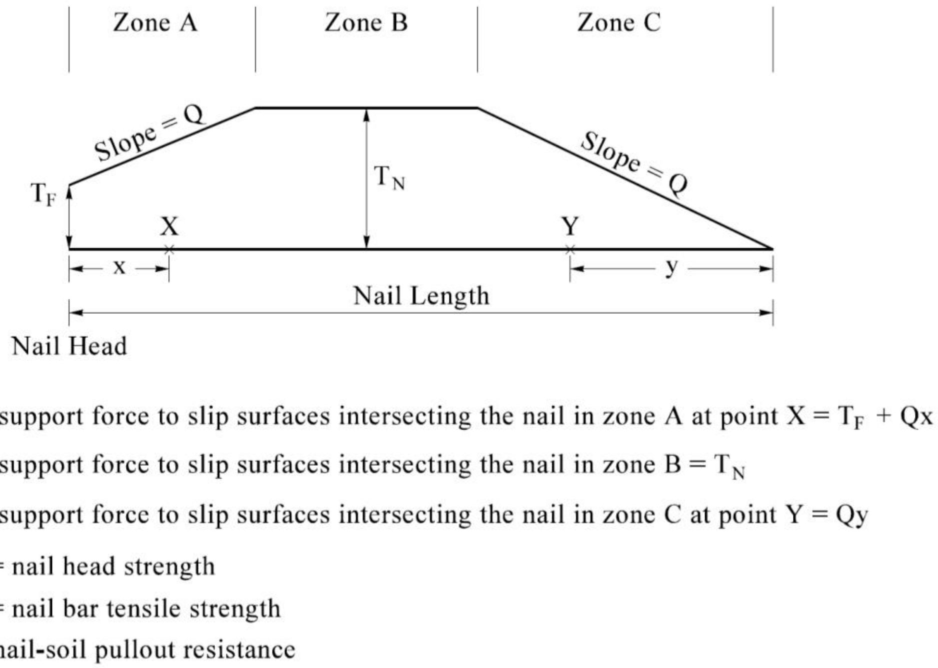


Figure 2-9. Allowable Nail Load Support Diagram (After FHWA)

As such, the strength of the nail head is a major contributing factor for calculating the tensile force of the nail, in this method. This guide also states that the distribution of earth pressure between the nail heads is non-uniform due to the arching effects developed. As such, the stresses are concentrated at the nail heads. The manual recommends adopting the nail head design load as the force attributable to 50% of the Coulomb active earth pressure (Byrne, Cotton, Porterfield, Wolshlag, & Ueblacker, 1998).

2.2.3 French approach

The design guides provided in the French National Research Project (1991), also describe designing full-face facings rather than isolated nail heads. The method requires the determination of nail tensile load at the facing (T_0 or T_F) as well as the soil pressure (P) acting on the facing as a result of the tensile load of the nail.

Similar to the US-FHWA manual, this French approach discusses the effect of soil arching between the nail heads, which results in concentrating the soil pressures towards the nail heads. Guidance for the structural design of the facing is also provided in this guide where bending, shearing, and punching of the facing at the nail head is taken into consideration (French National Project Clouterre, 1991). Most importantly, Clouterre 1991 prescribes design limit states and factors used in the structural design under the short term, medium-term, and long-term soil nail facings.

2.2.4 Japanese approach

This approach uses the nail head pressure for obtaining the distribution of tension force along the nails. Miki et al (1997) reported that the design axial nail load depends on the pullout resistance (loss) through the active zone, pullout resistance developed through the resistive zone, the allowable tensile load of the nail reinforcement, and the nailhead load. This conclusion is identical to that of FHWA on the pullout resistance of the soil nail illustrated in Figure 2-9 and also coincides with the FLAC analysis of GEO report 175 discussed in Section 2.9 and shown in

Figure 2-38. Thus, the conclusion of Miki et al (1997) states when the nail-head load is not considered, the pullout resistance through the active zone ($T_F + Qx$ in Figure 2-9) becomes much lower, resulting in the design pullout capacity of the nail to be much lower.

In addition, the Japan Highway Public Corporation (1998) introduces a slightly different approach to the US method for obtaining the tensile force at the soil nail head. This is discussed in Section 2.4 in detail.

2.2.5 Hong Kong approach

Hong Kong Geoguide 7 (Geotechnical Engineering Office, 2008) as well as several GEO reports provide detailed design guidance as well as typical structural details for designing facings.

The Geoguide 7 specifically prioritize isolated nailheads in providing detailed guides for designing and they do not address other facing designs in similar detail. Geoguide 7 refers to the GEO report No. 175 (2005) and presents design guides based on the results of its numerical modeling. Isolated nailhead design has been further discussed in Section 2.8 as well as in Chapter 7.

For the designing of facings other than isolated nailheads, Geoguide 7 does not provide specific guides. However, it recommends that the designers shall consider the facts such as erosion control, redistribution of soil nail forces between soil nails, preventing local failure between nail heads, ease of construction, time for vegetation growth, maintenance, and aesthetics.

In addition, Geoguide 7 directs the designers to other guidelines for facing designs which include CIRIA report c637 which has been discussed in Section 2.2.6.

2.2.6 The approach presented in CIRIA report C637

CIRIA report c637, Soil nailing – best practice guidance (Phear, et al., 2005) provides detailed design guidance and recommendations for nailhead as well as three types of head/ facings.

- Design of head plates for flexible or soft facings
- Design of soft (non-structural) facings
- Design of flexible (structural) facings
- Design of hard (structural) facings

For designing flexible structural facings, the CIRIA c637 proposes a method based on the local failure of soil between soil nails. The local soil mass is retained by the flexible mesh and the force applied to the mesh via the local failure of the slope will be transferred to the soil nails via bearing plates that fix the mesh to the nail. CIRIA c637 refers to the HA 68/94 method of two-wedge failure to assess the force induced in the mesh by soil.

For designing head plates, this guide describes a method similar to FHWA and French approach, considering the arching action between soil nails. Thus, the sizing of head plates to avoid bearing capacity failure is emphasized.

After obtaining this nailhead load, the nail head shall be sized to satisfy the bearing capacity and the resistance for punching through the facing. CIRIA guide recommends the above discussed HA 68/94 methods of upper and lower bound solutions for determining the bearing capacity of the nail heads. After obtaining the size of the plate, in the case of steel plates, the adequacy of thickness needs to be checked to avoid overstressing the head plate in bending (Phear, et al., 2005).

The guide states that the soft facings are primarily used to control erosion and to temporarily support the establishment of vegetation. Soft facing does not contribute to stabilize the slopes between the nails and does not ensure the group action of soil nails. Thus, nails act as individual components against destabilizing forces. Therefore, the guide recommends using the soft facings on slopes shallower than 30°.

For the design of hard (structural) facings, the CIRIA guide provides guidance as follows. For calculating the facing load, it recommends following the FHWA approach of using 50% of active earth pressure over the full height of the soil nail wall. Then the guide provides a detailed example for checking the proposed facing's structural adequacy in terms of flexural strength and punching shear capacity etc., in accordance with the relevant BS codes of practice (Phear, et al., 2005).

2.3 General Rules for Selecting the Facing Type

General rules or the prescriptive methods for soil nailing design are published in Hong Kong Geoguide 7 and in the pertinent GEO reports. CIRIA report c637 and FHWA soil nail reference manual 1998 have also presented some very important general criteria to follow before carrying out soil nail designs. The facts which are relevant for facing designs are discussed in this section.

Slope angle

Literature suggests that the slopes with no facing or having soft facings where the facing only acts to retain the vegetation layer and topsoil and to prevent surface erosion shall be limited to shallow slope angles not more than about 30° to the horizontal.

Design guides also suggest facings with isolated head plates with flexible structural mesh covering the slope should not be used for slopes steeper than about 60–70°. Also, a practical upper limit for the use of grillages on soil nailed slopes should be about 70°. Pedley and Pugh (1995), Johnson et al (2002), and Barley (1997) have investigated and reported on such slope mitigations.

As for hard structural facings, literature does not generally provide a slope angle limit and it usually goes up to 90°. Bruce and Jewell (1987), and Johnson and Card (1998) have reported on these slopes after collecting field data.

Additionally, recommendations for isolated nailhead designs for slopes steeper than 45° to the horizontal have been recommended in Hongkong Geoguide 7. Specially designed, effectively shaped isolated nailheads as discussed in Section 2.8.3 have been recommended in order to enhance slope face anchorage capacity for slopes not steeper than 45°.

The GEO report No. 175 has published a prescriptive method for designing isolated nailheads. In their method which is based on FLAC finite element models, they recommend using grillage structures instead of isolated nailheads where the slope angle is steeper than 65°.

Soil nail spacing

One other more important factor to consider before selecting the facing type is the soil nail spacing.

CIRIA c637 (Phear, et al., 2005) points out that the nail spacing affects not only the overall slope stability but the performance of the facing and therefore the spacing shall be a major concern when selecting the facing type. Similar to the slope angle criteria, CIRIA c637 presents the spacing requirement based on the flexibility of the facings.

Accordingly, CIRIA suggests using hard facings where nail spacings up to 6 m^2 per nail and using flexible structural facings where nail spacing is between 2-4 m^2. They further emphasize that if the spacing values exceed the specified for a given facing type, the soil nails behave as individual elements rather than a coherent soil nail system as they are supposed to be behaving. As such, they recommend using nail spacings in the range of 1-2 m as a general rule.

Meanwhile, the Hongkong Geoguide 7 (Geotechnical Engineering Office, 2008) also suggests that the nail spacing shall have an upper limit to ensure that the soil nailed slope is acting as an integral mass. They emphasize that the smaller the nail spacings, the better they are in preventing local instabilities in the soil nailed slope. Geoguide 7 also points out that nails with too little spacing may also be impractical and not cost-effective. As such, they state that in Hongkong mostly the nail spacings are kept within a range of 1-2 m. In addition, they also recommend increasing the integral action of soil nails by staggering the horizontal nail rows. However, it should be noted that it is not practical to use staggered nails with facing types such as Full face shotcrete and Grid beams.

FHWA 1998 guideline (Tei, Tayor, & Milligan, 1998) which mostly bases their studies on full-face shotcrete facings, consider the reduction of the structural capacity of the facing with the increased nail spacing in their designs. They also recommend having a nail spacing as large as possible for economic and practical reasons. They state that most designers practicing in the US keep the nail spacings within the 1.5 – 2 m range with 1.5 m being the most common for drilled nails. FHWA also suggests using the same value of vertical to horizontal spacing for achieving better structural integrity in the structural facings.

2.4 Nailhead Load – Tensile Force at the Nailhead

2.4.1 Introduction

To understand the amount of tensile force transferred to the nailhead, it is important to study the tensile force distribution along the soil nail behind the facing. From the global failure mechanism of soil nail slopes, it can be realized that the direction of the shear stress along the grout-soil interface is different on either side of the location where the slip surface cresses the soil nail.

However, because of this application of stress, the actual soil nail (rebar) inside the grout body is placed under a loading condition similar to when getting pulled from the two ends. As such, the tensile force along the soil nail has a distribution as shown in Figure 2-10.

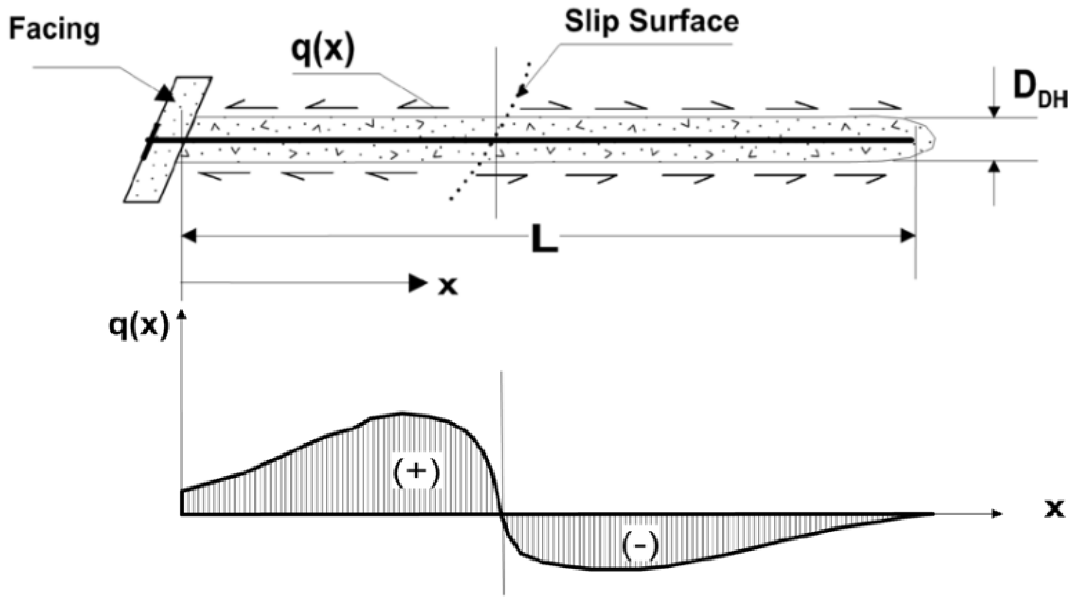


Figure 2-10. Shear stress distribution of grout-soil interface (Carlos A. Lazarte, et al., 2015)

It should also be noted the location of the maximum tensile force will be much closer to that of the failure surface of the slope. This distribution indicates how the nailhead load T_0 is a lower value than the maximum tensile force of the soil nail (Figure 2-11).

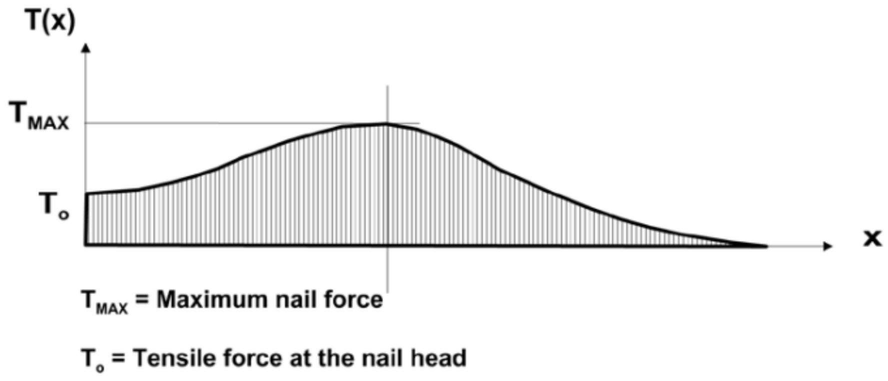


Figure 2-11. Distribution of tensile force along the soil nail (Carlos A. Lazarte, et al., 2015)

2.4.2 Methods proposed by FHWA

FHWA (1998) (Byrne, Cotton, Porterfield, Wolshlag, & Ueblacker, 1998) presents the results of a series of tests carried out on instrumented full-scale soil nail walls, with which they have proposed a method for obtaining the nailhead load. As seen in the chart presented in Figure 2-12, their approach introduces a parameter called the "normalized nailhead load" (T_0/ K_a γ_s H S_V S_H) which facilitates merging and comparing the test results from slopes with different soil properties, slope heights, and nail spacings.

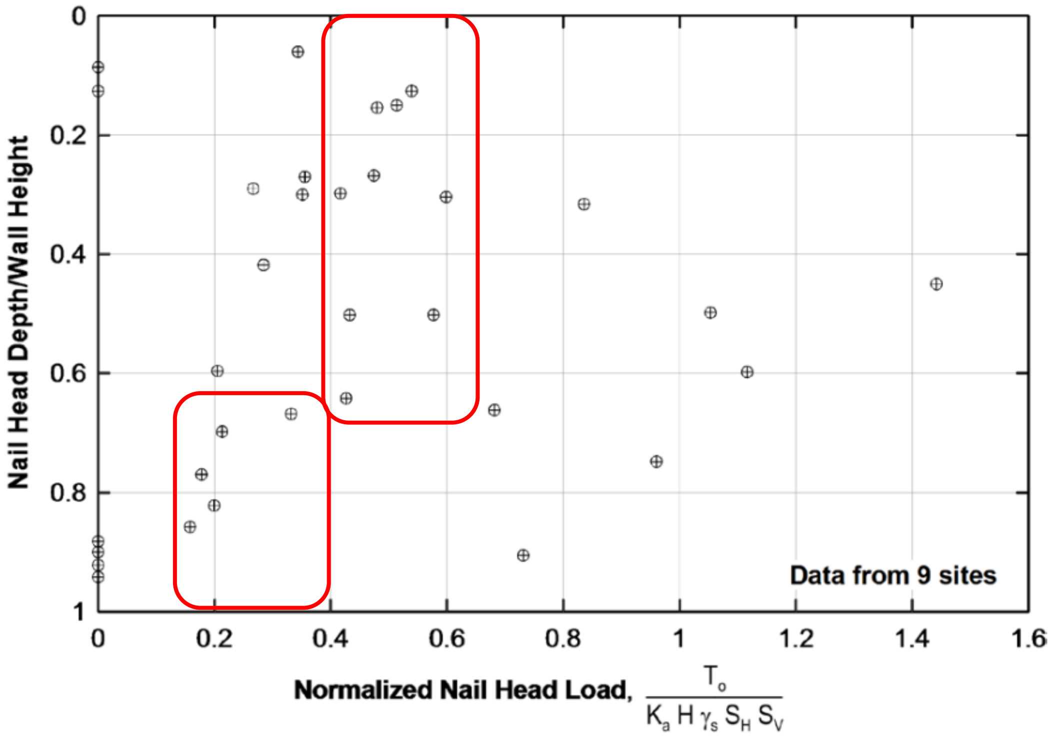


Figure 2-12. Normalized Nailhead Load (F_F) Distribution (Byrne, Cotton, Porterfield, Wolshlag, & Ueblacker, 1998)

These results indicate that the normalized nailhead forces at the upper two-thirds of the slope are about 50 % higher than that of the lower one-third. However, the average nailhead loads vary approximately from $T_o = 0.50\ K_a\ \gamma_s\ H\ S_V\ S_H$ to $T_o = 0.60\ K_a\ \gamma_s\ H\ S_V\ S_H$.

The above results have been used to develop the following empirical equation for estimating the nailhead service load:

$$T_0 \quad = F_F K_A \gamma H S_H S_V \tag{2.1}$$

The terms used in the expression for normalized nailhead load are:

T_0,	Nailhead load
F_F	The fraction of the active earth force on the tributary area of a soil nail transferred to the nailhead, (Normalized nailhead load)
γ_s	Unit weight of soil
H	Slope height
S_H, S_V	Nail spacing
K_A	Active earth pressure coefficient obtained by Coulomb method

$$K_A = \frac{\sin^2(\theta + \phi')}{\sin^2\theta \sin\ \theta - \delta\ \left[1 + \sqrt{\dfrac{\sin(\phi' + \delta) + \sin(\phi' - \beta)}{\sin\ \theta - \delta\ + \sin\ \theta + \beta}}\right]^2} \qquad (2.2)$$

Where:

θ	Angle of the slope face from outside
δ	Interface friction between slope material and nailhead material
β	Back slope angle
ϕ'	Friction angle of the slope material

Standard Specifications for Highways and Bridges (AASHTO, 2002), proposes that wall friction angle, δ can be obtained from the values presented in Table 2-1.

Table 2-1. Ultimate Friction Angles for Dissimilar Materials (AASHTO, 2002)

Interface Materials	Friction Factor f = tan δ (dim)	Friction Angle, δ (degrees)
Mass concrete or masonry on the following foundation materials:		
—Clean sound rock	0.70	35
—Clean gravel, gravel-sand mixtures, coarse sand	0.55 to 0.60	29 to31
—Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 to 0.55	24 to 29
—Clean fine sand, silty or clayey fine to medium sand	0.35 to 0.45	19 to 24
—Fine sandy silt, nonplastic silt	0.30 to 0.35	17 to 19
—Very stiff and hard residual or preconsolidated clay	0.40 to 0.50	22 to 26
—Medium stiff and stiff clay and silty clay	0.30 to 0.35	17 to 19
Steel sheet piles against the following soils:		
—Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	0.40	22
—Clean sand, silty sand-gravel mixtures, single size hard rock fill	0.30	17
—Silty sand, gravel or sand mixed with silt or clay	0.25	14
—Fine sandy silt, nonplastic silt	0.20	11
Formed concrete or concrete sheet piling against the following soils:		
—Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	0.40 to 0.50	22 to 26
—Clean sand, silty sand-gravel mixtures, single size hard rock fill	0.30 to 0.40	17 to 22
—Silty sand, gravel or sand mixed with silt or clay	0.30	17
—Fine sandy silt, nonplastic silt	0.25	14
Various structural materials:		
—Masonry on masonry, igneous, and metamorphic rocks		
• Dressed soft rock on dressed soft rock	0.70	35
• Dressed hard rock on dressed soft rock	0.65	33
• Dressed hard rock on dressed hard rock	0.55	29
—Masonry on wood (cross grain)	0.50	26
—Steel on steel at sheet pile interlocks	0.30	17

Additionally, FHWA (2003) has presented a method for obtaining maximum soil nail force ($T_{max\text{-}s}$) which has also been developed based on the experiments on instrumented soil nail walls discussed in this section. The experimental results show that the normalized average maximum nail force is around 0.75 in the upper two-thirds of the wall and a value close to 50% of that in the lower one-third. Moreover, FHWA (2003) also provides design charts for obtaining normalized maximum nail forces for given back slope and face batter angles which can be used in preliminary designs that have not been taken into this study because of its limitations. However, Briaud and Lim (1997) have suggested the use of the expression given in Equation (2.3) for obtaining the maximum nail force in the upper two-thirds of the nail wall.

$$T_{max\text{-}s} = 0.65\, K_A \gamma H S_H S_V \quad (2.3)$$

2.4.3 Method proposed by the Japan Highway Public Corporation

The Japan Highway Public Corporation (1998) introduces a force reduction coefficient, $\mu = T_0 / T_{max}$, for obtaining the tensile force at the soil nail head where T_0 is the nailhead load and T_{max} is the maximum soil nail load.

Similar to the FHWA method discussed in 2.4.2, the force reduction coefficient, $\mu = T_0 / T_{max}$ presented in Japan Highway Public Corporation (1998), has been obtained based on the measurements taken from full-scale soil nailed slopes and model tests. Contrary to the FHWA (1998) approach, Japan Highway Corporation (1998) uses a normalized value of $fa = L^2/BS$ to plot the T_0 / T_{max} value against, as shown in Figure 2-13, where B is the width of nailhead, S is the nail spacing and L is the length of reinforcement. Although this indicates a linear trend than what's reported in FHWA (1998), it is to be noted that the maximum soil nail force could be a function of the length of reinforcement, L obtained from the slope stability analysis which is a variable of slope height, angle, and material properties. Additionally, as discussed in this paper, the nailhead load T_0 is also a function of nail spacing and the nailhead size as per the findings of this paper. Therefore, using Figure 2-13 for designing nailheads could be considered an iterative process.

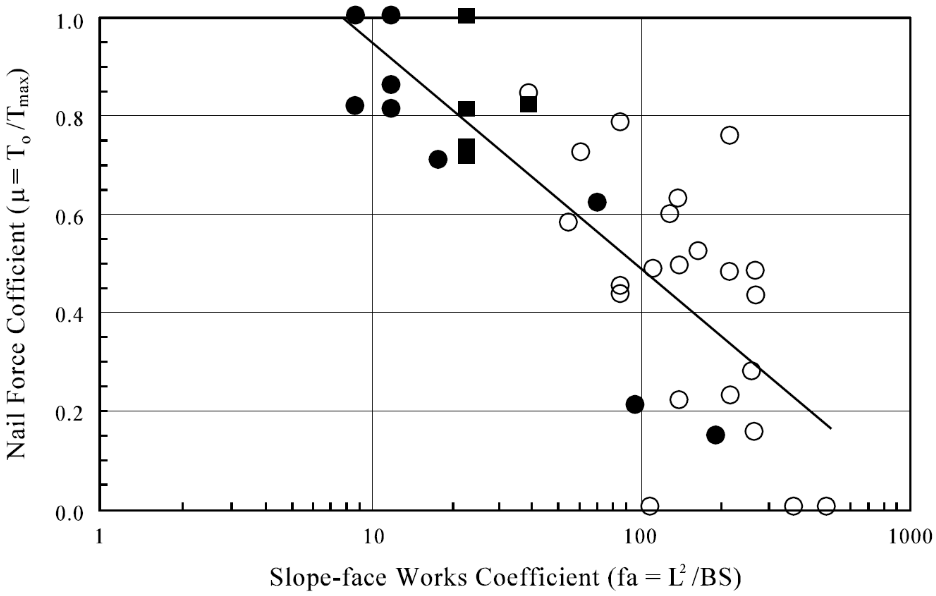


Figure 2-13. Nail Force Coefficient μ (after Japan Highway Public Corporation, 1998)

2.4.4 Suggestions in GEO Reports No. 175

One of the most consistent findings with the nailhead load of FHWA (1998) is the results of the finite element analysis with FLAC presented in GEO Report No. 175 (Shiu & Chang, 2004). Both the analyses carried out using different isolated nailhead sizes of 400 mm and 800 mm show values of T_0 / T_{max} varying around 0.55 to 0.7

along the slope height where T_0 is the nailhead load and T_{max} is the maximum soil nail load. Similar to the FHWA (1998), the top two-thirds of the nails show a higher ratio while the lower third show a lower ratio of T_0 / T_{max} as shown in Figure 2-14.

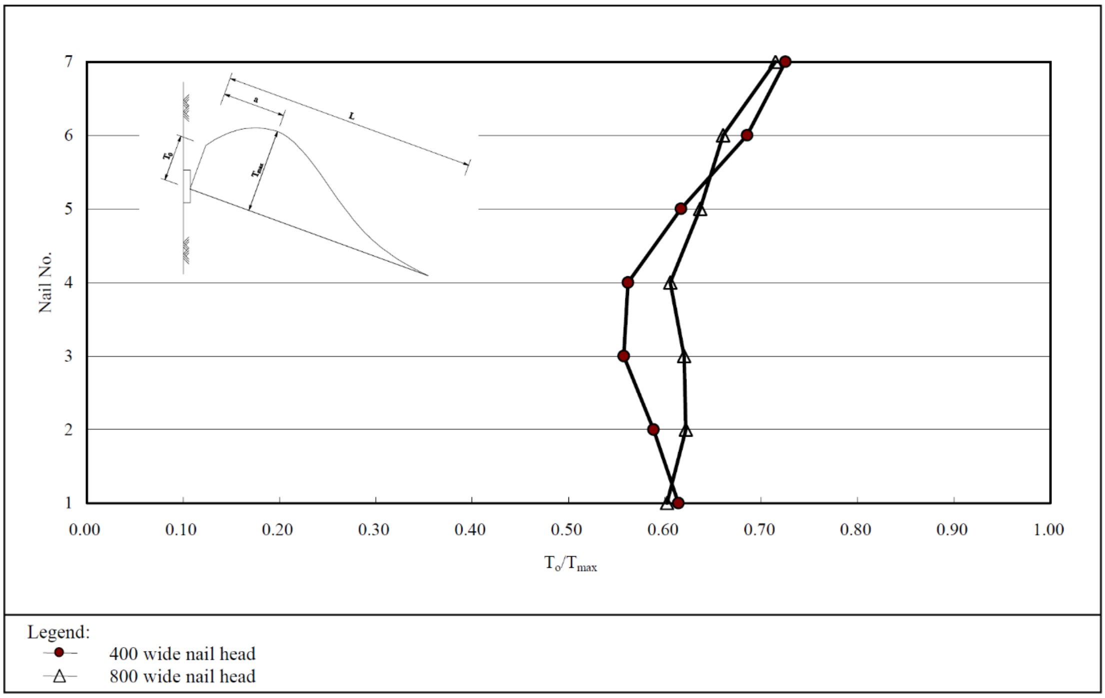


Figure 2-14. T_0/T_{max} distribution of isolated nailheads via finite element analysis (Shiu & Chang, 2004)

However, for the design of isolated nailheads, GEO report No. 175 recommends taking the maximum soil nail load as the soil nailhead load ($T_0 = T_{max}$). Additionally, the report defines the nailhead load which is based on the tensile strength of the soil nails (f_y) as shown in Equation (2.4).

$$T_0 = f_m A_s \ (kN) \tag{2.4}$$

$$f_m = 0.55 f_y \leq 0.23 \ (kN/mm^2)$$

Where:

A_s Cross-section area of soil nail reinforcement

f_y Yield strength of soil nail reinforcement

2.4.5 Method proposed by French National Project Clouterre

CIRIA c637 suggests an alternative but conceptual method for calculating the nail head load to maximum soil nail load ratio for shotcrete facings based on Clouterre. 1991 (French National Project Clouterre, 1991). Expressions used in the method are given in equation (2.5).

However, to use this method to determine the nailhead load, the maximum tensile load generated in soil nails must be known. As such the proposed method also presents an expression to obtain the maximum tensile load generated in a particular soil nail (Equation (2.6)). However, this method requires the effective soil nail length in the resistive zone (L_e), which has to be derived using a multi wedge or a slope stability analysis for the soil nailed slope.

$$\begin{aligned} \frac{T_o}{T_{\max}} &= 0.5 + \frac{s - 0.5}{5.0} \quad \text{when} \quad 1\text{m} \le s \le 3\text{m} \\ \frac{T_o}{T_{\max}} &= 0.6 \quad \text{when} \quad s \le 1m \\ \frac{T_o}{T_{\max}} &= 1.0 \quad \text{when} \quad s \ge 3m \end{aligned} \tag{2.5}$$

$$T_{\max} = \min\left[\frac{q_s \pi D L_e}{\gamma_d}, \frac{f_y}{\gamma_s}\right] \tag{2.6}$$

Where;

T_{max}	Maximum soil nail force
T_0	Nailhead load
s	Maximum of the vertical and horizontal nail spacing
q_s	Skin friction in the resistive zone
D	Diameter of borehole
L_e	Soil nail length in the resistive zone
f_y	Tensile strength of soil nail
γ_d	Partial safety factor for skin friction
γ_s	Partial safety factor for the tensile strength of the nail

The later released US Federal Highway manual, 2003 (Carlos A. Lazarte, Victor Elias, R. David Espinoza, & Paul J. Sabatini, 2003) has simplified the method proposed by Clouterre. 1991 to a single expression (Equation (2.7)). These findings have also been developed based on the findings of instrumented test results presented in FHWA 2003, discussed in Section 2.4.2.

$$T_o = T_{max}[0.6 + 0.2 \ S_v - 1 \ ] \quad (2.7)$$

The terms used in the expression are:

T_o	Nailhead load
T_{max}	Maximum soil nail force
S_V	Vertical soil nail spacing

The maximum soil nail force (T_{max}) used in these methods could be obtained either from Clouterre 1991's Equation (2.6), a detailed stability analysis of the soil nailed slope, or the method presented in the FHWA soil nail reference manual (Byrne, Cotton, Porterfield, Wolshlag, & Ueblacker, 1998) to calculate the maximum nailhead force presented in Section 2.4.2.

2.4.6 Safety factors used to be used with nailhead load or resistance

The safety factors recommended in the design methods published by, FHWA (1998), CIRIA c637 (2005), and Clauterre (1991) for the design of soil nail facings have been summarized in this section.

FHWA (1998)

FHWA's (1998) method uses several factors to calculate the allowable flexural/ shear resistance, T_0.

$$T_0 \le \frac{\phi_F}{\gamma} \times R_F \quad (2.8)$$

Where;

R_F	is the calculated nailhead resistance values	
ϕ_F	$= 0.9$	; resistance factor for bending/ flexure, shear in the facing
γ	$= 1.35$	; load factor for nailhead load variation

These ultimate resistance values will be directly compared with the nailhead load obtained using Equations (2.1) and (2.2).

CIRIA c637 (2005)

Whereas, CIRIA c637 (2005) uses factors based on the BS codes to modify the nailhead load, T_0 (calculated using the FHWA (1998) approach as given in 2.4.2) to obtain the ultimate nailhead load, $T_{0,ULT}$:

$$T_{0,ULT} = T_0 \times \gamma \times \gamma_p \tag{2.9}$$

Where; $\gamma = 1.35$; load factor for nailhead load variation (FHWA proposed factor)

$\gamma_p = 1.5$; load factor for Earth pressure

It is to be noted that the nailhead resistance values against flexure and shear calculated using CIRIA c637 are the ultimate values as partial factors have been used in the expressions presented by them discussed in Section 2.5.3.
This expression could also be written in the form:

$$T_0 \leq \frac{R_{F,ULT}}{1.35 \times 1.5} \tag{2.10}$$

Where; $R_{F,ULT}$ = ultimate nailhead resistance

T_0 = allowable nailhead load

Clauterre (1991)

Similarly, Clauterre (1991) also suggest using a safety factor γ for nailhead loads, calculated using the method proposed by them when checking for nailhead resistance against flexure:

$$T_{0,ULT} = \gamma \times T_0 \tag{2.11}$$

Where; T_0 = nailhead load calculated via Clouterre method

$T_{0,ULT}$ = ultimate nailhead load

γ = 1.35

Additionally, Clouterre recommends using T_{max} as the nailhead load instead of T_0 for designing or checking for nailhead resistance against punching.

2.5 Full-Face Shotcrete Facings

2.5.1 Introduction

Shotcrete facing is essentially a reinforced concrete wall that connects the nail heads and is constructed on the surface of the slope. The facing has an embedded reinforcement grid system that connects the nails at their heads to structurally integrate with the system of soil nails. Additionally, a flexible GI mesh is also cast into the facing to cover the space in between the reinforcement grid and provide the additional tensile capacity required.

Shotcrete, Sprayed Concrete, or Gunite is the technique that uses high-pressure equipment to firmly project concrete into non-horizontal surfaces so that the mixture self-compacts and adheres without additional compaction effort. Several facing types such as Grid Beams and Isolated heads can be constructed using Shotcreting. However, the content discussed in this section only applies to the type of facing that covers the full face of the slope, although it is often referred to as Shotcrete.

The loading mechanism is such that, the nail head loads resulting from the soil movements related to the overall stability are distributed all across the surface of the spacing starting from the nail head plate inside the facing. The two most critical failure mechanisms are the flexure and the punching shear in the facing that result from the nailhead load. The loads from potential local failures are not addressed separately in the design because of the non-flexible nature of the shotcrete. The design approach consists of the design for the nail head/ plates as well as the shotcrete and steel reinforcement system.

A typical detail of a shotcrete design along with soil nails used in Sri Lanka is shown in Figure 2-15.

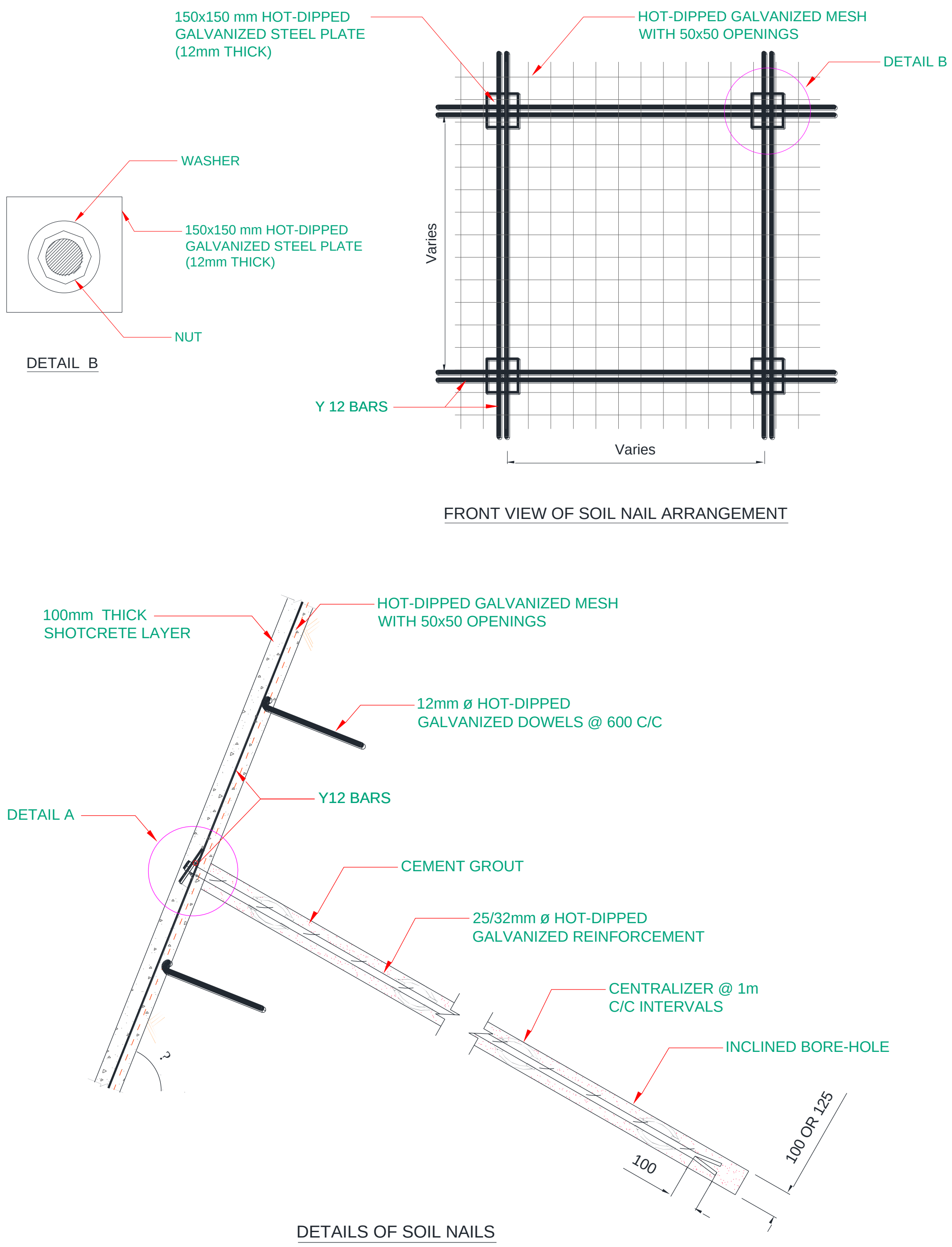


Figure 2-15. Typical details of shotcrete facing used in Sri Lanka (after NBRO design documents)

2.5.2 Method proposed by FHWA

US Federal Highway Design manuals (Carlos A. Lazarte, Victor Elias, R. David Espinoza, & Paul J. Sabatini, 2003); (Carlos A. Lazarte, et al., 2015) provide guidelines for designing soil nail facings where the nail heads are connected to the full face facings.

FHWA manuals describe 4 failure mechanisms that can be identified in full-face shotcrete facings out of which failure in flexure and punching shear are predominant. Flexure and the shearing of the bearing plates are the other 2 failure modes.

It was stated that the bearing plates of typical dimensions are used, the plate shear never takes place because of their ductile nature, according to the tests carried out. The mild steel plates used in these tests have continuously deformed, redistributing the pressures between the facing and the plate. The design recommendation of a minimum plate width of 200 mm and a minimum plate thickness of 19mm can be used in all shotcrete facings. However, where plates of dimensions less than the minimum dimensions are used, it is recommended to carry out calculation checks to demonstrate the adequacy of flexure and shear of the bearing plates.

Flexural resistance of shotcrete facing in accordance with FHWA (1998)

The flexural resistance of full shotcrete facing, R_{FF} is given in FHWA (1998) as the minimum of:

$$R_{FF} = C_F \times \; m_{vm} + m_{vn} \; \times \left(\frac{8S_H}{S_V}\right) \qquad (2.12)$$

$$R_{FF} = C_F \times \; m_{hm} + m_{hn} \; \times \left(\frac{8S_V}{S_H}\right)$$

(Byrne, Cotton, Porterfield, Wolshlag, & Ueblacker, 1998)

Where;

m_{vm}	moment resistance in vertical direction at midspan
m_{vn}	moment resistance in vertical direction at nailhead
m_{hm}	moment resistance in horizontal direction at midspan
m_{hn}	moment resistance in horizontal direction at nailhead

FHWA manuals use a generalized section for the estimation of moment resistance. This represents an equivalent beam sustaining pure bending in the context of reinforced concrete design. When the force equilibrium (c = t) and the moment equilibrium ($m_V = t \times z$) are considered, the expression for calculating the maximum bending moment can be developed. The mechanism adopted for obtaining the expressions is shown in Figure 2-16.

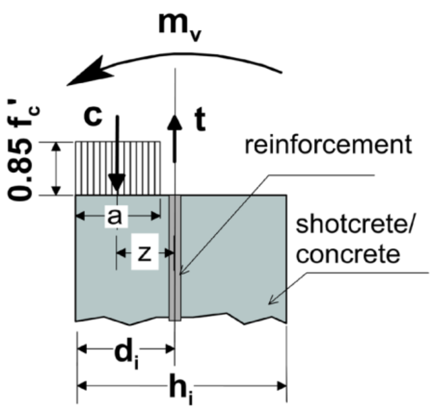


Figure 2-16. Bending mechanism of shotcrete facing (FHWA 1998)

Accordingly:

$$m = \frac{A_s F_y}{b}\left(d - \frac{A_s F_y}{1.7 f_c' b}\right) \tag{2.13}$$

Where;

A_s	area of tension reinforcement
F_y	yield strength of tension reinforcement
b	breadth of section considered
d	effective depth of section considered
f_c'	compressive strength of shotcrete/ concrete

Further, C_F is a factor that accounts for the effect of non-uniform soil pressure acting behind the facing. It can be observed that the soil pressure is considered to be concentrating more towards the location of the nailhead as shown in Figure 2-17, as the facing thickness reduces (Refer to Table 2-2 for recommended values of C_F which varies with facing thickness).

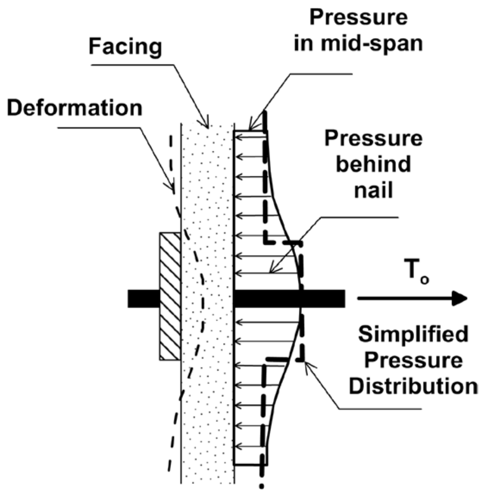


Figure 2-17. Varying soil pressure acting behind the facing (FHWA 1998)

Table 2-2. Facing Pressure Factors (FHWA,1998)

	Temporary Facings		Permanent Facings	
Nominal Facing Thickness (mm)	Flexure Pressure Factor C_F	Shear Pressure Factor C_S	Flexure Pressure Factor C_F	Shear Pressure Factor 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

It can be understood that the FHWA formulae have been generated assuming the facing acts as a two-way continuous slab with equal spans where the earth pressure is acting as a distributed load. Resultant moment resistance or the nailhead load shall be equal to the force excreted on the facing as earth pressure A sample derivation is carried out in Section 5.1.1 considering the vertical moment resistance.

Punching resistance of shotcrete facing in accordance with FHWA (1998)

Based on a system with a bearing plate connection as shown in Figure 2-18, FHWA (1998) has developed the following equation (2.14) to determine the internal punching shear strength, V_N of a shotcrete facing:

$$V_N = 0.33\sqrt{f'c}\ \pi\ \left(D'_C\right)\ h_C \qquad (2.14)$$

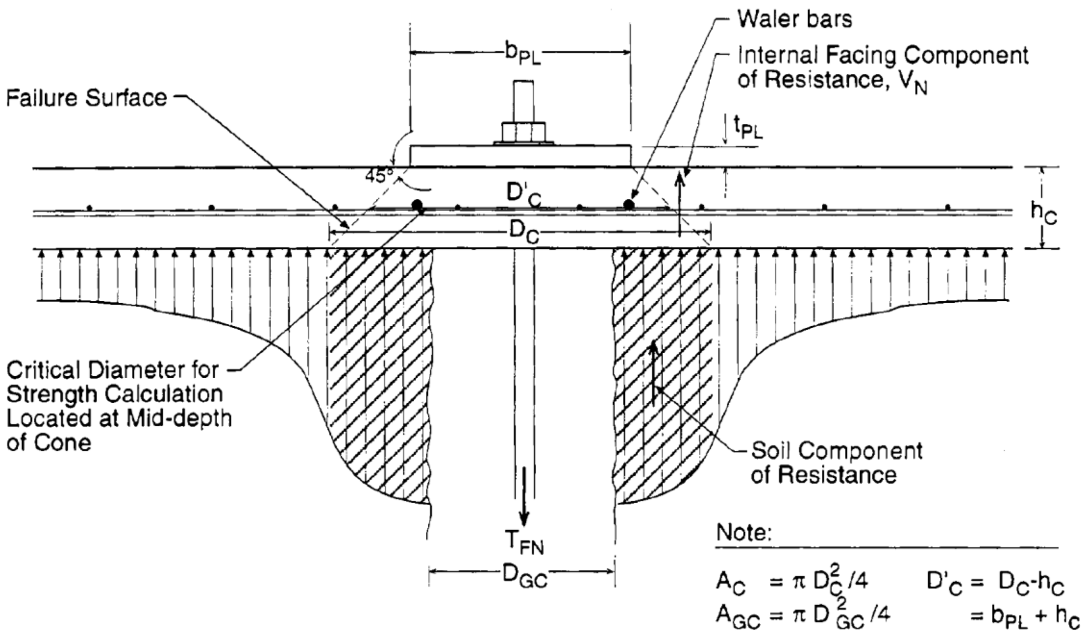


Figure 2-18. Punching Shear Mechanism of a Nailhead, (Byrne, Cotton, Porterfield, Wolshlag, & Ueblacker, 1998)

D'_C is defined as the effective shear cone-diameter which is set by considering the failure surface that initiates at the periphery of the bearing plate and propagates at an angle of 45°. Thus, D'_C is expressed as:

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

However, equation (2.14) can be considered conservative because it does not consider the contribution from the reaction of soil underneath. The following equation has been obtained by considering the force equilibrium to modify the V_N and calculate the nail head resistance against punching by including the soil reaction contribution:

$$R_{FP} \text{ or } T_{FN} = V_N \left(\frac{1}{1 - C_S\ A_C - A_{GC}\ /\ S_V S_H - A_{GC}} \right) \quad (2.15)$$

The terms used in (2.14) and (2.15) have been depicted in Figure 2-18.

It must be noted that both equations (2.14) and (2.15) have been developed via the results of the back analysis of case histories, full-scale testing, and finite element analysis. The values obtained for this shear pressure factor, C_S in the above manner have been presented in Table 2-2.

2.5.3 Methods presented in CIRIA c637 (2005)

For the design of hard (structural) facings, the CIRIA guide provides guidance as follows. For calculating the facing load, it recommends following the FHWA approach of using 50% of active earth pressure over the full height of the soil nail wall. Then the guide provides a detailed example for checking the proposed facing's structural adequacy in terms of flexural strength and punching shear capacity etc., in accordance with the relevant BS codes of practice (Phear, et al., 2005).

Flexural resistance of shotcrete facing using the CIRIA c637 (2005)

The FHWA (1998) approach presented above does not follow the principles of BS or Eurocode standards which are mostly used in Sri Lanka for structural purposes. However, CIRIA report c637 (2005) suggests using the following method which is more compatible with the Eurocode and BS8110 Part 1. Flexural resistance of the facing, R_{FF}:

$$R_{FF} = C_F \times n \times S_H \times S_V \qquad \text{Where;}$$

Load per unit area of shotcrete, n

$$= \min \begin{bmatrix} \dfrac{m_{vn}}{\beta_{sx} \times l_x^2} & or & \dfrac{m_{vn}}{\beta_{sy} \times l_x^2} \\ \dfrac{m_{hm}}{\beta_{sx} \times l_x^2} & or & \dfrac{m_{hm}}{\beta_{sy} \times l_x^2} \end{bmatrix} \tag{2.16}$$

β_{sx} and β_{sy} can be obtained from Table 3.14 of BS 8110:1997: Part 1 considering the facing as a continuous panel. l_x would be the smaller of the horizontal nail spacing and the vertical nail spacing.

The other abbreviation used;

m_{vm}	moment resistance in vertical direction at midspan
m_{vn}	moment resistance in vertical direction at nailhead
m_{hm}	moment resistance in horizontal direction at midspan
m_{hn}	moment resistance in horizontal direction at nailhead

The resistance moment of the shotcrete section can be calculated as per the standard code of practice BS 5400 part 4. Accordingly, clause 5.3.2.3 of BS 5400 part 4 can be used to determine the ultimate moment resistance as follows:

$$m = (0.87f_y)A_s z \qquad (2.17)$$

Where;

$$z = \left(1 - \frac{1.1f_y A_s}{f_{cu} bd}\right) d \qquad z \leq 0.95d \qquad (2.18)$$

m	ultimate moment resistance in the considered direction
A_s	area of tension reinforcement in the considered direction
f_y	yield strength of tension reinforcement
b	breadth of section considered
d	effective depth of section considered
f_{cu}	compressive strength of shotcrete/ concrete

Additionally, the following checks on reinforcement shall also be carried out as per BS 5400 part 4:

- Reinforcement ratio: 4%bh ≥ ρ ≥ 0.15%bd,

Where;

b	Breadth of the panel
h	Panel thickness
d	Depth to tension reinforcement

- Nominal cover to the reinforcement shall be determined as per Table 13, BS 5400. Grid beam facings when used in Sri Lankan conditions, fall into the “Severe” category in the above table because of the tropical climate with the recurring rainy and dry seasons.

The R_{FF} value calculated above is the ultimate value and thus shall be compared with the ultimate value of nailhead load as discussed in Section 2.4.6.

Punching shear resistance of shotcrete facing using the CIRIA c637 (2005)

Based on BS 5400: Part 4:1990, CIRIA report c637 presents the following expression to obtain the nailhead resistance against punching shear of shotcrete facings:

$$R_{\mathrm{FP}} \quad = [4 \times \xi_s v_c \times \ L_{HB} + h \ \ \times d]/\ 1000 \tag{2.19}$$

Where, (f_{cu}) is concrete strength, (h) is facing thickness, (L_{BP}) is the bearing plate length, and (d) is the effective depth of the section.

and the ultimate shear strength of concrete, v_c;

$$v_c \quad = \frac{0.27}{\gamma_{\mathrm{m}}}\left(\frac{100A_{\mathrm{s}}}{b_{\mathrm{w}}d}\right)^{1/3} \ f_{\mathrm{cu}}\ ^{1/3} \tag{2.20}$$

and the depth factor, ξ_s;

$$\xi_s \quad = \ 500/d\ ^{1/4} \text{ or } 0.70 \qquad \text{; Whichever is greater} \tag{2.21}$$

It should be noted that this expression for the punching shear has been obtained by considering the shear resistance contribution of concrete and the longitudinal reinforcement only. The contribution from the earth pressure behind the facing has not been taken into consideration.

The ultimate resistance has been derived pursuant to Clause 5.4.4.1 of BS 5400: Part 4:1990 where the maximum shear stress is given as $\xi_s v_c$ for slabs with no shear reinforcement. Further, it can be seen that the shear surface considered is a truncated pyramid of base length L_{HB} + 2h, to obtain the shear resistance at the nailhead.

2.5.4 Method proposed in Clouterre (1991)

The design guides provided in the French National Research Project (1991), also describe designing full-face facings rather than isolated nail heads. The method requires the determination of nail tensile load at the facing (T_0 or T_F) as well as the soil pressure (P) acting on the facing as a result of the tensile load of the nail.

Clouterre discusses how the facing is utilized to maintain the equilibrium of soil pressure and the soil nail tension at the nailhead to provide lateral confinement for the local soil. This approach is very much similar to the fundamentals of the facing design of the US-FHWA manual. This is further confirmed as the French approach also states that the soil pressure behind the facing between the nails is not uniform and depending on the rigidity of the facing, the pressure concentrates locally around the nailheads as a result of the arching effects to develop between the nails.

However, unlike the FHWA, Clouterre 1991 recommends considering the soil pressure behind the facing as a uniformly distributed load and states that it is conservative. They present a calculation model for determining the structural resistance of the facing similar to a slab loaded uniformly and supported by concentrated loads at the nailheads.

Guidance for the structural design of the facing is also provided in this manual where bending, shearing, and punching of the facing at the nail head is taken into consideration. Most importantly, Clouterre 1991 prescribes design limit states and factors used in the structural design under the short term, medium-term, and long term soil nail facings. They recommend that the facing should be designed at ultimate limit states and the nailhead load T_0 and the soil pressures behind the facing are therefore considered as permanent and external forces on the facing.

Designing for flexure (Clouterre 1991)

Nailhead forces for the design are recommended to be obtained from the method discussed in 0. The design shall be carried out based on the design service life of the facing. Given the facing is built with continuity with recommended overlapping lengths following two categories are defined:

1. Short term facing (service life less than 18 months)
   Facing is designed as a continuous two-way spanning slab and the calculation shall be done for Ultimate Limit State. The partial factors recommended are;
   Permanent loads (Nailhead load) γ_g= 1.35

   Concrete, γ_{mb} = 1.5

   Steel, γ_{ms} = 1.15
2. Medium-term (service life more than 18 months and less than 30 years) and Long term (service life more than 30 years): Consider the fragile nature of facing around zones of construction joints in the design.
   Calculation shall be done for both Serviceability and Ultimate Limite State. The calculation for Serviceability Limit is similar to the ULS design, except the cracking considered in SLS and the permanent loads are not factored.

Apart from these essential guides, the Clouterre 1991 does not provide detailed guidelines on structural designs. However, for structural calculations, it is recommended to use the BAEL 83 (Béton armé aux états limites/ Reinforced concrete at limit states - France) code which was the French code of practice regarding the use of reinforced concrete in construction.

Designing for punching shear (Clouterre 1991)

To account for the concentrated forces around the nailheads that affect the punching shear failure, Clauterre recommends using the maximum nailhead load, T_{max} value in place of the nailhead load T_{0}.

For the punching as well, they recommend using BAEL 83 (Béton armé aux états limites/ Reinforced concrete at limit states - France) for structural calculations. Apart from that, no other details of the structural calculation for punching are offered in this guide.

2.6 Grid Beam Facings

2.6.1 Introduction

Grid Beams are is typically a system of reinforced concrete beams constructed across the surface of the slope. The beam system connects the nail heads at their heads to integrate with the system of soil nails. Typically, in most cases, a flexible high tensile steel mesh is also used on top of the beam system to stabilize the surface soil mass in between the grid system. In addition to the net, a vegetation cover is also nurtured in the slope surface in between the grid system, to protect the slope from erosion failures.

The loading mechanism is such that, the nail head loads resulting from the soil movements related to the overall stability are predominantly carried by the nail head plate inside the beam and then transferred through the grid beam system to the slope face. The two most critical failure mechanisms of the beam are the flexure and the shear in the beam system. Whereas, the loads from potential local failures are carried by the flexible steel mesh, where the potential failure modes are the punching shear of the mesh. Thus, the design approach must consist of the design for the nail head/ plates, and the design for the steel mesh which will be discussed in Chapter 6.

A typical detail of a grid beam design along with soil nails used in Sri Lanka for slope protection is presented in Figure 2-19. The use of reinforced concrete/ shotcrete with steel plates and flexible mesh can be observed in these drawings.

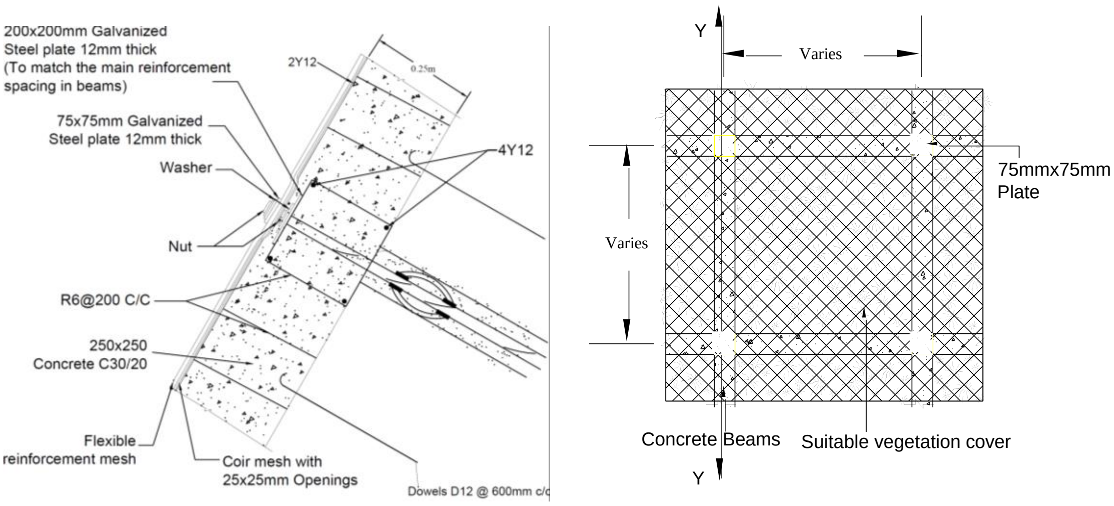


Figure 2-19. Details of a typical Grid Beam system used in Sri Lanka

Although various design guides are available for Shotcrete facings, very little discussion on Grid Beam facing systems are available on common design guides. It is understood that in countries like Japan, Grid Beam systems are frequently used with soil nailing as ground anchoring works in slope mitigation. However, any of the Japanese guides to carryout Grid Beam designs were unavailable to be found during the period of this study, maybe because most of them are published in the Japanese language.

However, the Hongkong Geotechnical Engineering Office and The Hong Kong Institution of Engineers have published a sample design calculation in one of their design guides, " Use of Soil Nails to Upgrade Loose Fill Slopes" (Geotechnical Engineering Office, Hong Kong Institution of Engineers, 2013). Although this guide is meant for designing Grillage Structures for loose fills where the liquefaction of the slope is considered critical, the fundamentals could be adopted for regular facing design as well.

2.6.2 Method proposed by GEO and HKIE for loose fills

The "Use of Soil Nails to Upgrade Loose Fill Slopes" report published by Hongkong Geotechnical Engineering Office (GEO) and The Hong Kong Institution of Engineers (HKIE) presents design examples with reference to their previous publications "Design of Soil Nails for Upgrading Loose Fill Slopes" (2011) and "Hongkong Geoguide 7" (2008).

The calculations in this report present methods for calculating the stabilizing pressure at the slope surface and the structural design of the "Grillage structure" (Grid Beam) facing they adopt.

General Rules

It is recommended to use the grid beam in such a way as to achieve a minimum slope coverage of 50% to minimize the risk of liquefaction of the loose fill. Based on this limitation and the nail spacing, the width of the beam could be determined. Further, it is recommended to use a minimum embedment of 300mm.

However, it is understood that these guides are useful for facings on the filled slopes and not for the general slopes with relatively stiffer material with lesser potential of liquefaction.

Limit Equilibrium Method to Identify Facing Load

In many other studies, the nailhead load obtained considering the global stability analysis of the slope was used for designing the facing structure. However, GEO & HKIE (2013) use the limit equilibrium method without using soil nails to determine the load exerted to the facing (in place of nailhead load) as shown in Figure 2-20. Using a series of point loads distributed to represent a triangular earth pressure on the slope, the Grid and Radius analysis is carried out in a trial and error procedure to determine the required pressure to stabilize the filled slope's surface soil.

Structural Design of Grillage Structures

The grid beam is considered as a simply supported beam with a uniformly distributed load in the GEO & HKIE (2013) report for the structural design. Moreover, the point load at the bottom of the slope obtained from the Limit equilibrium analysis is used to calculate the uniformly distributed load under the grid beam. Even though it could be considered using the previously obtained triangular load distribution itself on the beam to be more accurate, analysis carried out in the GEO & HKIE (2013) report using both the above assumptions (simply supported with triangular force distribution and continuous beam with a uniform load distribution) has shown that the method which assumes a simply supported beam with uniformly distributed loading arrangement generates larger bending moments than the other. As such, they recommended that this approach is conservatively applicable for most general cases.

However, the approach practiced in the FHWA soil nail guide and CIRIA c637 consider the shotcrete facing as a continuous panel as opposed to the simply supported approach used here.

Structural analysis has been carried out in this guide based on the above assumptions using the "Code of Practice of Structural use of Concrete 2013" by the Buildings Department, Hong Kong. Similar to the methods proposed for Shotcrete by CIRIA c637 and FHWA soil nail guide, checks for bending at midspan as well as shear checks at the nailhead have been carried out. Additionally, the method of checking the adequacy of the facing under tension and bending moment at the same time using a bending moment axial force interaction diagram has been demonstrated in this report. It is understood that this check is useful for conditions where the facing is under tension due to the forces generated from sub-horizontal soil nails in particular.

Calculation Steps presented in GEO & HKIE (2013)

Initially, the limit equilibrium analysis is carried out to determine the stabilizing earth pressure by applying a triangular distributed load to the slope face using a series of point loads (Figure 2-20). The minimum triangular earth pressure required to achieve a FOS of 1.1 is determined in this analysis. For loose fills, a minimum basal shear of 3 kPa is assumed and point loads parallel to the slope face will be also used to represent it as a surface shear force.

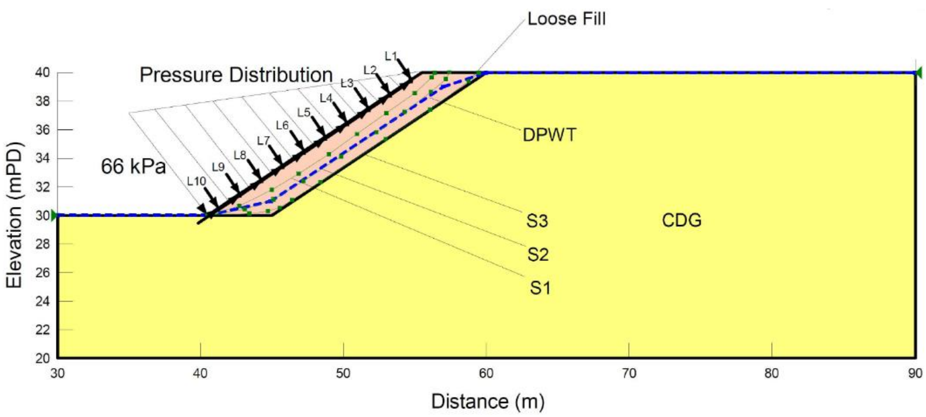
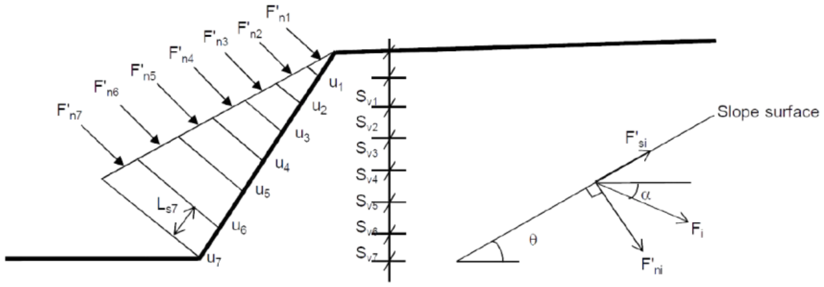


Figure 2-20. Limit equilibrium model to determine stabilizing surface pressure (Geotechnical Engineering Office, Hong Kong Institution of Engineers, 2013)

Using the stabilizing triangular pressure distribution obtained from the initial step (U_i), the soil nailhead forces (F_i) at each level are calculated by multiplying the earth pressure and the tributary area of each soil nailhead. Then the level of maximum nailhead load is determined and the earth pressure at that level (u') is obtained as shown in Figure 2-21.



	Vertical distance from crest (m)	α (°)	S_{vi} (m)	ΣS_{vi} from crest (m)	u_i (kPa)	L_{si} (m)	F'_{ni} (kN/m)	F_i (kN/m)	F'_{si} (kN/m)	Individual nail force (kN)
Row 1	0.50	20.00	1.25	1.25	8.25	2.25	9.30	11.54	6.83	17.30
Row 2	2.00	20.00	1.50	2.75	18.15	2.70	35.69	44.30	26.23	66.44
Row 3	3.50	20.00	1.50	4.25	28.05	2.70	62.47	77.52	45.90	116.28
Row 4	5.00	56.00	1.50	5.75	37.95	2.70	89.24	89.24	0.00	133.86
Row 5	6.50	56.00	1.50	7.25	47.85	2.70	116.01	116.01	0.00	174.02
Row 6	8.00	56.00	1.50	8.75	57.75	2.70	142.78	142.78	0.00	214.17
Row 7	9.50	56.00	1.25	10.00	66.00	2.25	139.43	139.44	0.00	209.15
						Total=	594.92	620.82	78.96	931.22
						Max. value=		142.78		214.17

Figure 2-21. Example calculation of earth pressure presented in GEO & HKIE (2013) report

Structural calculation proposed to be carried out is as follows;

a. Parameters of the soil nailed slope

Slope angle	β
Vertical and Horizontal nail spacings	S_H, S_V
Calculate spacing along slope face	$L = S_V / sin\beta$

b. Grillage section properties and reinforcement amounts

Breadth of grillage beam	b	No. of main bars	N
Depth of grillage beam	h	Dia. of main bars	D
Minimum cover	C	Rebar yield strength	f_y
Concrete strength	f_{cu}		

Calculated area of tor steel	$A_{s\ prov}$
Calculated effective depth	d

Design of Bending Reinforcement (longitudinal span)

c. Design flexure is calculated assuming a simply supported beam

$$M = u' \times L^2 \times \frac{S_H}{8}$$

Where; u' is the earth pressure corresponding to the maximum estimated nailhead load

d. The requirement for compression reinforcement is checked

$$k = M/bd^2 f_{cu}$$

Where $k < 0.156$ compression reinforcement not necessary

e. The tension reinforcement requirement is calculated for each location, nailhead, and midspan for larger and smaller spans

$$A_{s\ req} = M/(0.87 f_y \times z)$$

Where;

$$z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right)$$

And check $A_{s\ req} < A_{s\ prov}$ criteria

f. Minimum reinforcement requirement is calculated

$$A_{s\ min} = 0.13\% bd$$

And check $A_{s\ min} < A_{s\ prov}$ criteria

Design of Shear Reinforcement

g. Additional parameters required for shear design is obtained

No. of shear bars	N_s	Calculated area of shear steel	$A_{sv(prov)}$
Dia. of shear bars	D_s	Yield strength of shear links	f_{yv}
Spacing of shear bars	S_s		

h. Design shear is calculated:

$$V = u' \times L \times \frac{S_H}{2}$$

i. Design shear stress is calculated:

$$\nu = V/bd$$

j. Shear strength of concrete is calculated:

Table 6.3 of Code of Practice of Structural use of Concrete 2013 (Buildings Department, Hong Kong, 2013):

$$\nu_c = \frac{0.79 \times (100 A_s / bd)^{1/3} \times (400 / d)^{1/4} \times (f_{cu} / 25)^{1/3}}{\gamma_m}$$

Where; $\gamma_m = 1.25$

k. The form of reinforcement is determined

Table 6.2 of Code of Practice of Structural use of Concrete 2013 (Buildings Department, Hong Kong, 2013):

$0.5v_c < v < v_c + 0.4$ criteria is assessed

Where; v_c is the design concrete shear strength and v is the design shear stress calculated in the preceding step. The form of shear reinforcement is determined based on this check and as given in Table 6.2 of Code of Practice of Structural use of Concrete 2013.

l. Minimum shear reinforcement is calculated:

$A_{sv} \geqq 0.4 \times b \times S_v / 0.87 f_{yv}$

Same steps could be followed to carry out the structural design for transverse span interchanging S_V and S_H values accordingly.

Bearing Failure of the Base Plate

m. Additional section properties necessary for the design for bearing is defined

Base plate width	b_1	
Calculate the base plate perimeter	u_o	$= b_1 \times 4$
Effective depth at nailhead	d_e	

n. Maximum design shear capacity is calculated

Cl 6.1.5.7 b) of Code of Practice of Structural use of Concrete 2013 (Buildings Department, Hong Kong, 2013):

$$v_{\max\ all} = 0.8\sqrt{fcu}\ , or\ 5\ N/mm^2\ if\ less$$

o. Design shear stress at the bearing plate is calculated

$$v_{\max} = \frac{V}{u_o d_{peff}}$$

$$V = \max nailhead\ load\ \ (F_{nail})$$

And check $v_{\max} < v_{\max\ all}$ criteria

Punching Shear at Nailhead

p. The soil pressure value at the top and bottom of the nailhead plate (U_a & U_b respectively) is obtained from the triangular pressure distribution considered previously. Then the average soil pressure behind the nailhead is calculated.

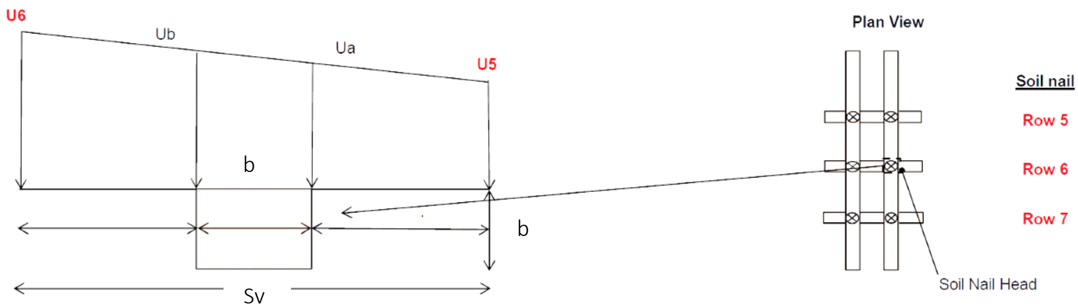


Figure 2-22. Soil pressure distribution behind the nailhead (After GEO & HKIE (2013))

$$F_{soil} = \frac{U_a + U_b}{2} \times Area\ of\ nailhead$$

Punching force is the maximum soil nail force obtained initially, F_{nail}

q. Resultant force resisted by facing is calculated

$$F_{res} = F_{nail} - F_{soil}$$

r. The design shear at 1.5d from loaded area is calculated

Cl 6.1.5.7 b) of Code of Practice of Structural use of Concrete 2013 (Buildings Department, Hong Kong, 2013):

$$v_{1.5} = \frac{F_{res}}{u_{1.5} \times d_{effn}}$$ Where;

$$u_{1.5} = Perimeter\ 1.5d\ from\ loaded\ area$$

$v_{1.5} < v_c$ criteria is checked. v_c obtained in step (j) is used here.

Analyses for the force equilibrium of the facing is also carried out in this report. However, this calculation is targeted towards a grillage on a loose fill where liquefaction is also considered. In the general soil nailing practice where the slope underneath is providing sufficient frictional resistance, such calculations are not necessarily carried out.

2.6.3 The usefulness of FHWA (1998) and CIRIA c637 methods of Shotcrete facing design

Apart from the guideline from GEO and HKIE, the guidelines for designing Shotcrete facings discussed deeply in Section 2.5 were found to be potentially useful for developing a design procedure for Grid Beams. As such, as a part of this study, a few methods for Grid Beam designs have been developed by modifying the methods proposed for the design of full-face shotcrete facings. The development procedure of these methods has been presented in Chapter 5 in detail.

2.7 Flexible Structural Facings

2.7.1 Introduction

These systems consist of a flexible metallic mesh or a geosynthetic which is fixed against the slope by head plates bolted to the soil nails/ anchors. Underneath the flexible facing system, a vegetation layer is prepared. A typical illustration of a flexible facing system is given in Figure 2-23. Although in this illustration, steel plates have been used as nailheads, concrete blocks are also used as isolated nailheads. Additionally, the flexible facings are also used along with Grid Beam facings to protect the unsupported soil mass between the grillage structure.

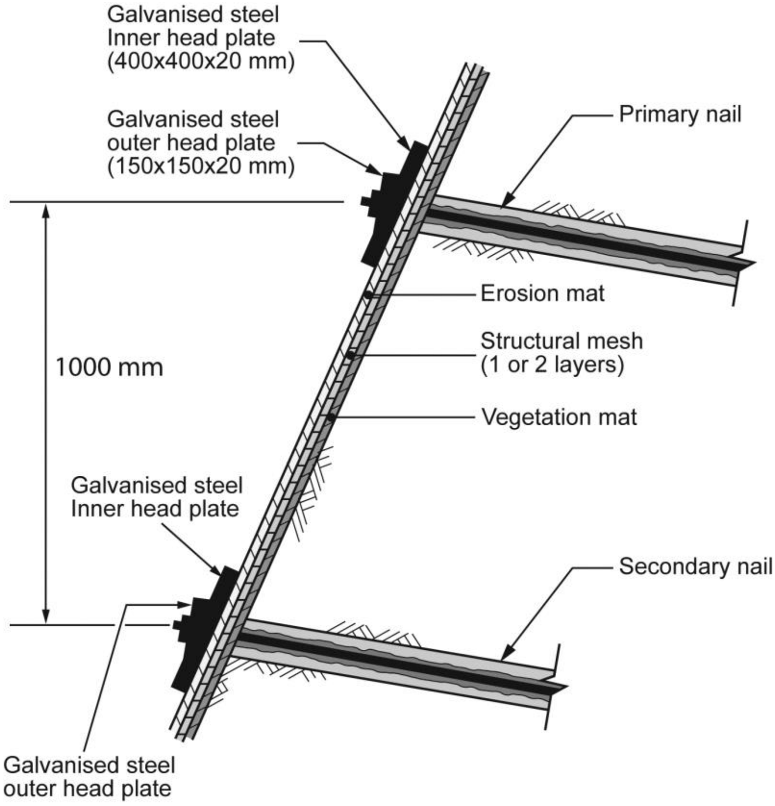


Figure 2-23. Typical details of a flexible facing system (Phear, et al., 2005)

However, these vegetated systems need to be established on the slopes without delay and shall be maintained frequently. If vegetation fails to grow, the slopes will always start to erode and soften underneath the flexible systems overtime. Therefore, on slopes steeper than 60–70°, where maintenance is tedious, and the growth is unreliable, a grid beam or another hard facing is advisable to be used as per CIRIA C637 (2005).

When used with adequate strength and nail spacing, flexible nets/ meshes can be used along with steel head-plates as structural facings in place of hard facings for long term use. However, flexible meshes, when used with isolated nailheads such as concrete blocks (Pillows) or plates as nailheads, allow for soil movements in between the nail heads in extents larger than that of hard structural facings such as a Grid Beam. Therefore, smaller nail spacings are always recommended when flexible facings are used with isolated nailheads or plates.

Flexible meshes must be designed to withstand the punching force exerted at the nailheads caused by the local instability of the soil mass underneath. Thus, the material must be selected considering its capacity to withstand rupture under punching load. Ordinary Geosynthetics are rarely capable of withstanding the estimated punching loads on flexible facings. Therefore, a number of metallic mesh products have been developed to serve this purpose.

One of the few examples of local instabilities presented in GEO report No. 175, is shown in Figure 2-24.

Figure 2-24. Local instability between nailheads (Shiu & Chang, 2004)

2.7.2 Suggestions in CIRIA c637 for designing flexible structural facings

CIRIA c637 (2005) guide recommends not to use flexible (structural) facings for slopes with large spacing between the nails. Flexible mesh design can be carried out using the approaches described in CIRIA report C637 (Phear, et al., 2005), which is mostly based on the UK guide available on HA 68/ 94 (The Highways Agency, 1994).

The sum forces transferred to the nailhead from the failure of soil mass in-between the nailheads through the facing is considered as the out of balance force. To obtain this force on facing, as shown in Figure 2-25, in the case of shallow slopes, a two-part wedge failure mechanism between rows of nails which is based on HA 68/94 is used, whereas, in the case of steeper slopes, a one-wedge failure mechanism which is based on Ruegger et al, 2000 is used. After obtaining the out of balance force, the only calculation that needs to be done is the punching capacity of the facing material against this load. However, due to the complexity of the behavior of flexible facing materials, for high-risk slopes, CIRIA c637 recommends carrying out rigorous calculation methods such as numerical modeling.

When used for the calculation of facing load, one major assumption made in this method is none of the out-of-balance forces is directly carried by the head plates and the forces always get transferred through the flexible facing. Another assumption is taking the porewater pressure ratio, r_u as 0.5 to address the worst case of tension cracks in the slope face.

In these analysis wedge angles θ, θ_1 and θ_2 must be varied to obtain the maximum force induced on the mesh, for a given slope.

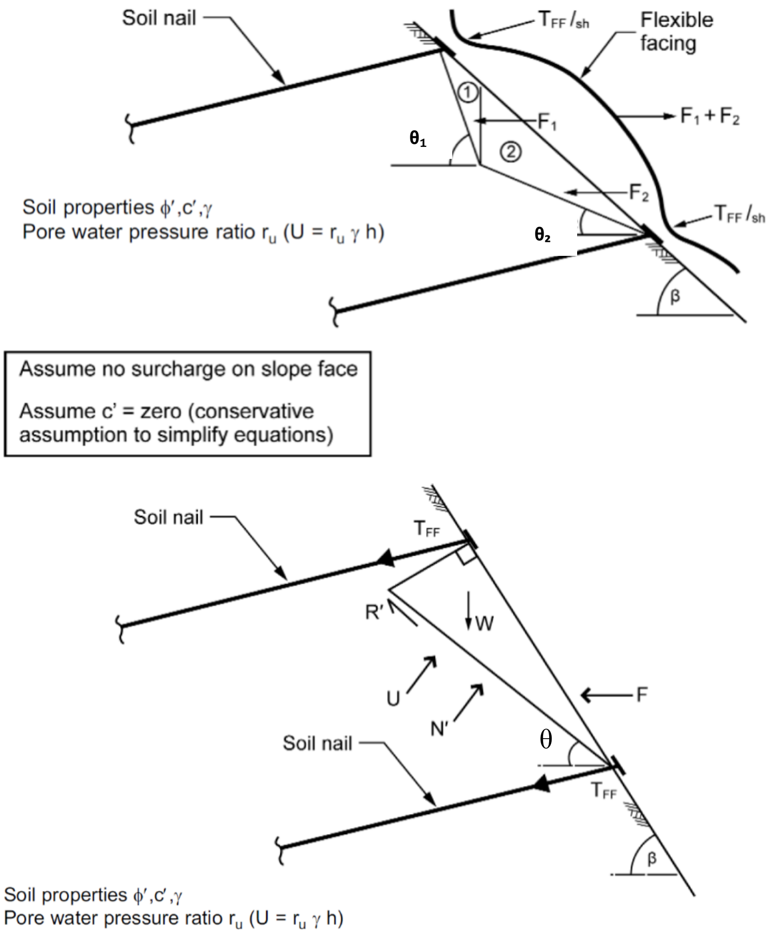


Figure 2-25. Ways to estimate potential out-of-balance forces for the design of flexible facings (CIRIA, 2005)

Two wedge approach

Here, the out of balance force $F_1 + F_2$ or $(T_1 + T_2)$ is taken as:

$$F_1 + F_2 = \frac{W_1(\tan\theta_1 - \tan\phi') + U_1 \tan\phi' / \cos\theta_1}{1 + \tan\theta_1 \tan\phi'} + \frac{W_2(\tan\theta_2 - \tan\phi') + U_2 \tan\phi' / \cos\theta_2}{1 + \tan\theta_2 \tan\phi'} \tag{2.22}$$

This simplified equation presented in CIRIA c637 (2005) for obtaining $F_1 + F_2$ omits the cohesion of soil. However, the original equation presented in HA 68/94 (Equation (2.23)) can be used for obtaining out of balance force. This expression takes the effect of cohesion into account.

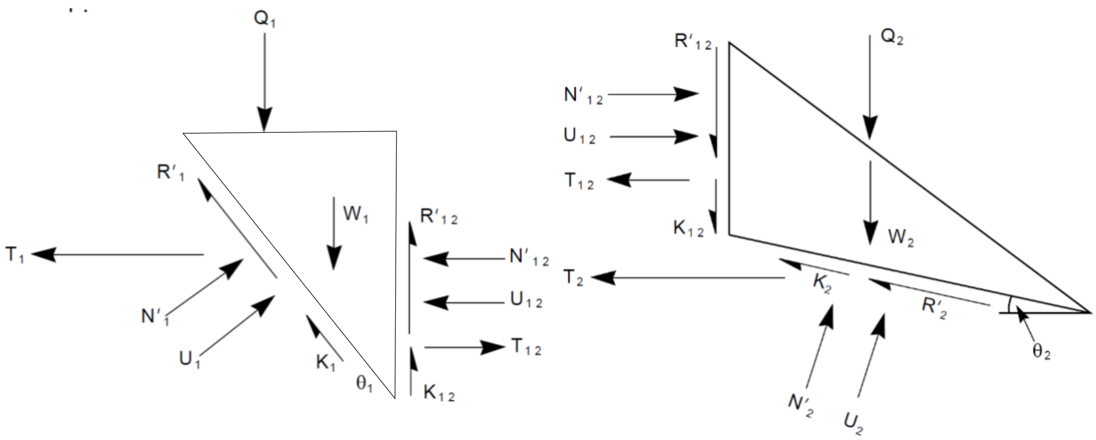


Figure 2-26. Forces on the two-wedge mechanism (The Highways Agency, 1994)

$$T_1 + T_2 = \frac{[W_1(\tan\theta_1 - \tan\phi_1') + (U_1 \tan\phi_1' - K_1)/\cos\theta_1]}{(1 + \tan\theta_1 \tan\phi_1')} + \frac{[W_2(\tan\theta_2 - \tan\phi_2') + (U_2 \tan\phi_2' - K_2)/\cos\theta_2]}{(1 + \tan\theta_2 \tan\phi_2')} \quad (2.23)$$

The expressions used in Equation (2.23) have been illustrated in Figure 2-26. It must be noted that the tensile forces (out of balance force) in this expression are horizontal. However, when the nail inclination is incorporated into this equation, it becomes much more complicated. Thus, it is assumed that all horizontal forces are acting on the upper wedge instead of both. Thereby, this simple modification (Equation (2.24)) can be done to derive the tensile forces in the direction of soil nails.

$$T_1 + T_2\ _{\delta} = \zeta \cdot\ T_1 + T_2\ _{hori} \qquad \textit{Where}: \quad (2.24)$$

$$\zeta = [\cos(\theta_1 - \phi_1') / \cos(\theta_1 - \phi_1' + \delta)]$$

Single wedge approach

Based on the same principles used in the CIRIA c637's two-wedge method, an expression could be developed for assessing the out-of-balance force "F" of a two-dimensional single wedge failure mode.

$$F = \frac{[W(\tan\theta - \tan\phi') + (U \tan\phi' - K)/\cos\theta]}{(1 + \tan\theta \tan\phi')} \quad (2.25)$$

The higher of the "F" value of equation (2.25) and the "$T_1 + T_2$" obtained from (2.24) will be used in the design of flexible structural facing.

2.7.3 Analysis carried out in GEO report No. 175

Although does not include flexible mesh facing designs, the GEO report No. 175 (Shiu & Chang, 2004) presents the results of the analysis carried out on local instabilities between isolated nailheads. The results of the GEO report have been based on the finite element analysis carried out with FLAC on variable slope conditions.

The simplified model considered in the FLAC analysis is as shown in Figure 2-27. It is to be noted that the water table is considered to be at the slope face and the water has been simulated to flow in a direction parallel to the slope face. This has been done to simulate the transient perched water table which can form during heavy rainfall. The report mentions the heterogeneity of soil permeability, the opening of relic joints and cracks could in reality form this type of a transient water table at the slope face.

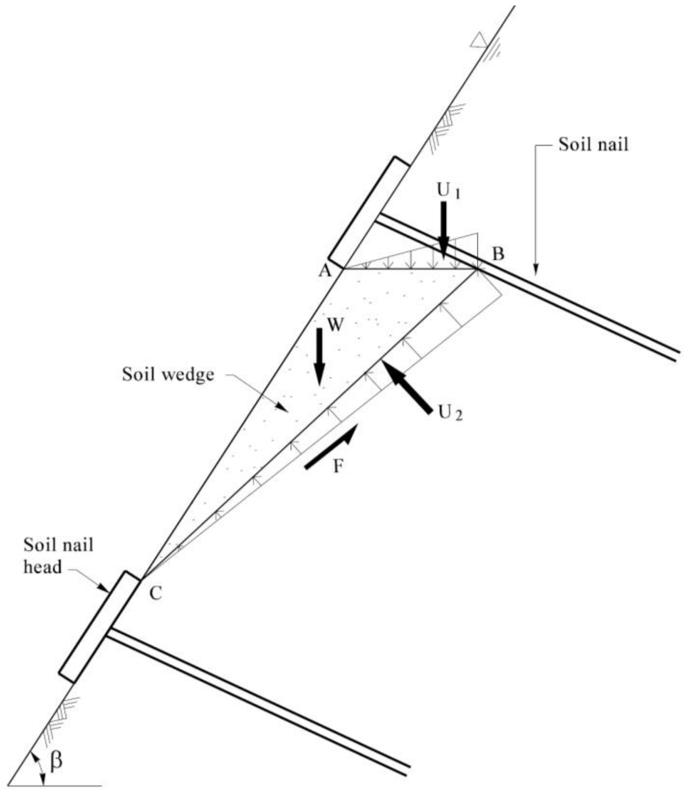


Figure 2-27. Failure mode of local instabilities considered in the GEO report No. 175

However, the report has carried out the analysis for dry slope conditions as well as for the wet conditions. The output of the analysis is the maximum vertical distances (Vertical critical distance) between nailheads up to which the slope can sustain the local instability between nailheads where no additional mesh element has been installed. Accordingly, the analysis has been carried out for various cohesion and friction angles of surface soil and slope angles from 45° and 75°. Results of the analysis have been depicted in Figure 2-28.

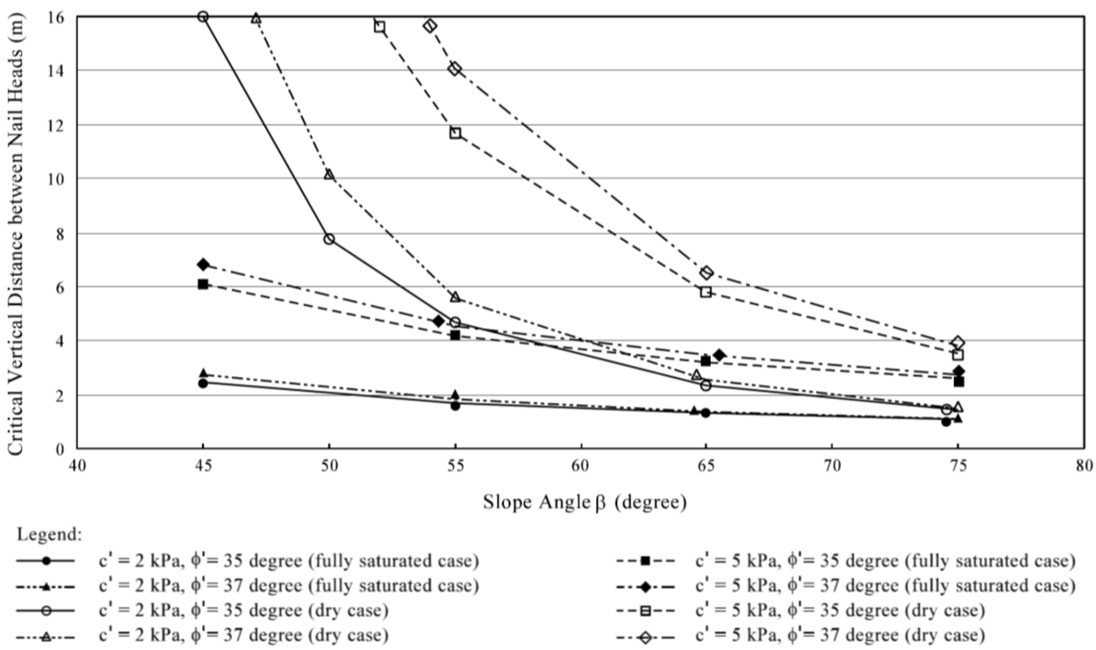


Figure 2-28. Vertical critical distance (after GEO report No. 175)

2.7.4 The design approach presented by Ruegger et. al

Modern approaches of flexible facing have been developed with the advancement of material technology for the purpose of overcoming a few drawbacks of the traditional soil nail facings. The most prominent publications in this regard have been done by Ruegger & Flum (Ruegger & Flum, 2000). Their work has been done side by side with the development and testing of a high tensile mesh product range based in Switzerland. Rugger et.al state that although their studies are gravitating around high-tensile steel wire meshes, their concept could be applied to assess the performance of any flexible mesh system.

Ruegger & Flum states that when the traditional flexible protection systems are used with steel plates as nailheads, either the distance between the nailheads should be controlled or additional wire ropes should be fitted to the nailheads to absorb the excess forces on the mesh. However, when used with high tensile steel meshes, nail spacings could be used with more freedom and there is no need of using additional ropes connecting the nailhead. Moreover, the ability to pretension the flexible mesh at the nailhead is considered as an added capability of the high-tensile meshes. In addition to the above, the durability and the flexibility of these high-performance meshes make them an economical facing solution than the traditional facing methods as per their findings.

The introduced new methods for the analysis of flexible mesh facings are based on the traditional methods for designing meshes. It is to be noted that CIRIA C637 (2005) has also adopted the method of single wedge mechanism for analyzing local instabilities between nailheads based on Ruegger & Flum's publication from 2000. Two modes of failures discussed by them are the superficial instability parallel to the slope and the local instability between the nailheads. It is realized that the CIRIA C637 (2005) does not take the superficial instability into account in the design of flexible facings, as discussed in Section 2.7.2. The. It is to be noted that the original work carried out by Ruegger and Flum (Ruegger & Flum, 2000) has been updated in the later publications (Cala, Flum, Rüegger, Roduner, & Wartmann, 2020) and the updated analysis methods have been discussed in detail hereunder.

Analysis of superficial instabilities parallel to the slope

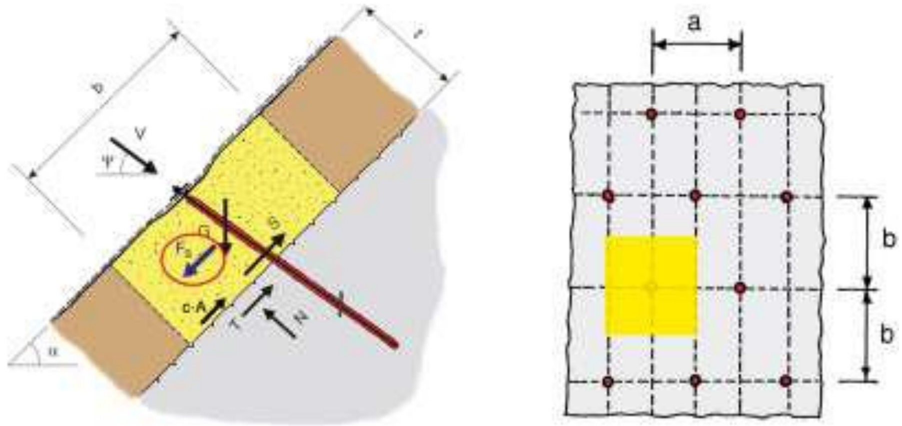


Figure 2-29. Forces acting on a sliding body soil parallel to the slope face (Cala, Flum, Rüegger, Roduner, & Wartmann, 2020)

The annotations used in Figure 2-29 are as follows;

G	deadweight of the sliding body
S	resisting shear force required to be absorbed by the nail
V	Pre-tensioning force
$c \times A$	cohesion of the cover layer × ground surface of the body liable to break out, whereby $A = a \times b$
T, N	reaction forces from the subsoil
α	inclination of the sloping front and sliding surface, respectively, from horizontal
Fs	Force-induced by streaming of water parallel to the slope
Ψ	inclination of the nail relative to the horizontal
φ	effective friction angle of the cover layer
γ_{mod}	model uncertainty correction value
a, b	Horizontal and vertical spacing of nailing arrangement along the slope
t	Thickness of the potential sliding body

As shown in Equation (2.26) shear force absorbed by the soil nail could be obtained. The external pre-tensioning force of the head-plate/ nut is a novel addition in these high-tensile mesh systems which has not been discussed in the traditional mesh designs.

$$S = G \cdot \sin\alpha - V \cdot \cos(\psi + \alpha) + F_s - \frac{[V \cdot \sin(\psi + \alpha) + G \cdot \cos\alpha] \cdot \tan\varphi + c \cdot A}{\gamma_{mod}} \qquad (2.26)$$

Where;

$$F_s = \gamma_w \cdot \sin\alpha \cdot a \cdot b \cdot t$$

Where, γ_w is the density of water

Iterative calculations must be carried out to obtain the maximum value of "S" by changing the thickness of the superficial layer "t".

Analysis of local instabilities between the nailheads

Similar to the method proposed in CIRIA c637, two failure mechanisms have been considered in the analysis of local instabilities namely, one-body mechanism and two-body mechanism.

<u>One body mechanism</u>

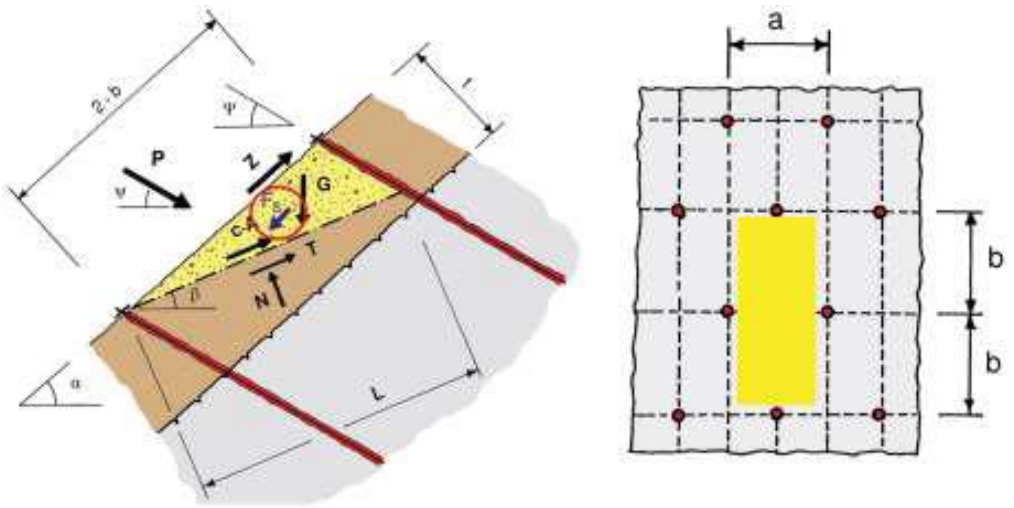


Figure 2-30. Forces considered for the one-body mechanism (Cala, Flum, Rüegger, Roduner, & Wartmann, 2020)

There are two different expressions presented for dry condition (2.27) and for considering streaming parallel to the slope (2.28) to obtain the out-of-balance force on the mesh.

For dry condition;

$$P = \frac{G \cdot [\gamma_{\mathrm{mod}} \cdot \sin\beta - \cos\beta \cdot \tan\varphi] - Z \cdot [\gamma_{\mathrm{mod}} \cdot \cos(\alpha - \beta) - \sin(\alpha - \beta) \cdot \tan\varphi] - c \cdot A}{\gamma_{\mathrm{mod}} \cdot \cos(\beta + \psi) + \sin(\beta + \psi) \cdot \tan\varphi} \tag{2.27}$$

For streaming condition;

$$P = \frac{F_s \cdot \cos(\alpha - \beta) + G \cdot \sin\beta - Z \cdot \cos(a - \beta)}{\cos(\psi + \beta) + \sin(\psi + \beta) \cdot \dfrac{\tan\varphi}{\gamma_{mod}}} - \frac{\dfrac{[G \cdot \cos\beta - Z \cdot \sin(\alpha - \beta)] + F_s \cdot \sin(\alpha - \beta)] \cdot \tan\varphi + c \cdot A_c}{\gamma_{\mathrm{mod}}}}{\cos(\psi + \beta) + \sin(\psi + \beta) \cdot \dfrac{\tan\varphi}{\gamma_{mod}}} \tag{2.28}$$

Two body mechanism

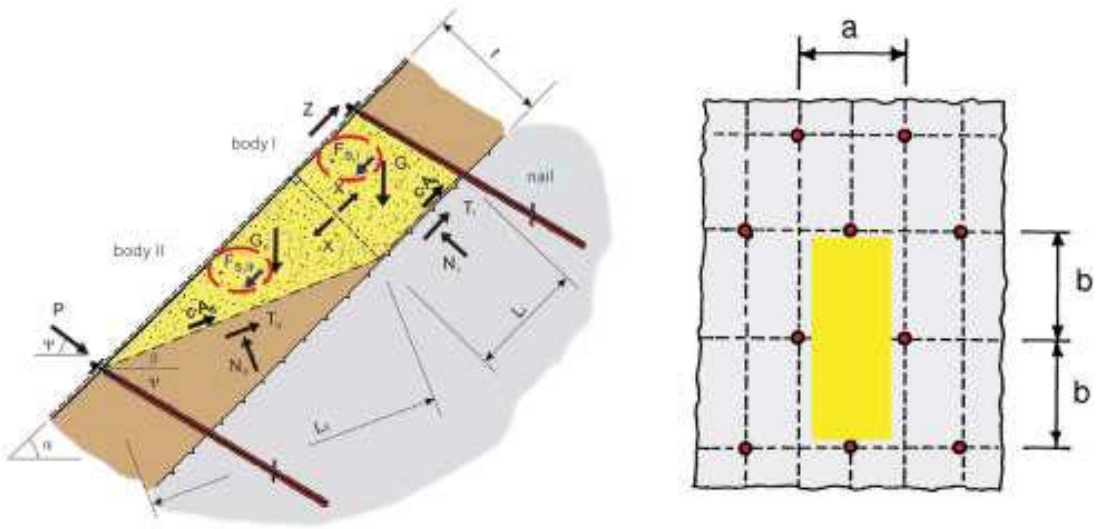


Figure 2-31. Forces considered for the two-body mechanism (Ruegger & Flum, 2000)

The contact force between the two bodies can be obtained by the following expressions.

For dry condition;

$$X = 1/\gamma_{\mathrm{mod}} \cdot \{G_{\mathrm{I}} \cdot (\gamma_{\mathrm{mod}} \cdot \sin\alpha - \cos\alpha \cdot \tan\varphi) - c \cdot A_{\mathrm{I}}\} \tag{2.29}$$

For streaming condition;

$$X = F_{s,\mathrm{I}} + G_{\mathrm{I}} \cdot \sin\alpha - \frac{G_{\mathrm{I}} \cdot \cos\alpha \cdot \tan\varphi + c \cdot A_{c,\mathrm{I}}}{\gamma_{\mathrm{mod}}} \tag{2.30}$$

The forces are resolved to obtain expression which calculates the resisting out of balance force by the mesh.

For dry condition;

$$P = \frac{G_{\mathrm{II}} \cdot [\gamma_{\mathrm{mod}} \cdot \sin\beta - \cos\beta \cdot \tan\varphi] + (X - Z) \cdot [\gamma_{\mathrm{mod}} \cdot \cos(\alpha - \beta) - \sin(\alpha - \beta) \cdot \tan\varphi] - c \cdot A_{\mathrm{II}}}{\gamma_{\mathrm{mod}} \cdot \cos(\beta + \psi) + \sin(\beta + \psi) \cdot \tan\varphi} \tag{2.31}$$

For streaming condition;

$$P = \frac{G_{\mathrm{II}} \cdot \sin\beta + X \cdot \cos(\alpha - \beta) + F_{s,\mathrm{II}} \cdot \cos(\alpha - \beta) - Z \cdot \cos(\alpha - \beta)}{\cos(\psi + \beta) + \sin(\psi + \beta) \cdot \frac{\tan\varphi}{\gamma_{mod}}} - \frac{\frac{[G_{\mathrm{II}} \cdot \cos\beta - Z \cdot \sin(\alpha - \beta) + X \cdot \sin(\alpha - \beta) + F_{s,II} \cdot \sin(\alpha - \beta)] \cdot \tan\varphi + c \cdot A_{c,\mathrm{II}}}{\gamma_{mod}}}{\cos(\psi + \beta) + \sin(\psi + \beta) \cdot \frac{\tan\varphi}{\gamma_{mod}}} \tag{2.32}$$

For the design checks, the maximum of the resisting forces P obtained from the one body and two body methods, P_d will be used.

It should be noted that a few default values are used in the calculation method, the slope parallel force has been taken to support the equilibrium, and the pretension force is used to tension the mesh against the slope at the nailhead. However, the method of determining these values has not been clearly presented. It is learned that the slope parallel force could be a safer value within the safety margins of the bearing resistance of the mesh to selective, slope-parallel tensile stress and the pretension force could also be within the safety margins of the bearing resistance of the mesh to pressure stress in nail direction. Both of these complications are not present in the method proposed by CIRIA c637.

Design checks

Using the outcomes of the above calculations, several design checks are carried out to assess the suitability of a particular mesh and nail. These checks must be done using the design values of all parameters and also using partial factors as per Eurocode 7 which will be detailed in Chapter 6.

- Checking **nail** against sliding-off of a superficial layer parallel to the slope

The bearing resistance of the soil nail against shearing, S_R must be adequate to sustain the unbalance superficial sliding force, S_d.

Shear strength of re-bar, $= \frac{f_y}{\sqrt{3}} \times A_n$

Where; f_y Yield strength of soil nail rebar

A_n Cross-section area of soil nail rebar

- Check for the **mesh** against puncturing

Bearing resistance of the mesh against pressure stress in nail direction, D_r must be adequate to sustain the pretension force, V.

- Check for the **nail** to combined stress of both puncturing and sliding off

Two expressions that utilize the above outputs, the parameters of the mesh and the tensile capacity of the nail T_R is presented by Ruegger et. al. for this check.

$$\left(\left(\frac{V}{T_R}\right)^2 + \left(\frac{S_d}{S_R}\right)^2\right)^{0.5} \leq 1 \qquad (2.33)$$

$$\left(\left(\frac{P_d}{T_R}\right)^2 + \left(\frac{S_d}{S_R}\right)^2\right)^{0.5} \leq 1 \qquad (2.34)$$

- Checking the **mesh** against shearing-off at the upslope edge of the nailhead plate

The bearing resistance of the mesh against sharing off, P_R must be adequate to sustain the maximum out of balance force obtained from one body and two body analysis, P_d.

- Checking the adequacy of the **mesh** to transmit the force, Z_d onto the nail

The bearing resistance of the mesh against slope parallel tensile stress must be adequate to sustain the slope parallel force input in the one-body and two-body equilibrium calculations, Z_d.

2.7.5 Mechanical properties of the mesh

As discussed in the preceding section, several different mechanical properties of flexible meshes have been discussed in the design of facings.

However, punching shear resistance is the primary consideration in the design of soil nail facings. This is the property discussed as the bearing resistance of the mesh against pressure stress in nail direction, D_r presented by Ruegger et. al in their publication. In the CIRIA c637, the out of balance force obtained from the analysis is compared against the punching capacity of the facing material as discussed in 2.7.2.

Generally, the punching shear strength is measured in the direction perpendicular to the plane of the mesh by applying a force until the point of failure. ASTM A975, EN 10223-3, ISO 17746, and EAD 230025-00-106 could be considered as some of the accepted test standards available for testing punching shear strength. However, these methods use different test setups to each other which makes it impossible to compare the strength results obtained from these tests. Particularly, the form and size of the device/ plate used to apply the load onto the mesh cause the test results to differ from each other. In addition to the plate, the supporting material underneath the mesh during the test also affects the punching strength result and some test methods use a deformable material underneath and others test on open air. However, the best results could be achieved when the plate used in the actual mitigation work is represented in the actual testing.

It must be noted that the tensile capacity of the mesh wire can be tested without any complication of test methods as in the case of testing for punching shear strength. Moreover, the punching shear strength of the mesh could also be calculated theoretically via principles of structural mechanics based on the tensile capacity of the mesh wire. This method has been discussed in detail in Section 6.2.

2.8 Isolated Nailheads

2.8.1 Introduction

Concrete Pillows

On slopes where erosion is not present it is a common practice to use a square-shaped concrete block which is often referred to as a "Pillow" at the nailhead to anchor the nail at the slope surface. Additionally, a flexible steel mesh is installed to protect the slope in between the pillows. Accordingly, the Pillow-Metallic Mesh system will act as the soil nail facing. Special attention must be paid to the sizing of the plate when the mesh is fixed to the nailheads via a steel plate. The sizing details of the mesh plate have been discussed in Chapter 6. Although in the local practice the pillow is embedded into the slope, there are several other parts of the world where the pillow is cast extruded to the slope surface. A typical detail of a Pillow used in the local practice is shown in Figure 2-32.

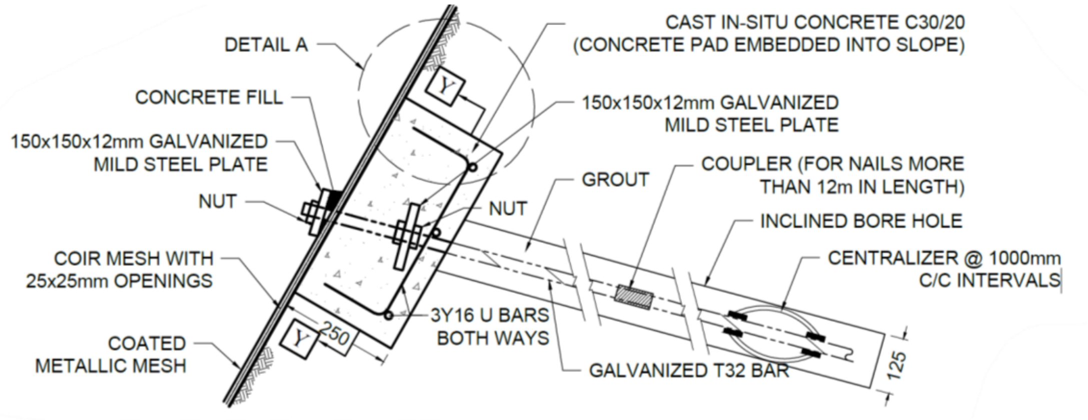


Figure 2-32. Typical Detail of an Isolated Nailhead (Pillow) (NBRO Design Documents)

Where isolated nailheads are used as the preferred facing type, 2 major requirements must be checked in the design process;

- Bearing capacity of the pillow for anchoring the soil nails at the slope surface,
- Adequacy of the bearing plate used to fix the steel mesh into the nailhead,

Adequacy of the bearing plate shall be evaluated considering two major failure modes as discussed in Chapter 6;

a) Flexural failure of the plate itself, and

b) Punching failure of the steel mesh around the steel bearing plate.

The method for evaluating the structural capacity of the plate is described in Section 2.8.2.

Bearing plate nailheads

It should be noted that adequately sized steel plates could also be used in place of concrete pillows at the nailhead. This method is commonly referred to as the “Bearing plate method”. In this case, the bearing plate is used to fix the flexible mesh against the slope at the nailheads. However, the bearing plate will additionally serve the purpose of transmitting the nailhead load onto the soil underneath. The bearing capacity of the plate on the soil slope could be calculated using the same methods presented in CIRIA c637 discussed in 2.8.3,

The design of these nailheads is discussed in detail in Section 7.3.1. The important factor to note here is the flexible mesh is also capable of carrying the nailhead load in this instance. As such, the bearing resistance of meshes against punching should be taken into account when sizing the bearing plates. This means that the effective nailhead load transferred to the soil underneath by the bearing plate is less than the actual nailhead load. Also, the flexural capacity of the bearing plate must be assessed in the same manner presented in Section 2.8.2.

2.8.2 Method for obtaining the capacity of the bearing plate

CIRIA c637 (2005) in their design examples, presents a method for obtaining the size of steel bearing plate to be used as the clamping element to fix the flexible mesh to each nailhead. This calculation could be used with modification, to assess the plate size requirements on instances where the plate is used with a hard facing or without a hard facing at the nailhead. In the case of the presence of a hard facing, the stress exerted on the plate will be by the punching force generated by the out of balance of local instabilities.

As per the force diagram and the location of section modulus (x-x) in Figure 2-33, the following expression can be developed using principles of structural mechanics.

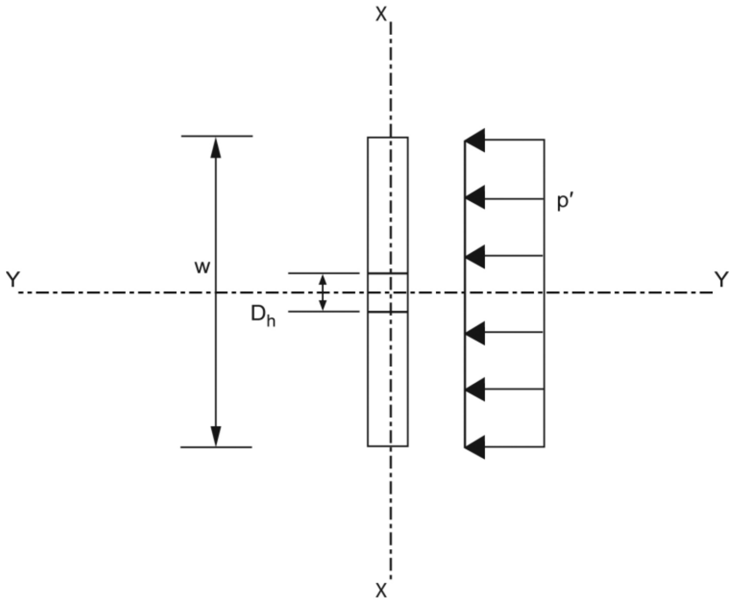


Figure 2-33. Forces acting on the bearing plate, Cross-section

Pressure acting on the plate, p'	$= \frac{T_{max}}{A_{plate}}$	(2.35)
Section modulus of the plate at the centerline, Z	$= \frac{w - D_h \quad \times t^2}{6}$	(2.36)
Moment exerted at the centerline, M_{zz}	$= \frac{p' \times w^2}{2} \times \left(\frac{w}{4}\right)$	(2.37)
Maximum stress on the plate, σ	$= \frac{M_{zz}}{Z}$	(2.38)

Where;

A_{plate}	Gross area of the plate
w	Width of the plate
t	Thickness of the plate
D_h	Diameter of the hole of the plate

It must be noted that the maximum soil nail load, T_{max} value in Equation (2.35) must be taken as per the context. Where the plate is used as the only element in the nailhead without having any other facing-element such as a grid beam, pillow, or shotcrete, (bearing plate method) maximum nailhead load should be used as T_{max}. Whereas, in the case where the plate is used to fix the metallic mesh to the nailhead underneath, the out of balance force from the soil in-between the nailheads calculated as per Section 6.3 must be used as T_{max}.

The maximum stress in the plate could be calculated at the design stage and compared with the yield strength of the steel used. A suitable material factor of safety should be used in this assessment as per the codes used for the design. Accordingly, the plate size and thickness could be determined.

2.8.3 Bearing capacity of isolated nailheads

Mainly, two different methods have been presented in the literature for obtaining the bearing capacity of an isolated nailhead. It must be noted that these methods have been developed considering the nailhead similar to a pad foundation loaded by a single column. In this context, the soil nail is idealized as the column of the footing element.

CIRIA c637 (2005) (Phear, et al., 2005) mentions two possible methods of evaluating the bearing capacity of nailhead plates used in flexible facing designs. These methods can be used to design isolated (Pillow) nailheads similarly.

The lower bound solution from HA 68/94

Out of the two presented methods the lower bound solution considers the soil overburden above the failure to calculate the bearing resistance. This method has been originally proposed in HA 68/94 (The Highways Agency, 1994) nailing design guide as discussed in 2.2.1 and recommends for steeper slopes up to 60 degrees.

The failure mechanism and parameters considered in the lower bound method proposed in HA 68/64 are depicted in Figure 2-34. Design formulae are given in Equations (2.39) and (2.40).

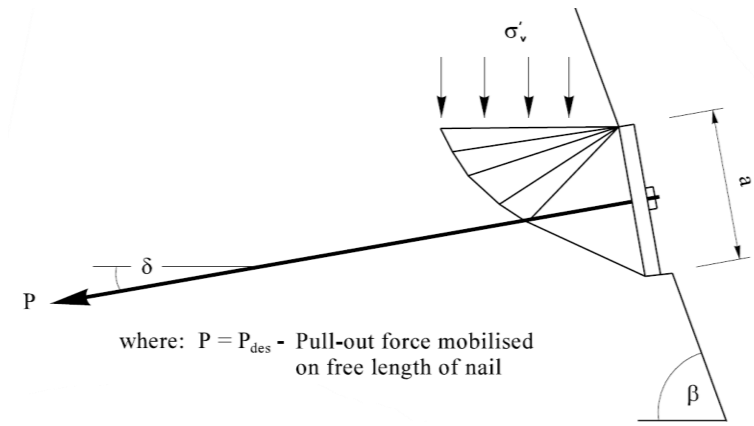


Figure 2-34. Nailhead Bearing Capacity (lower bound solution) (after HA 68/94)

$$a_{req} = \left(\frac{P}{\eta}\right)^{\frac{1}{3}} \quad \text{Where;} \qquad (2.39)$$

$$\eta = \frac{\gamma(1 - r_u)\tan\beta e^{3\left(\frac{\pi}{4} - \frac{\phi'}{2} + \delta\right)\tan\phi'}}{2\cos\left(\frac{\pi}{4} + \frac{\phi'}{2}\right)(1 - \sin\phi')} \qquad (2.40)$$

Where;

a_{req}	Length & width of the nailhead required
P	Nailhead load
γ	Unit weight of soil
r_u	Pore-water pressure ratio
β	Slope angle
ϕ'	Effective angle of friction resistance
δ	Angle of the soil nail

It is seen that this calculation is based on an empirical correlation. As such, it could be considered similar in principle to Terzaghi's bearing capacity equation. However, the GEO report No. 175 has shown that the values obtained through this method are similar to that of finite element analysis.

The parameter *P*, used in this calculation which is related to the soil nail load has been discussed and evaluated in this study. The detailed method of bearing capacity calculation has been presented in Chapter 7.

The upper bound solution from HA 68/94

This method has also been mentioned in the CIRIA c637 (2005) and reference has been made to the HA 68/94 manual for detailed evaluation methods. The concept presented in HA 68/94 is based on the two-part wedge method discussed in 2.7.2. However, this method analyses the passive action of the two-part wedge method and is recommended to be used for shallower slopes (Figure 2-35) for obtaining the bearing capacity of slope faces.

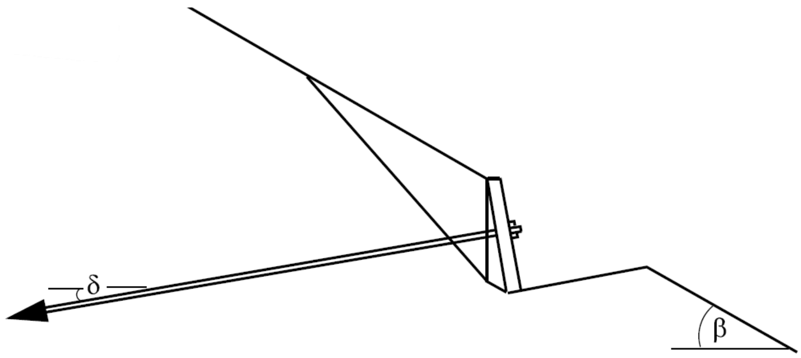


Figure 2-35. Failure mode considered in the upper bound solution for bearing resistance of nailhead (The Highways Agency, 1994)

The spreadsheets developed for obtaining the out of balance force using the two-part wedge method could be modified for calculating the bearing capacity.

In the case of bearing plate nailheads, the bearing capacity could be calculated using the same methods discussed in this section. However, as pointed out in 2.8.1, the effective nailhead load must be used instead of the full nailhead load to assess the adequacy of bearing capacity.

Design guides on Hong Kong GEO publications

The current design guides available in Hong Kong Geoguide 7 for the isolated nail heads (Pillows) are based on the findings of GEO report No. 175 (Shiu & Chang, 2004) where they have carried out FLAC finite element modeling to obtain the bearing resistance of isolated nailheads. These nailhead sizes have been published in Geoguide 7 in a tabulated form. The selection of the nailhead size in this table depends on the shear strength parameters of soil, the angle of slope, and the diameter of soil nail reinforcement.

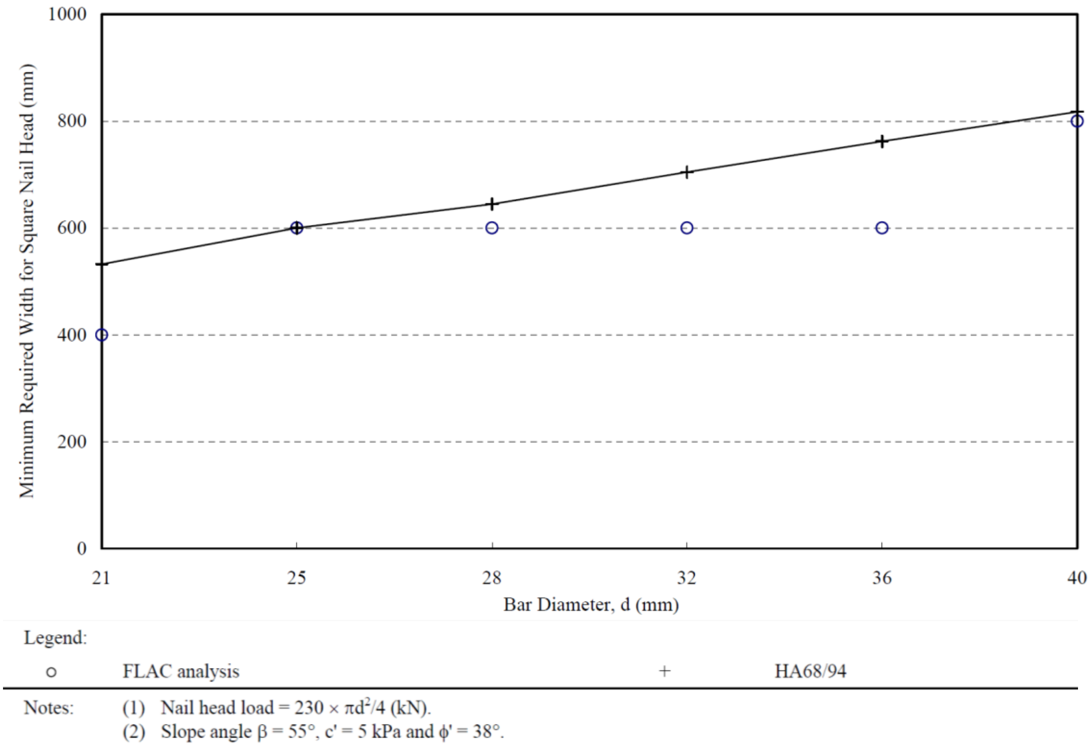


Figure 2-36. Nailhead size requirement FLAC analysis vs Lower bound method HA 68/94 (Shiu & Chang, 2004)

Additionally, the lower bound method discussed in 2.2.1 which has been developed in the UK has been also recommended by Geoguide 7 for obtaining the bearing capacity of isolated nailheads. This recommendation has been based on the results published in GEO report No. 175 where they had obtained comparable nailhead size results to the finite element analysis from the lower bound method. The comparison made by the GEO report is shown in the chart in Figure 2-36.

Moreover, Geoguide 7 recommends considering the decreased contribution of the nailhead into the development of tensile forces along the soil nail at shallower slope angles. Thus, the guide provides the special design of isolated nail heads (Figure 2-37) which can be used to enhance the effect of generating tensile forces in the soil nails in shallower slopes.

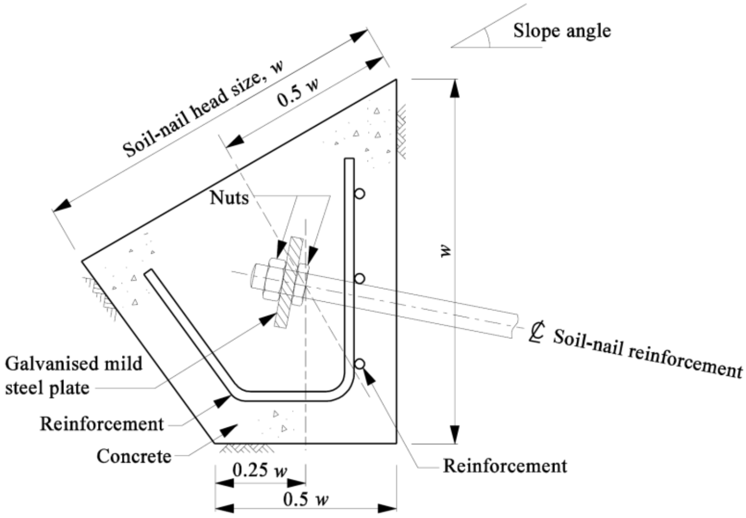


Figure 2-37.Typical Details of a Soil-nail Head for a Shallower Slope (Geotechnical Engineering Office, 2008)

2.9 Relationship Between Soil Nail facing and the Factor of Safety of the Soil Nail system

Initially, the observations made by Gutierrez & Tatsuoka (1988) discussed in 2.1.1 disclosed that the contribution of a facing on the magnitude of tensile forces generated along soil nails is significant and thus increases the overall stability of the slope as shown in Figure 2-4.

Later the findings by Muramatsu et al (1992) in their field studies shown in Figure 2-8 and discussed in 2.1.2 concluded that more flexible nailheads tend to mobilize lower T_0 / T_{max} values, whereas stiffer nailheads mobilize higher T_0 / T_{max} values. Similar observations have been made by Muramatsu et al (1992) in their model studies as shown in

Figure 2-5, where the results indicate how the tensile loads at nailhead in slopes with rigid facings are higher than that of relatively flexible facings (Figure 2-6).

The findings of model studies by Tei et al (1998) also indicate that the slopes with flexible facings undergo higher displacements than those with relatively rigid facings as shown in Figure 2-7 and discussed in Section 2.1.1.

Finite element analysis (FLAC) results of a soil nailed slope with different types of facings presented in GEO report 175, also show that the presence of nailhead load affects the tensile forces developed along soil nails. The tensile force distribution diagram obtained by the FLAC analysis of GEO report 175, shown in Figure 2-38 clearly illustrates that the soil nail force is significantly affected by the facing used. GEO report 175 further supplements this finding by carrying out analysis on different types of facings that have different slope coverage ratios. The results clearly show how the tensile force along the soil nail is increasing with the increased slope surface area covered by the facing (Figure 2-39).

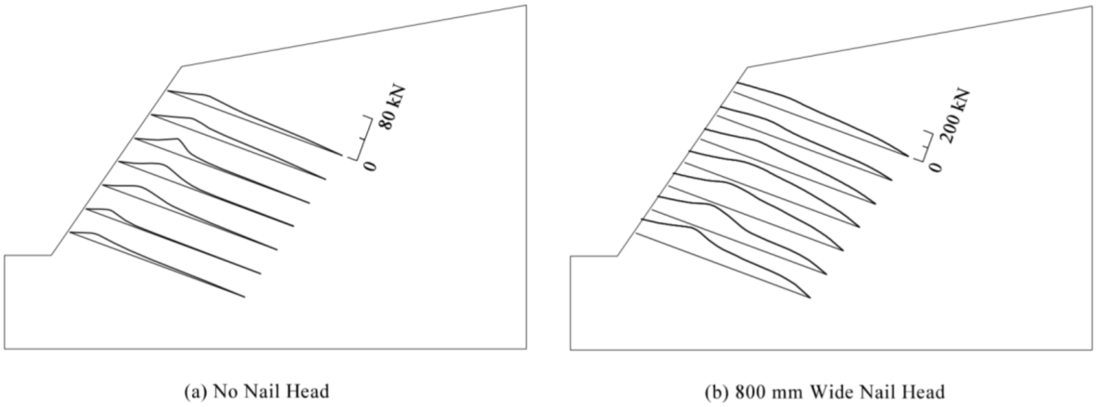


Figure 2-38. Effect of nailhead on the distribution of tensile forces along soil nails (Shiu & Chang, 2004)

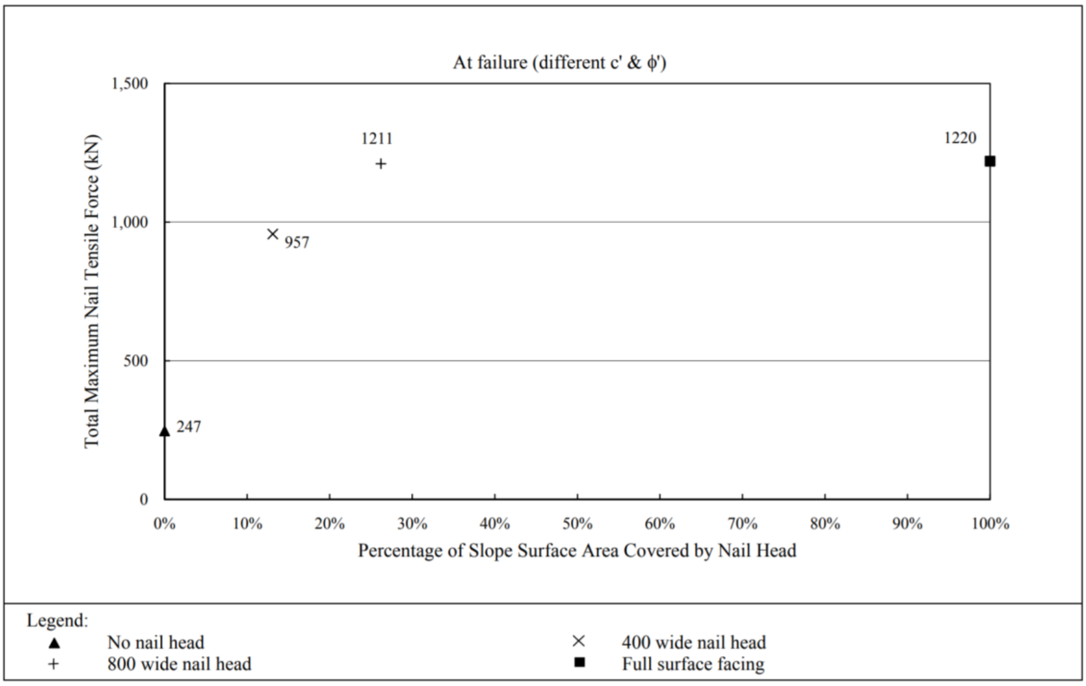


Figure 2-39. Effect of the types of facing on the maximum tensile force along soil nails (Shiu & Chang, 2004)

CHAPTER 3. ESTIMATION OF THE NAILHEAD LOAD

This chapter compares the different approaches presented in Chapter 2 for obtaining the nailhead loads. Also, the most suitable method for obtaining nailhead load in the context of soil nailing practice in Sri Lanka as per the findings of this study is presented. Lastly, the methods discussed are used in a case history in Sri Lanka to obtain the nailhead load value for the design of facing systems

3.1 Background

As discussed in Section 2.4, FHWA, French National Project Clouterre, and the GEO reports address the nail head load evaluation using two distinct contexts. The previous work by FHWA 2003 and by French National Project Clouterre 1991 are based on studies using full-face shotcrete facings whereas GEO report 175 (Shiu & Chang, 2004), has focussed on isolated nail heads of different sizes. However, both facing types (Shotcrete, and isolated nail heads) show similar trends in T_0 / T_{max} or normalized nailhead forces.

Where the precise values of F_F cannot be obtained, FHWA recommends the designers use $F_F = 0.5$ in Equation (2.1) for obtaining the nailhead force (Byrne, Cotton, Porterfield, Wolshlag, & Ueblacker, 1998). Full-scale tests by Gässler & Gudehus (1981) on shotcrete facings in Germany (2.1.1), field studies by Stocker & Riedinger (1990) (2.1.2), French National Project Clouterre (1991), and also the results of instrumented soil nail walls by Holman and Tuozzolo (2009), could be used to justify the above recommendation.

This FHWA approach is recommended for estimating the nailhead load in the case of hard facings in CIRIA c637 (2005) (Design Example 1 (Page 235)) as well. Thus, the FHWA method can be considered as globally accepted for estimating the nailhead load. As such, the method given in FHWA (1998) (Byrne, Cotton, Porterfield, Wolshlag, & Ueblacker, 1998), could be considered as the most appropriate method for carrying out hand calculation of the tensile load at facing or the nailhead force on structural facings.

However, in cases where an analysis of stability has been carried out on the proposed soil nailed slope, the method proposed in Clouterre 1991 (French National Project Clouterre, 1991) discussed in Section 2.4.5 could be used to obtain the maximum nail load, which could then be used for obtaining nailhead load using FHWA 2003's Equation (2.7). When the nailhead values have been obtained from stability analysis, they could be considered as the ultimate value unlike in the case of FHWA and CIRIA c637 methods. This will be referred to as the Clouterre 1991 method since all of its roots are from the French manual.

3.2 Procedure for Obtaining Nailhead Load

The procedure described in the preceding paragraph is summarized in this flow chart.

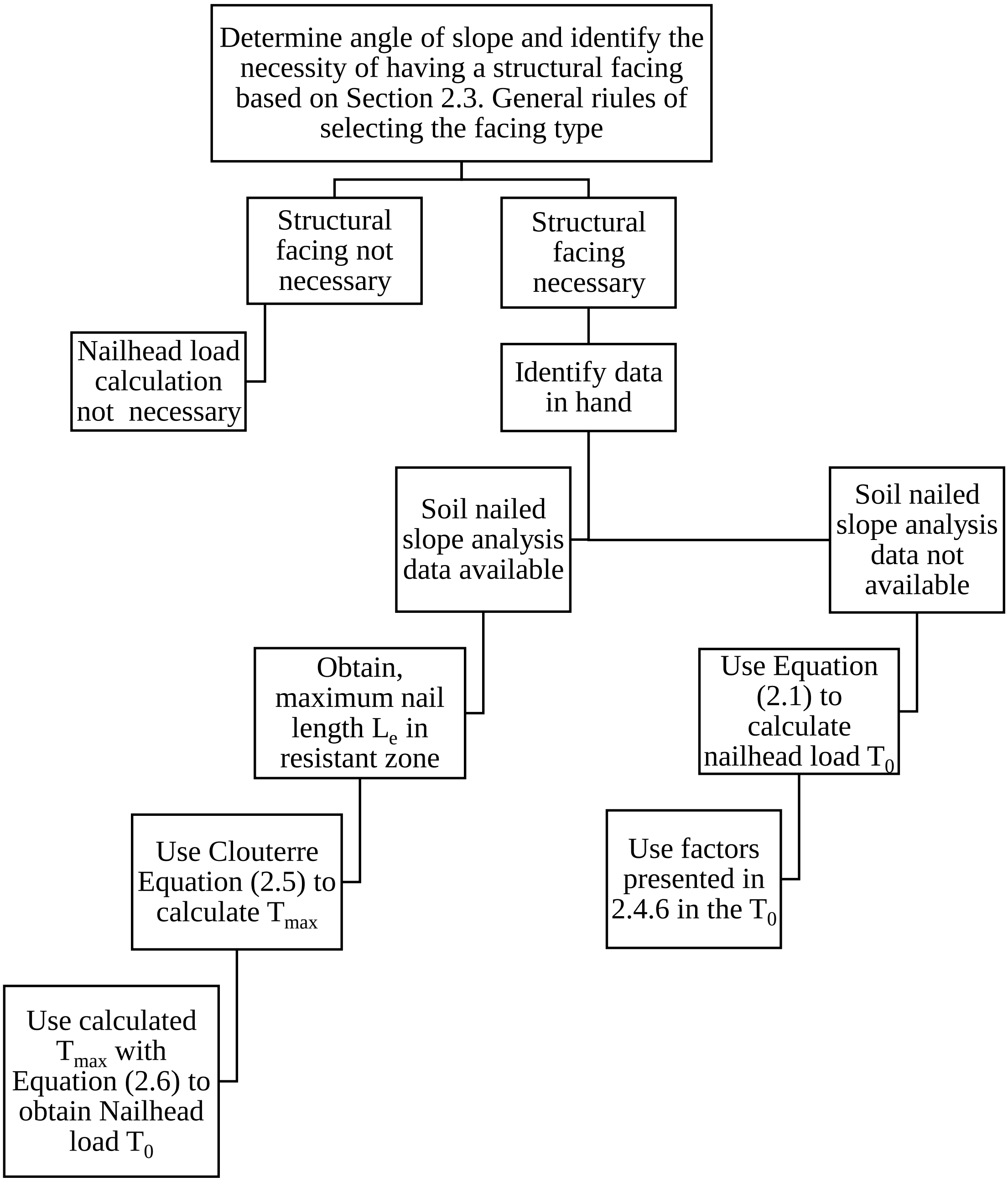


Figure 3-1. Nailhead load calculation procedure

3.3 Application for a Case History – Landslide Rectification at Hakgala

3.3.1 Obtaining the nailhead load

Methods discussed in the preceding chapter will be applied for a project of rectification of Landslide at Hakgala. This project has some of the steepest and tallest soil nailed slopes in the country.

Several landslides had taken place along the Budulla – Nuwara Eliya main road between Culvert 75/5 and Culverts 76/5 and there was a potential for further progressive failures. This failure was triggered by rainfall after the widening of the road. During the widening, some gravity retaining walls at the toe had been removed and no drainage measures were provided. As such, appropriate stabilizing measures were adopted to ensure that further progressive failure is prevented, and safety margins of all existing slopes are enhanced to an acceptable level.

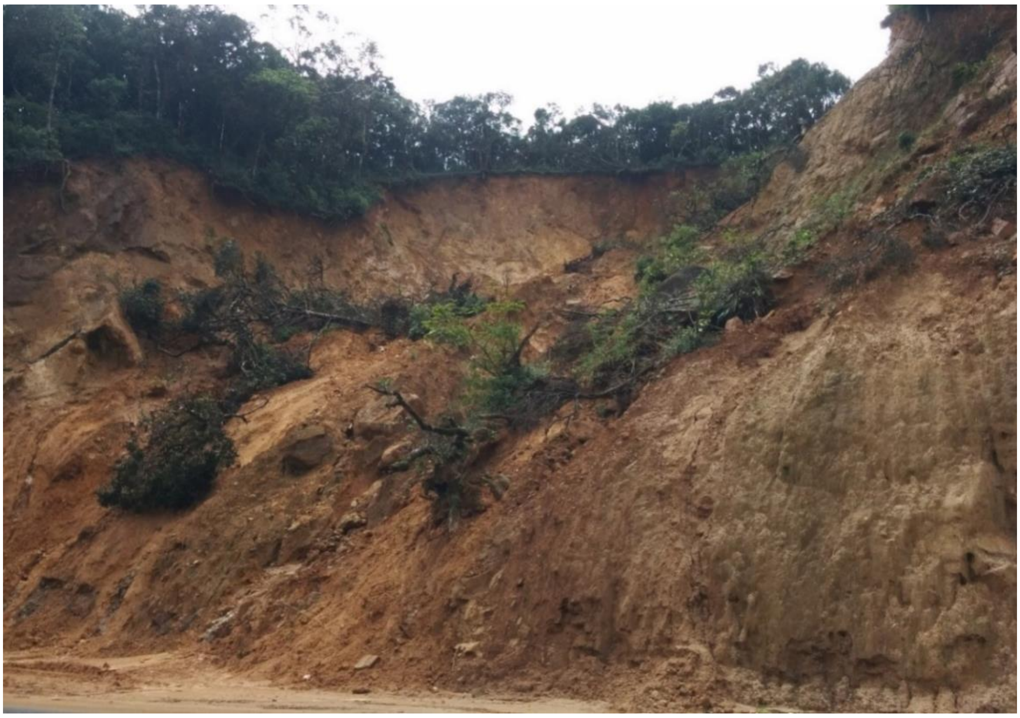


Figure 3-2. Image of Hakgala major landslide before implementing mitigations

To carry out the stability analysis, detailed surveys and borehole investigations were conducted at the main landslide locations and several cross-sections were taken along the road. Rectification designs had been then carried out to enhance the stability to the recommended level at the landslide locations.

The subsoil of the area was found to consist of residual formations of brownish sandy/silt clay. Outcrops of fractured rock are also present at the toe region of the slope, particularly at the major landslide. Some of the shallow failures of the cut slope appeared to have propagated further developing into large-scale landslide at Culvert 75/10 as shown in the contour map Figure 3-3.

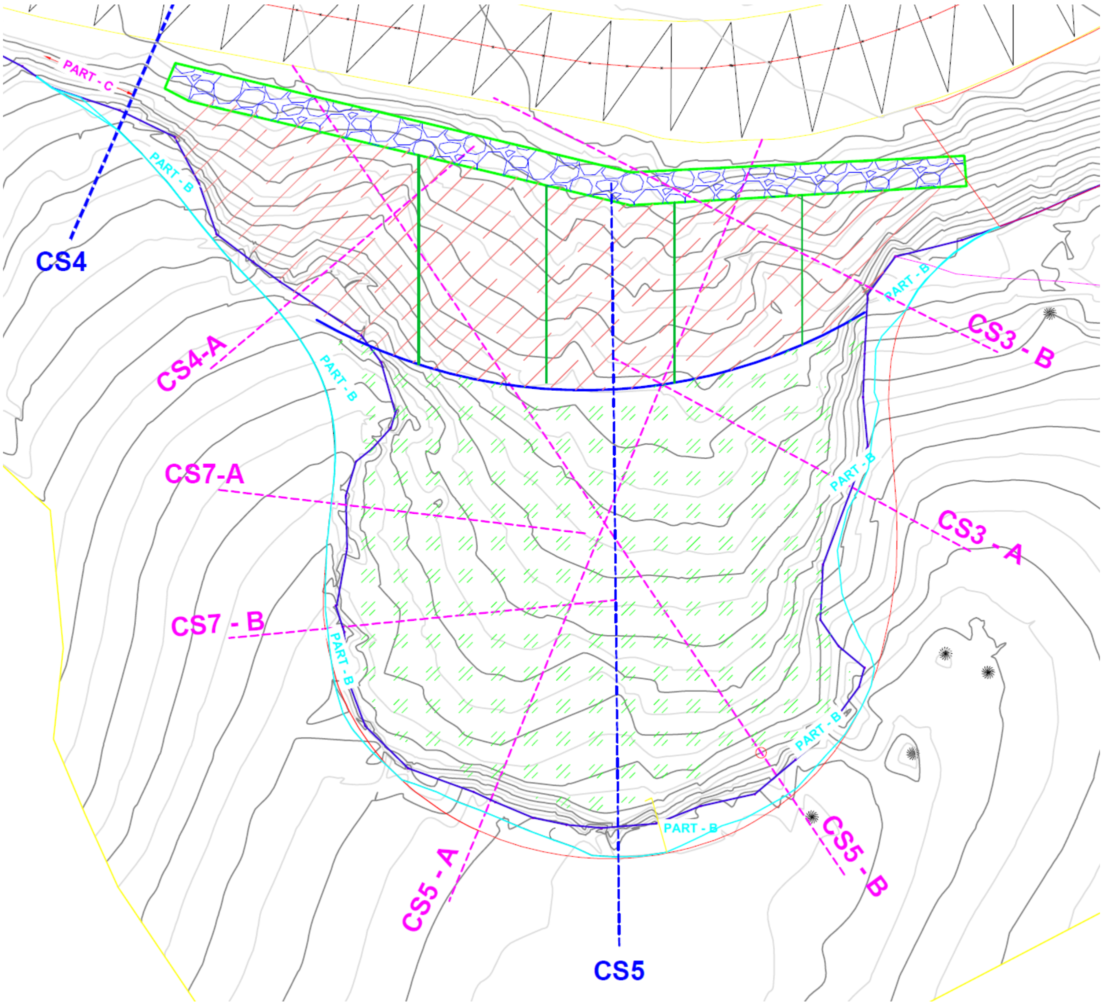


Figure 3-3. Contour map and plan view of cross-sections – Hakgala landslide

Height and the angle of the cut slope are much higher in the segment from Culvert 75/10 to Culvert 76/2 than in the other areas. The steep escarpments developed due to the landslide has to be stabilized to prevent further propagation. The rectification design was done for a number of slope across sections presented in Figure 3-3 (CS4, CS7-B, CS5-B, CS3-B). The stability analysis of Section 5 is elaborated in this chapter.

The analysis of the stability of the escarpment (Figure 3-4) yielded a FOS of 0.811 by the Spencer method of limit equilibrium analysis using Slopw/ W software. Hence a nailing arrangement was proposed to enhance the stability of this section. Soil parameters used in this analysis are presented in Table 3-1.

Table 3-1. Soil parameters used in CS5 stability analysis

Soil Type	Notation	γ (kN/m^3)	C$^/$(kPa)	$\Phi^/$(degree)
Residual Soil Layer 1	(yellow)	18	7	28
Residual Soil Layer 2	(pink)	18.5	10	30
Highly Weathered Rock	(orange)	19.5	14	35
Fractured Rock	(brown)	22	50	45

Soil nailing arrangement in the form of three numbers of 16m long and three numbers of 12m long 32mm diameter soil nails at an angle of 15° to the horizontal was used in this analysis. Partial factors used for this analysis are as follows:

γ_d	Partial safety factor for skin friction	= 1.5
γ_s	Partial safety factor for the tensile strength of the nail	= 2

This proposed arrangement resulted in horizontal spacing $S_H = 2.0\, m$ and vertical spacing $S_V = 2.0\, m$. This resulting arrangement increased the Factor of Safety (FOS) up to **1.335** as presented in Figure 3-5**,** which was deemed acceptable.

The detailed facing design for this nailing arrangement using the methods discussed in Chapter 2 will be elaborated.

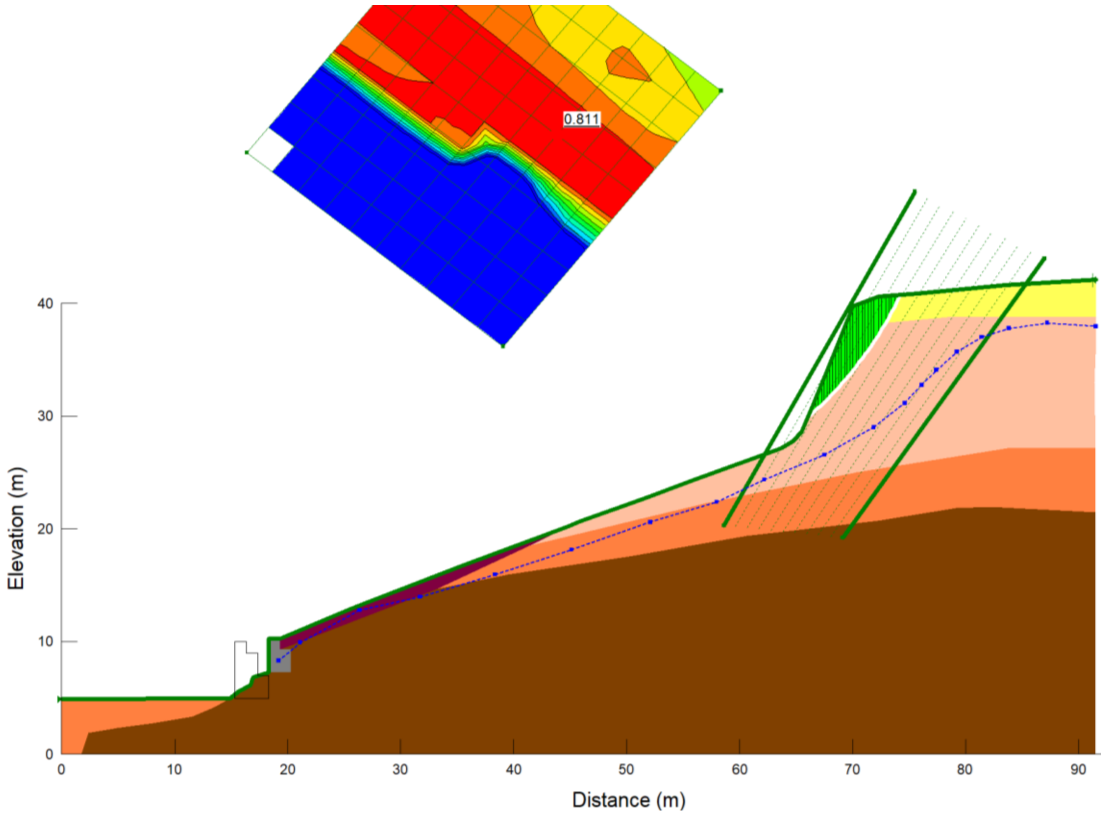


Figure 3-4. Existing Stability analysis - CS5 – Hakgala landslide

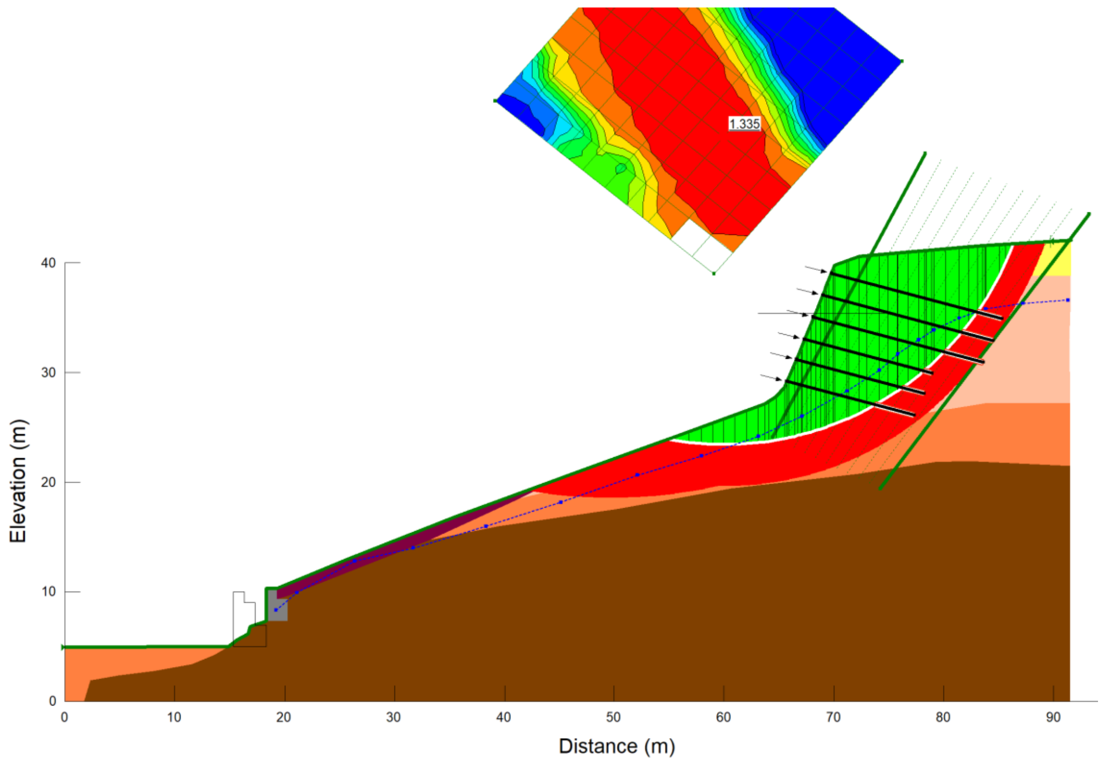


Figure 3-5. Stability analysis soil nail at 2m - CS5 – Hakgala landslide

1. **Obtaining maximum soil nail force**

The method proposed in Clouterre 1991 discussed in Section 2.4.5 given in Equation (2.6) is generally used to estimate the maximum nail force at the surface. However, the Slope/W software automatically calculates the forces generated in the soil nails. These values can be obtained by using the "View Object Information" function in Slope/W (Figure 3-6). However, it should be noted that these loads depend on the "Pullout resistance" or "Skin friction" input and the partial FOS values used by the designer. The most common method of calculating Pullout resistance is the one presented in Hongkong Geoguide 7 (Geotechnical Engineering Office, Hong Kong Institution of Engineers, 2013).

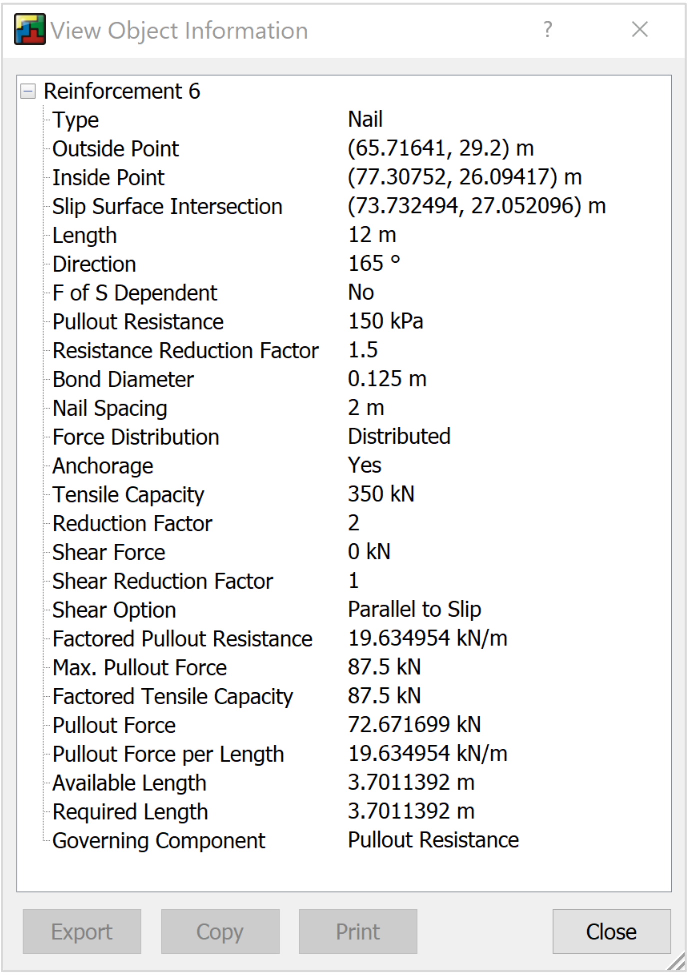


Figure 3-6. Soil Nail forces obtained from Slope/ W software

The pullout resistance is calculated using the method given in Equation (3.1):

$$\text{Pullout resistance } (t_f) = C_w + \sigma_{av} \times \tan\delta \tag{3.1}$$

C_w Cohesion [kPa]

δ Angle of interface friction between the grouted body

σ_{av} $= (\sigma_v + \sigma_h)/2$

Accordingly, the "Pullout Force" value represents $(qs\ \pi DLe)/\gamma d$ component, and the "Factored tensile capacity" represents the $fy/\gamma s$ component of Equation (2.6). The values obtained for each nail from the results of stability analysis to determine $T_{\max}$ are shown in Table 3-2.

Table 3-2. Soil nail forces obtained from the stability analysis (2.0 m x 2.0 m arrangement)

Nail No. (from top)	Factored Pullout Force (kN/m)	Factored tensile capacity (kN/m)	$T_{\max}$ (kN/m)	$T_{\max}$ (kN) (x S_H)
1	14.80	87.5	14.80	29.6
2	27.40	87.5	27.40	54.8
3	49.51	87.5	49.51	99.02
4	12.61	87.5	12.61	25.22
5	39.08	87.5	39.08	78.16
6	**72.67**	**87.5**	**72.67**	**145.34**

As such, for the facing design, $T_{\max}$will be taken as 145.34 kN.

2. **Calculating nailhead force**

Having found maximum soil nail force, Equation (2.7) can be used to calculate the nailhead force as discussed.

$$\begin{aligned} T_o &= T_{\max}[0.6 + 0.2\ (S_v[m] - 1\ )] \\ &= 145.34 \times [0.6 + 0.2 \times (\ 2 - 1\ )] \\ &= \mathbf{116.27\ kN} \end{aligned}$$

The obtained value of the nailhead load is based on the forces obtained from a limit equilibrium analysis where an acceptable factor of safety is already applied. As such, this value could be considered as the ultimate nailhead load, without applying the factors discussed in 2.4.6. Therefore:

$$T_{0,ULT} = \mathbf{116.27\ kN}$$

This value is used for facing designs in the latter sections accordingly.

Further analysis has been carried out in the following sections to assess the nailhead loads of the same slope segment for several design variations.

3.3.2 Evaluation of nailhead forces for different nailing arrangements

Variation of nail spacing

If a different nailing arrangement was used with increasing the spacing of nails ($S_V = S_H = 2.5\ m$), the resulting FOS will be **1.289** (Figure 3-7).

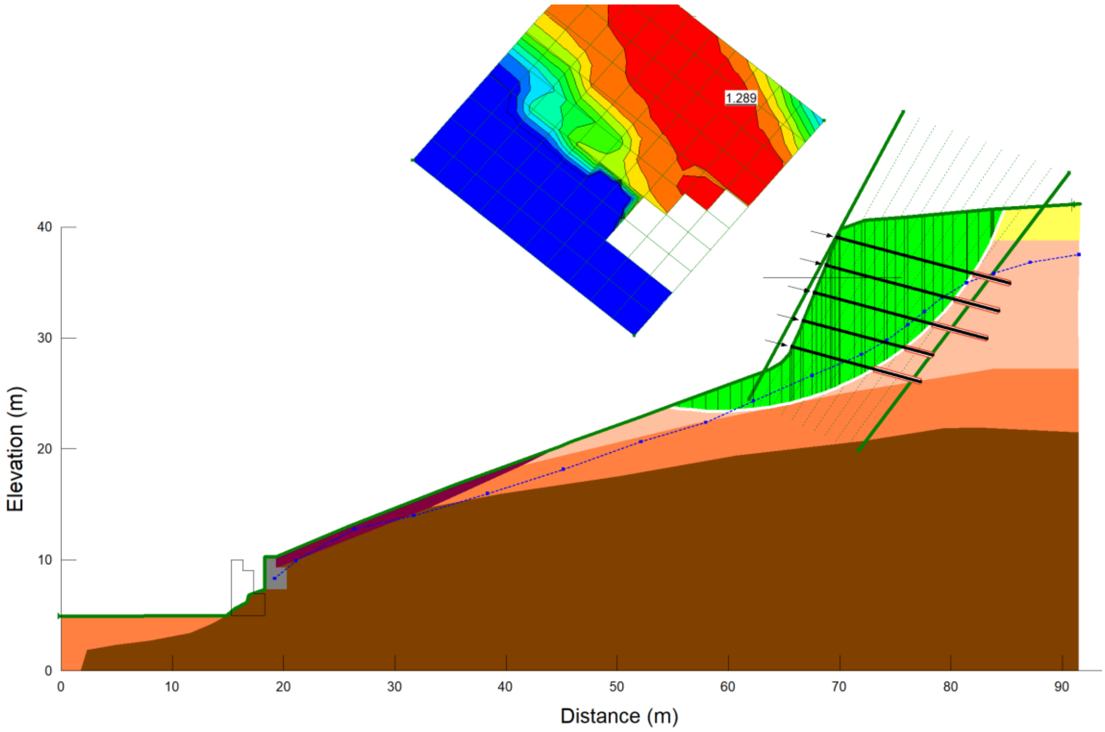


Figure 3-7. Stability analysis soil nails at 2.5m - CS5 – Hakgala landslide

The nail forces for this arrangement are presented in Table 3-3.

Table 3-3. Soil nail forces obtained from the stability analysis (2.5 m x 2.5 m arrangement)

Nail No. (from top)	Factored Pullout Force (kN/m)	Factored tensile capacity (kN/m)	T_{max} (kN/m)	T_{max} (kN) (x S_H)
1	21.40	70	21.40	53.50
2	34.40	70	34.40	86.00
3	58.65	70	58.65	146.62
4	40.75	70	40.75	101.87
5	68.34	70	68.34	170.85

With this arrangement with increased nail spacing, the forces on the nails are increased.

As such, for facing design $T_{\max}$ will be taken as **170.85 kN.**

Accordingly, the nailhead force using Equation (2.7):

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

$$= 170.85 \times [0.6 + 0.2 \times (2.5 - 1)]$$

$$\underline{\underline{= \mathbf{153.76\ kN}}}$$

- Taking $S_V = S_H = 1.5\ m$: **(FOS 1.401)**

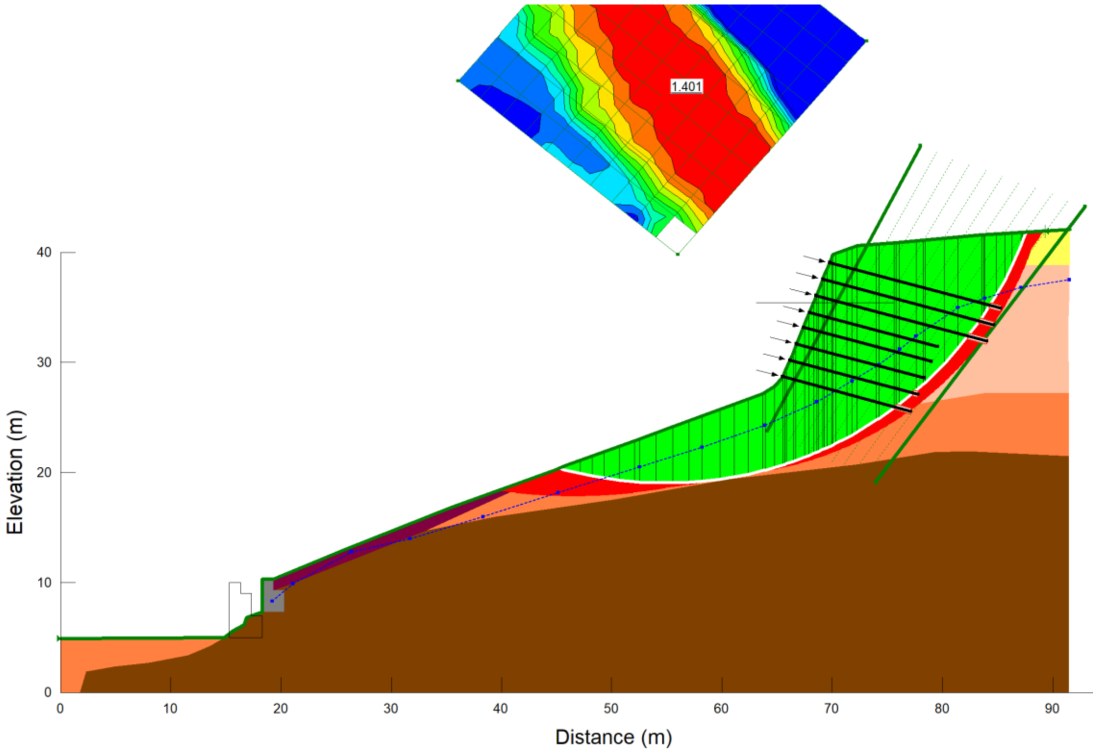


Figure 3-8. Stability analysis with Sv=1.5 m - CS5 – Hakgala landslide

Similarly, from Slope/W analysis:

$$T_{max} = 99.80\ \text{kN}$$

Accordingly, the nailhead force using Equation (2.7):

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

$$= 99.80 \times [0.6 + 0.2 \times (1.5 - 1)]$$

$$\underline{\underline{= \mathbf{69.86\ kN}}}$$

The summary of the analysis results is as follows:

Table 3-4. Variation of Nailhead Load with Nail Spacing

Vertical spacing (S_V) (m)	Horizontal spacing (S_H) (m)	FOS	T_{max} (kN)	T_o (kN)
2.5	2.5	1.289	170.85	153.76
2	2	1.335	145.34	116.27
1.5	1.5	1.401	99.8	69.86
2.5	2	1.324	170.86	136.69
1.5	2	1.372	164.62	115.23

The overall stability of the slope (FOS) after mitigation should be raised to an acceptable value on a site-based risk analysis. For Hakgala landslide mitigation, $FOS \geq 1.3$ has been deemed acceptable.

The minimum acceptable FOS after rectification of 1.324 was achieved for the nailing arrangement with $S_V = 2.5\ m$ and $S_H = 2.0\ m$. However, it is seen that the nailhead load of this arrangement is 17% higher than that of $S_V = 2.0\ m$ and $S_H = 2.0\ m$ spacing arrangement which also yields an acceptable FOS. Moreover, the structural integrity of a given facing with $S_V = S_H = 2\ m$ arrangement will be greater than that of $S_V = 2.5\ m$ and $S_H = 2.0\ m$ arrangement as discussed in Section 2.3. The $S_V = 2.5\ m$ and $S_H = 2.0\ m$ arrangement results in a relatively more expensive and heavier facing design than the other. As such, the nail spacing arrangement for the most economical and lighter facing design could be taken as $S_V = 2.0\ m$ and $S_H = 2.0\ m$.

It should be also noted that since the nailhead values have been obtained from limit equilibrium analysis, they could be considered as the ultimate value. As such, factors discussed in Section 2.4.6 need not be used on these nailhead values.

3.3.3 Evaluation of nailhead forces for different methods and slope conditions

Nailhead load using FHWA (1998) method

As discussed in Section 2.4, the method proposed by FHWA (1998) could also be used to calculate the nailhead force of the above study. The nailing arrangement used in the case history discussed in 3.3.1will be considered for evaluating the nailhead load using this method in the following section.

Equation (2.2);

$$K_A = \frac{\sin^2(\theta + \phi')}{\sin^2\theta \sin(\theta - \delta)\left[1 + \sqrt{\frac{\sin(\phi' + \delta) + \sin(\phi' - \beta)}{\sin(\theta - \delta) + \sin(\theta + \beta)}}\right]^2}$$

θ	Angle of the slope face from outside	= 180° − 66° = 114°
δ	Interface friction between slope material and nailhead material	= 21.7°
β	Back slope angle	= 5°
ϕ'	Friction angle of the slope material	= 28°

$$K_A = \frac{\sin^2(114 + 28)}{\sin^2 114 \sin(114 - 21.7)\left[1 + \sqrt{\frac{\sin(28 + 21.7) + \sin(28 - 5)}{\sin(114 - 21.7) + \sin(114 + 5)}}\right]^2}$$

$$= 0.181$$

Equation (2.1);

$$T_0 = F_F K_A \gamma H S_H S_V$$

$$= 0.5 \times 0.181 \times 18 \times 13 \times 2 \times 2$$

$$= 84.71\ kN$$

As discussed in Section 2.4.6 factor for calculating the ultimate nailhead load could be taken as **1.35**;

$$T_0, ult = 1.35 \times 84.71$$

$$= \mathbf{114.36\ kN}$$

It can be seen that this nailhead value is much similar to the value obtained from the method discussed in 3.3.1.

Similar to the above, the analysis could be done for different nail spacings using the same FHWA (1998) method. The summary of the analysis results is presented in Table 3-5.

Table 3-5. $T_{0,ult}$ values obtained from FHWA (1998) method for different nail spacings

Vertical spacing (S_V) (m)	Horizontal spacing (S_H) (m)	T_o (kN)	T_0, ult (kN)
2.5	2.5	132.45	178.81
2	2	84.71	114.36
1.5	1.5	47.68	64.37
2.5	2	105.96	143.05
1.5	2	63.58	85.83

A comparison of the two methods considered for obtaining the nailhead loads thus far is shown in Table 3-6.

Table 3-6. Comparison of the T_o,ult obtained from the two methods Clouterre (1991) and FHWA (1998)

Vertical spacing (S_V) (m)	Horizontal spacing (S_H) (m)	T_0, ult (kN)	
		Clouterre (1991) based method	FHWA (1998) based method
2.5	2.5	153.76	178.81
2	2	116.27	114.36
1.5	1.5	69.86	64.37
2.5	2	136.69	143.05
1.5	2	115.23	85.83

From the results, it could be concluded that the two methods produce comparable results up to $S_V = S_H = 2\ m$, whereas S_V & S_H go beyond 2m or when $S_V \neq S_H$ the results produced from two methods deviate from one another.

However, the Clouterre (1991) based method could be considered more accurate because the maximum soil nail force has been obtained from a dedicated limit equilibrium model, whereas the FHWA (1998) method largely relies on generalized data obtained from experimental studies as discussed in Section 2.4.2.

Variation of slope angle and friction angle of soil

The purpose of this analysis is to identify how the parameters other than the nail spacing are affecting the nailhead load.

The nailhead loads of the same slope segment at CS5 of Hakgla landslide have been calculated for several excavation cut angles and soils with different friction angles, using the FHWA (1998) method using Equation (2.2) and Equation (2.1). Slope height of 13m, γ=18kN/m^3, Back slope angle $\beta = 5^0$ and $S_V = S_H = 2\ m$ has been used in these calculations, similar to the above method;

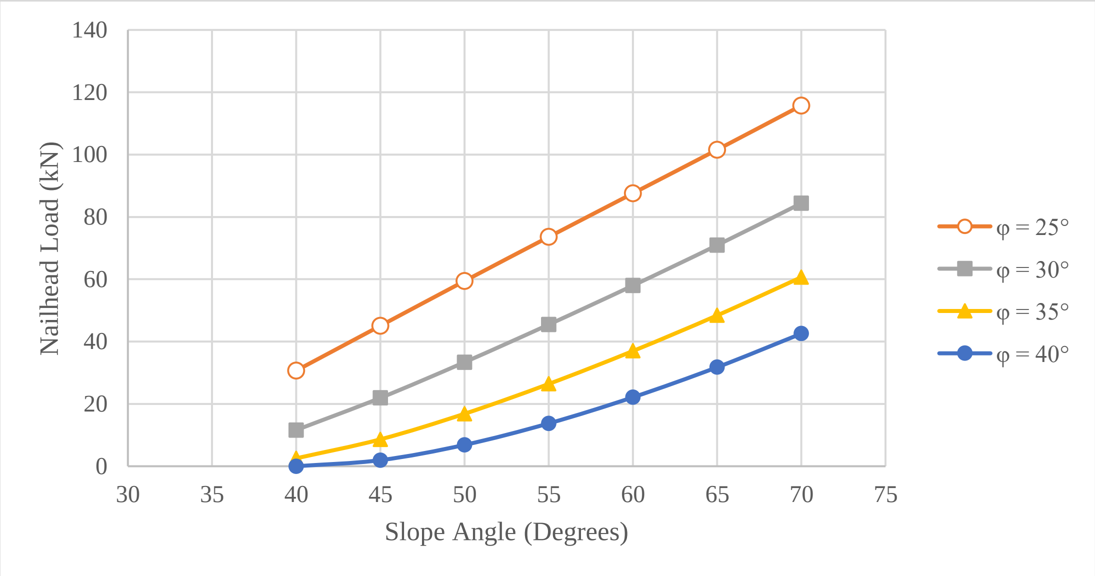


Figure 3-9. Nailhead Load vs Slope angle for different ϕ' values of soil (FHWA 1998 method)

It can be realized that the nailhead load, similar to the active force of a slope is increasing with slope angle and decreasing with a friction angle of soil.

Although it is possible to obtain a chart similar to this could be obtained for the Clouterre (1991) based method, its results could vary based on the soil layering and nail arrangement used.

CHAPTER 4. DESIGN OF SHOTCRETE FACINGS

Different methods of designing shotcrete facings discussed in Chapter 2 are compared in this chapter. The appropriateness of different methods to Sri Lankan conditions is discussed. Eventually, calculation steps are carried out to assess the suitability of a Shotcrete facing and its requirements for a case history in Sri Lanka.

4.1 Background

Design methods for full-face shotcrete facings are directly presented in FHWA (1998/ 2015) manuals as well as the European guidelines CIRIA c637 (2005) and Clouterre (1991) as discussed in Section 2.5. In this chapter, the above approaches are analyzed and the most appropriate approach for local practice is identified, reviewed, and recommended. When choosing the methods, the following facts were mainly considered:

- Compatibility with BS EN codes of practice that are commonly accepted in Sri Lanka
- Common construction methods that have been nurtured to suit Sri Lankan site conditions

4.1.1 Design for flexure

Flexural moments are induced inside shotcrete facings due to the combined action of earth pressure behind the facing and nailhead loads. At the midspan of facing between nailheads, positive moments generate whereas, around the nail heads, negative moments generate. When the lateral earth pressure increases, the bending limit state is reached, and a yielding mechanism initiates forming plastic hinges after the facing starts to deform.

As described in both Clauterre 1991 and other guides, the mechanism of shotcrete facing interaction can be idealized by a system of a loaded flat slab supported by columns. Except, in this case, the nail head load of soil nails is equivalent to the axial load of the columns. Both methods used in FHWA (1998) and CIRIA c637 (2005) could be readily used for designing the facing. As such, a comparison between the two methods is carried out as follows to understand the key differences between the two methods.

4.1.2 Design for shear

Punching shear becomes the more critical mode of shear failure in typical shotcrete-facing systems. Where the nailhead connects to the facing, a shear zone will be formed which in turn causes punching shear failure. However, line shear failure checks are usually carried out around the nailhead.

As discussed in 2.5, all soil nail design manuals including Cluterre (1991), FHWA (1998), and CIRIA (2005) guide carrying out calculations for shear capacity of shotcrete facings. However, only the latter two provide comprehensive guidance step by step. However, as discussed, Clouterre (1990) has presented important facts about safety margins and limit states for designing for shear as well.

4.2 Proposing a procedure for the design of shotcrete facings

Having studied the two major methods proposed in the literature for the structural design of Shotcrete facings, it is clear that both methods rely on the same principle. It is understood that the CIRIA c637 (2005) presents the most familiar British design practices as discussed before in 2.5.3. However, the method used to calculate the ultimate resistance moment and ultimate shear strength of concrete in 2.5.3, in particular, is slightly different from the methods given in the more familiar BS 8110: Part 1. Besides, both methods use an allowable limit approach which is not commonly promoted in modern structural designs.

As such, as a part of this study, a new set of calculations for the structural design of shotcrete is presented purely based on BS 8110:1997: Part 1. These steps are based on Clause. 3.7 Design of flat slabs in BS 8110:1997: Part 1 that resembles the loading mechanism of shotcrete facing very closely and covers design for flexure, shear, as well as all structural checks permitted by the British code of practice.

Basic parameters for design

1. Acquire the parameters of the soil nailed slope

Slope angle	β
Vertical and Horizontal nail spacings	S_H, S_V
Calculate spacing along slope face	$L = S_V/sin\beta$
Angle of soil nail	α

2. Obtain the nailhead load T_0 using the method recommended in Section 2.4.5
3. Predefine Shotcrete section properties and reinforcement amounts

Section thickness	d	No. of main bars	N
Minimum cover	C	Dia. of main bars	D
Minimum cover at buried face	C_b	Mesh diameter	D_m
Concrete strength	f_{cu}	Mesh spacing	S_m
Rebar tensile strength	f_y		
GI mesh tensile strength	f_{ym}		

Calculate Area of tor steel	$A_{s\ prov\ 1} = \pi\left(\frac{D}{2}\right)^2 \times N$
Calculate Area of mesh steel along the short span	$A_{s\ prov\ 2} = \pi\left(\frac{D_m}{2}\right)^2 \times \frac{\max(S_H, S_V)}{S_m}$
Calculate Area of mesh steel along the long span	$A_{s\ prov\ 2}' = \pi\left(\frac{D_m}{2}\right)^2 \times \frac{\min(S_H, S_V)}{S_m}$
Calculate Effective depth at nailhead	$d_{effn} = d - C_b - D/2$
Calculate Effective depth at midspan	$d_{effm} = C_b + \frac{D}{2}$

4. Calculate the load per unit area behind the facing

$$n = T_0 sin(\alpha + \beta)/(S_H \times S_V)$$

Design for flexure using BS 8110: Part 1

5. Calculate design flexure assuming a two-way spanning slab

$$M_x = \beta_{sx} n l_x^2$$
$$M_y = \beta_{sy} n l_x^2$$

BS 8110-1, Cl. 3.5.3.4

Calculation shall be done at nailhead and midspan for both larger spacing and smaller spacing. 4-moment values shall be obtained.

Where:

Smaller nail spacing l_x

Larger nail spacing	l_y	Typically equals the spacing along slope face $L = S_V/sin\beta$

β_{sx} and β_{sy} could be obtained from BS 8110-1, Table 3.14, for the interior panel; for nailhead and midspan accordingly.

6. Check for the requirement for compression reinforcement

$$k = M/bd^2 f_{cu}$$ *BS 8110-1, Cl. 3.4.4.4*

Where $k < 0.156$ compression reinforcement not necessary

7. Calculate the tension reinforcement requirement for each location, nailhead, and midspan for larger and smaller spans

$$A_{s\ req} = M/(0.87f_y \times z)$$ Where;

$$z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right)$$ *BS 8110-1, Cl. 3.4.4.4*

And check $A_{s\ req} < A_{s\ prov}$ criteria

8. Check minimum reinforcement requirement

$$A_{s\ min} = 0.13\%bd$$ *BS 8110-1, Table 3.25*

And check $A_{s\ min} < A_{s\ prov}$ criteria

<u>Design for shear using BS 8110: Part 1</u>

Design for line shear around nailhead

9. Predefine additional section properties necessary for the design for shear

Base plate width	b_p	
Calculate the base plate perimeter	u_o	$= b_p \times 4$
Base plate thickness	t_p	
Depth of beam below the base plate	d_p	$= d - C$
Effective depth at nailhead	d_{effp}	$= d_p - C_b - 0.5D$

10. Calculate effective shear force from the nail

$$V_{eff} = 1.15 \times V$$ Where;

$$V = T_o \times \sin\ \alpha + \beta$$ *BS 8110-1, Cl. 3.7.6.2*

11. Calculate maximum design shear capacity

$$v_{\max\ all} = 0.8\sqrt{fcu}\ ,or\ 5\ N/mm^2\ if\ less$$ *BS 8110-1, Cl. 3.4.5.2*

12. Calculate design shear stress at bearing plate

$$v_{\max} = \frac{V_{eff}}{u_o d_{effp}} \qquad \textit{BS 8110-1, Cl. 3.7.7.2}$$

And check $v_{\max} < v_{\max\ all}$ criteria

Design for punching failure of the nail-facing connection

13. Calculate design concrete shear strength v_c as per BS 8110-1, Cl. 3.7.7.4 and Table 3.8
14. Calculate force from the soil at a nailhead

$$F_{sh} = n \times A_{1.5} \qquad \text{Where;}$$
$$A_{1.5} = Area\ of\ perimeter\ 1.5d\ from\ plate$$

15. Calculate resultant force resisted by facing

$$F_{res} = V_{eff} - F_{sh}$$

16. Calculate design shear at 1.5d from loaded area

$$v_{1.5} = \frac{F_{res}}{u_{1.5} \times d_{effp}} \qquad \text{Where;}$$
$$u_{1.5} = Perimeter\ 1.5d\ from\ loaded\ area$$

And check $v_{1.5} < v_c$ criteria.

The adequacy of the parameters defined in step 3 shall be assessed at each relevant step above and the parameters must be readjusted accordingly.

4.3 Application for a Case History

4.3.1 Shotcrete facing design for Hakgala landslide mitigation project using the proposed method

Continuing from Section 3.3 analysis could be carried out to access the applicability of a Shotcrete facing for the Hakgala Landslide mitigation project’s CS5 area. Additionally, calculations are performed to determine the required sectional and reinforcement properties for the area in question as per the recommended design procedure in Section 4.2. The soil nail arrangement was also considered as $S_V = S_H = 2\ m$, as determined most suitable in the preceding chapter.

1. Parameters of the soil nailed slope

Slope angle β $= 66^0$

Vertical and Horizontal nail spacings	S_H	= 2 m
	S_V	= 2 m
Calculate spacing along slope face	$L = S_V / sin\beta$	= 2.19 m
Angle of soil nail	α	$= 10^0$

2. Nailhead Load

$T_0 = 116.27\ kN$ From Section 3.3

3. Shotcrete section properties and reinforcement amounts

Section thickness	d	$= 150\ mm$
Minimum cover	C	$= 30\ mm$
Cover at buried faces	C_b	$= 40\ mm$
Concrete strength	f_{cu}	$= 30\ N/mm^2$
Rebar tensile strength	f_y	$= 460\ N/mm^2$
GI mesh tensile strength	f_{ym}	$= 250\ N/mm^2$
No. of main bars	N	$= 2\ nos$
Dia. of main bars	D	$= 16\ mm$
Mesh diameter	D_m	$= 3\ mm$
Mesh spacing	S_m	$= 50\ mm$
Calculate Area of tor steel	$A_{s\ prov\ 1}$	$= 402\ mm^2$
Calculate Area of mesh steel along the short span	$A_{s\ prov\ 2}$	$= 283\ mm^2$
Calculate Area of mesh steel along the long span	$A_{s\ prov\ 2}'$	$= 283\ mm^2$
Calculate Effective depth at nailhead	d_{effn}	$= 86\ mm$
Calculate Effective depth at midspan	d_{effm}	$= 48\ mm$

4. Calculate the load per unit area behind the facing

$$\begin{aligned} n &= T_0 sin(\alpha + \beta)/(S_H \times S_V) \\ &= 116.27 \times sin(15 + 66)/(2 \times 2) \\ &= 26.23\ kPa \end{aligned}$$

Design for flexure

5. Design flexure at the critical location (at midspan along the shorter spacing)

$$\begin{aligned} M_x &= \beta_{sx} n l_x^2 \\ &= 0.028 \times 26.23 \times 2^2 \times 2.19 \quad \textit{BS 8110-1, Table 3.14} \\ &= 6.43\ kNm \end{aligned}$$

6. The requirement for compression reinforcement

$$k = M/bd^2 f_{cu} \qquad \textit{BS 8110-1, Cl. 3.4.4.4}$$
$$= \frac{6.43 \times 10^6}{2.19 \times 48^2 \times 460}$$
$$= 0.042 < 0.156 \qquad \textit{BS 8110-1, Table 3.6}$$

$k < 0.156$, compression reinforcement not necessary

7. Reinforcement requirement at the critical location (at midspan along the shorter spacing)

$$z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right) \qquad \textit{BS 8110-1, Cl. 3.4.4.4}$$
$$= 48\left(0.5 + \sqrt{0.25 - \frac{0.042}{0.9}}\right) \qquad = 45.61\ mm$$
$$A_{s\ req} = M/(0.87 f_y \times z)$$
$$= 6.43/(0.87 \times 460 \times 45.61)$$
$$= 352.28\ mm^2$$
$$A_{s\ prov} = A_{s\ prov\ 1} + A_{s\ prov\ 2}{}'$$
$$= 402 + 283 \times 250/460 \qquad \textit{Equivalent tor steel area}$$
$$= 556\ mm^2 > A_{s\ req} \qquad \textit{OK}$$

8. Minimum reinforcement requirement

$$A_{s\ min} = 0.13\% bd \qquad \textit{BS 8110-1, Table 3.25}$$
$$= 0.0013 \times 2 \times 150 \times 10^6$$
$$= 390\ mm^2$$

$A_{s\ \min} < A_{s\ prov} = 556\ mm^2$, minimum tension reinforcement requirement satisfied.

Design for shear

<u>Design for bearing failure around nailhead</u>

9. Section properties necessary for the design for shear

Base plate width	b_p	$= 200\ mm$
Calculate the base plate perimeter	u_o	$= 800\ mm$
Base plate thickness	t_p	$= 12\ mm$
Depth of section below nailhead	d_p	$= 120\ mm$
Effective depth at nailhead	d_{peff}	$= 120 - \frac{16}{2} - 40$
		$= 72\ mm$

10. Calculate effective shear force from the nail

$$V = T_o \times \sin(\alpha + \beta)$$
$$= 116.27 \times \sin(15 + 66)$$
$$= 114.84\ kN$$
$$V_{eff} = 1.15 \times V \qquad \textit{BS 8110-1, Cl. 3.7.6.2}$$
$$= 132.06\ kN$$

11. Maximum design shear capacity

$$v_{\max\ all} = 0.8\sqrt{fcu}\ ,or\ 5\ N/mm^2\ if\ less \qquad \textit{BS 8110-1, Cl. 3.7.6.4}$$
$$= 4.38\ N/mm^2$$

12. Design shear stress at bearing plate

$$v_{\max} = \frac{V_{eff}}{u_o d_{peff}} \qquad \textit{BS 8110-1, Cl. 3.7.7.2}$$
$$= \frac{132.06}{800 \times 72}$$
$$= 2.29\ N/mm^2$$

$v_{\max} < v_{\max\ all} = 4.38\ N/mm^2$, bearing failure around nailhead criteria is satisfactory.

<u>Design for punching failure of the nail-facing connection</u>

13. Design concrete shear strength

$$b_v = 3 \times d_{peff} + b_{PL} \qquad = 3 \times 72 + 200$$
$$= 366\ mm \qquad \text{BS 5400:Part 4:1990}$$
$$A_{sv} = A_{s1} + A_{s2} \times b_v/S_H$$
$$= 416\ mm^2$$

Accordingly;

$$v_c = 1.17 N/mm^2 \qquad \text{BS 8110-1, Cl. 3.7.7.4 and Table 3.8}$$

14. Force from the soil at the nailhead

$$A_{1.5} = Area\ of\ perimeter\ 1.5d\ from\ plate$$
$$= 0.173\ m^2$$
$$F_{sh} = n \times A_{1.5}$$
$$= 26.23 \times 0.173$$
$$= 4.54\ kN$$

15. Calculate resultant force resisted by facing

$$F_{res} = V_{eff} - F_{sh}$$

$$= 132.06 - 4.54$$
$$= 127.53\ kN$$

16. Calculate design shear at 1.5d from loaded area

$$u_{1.5} = 1664\ mm$$
$$v_{1.5} = \frac{127.53}{1664 \times 72}$$
$$= 1.06\ N/mm^2$$

$v_{1.5} < v_c = 1.17\ N/mm^2$, punching shear around nailhead criteria is satisfactory.

As such, Full-Face Shotcrete facing with 150mm thickness, 2 x 16mm dia. main bars, and a 3mm dia. wired 50mmx50mm GI mesh and a 200 mm x 200 mm bearing plate can be satisfactorily used as the soil nail facing at CS5 area of Hakgala Landslide Project.

It was noted that that the most critical mode of failure was the punching shear at 1.5d from the loaded area. As such, several critical improvements to optimize the current design could be proposed.

4.3.2 Design of shotcrete facing for different nailing arrangements

Other than the above parameters used, the design could also be altered to suit variations in soil nailing. The same method used in Section 4.3.1 for $S_V = S_H = 2\ m$ could be used to assess the nailhead loads at different nail spacings. Accordingly, the practical minimum structural requirements for each case are obtained and tabulated. The nailhead values for the calculations were obtained from the calculation presented in Section 3.3.1.

Table 4-1. Summary of possible Shotcrete designs for alternative nail spacings

Vertical spacing (S_V) (m)	Horizontal spacing (S_H) (m)	T_{oULT} (kN)	Facing thickness (mm)	Main reinforcement	Bearing plate size (mm)
2.5	2.5	153.76	175	2Y16	175x175
2	**2**	**116.27**	**150**	**2Y16**	**200x200**
1.5	1.5	69.86	125	2Y10	200x200
2.5	2	136.69	150	2Y16	250x250
1.5	**2**	**115.23**	**150**	**2Y16**	**150x150**

It can be observed that the maximum nail spacing, be it vertical or horizontal, has a clear impact on the structural requirement of the spacing. This can be identified from the 2 m x 2 m arrangement and the 1.5 m x 2 m nail arrangement. As such, reducing only the vertical spacing, to reduce facing requirement is not effective. On the other hand, reducing both spacings will increase the soil nailing cost and the design would be highly uneconomical. As such, using $S_V = S_H$ arrangement and staying inside the recommended spacing values as given in Section 2.3 is recommended.

4.3.3 Design of Shotcrete facing using alternative methods

Additionally, the alternative methods of design available in the literature discussed in Section 2.5 could also be used to carry out the design of the Hakgala landslide mitigation project - CS5 area.

Design for flexure using FHWA (1998) method

The design parameters and abbreviations used are the same as Section 4.3.1 unless otherwise stated. Calculations have to be carried out for both the midspan of the section as well as the nailhead for the flexural capacity of the section, unlike the previous method.

Equation (2.13) for the midspan;

$$m = \frac{A_s F_y}{b}\left(d - \frac{A_s F_y}{1.7 f_c' b}\right) = \frac{556 \times 460}{2000} \times \left(48 - \frac{556 \times 460}{1.7 \times 30 \times 2000}\right)$$

$$m_{vm} = m_{hm} = 5.82\ kN.m/m$$

Similarly, for the nailhead area;

$$m = \frac{A_s F_y}{b}\left(d - \frac{A_s F_y}{1.7 f_c' b}\right) = \frac{556 \times 460}{2000} \times \left(86 - \frac{556 \times 460}{1.7 \times 30 \times 2000}\right)$$

$$m_{vn} = m_{hn} = 10.67\ kN.m/m$$

Flexural resistance of the facing:

The flexural resistance of the shotcrete shall be the minimum of calculated vertical and horizontal resistance values. However, in this case, where $S_V = S_H$, both horizontal and vertical values are the same.

Equation (2.12);

$$R_{FF} = C_F \times \; m_m + m_n \; \times \frac{8 \; S_H}{S_V}$$

$$= \left(1 \times \; 5.82 + 10.67 \; \times \frac{8 \; 2}{2}\right) \qquad C_F = 1; \textit{Permanent facing}$$

$$= 131.91 \; kN$$

Application of factors for design as discussed in Section 2.4.6 and Equation (2.8);

Resistance factor for bending/flexure in the facing ϕ_F= 0.9. As such;

$$R_{FF,ult} = 0.9 \times 131.91$$

$$= \mathbf{118.72} \; \boldsymbol{kN}$$

As per the FHWA (1998) method, the ultimate flexural resistance is adequate to cater to the induced ultimate nailhead load perpendicular to the facing, $T_{0,eff}$;

$$T_{0,eff} = T_0 \times \sin(\delta + \beta)$$

Where;

T_0	Nailhead load
δ	Angle of soil nail to the horizontal
β	Slope angle to the horizontal

Substituting

$$T_{0,eff} = 116.27 \times \sin(15 + 66)$$

$$= 113.81 \; kN \leq \; R_{FF,ult} = 118.72 \; kN$$

Design for flexure using CIRIA c637 (2005) method

- Flexural capacity of the section

Equation (2.18) for the midspan;

$$z = \left(1 - \frac{1.1 f_y A_s}{f_{cu} b d}\right) d = \left(1 - \frac{1.1 \times \; 460 \times \; 556}{30 \times 2000 \times 48}\right) \times 48$$

$$= 43.31 \; mm$$

$$0.95d = 0.95 \times 48 = 45.6 \; mm$$

$$z \leq 0.95d$$

Equation (2.17);

$$m = (0.87 f_y) A_s z = (0.87 \times 460 \times 556 \times 43.31)$$

$$m_{vm} = m_{hm} = 9.64\ kNm = 4.82\ kN.m/m$$

Equation (2.18) for the nailhead area;

$$z = \left(1 - \frac{1.1 f_y A_s}{f_{cu} b d}\right) d = \left(1 - \frac{1.1 \times 460 \times 556}{30 \times 2000 \times 86}\right) \times 86$$

$$= 81.31\ mm$$

$$0.95d = 0.95 \times 86 = 81.70\ mm$$

$$z \leq 0.95d$$

Equation (2.17);

$$m = (0.87 f_y) A_s z = (0.87 \times 460 \times 556 \times 81.31)$$

$$m_{vn} = m_{hn} = 18.08\ kN.m = 9.04\ kN.m/m$$

- Flexural resistance of the facing:

Equation (2.16);

$$w = \min \begin{bmatrix} \dfrac{m_{vn}}{(0.031 \times S_H^2)} & or & \dfrac{m_{vm}}{(0.024 \times S_H^2)} \\ \dfrac{m_{hn}}{(0.031 \times S_H^2)} & or & \dfrac{m_{hm}}{(0.024 \times S_H^2)} \end{bmatrix}$$

$$w = \min \begin{bmatrix} \dfrac{9.04}{(0.031 \times 2^2)} & or & \dfrac{4.82}{(0.024 \times 2^2)} \\ \dfrac{9.04}{(0.031 \times 2^2)} & or & \dfrac{4.82}{(0.024 \times 2^2)} \end{bmatrix}$$

$$= 50.18\ kN/m^2$$

$$R_{FF} = w \times S_H \times S_V = 50.18 \times 2 \times 2$$

$$= \mathbf{200.71\ kN}$$

- The ultimate moment capacity of the facing:

It should be noted that the obtained R_{FF} value is the ultimate value as partial factors are embedded in the expressions. Therefore, no modifications required;

$$R_{FF,ult} = R_{FF} = \mathbf{200.71\ kN}$$

As such, as per the CIRIA c637 (2005) method, the flexural resistance is adequate to cater to the induced ultimate nailhead load perpendicular to the facing of **113.81 kN.**

Design for punching using FHWA (1998) method

Punching capacity of the section using Equation (2.14);

V_N	$= 0.33\sqrt{f'c}\ \pi\ (D'_C)\ h_C$	Where;
h_c	$= 2 \times d_{effp}$	$= 144\ mm$
D'_C	$= b_{PL} + h_C$	$= 200 + 144$
	$= 344\ mm$	
V_N	$= 0.33\sqrt{30}\ \pi\ \ 344\ \ 120$	$= 234.40\ kN$

Nailhead resistance for punching using Equation (2.15);

$T_{FN} = R_{FP}$	$= V_N\left(\dfrac{1}{1 - C_S\ A_C - A_{GC}\ /\ S_V S_H - A_{GC}}\right)$	Where;
A_c	$= \dfrac{\pi D_C^2}{4}$	$= \dfrac{\pi\ 200 + 120 \times 2\ ^2}{4}$
	$= 0.152\ m^2$	
A_{GC}	$= \dfrac{\pi D_{GC}^2}{4}$	$= \dfrac{\pi \times 125^2}{4}$
	$= 0.0123\ m^2$	

Nailhead resistance against punching is calculated using Equation (2.15):

$$R_{FP} = 234.40 \times \frac{1}{1 - 1 \times (0.152 - 0.0123)/(2 \times 2 - 0.0123)}$$

$$= 242.92\ kN$$

As per Section 2.4.6;

$$R_{FP,ULT} = 0.9 \times R_{FP} \qquad = \mathbf{218.63\ kN}$$

As such, as per the FHWA (1998) method, the ultimate punching resistance is adequate to cater to the induced ultimate nailhead load perpendicular to the facing of **113. 81 *kN*.**

Design for punching using CRIA c637 (2005) method

Calculating the ultimate shear strength of concrete using (2.20);

v_c	$= \dfrac{0.27}{\gamma_m}\left(\dfrac{100A_s}{b_w d}\right)^{1/3}\ f_{cu}\ ^{1/3}$	Where;
b_w	$= 3 \times d + b_{PL}$	$= 3 \times 72 + 200$

$= 416\ mm$; BS 5400:Part 4:1990

$A_{sv} = A_{s1} + A_{s2} \times b_w / S_H$

$= 434.09\ mm^2$

$$v_c = \frac{0.27}{1.25} \times \left(\frac{100 \times 434.09}{416 \times 72}\right)^{\frac{1}{3}} \times 30^{\frac{1}{3}}$$

$= 0.7595$

From Equation (2.21);

$\xi_s = 500/d^{\ 1/4}$ or 0.70 ; Whichever is greater

$$= \left(\frac{500}{72}\right)^{\frac{1}{4}}$$ *$d = d_{effp}$ as calculated before*

$= 1.62$

Obtain nailhead resistance from Equation (2.19);

$R_{FP} = [4 \times \xi_s v_c \times\ L_{HB} + h\ \times d]/\ 1000$

$= [4 \times 1.62 \times 0.759 \times\ 200 + 120\ \times 72]/\ 1000$

$R_{FP,ULT} = \mathbf{113.63\ kN}$

As such, unlike the proposed BS 8110 based method, as per the CIRIA c637 method, the ultimate punching resistance of the full-Face Shotcrete facing with 150 mm thickness, 2 x 16 mm dia. main bars, and a 3mm dia. wired 50mmx50mm GI mesh and a 200 mm x 200 mm bearing plate is not adequate to cater to the induced ultimate nailhead load perpendicular to the facing of **113. 81 *kN*.**

Conclusion on the outcomes of the three design approaches

Design for Flexure

Presented in Figure 4-1 is a comparison of the flexural resistance, R_{ff} outputs of the shotcrete facing system used in the case history when installed on soil nail systems with different spacings. The difference between the outcomes of the methods used above can be identified from this illustration. Additionally, the effect of nail spacing on the variation of nailhead load, T_0, and the resistance against flexure, R_{ff} can be identified by the illustration. At a given soil nail spacing, the R_{ff} value could be compared against the T_0 values to check the adequacy of the facing design.

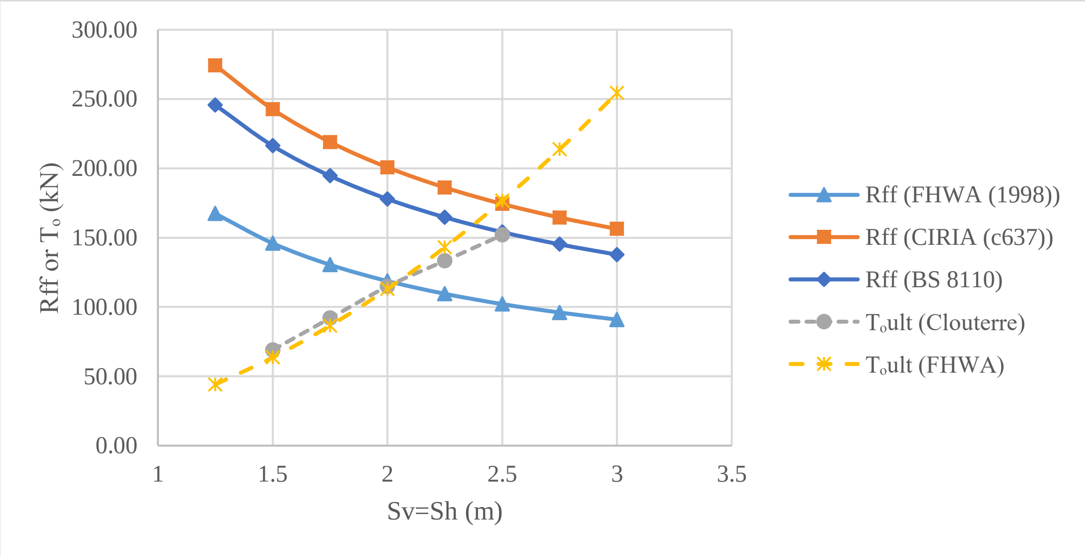


Figure 4-1. Flexural resistance, $T_o$ult vs Nail Spacing for the Shotcrete section considered for CS5 of Hakgala landslide mitigation

However, since the proposed BS 8110 based method does not use an allowable nailhead load approach, one cannot directly compare its results with the flexural resistance results of the FHWA (1998) and CIRIA c637 methods. As such, a back-calculation to determine the maximum flexural resistance has been carried out and illustrated in the same chart along with the results of the other two approaches. The nailhead loads calculated in Chapter 3 have also been added to the chart for comparison.

The flexural resistance of the facing reduces as the nail grid spacing increases similar to a beam-column system of a building. Thus, the effect of nail spacing is significant in the flexural performances of soil nail facings. As such, adhering to the general rules for spacing discussed in 2.3 for preliminary design is highly recommended.

It is understood that the FHWA (1998) method is using a fundamentally different method to the other two methods for calculating the bending moments at the nailhead and the mid-span. Specifically, FHWA (1998) is assuming the facing as a one-way spanning continuous panel whereas the other two assume a two-way spanning continuous panel. This may be one of the reasons for the lower values of the flexural resistance of the FHWA method obtained in this analysis.

The results obtained suggest that out of the two alternative approaches, the CIRIA method estimates a higher flexural resistance than the FHWAs method, although both results show a similar variation with nail spacing. On the other hand, the proposed BS 8110 based-proposed method has yielded comparable nailhead resistance results with that of the CIRIA approach.

Design for Punching Shear

Presented in Figure 4-2 is a comparison of the punching shear resistance, R_{fp} outputs of the shotcrete facing system used in the case history when installed on soil nail systems with different spacings.

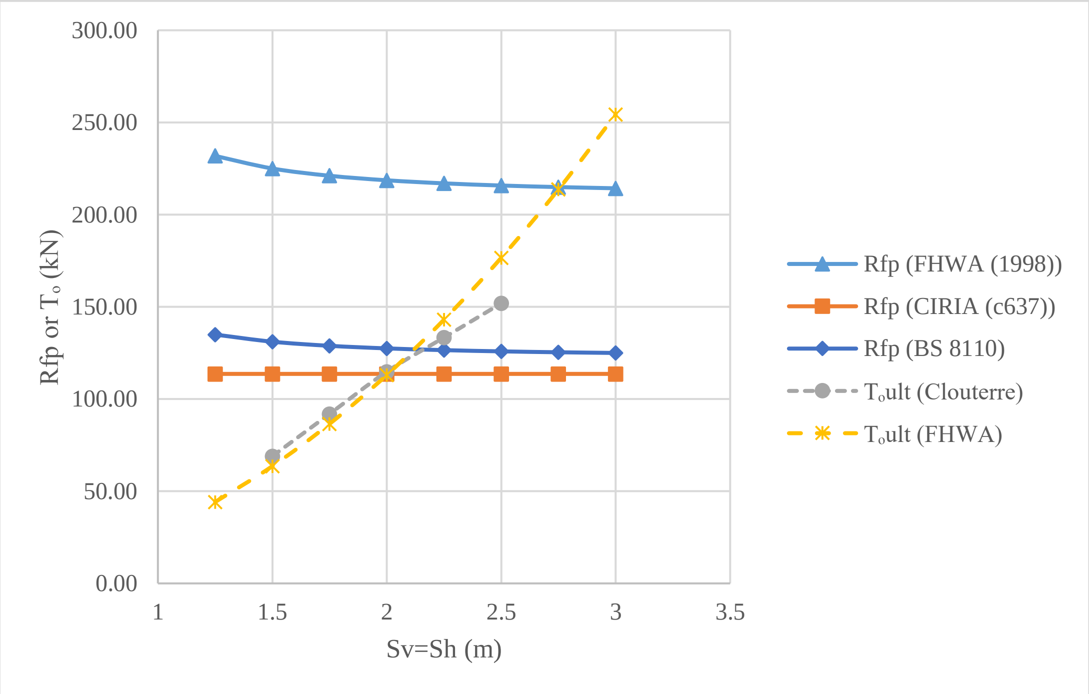


Figure 4-2. Punching shear resistance, $T_o$ult vs Nail Spacing for the Shotcrete section considered for CS5 of Hakgala landslide mitigation

Additionally, the variation of nailhead load, T_0 against the soil nail spacing is also presented in the illustration. At a given soil nail spacing, the R_{fp} value could be compared against the T_0 values to check the adequacy of the facing design.

It can be identified that the nail spacing has a minimal effect on the punching shear resistance, whereas, the bearing plate size and the facing thickness were found to be more sensitive parameters. Unlike in the case of flexure, the CIRIA method appears to produce lower values than that of the FHWA method. This is predominantly due to the fundamental approach of the two methods being somewhat different from each other. This can be identified by the fact that the CIRIA and BS 8110 methods taking the reinforcement ratio into account for its shear capacity calculation, whereas, the FHWA doesn't. Also, FHWA idealizes a circular failure pattern, whereas, the CIRIA and BS 8110 approaches consider a square pattern.

However, similar to in the case of punching, CIRIA, and BS 8110 based methods produce comparable results of punching shear resistance. However, interestingly, the CIRIA method's punching shear resistance values do not appear to vary with spacing.

Additionally, a comparison is made to identify the effect of plate sizing and also the facing thickness. Given in Figure 4-3 is a comparison of the punching shear resistance outputs of the shotcrete face system considered in the case history when used with bearing plates with different plate dimensions, b_p.

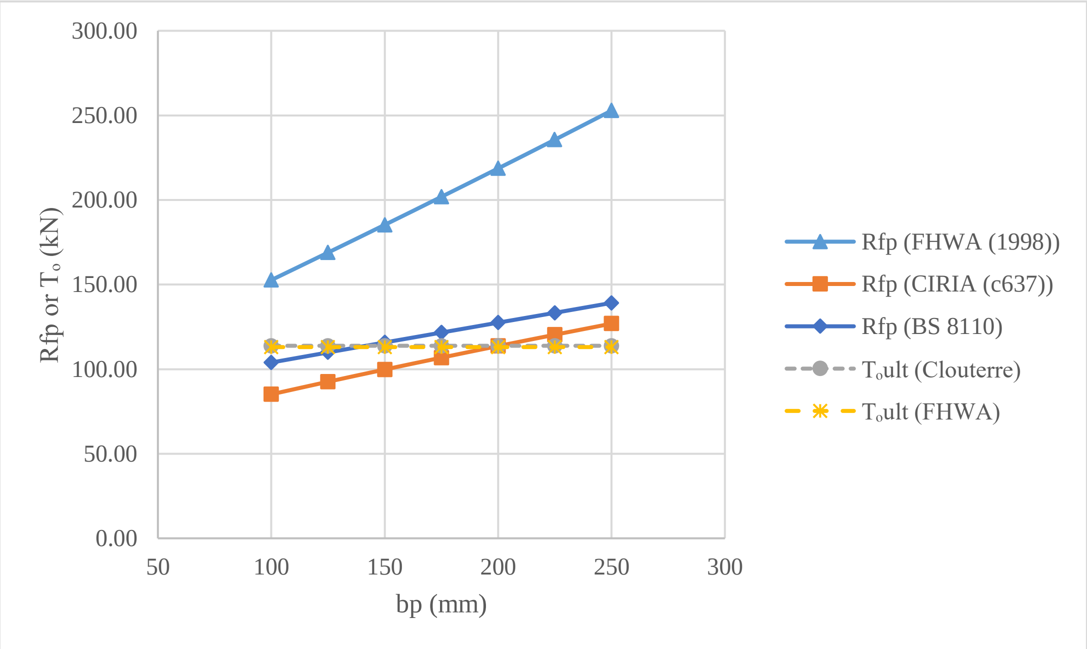


Figure 4-3. Punching shear resistance, $T_o$ult vs Bearing plate size for the Shotcrete section considered for CS5 of Hakgala landslide mitigation

The nailhead load, T_0 does not vary with the bearing plate dimension as presented in the illustration. As such, the R_{fp} value could be compared against this constant, T_0 values to check required bearing plate size, b_p for the facing design.

Unlike with nail spacing, the punching shear resistance of the CIRIA method varies with the plate size used. Also, the variation is comparable with the results of the BS 8110 method.

It must be noted that, in the case of shear, both these alternative methods do not check for the bearing failure around the nailhead. However, the proposed BS8110 based method carries out checks for shear around the nailhead connection and it is seen that this mode of failure could also be critical particularly in the case of facings with smaller plate sizes.

Additionally, the variation of facing resistance against punching shear was also compared with the thickness of the facing, d as shown in Figure 4-4. The facing thickness was found to be the most sensitive parameter to the nailhead resistance in the case of Shotcrete facings.

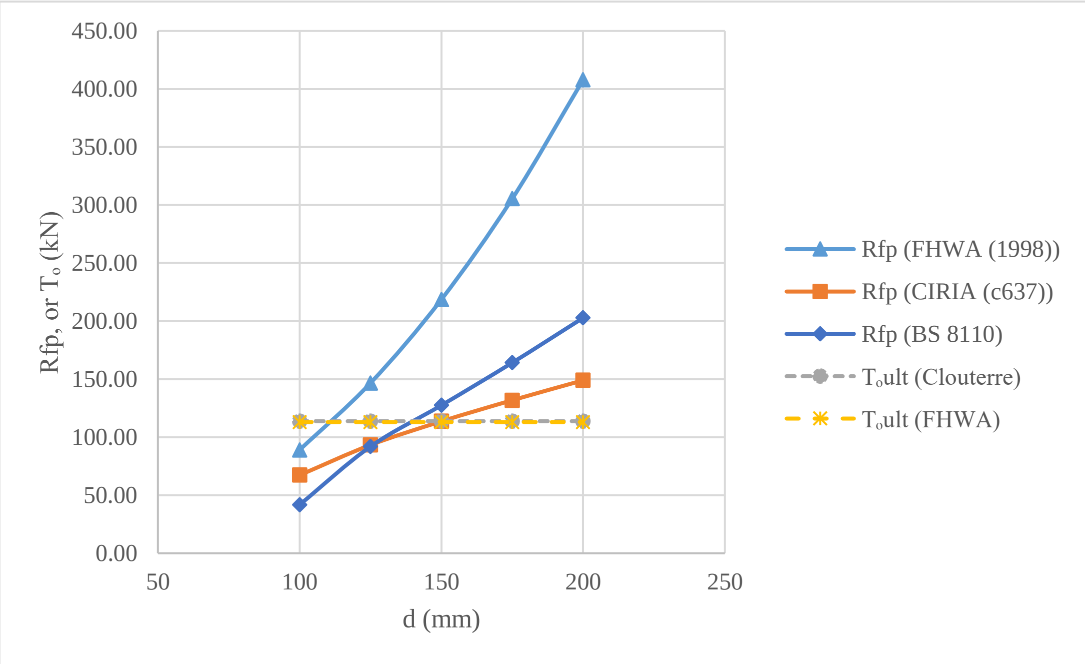


Figure 4-4. Punching shear resistance, $T_o$ult vs Facing thickness for the Shotcrete facing considered for CS5 of Hakgala landslide mitigation

The nailhead load, T_0 does not vary with the facing thickness, d as presented in the illustration. As such, the R_{fp} value could be compared against this constant, T_0 values to check the required facing thickness, d for the facing design.

Moreover, similar to the previous comparison, the BS 8110 based method is producing comparable results to the CIRIA method. Also, the results of the FHWA approach deviate away from the other two, as the facing thickness increases.

<u>Conclusion</u>

In general, the CIRIA c637 in flexure and the FHWA method in shear produces nailhead resistance values that are higher than the other methods. However, in all cases, CIRIA and BS 8110 methods produce comparable results.

However, it must be noted that the outcome of the nailhead resistance must be the lower of flexural and punching resistance values. As such the minimum of the two resistance values, flexure and punching shear at each variation (envelope values of nailhead resistance) were obtained for comparison and presented in Figure 4-5.

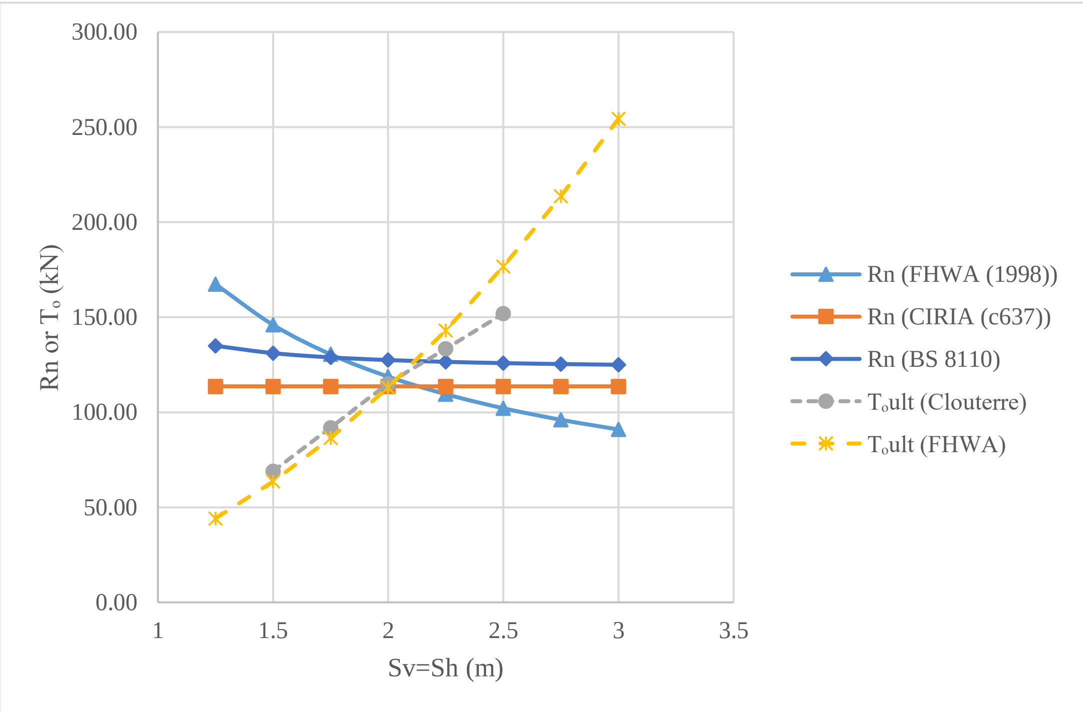


Figure 4-5. Envelope Nailhead resistance, $T_o$ult vs Facing thickness for the Shotcrete facing considered for CS5 of Hakgala landslide mitigation

With these results, the nailhead resistance of FHWA and CIRIA methods could be directly compared to the nailhead resistance obtained from the BS 8110 approach.

There were major differences in results at flexural resistance and punching resistance, between the methods considered. However, from this final comparison, it is evident that the three methods produce similar results despite their fundamental differences.

From the values obtained via the BS 8110 approach, it is evident that the punching shear resistance plays a governing role in the overall structural resistance of full-face shotcrete facings. Further, it is also evident that the punching resistance is more sensitive to the thickness of the facing at and around the nailhead. Thus, it could be proposed to consider full-face shotcrete systems that have increased thickness at the nailhead area compared to the rest as shown in Figure 4-6 to enhance the efficiency of the facing designs. Systems similar to this have been used in Sri Lanka in few instances. However, due to the aesthetic preferences, and ease of construction, most clients prefer shotcrete facings with uniform thickness.

Figure 4-6. Shotcrete facings with strengthened nailhead area

CHAPTER 5. DESIGN OF GRID BEAMS (CONCRETE CRIB)

Only a handful of methods on Grid Beam designs are available in literature as discussed in Chapter 2. However, the design methods available for Shotcrete facings could be modified to develop reasonably accurate design methods for Grid Beam facings. In this chapter, attempts have been made to develop such methods in detail.

Moreover, the most suited Grid Beam facings design method is presented in this chapter based on the findings of this study to suit the usability in Sri Lanka. A case study has also been carried out to assess the applicability of different methods considered in this study.

5.1 Background

A critically reviewed calculation method for Grid Beam structures is presented in the report published by GEO and HKIE for the design of loosely filled slopes. Although design methods for full-face shotcrete facings are presented in FHWA (1998/ 2015) manuals as well as the European guidelines (CIRIA c637), none of them address the Grid Beam applications or designs. As discussed, the Shotcrete facings demonstrate similar characteristics to Grid Beams in structural performance. As such, the adaptability of the FHWA (1998) and CRIA c637 methods for designing Shotcrete to Grid Beams will be assessed in this section.

Ultimately, the above approaches are analyzed and their applicability to local practice is reviewed and recommended. When modifying the methods, similar to the Shotcrete facings the following facts were mainly considered:

- Compatibility with BS EN codes of practice that are commonly accepted in Sri Lanka
- Common construction methods that have been nurtured to suit Sri Lankan site conditions

In grid beam facing systems, 4 failure mechanisms can be identified similar to the Shotcrete facings. Concrete/ shotcrete beams mainly fail in flexure or shear. Flexure and the shearing of the bearing plates are the other 2 failure modes. As discussed in Full-face shotcrete facings, the use of recommended-sized plates can easily prevent plate failures. Bearing failure and punching shear failure at the nailhead is usually checked. However, these failure modes rarely become critical.

5.1.1 Flexural Resistance of Grid Beams

The combined action of lateral earth pressure and the reaction forces of soil nails give rise to flexural moments in the grid beam facing system. In the midspan of beams, positive moments generate whereas, around the nail heads, negative moments generate. Also, the moments which resisted by vertical reinforcement act around a horizontal axis, and the moments which resisted by horizontal reinforcements generate around a horizontal axis. When the lateral earth pressures increase, the bending limit state is reached, the facing is deformed, and a yielding mechanism is produced forming plastic hinges.

The mechanism can be idealized similar to a system of loaded beams supported by columns. Except, in this case, the nail head load of soil nails is equivalent to the axial load of the columns. Nominal flexural resistance is calculated as the resultant of the contributing bending capacities of the facing around its nail heads. Nominal flexural resistance is equal to the nail head force when the equilibrium is reached. Therefore, the flexural resistance capacity must be higher than the potential nail head load.

Principle behind FHWA (1998) method for flexural resistance

Further to the discussions made in the literature survey, an attempt was made to further review the methods proposed by FHWA for realizing the principles behind the expressions published. Accordingly, the expressions used have been derived using the first principles of structural mechanics.

For shotcrete facings, the moment exerted per unit width generated at the nail head and the mid-span which corresponds to the force R_{FF} (nailhead flexural resistance) can be obtained by considering the facing as a continuous beam system of unit width with equal spans. As shown in Figure 5-1, considering an interior span of length L ($= S_v$), the FHWA (1998) equation can be formulated using the approximate methods of calculating the bending moments. It is to be noted that the facing is considered a two-way spanning slab for the calculation of moment distribution.

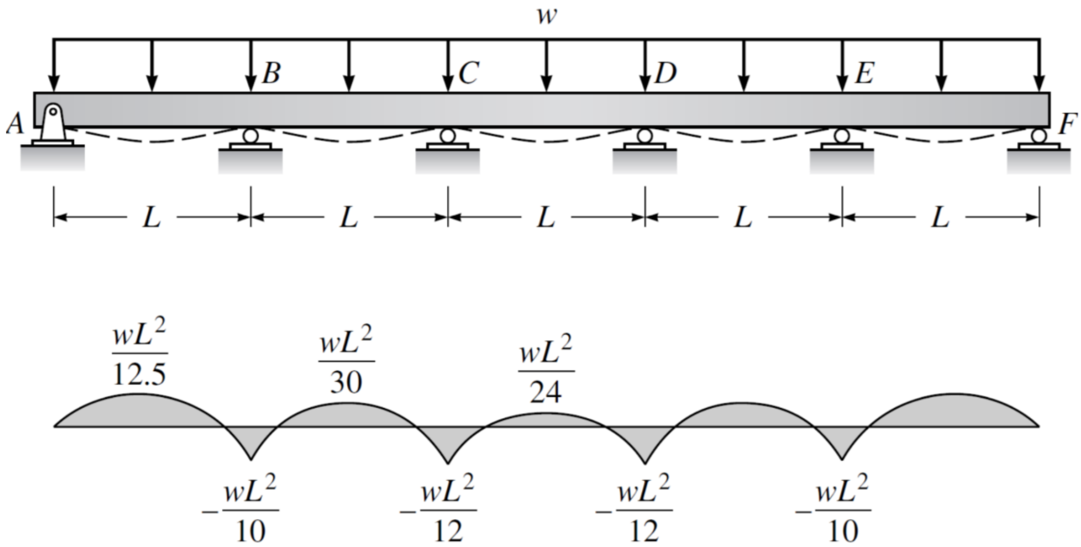


Figure 5-1. Approximate bending moments of a continuous slab

$$m_n = \frac{wS_V^2}{12} \qquad \text{And;} \qquad m_m = \frac{wS_V^2}{24}$$

Addition; $$m_n + m_m = \frac{wS_V^2}{8}$$

Rearrange; $$w = m_n + m_m \times \left(\frac{8}{S_V^2}\right)$$

Where UDL; $$w = \frac{R_{FF}}{S_V \times S_H}$$ (Considering static equilibrium)

$$R_{FF} = w \times S_V \times S_H$$

Substitute for w; $$R_{FF} = m_n + m_m \times \left(\frac{8S_H}{S_V}\right)$$ This is in the form of the FHWA (1998) equation

It is understood that the FHWA has used the factor C_F to modify the corresponding force R_{FF} in the above expression, to address the non-uniform distribution of UDL, w. Recommended values for C_F are presented in Table 2-2.

Substitute w with $C_F \times w$:

$$R_{FF} = C_F \; m_n + m_m \times \left(\frac{8S_H}{S_V}\right) \qquad (5.1)$$

This is the FHWA (1998) formula for flexural resistance of nailhead of a shotcrete facing. The above calculation shall be carried out for both horizontal and vertical directions across a given nailhead and the minimum of the two shall be taken as the flexural resistance.

Having understood the principle behind the FHWA approach for determining nominal nailhead resistance of shotcrete, using the same principles, we can modify the same to derive a set of equations for the grid beams, assuming a distributed load (earth pressure) acting along the beams.

Modifying the FHWA (1998) method for calculating flexural resistance of shotcrete to be used with grid beams:

In the case of grid beams, this equation must be used accordingly, to determine the flexural strength.

As depicted in Figure 5-2, the influence area of the lateral earth pressure is limited to the width of the grid beam and therefore, the crack formation (hinges) is also limited to the width of the grid beams instead of the whole planar surface around a nail.

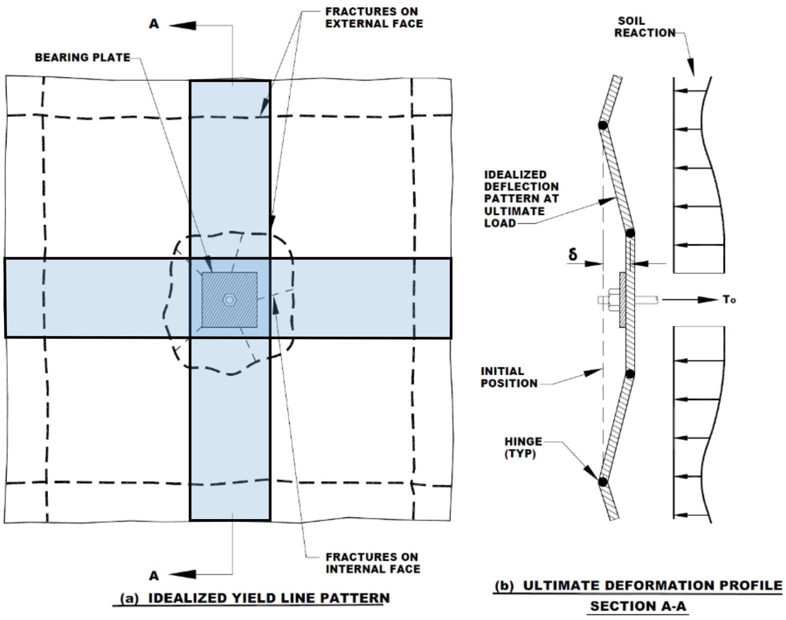


Figure 5-2. Yield pattern of grid beams at the nailhead (modified after FHWA 1998)

However, the generalized assumptions made while generating the FHWA formulae have made them directly applicable to the grid beam systems. A simple calculation following the same steps used to derive the FHWA formula for shotcrete can produce a formula to be used for grid beam design.

Considering the static equilibrium of grid beams:

$$\text{Udl along beams} = \frac{R_{FF}}{S_V + S_H} = w \qquad (5.2)$$

From the approximate methods of calculating bending moments:

$$\text{BM at n, vertical direction, } m_n = \frac{w \times S_V^2}{12} = \frac{R_{FF} \times S_V^2}{12 \times \ S_V + S_H} \qquad (5.3)$$

$$\text{BM at m, vertical direction, } m_m = \frac{w \times S_V^2}{24} = \frac{R_{FF} \times S_V^2}{24 \times \ S_V + S_H} \qquad (5.4)$$

$$(5.3)+(5.4) \quad m_n + m_m = \frac{3 \times R_{FF} \times S_V^2}{24 \times \ S_V + S_H}$$

$$R_{FF} = \ m_n + m_m \ \times \frac{8 \ S_V + S_H}{S_V^2}$$

Substitute w with $C_F \times w$, modified equation for obtaining flexural resistance of grid beams;

$$R_{FF} = C_F \ m_n + m_m \ \times \frac{8 \ S_V + S_H}{S_V^2} \quad \text{; Vertical direction} \qquad (5.5)$$

Similar steps can be executed to obtain the equation in the horizontal direction:

$$R_{FF} = C_F \ m_n + m_m \ \times \frac{8 \ S_V + S_H}{S_H^2} \quad \text{; Horizontal direction} \qquad (5.6)$$

The flexural resistance of the grid beam shall be the minimum of calculated vertical and horizontal resistance values.

The beam must satisfy the moment capacity requirement. Thus, the moments m_n and m_m in the above equation can be expressed as the ultimate moment capacity at the nailhead and the midspan. The moment capacity of the grid beam at nails and midspan can be calculated similarly to the FHWA, (1998/ 2015) approach for Shotcrete walls.

moment resistance per unit width; $m_V = \frac{A_s F_y}{b}\left(d - \frac{A_s F_y}{1.7 f_c' b}\right)$

moment resistance of the full beam width; $m_V = A_s F_y\left(d - \frac{A_s F_y}{1.7 f_c' b}\right)$ (5.7)

The above set of equations derived based on FHWA (1998) (Byrne, Cotton, Porterfield, Wolshlag, & Ueblacker, 1998) can be directly used for designing grid beam facings for soil nailing works.

Modifying the CIRIA c637 method for calculating flexural resistance of shotcrete to be used with grid beams:

The method given in CIRIA c637 for designing shotcrete considers the facing as a two-way spanning slab. Additionally, it considers the area of the Nailhead as the continuous interior edge and the midspan as an interior span as discussed in Section 2.5.3.

However, the Grid Beam systems shall be designed similar to a structural beam or a one-way spanning slab. As such, an alternative method available in BS 8110 for continuous beams will be used for this calculation.

Clause 3.4.3 BS 8110:1997: Part 1 provides guidelines for carrying out designs for uniformly loaded continuous beams. Table 3.5 of BS 8110:1997: Part 1 shown below provides the ultimate moments at interior supports in terms of uniformly distributed load and effective spacing.

Table 5-1. Design bending moments and shear forces (BS 8110, Part 1, 1997)

	At outer support	Near middle of end span	At first interior support	At middle of interior spans	At interior supports
Moment	0	0.09Fl	–0.11Fl	0.07Fl	–0.08Fl
Shear	0.45F	—	0.6F	—	0.55F
NOTE l is the effective span; F is the total design ultimate load ($1.4G_k + 1.6Q_k$). No redistribution of the moments calculated from this table should be made.					

The m_n (moment capacity at nailhead) and m_m (moment capacity at midspan) in grid beams can be obtained from bending moments (resistance) at the "interior supports" and the "middle of interior spans" in the above table respectively. Accordingly:

$$m_n = 0.08\, Fl \qquad = 0.08 \times \; w_1 S_V \; \times S_V$$

$$w_1 = \frac{m_n}{(0.08 \times S_V^2\ )}$$

$$m_m = 0.07\ Fl \qquad = 0.07 \times \ w_2 S_V \ \times S_V$$

$$w_2 = \frac{m_m}{(0.07 \times S_V^2\ )}$$

The calculation shall also be carried out to obtain w_3, and w_4, for horizontal direction replacing S_V with S_H.

Accordingly, w shall be taken as the smaller of w_1, w_2, w_3, and w_4 similar to how the calculation was done for Shotcrete in Section 4.1.1.

$$w = \min\begin{bmatrix} \dfrac{m_{vn}}{(0.08 \times S_V^2\ )} & or & \dfrac{m_{vm}}{(0.07 \times S_V^2\ )} \\ \dfrac{m_{hn}}{(0.08 \times S_H^2\ )} & or & \dfrac{m_{hm}}{(0.07 \times S_H^2\ )} \end{bmatrix} \tag{5.8}$$

Considering the static equilibrium of grid beams:

$$R_{FF} = \mathrm{C_F} \times w \times \ S_H + S_V \tag{5.9}$$

When calculating the R_{FF} the factor C_F used in FHWA (1998) is used in this method as well.

Lastly, Equation (2.17) and (2.18) used in this research for Shotcrete design had been presented in BS 5400:1990: Part 4 in Clause 5.3.2.3 and they recommend using them to calculate the ultimate moment resistance of solid slabs as well as rectangular beams, given that the amount of redistribution of the elastic ultimate moments is less than 10 %. Therefore, similar to as used with shotcrete, the ultimate moment capacity, m_V, m_h shall be calculated using the same formulae. i.e:

$$m_V, m_h = \left(0.87 f_{\mathrm{y}}\right) A_{\mathrm{sv}} z \tag{5.10}$$

Where;
$$z = \left(1 - \frac{1.1 f_{\mathrm{y}} A_{\mathrm{sv}}}{f_{\mathrm{cu}} b d}\right) d \qquad z \leq 0.95d \tag{5.11}$$

5.1.2 Shearing Resistance of Grid Beams

When the soil nail load is applied through the nailhead, a possible shear zone will generate around the nailhead which in turn causes punching shear failure. Unlike in shotcrete, the possible punching shear surface may not be of a continuous truncated cone shape around the nailhead. Instead, each beam may have individual failures.

In addition to the internal punching strength of the facing, the soil reaction behind the facing will contribute to the nailhead resistance against punching. Loads and the formation of a rupture surface on a nailhead under punching shear are shown below. These data, the shape of the failure surface, in particular, have been based on the tests carried out on laboratory models.

Modifying the FHWA (1998) method for calculating punching shear resistance of shotcrete to be used with grid beams:

As shown in Figure 5-3, the shear failure surface of a grid beam system may be slightly different from that of shotcrete. Failure of each beam at the beam-beam connection instead of a truncated cone shape could be considered for modifying. Based on the beam-column analogy described in 5.1.1, the following alterations can be performed.

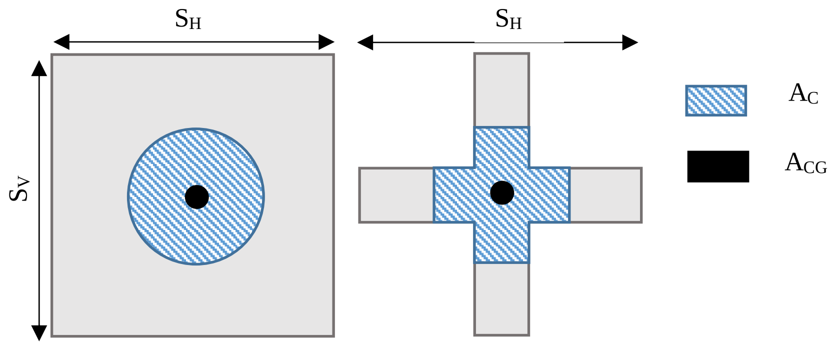


Figure 5-3. Punching shear area of shotcrete vs grid beam

From (2.8);

$$V_N\ (Shotcrete) = 0.33\sqrt{f'c}\ \pi\ (D'_C)\ h_C = 0.33\sqrt{f'c} \times (Area\ of\ the\ shear\ surface)$$

$$V_N\ (Beams) = 0.33\sqrt{f'c} \times (Area\ of\ the\ shear\ surface\ of\ each\ beam \times 4)$$

$$= 0.33\sqrt{f'c} \times \ Cross\ Sectional\ Area\ of\ Grid\ Beam \times 4$$

As such, the internal shear resistance of a grid beam at the nailhead;

$$V_N = 1.32\sqrt{f'c} \times bd \qquad (5.12)$$

Similar to the above, the nail head resistance against punching by including the soil reaction contribution can be obtained for grid beams similar to (2.15). Moreover, the shear pressure factor, C_S can be considered similar to that of shotcrete conservatively.

However, the terms in (2.15) must be modified considering the area of soil pressure behind the grid beam. Therefore, in (2.15) for Grid beams;

$$A_C = (2D_C - b) \times b$$

$$A_{GC} = \frac{\pi D_{GC}^2}{4} \quad \text{; No change}$$

$$S_V S_H = S_V + S_H \times b$$

Modifying the CIRIA c637 method for calculating punching resistance of shotcrete to be used with grid beams:

It is noted that the CIRIA has used the expressions for obtaining shear capacity of slabs with no shear reinforcement given in BS 5400: Part 4:1990.

Similarly, for Grid Beams with shear reinforcement, Clause 5.3.3.2 can be used to derive the shear resistance, R_{FS} at the nailhead. Accordingly, based on Table 7 of BS 5400: Part 4:1990 two cases are considered;

Case 1:

$$\frac{R_{FS}}{4 \times bd} \le \xi_s v_c \quad \text{; Rearrange}$$

$$R_{FS} \le 4bd\xi_s v_c \qquad A_{SV} \ge 0.4bs_v/0.87f_{yv} \qquad (5.13)$$

Case 2:

$$\frac{R_{FS}}{4 \times bd} \ge \xi_s v_c \quad \text{; Rearrange}$$

$$R_{FS} \ge 4bd\xi_s v_c \qquad A_{SV} \ge bs_V\left(\frac{R_{FS}}{4 \times bd} + 0.4 - \xi_s v_c\right)/0.87f_{YV} \qquad (5.14)$$

$$R_{FS} \le 4bd\left(\frac{0.87f_{YV}A_{SV}}{bs_V} - 0.4 + \xi_s v_c\right)$$

5.2 Proposing a method for designing Grid Beam facings

As discussed in Chapter 4, the CIRIA c637 (2005) presents somewhat similar methods for designing structural facing to British standards of the structural design. It is noted that this method was based on BS 5400:1990: Part 4. Additionally, if one desires, both FHWA and CIRIA methods can be used to design Grid Beams as shown in detail in 5.1. However, the method proposed by GEO and HKIE for Grillage Structure (Grid Beam) design could be considered as the approach which is most closely resembling the modern structural design concepts mainly because their approach is not using an allowable limit design. Allowable nailhead load design is the approach used by both major pieces of literature available for structural facing designs, FHWA soil nail reference manual, and CIRIA c637 as discussed in Section 2.5.

However, GEO and HKIE method is also not directly based on BS 8110:1997: Part 1. They use their own Hong Kong code of practice for structural design (Buildings Department, Hong Kong, 2013) for Grid Beam designs as discussed in Section 2.6.2. Hong Kong code of practice for structural use is mostly based on BS 8110, as such, the method presented by GEO and HKIE is useful in developing a procedure based on BS 8110 for designing Grid Beams.

Accordingly, in this research, a new procedure for calculating the structural design of Grid Beams is presented; which is purely based on BS 8110:1997: Part 1. Clause. 3.4, Design of Beams. The design checks used in this clause resemble the loading mechanism and the performance of a Grid Beam facing and covers design for flexure, shear, as well as all structural checks stipulated by the British code of practice.

Basic parameters for design

1. Acquire the parameters of the soil nailed slope

Slope angle	β
Vertical and Horizontal nail spacings	S_H, S_V
Calculate spacing along slope face	$L = S_V / sin\beta$
Angle of soil nail	α

2. Obtain the nailhead load T_0 using the method recommended in Section 3.1.
3. Predefine sectional properties and reinforcement amounts of Grid Beam

Section depth and width	d, b	No. of main bars	N
Minimum cover	C	Dia. of main bars	D
Minimum cover at buried face	C_b	Dia. of shear links	D_s
Concrete strength	f_{cu}		
Rebar tensile strength	f_y		

Calculate Area of tor steel $\quad A_{s\ prov} = \pi\left(\frac{D}{2}\right)^2 \times N$

Calculate Effective depth at nailhead $\quad d_{effn} = d - C_b - D_s - 1.5D$

Calculate Effective depth at midspan $\quad d_{effm} = d - C - D_s - 0.5D$

4. Calculate the load per unit area behind the facing

$$w = T_0 sin(\alpha + \beta)/(S_H + L)$$

Design for flexure using BS 8110: Part 1

5. Calculate design flexure assuming a non-continuous simply supported beam

$$M = w \times l^2/8$$

Here, the critical flexure will be induced on vertical beam segments. GEO & HKIE (Geotechnical Engineering Office, Hong Kong Institution of Engineers, 2013) in their review process proves that considering the beam segments as non-continuous segments instead of continuous segments, will deliver the most critical loading scenario.

6. Check for the requirement for compression reinforcement

$$\text{k} = M/bd^2 f_{cu}$$ *BS 8110-1, Cl. 3.4.4.4*

Where $k < 0.156$ compression reinforcement not necessary

7. Calculate the tension reinforcement requirement for each location, nailhead, and midspan for larger and smaller spans

$$A_{s\ req} = M/(0.87 f_y \times z)$$ Where;

$$z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right)$$ *BS 8110-1, Cl. 3.4.4.4*

And check $A_{s\ req} < A_{s\ prov}$ criteria

8. Check minimum reinforcement requirement

$$A_{s\ min} = 0.13\% bd$$ *BS 8110-1, Table 3.25*

And check $A_{s\ min} < A_{s\ prov}$ criteria

Design for shear using BS 8110: Part 1

Design for line shear around nailhead

9. Predefine additional section properties necessary for the design for shear

Base plate width	b_p	
Calculate the base plate perimeter	u_o	$= b_p \times 4$
Base plate thickness	t_p	
Effective depth at nailhead	d_{peff}	
Shear link Tensile Strength	f_{yv}	
No. of bars	N'	
Spacing along the beam	s_v	
Area of steel	A_{sv}	$= N_s \times \pi \times D_s^2/4$

10. Calculate design shear assuming the beam segments as non-continuous simply supported

$$V \quad = w \times L \times S_H/2$$

11. Calculate allowable design shear stress

$$v_{all} \quad = 0.8\sqrt{f_{cu}}\ or\ 5\ N/mm^2\ if\ less$$

12. Calculate design shear stress

$$v \quad = V/bd$$

And check $v < v_{all}$criteria

This check will define the beam sizing in general

13. Calculate design concrete shear stress v_c as per BS 8110-1, Cl. 3.4.5.4 and Table 3.8 and provide shear links as follows;

$v < 0.5v_c$	Provide nominal shear r/f (maximum spacing = $0.75d$)
$0.5v_c < v < v_c + 0.4$	Provide minimum shear link r/f
$v_c + 0.4 < v < 0.8\sqrt{f_{cu}}\ or\ 5\ N/mm^2\ if\ less$	Links or links combined with bent-up bars.

14. Calculate minimum shear link requirement as per BS 8110-1 Cl. 3.4.5.3 and Table 3.7

$$A_{sv,min} = 0.4 b_v s_v / 0.95 f_{yv}$$

Provide shear links accordingly.

Design for bearing failure of the nail-facing connection

Although this check is usually carried out for flat slabs, GEO & HKIE recommends this check for proper nailhead plate sizing in Grillage Structures.

15. Additional parameters for design

Depth of beam below the base plate	d_p	$= d - C$
Effective depth at nailhead	d_{effp}	$= d_p - C_b - 0.5D$
Calculate the base plate perimeter	u_o	$= b_p \times 4$

16. Calculate maximum design shear capacity

$$v_{\max\ all} = 0.8\sqrt{fcu}\ \ , or\ 5\ N/mm^2\ if\ less$$ *BS 8110-1, 3.7.7.2*

17. Calculate design shear stress at bearing plate

$$v_{\max} = \frac{V}{u_o d_{effp}}$$ *BS 8110-1, Cl. 3.7.7.2*

And check $v_{\max} < v_{\max\ all}$ criteria

18. Calculate force from the soil at a nailhead

$$F_{sh} = n \times A_{1.5}$$ Where;

$$A_{1.5} = Area\ of\ perimeter\ 1.5 d_{effp}\ from\ plate$$

$A_{1.5}$ maybe capped by the smaller grid beam crossing area.

In FHWA (1998) method the area covered by the grout column may be subtracted from the loaded area. However, this only makes a small increase in shearing force. As such, this step is omitted in this proposed approach.

19. Calculate resultant force resisted by facing

$$F_{res} = V_{eff} - F_{sh}$$

20. Calculate design shear at 1.5d from loaded area

$$v_{1.5} = \frac{F_{res}}{u_{1.5} \times d_{effn}}$$ Where;

$$u_{1.5} = Perimeter\ 1.5d\ from\ loaded\ area$$

$u_{1.5}$ maybe capped by the smaller grid beam crossing area.

And check $v_{1.5} < v_c$ criteria.

The adequacy of the parameters defined in each step shall be assessed at each relevant step above and the parameters must be readjusted accordingly.

5.3 Application for a Case History

5.3.1 Grid Beam facing design for Hakgala landslide mitigation project using the proposed method

Continuing from Section 3.3 analysis has been carried out to access the applicability of a Grid Beam facing for the Hakgala Landslide mitigation project's CS5 area. In this section, the required sectional and reinforcement properties for the area in question as per the recommended design procedure in Section 5.2. The soil nail arrangement was also considered as $S_V = S_H = 2\ m$ similar to the previous previous instances.

1. Parameters of the soil nailed slope

Slope angle	β	$= 66°$
Vertical and Horizontal nail spacings	S_H, S_V	$= 2\ m$
Calculate spacing along slope face	$L = S_V / sin\beta$	$= 2.19\ m$
Angle of soil nail	α	$= 15°$

1. Nailhead Load

$T_0 \quad = 116.27\ kN$ From Section 3.3

2. Predefine sectional properties and reinforcement amounts of Grid Beam

Section depth and width	d, b	$= 250\ m \times 250\ m$
Minimum cover	C	$= 30\ mm$
Minimum cover at buried face	C_b	$= 40\ mm$
Concrete strength	f_{cu}	$= 30\ N/mm^2$
Rebar tensile strength	f_y	$= 460\ N/mm^2$
No. of main bars	N	$= 2\ per\ face$
Dia. of main bars	D	$= 12\ mm$
Dia. of shear links	D_s	$= 10\ mm$

Calculate Area of tor steel $\quad A_{s\ prov} = \pi \left(\frac{D}{2}\right)^2 \times N = 226.19\ mm^2$

Calculate Effective depth at nailhead $\quad d_{effn} = d - C_b - D_s - 1.5D = 182\ mm$

Calculate Effective depth at midspan $\quad d_{effm} = d - C - D_s - 0.5D = 204\ mm$

3. Load per unit area behind the facing

$$w = T_0 sin(\alpha + \beta)/(S_H + L)$$
$$= 116.27 \times \sin(66 + 15)/(2 + 2.19)$$
$$= 27.41\ kN/m^2$$

Design for flexure

4. Design flexure assuming a non-continuous simply supported beam

$$M = w \times l^2/8 \qquad = 27.41 \times 2.19^2/8$$
$$= 16.42\ kNm$$

5. Requirement for compression reinforcement

$$k = M/bd^2 f_{cu}$$ *BS 8110-1, Cl. 3.4.4.4*

$$= \frac{16.42}{250 \times 204^2 \times 30} \times 10^6$$
$$= 0.053$$

$k < 0.156$ compression reinforcement not necessary

6. Tension reinforcement requirement for each location, nailhead, and midspan for larger and smaller spans

$$A_{s\ req} = M/(0.87 f_y \times z)$$ Where;

$$z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right)$$ *BS 8110-1, Cl. 3.4.4.4*

$$= 204 \times \left(0.5 + \sqrt{0.25 - \frac{0.053}{0.9}}\right)$$
$$= 191.28\ mm$$
$$A_{s\ req} = 16.42/(0.87 \times 460 \times 191.28)$$
$$= 214.54\ mm^2 \qquad < A_{s\ prov} = 402.12\ mm^2$$

7. Check minimum reinforcement requirement

$$A_{s\ min} = 0.13\%bd$$ *BS 8110-1, Table 3.25*

$$= 0.0013 \times 250 \times 250$$
$$= 81.25\ mm^2 \qquad < A_{s\ prov} = 226.19\ mm^2$$

As such, a Grid Beam facing with 4Y12 bars and a 250 mm x 250 mm section of c30 concrete section can be satisfactorily used to counter flexure induced by soil nails at the CS5 area of Hakgala Landslide Project.

Design for shear

Design for line shear

8. R/f properties necessary for the design of shear

Shear link Tensile Strength	f_{yv}	$= 250\ N/mm^2$
No. of bars	N_s	$= 1\ per\ link$
Spacing along the beam	s_v	$= 150\ mm$
Area of steel	$A_{sv(prov)}$	$= N_s \times \pi \times D_s^2/4$
		$= 78.54\ mm^2\ per\ link$

9. Design shear assuming the beam segments as non-continuous simply supported

$V = w \times L/2 \qquad = 27.41 \times 2.19/2$

$= 30.01kN$

10. Allowable design shear stress

$v_{all} = 0.8\sqrt{f_{cu}} \qquad = 0.8\sqrt{30}$

$= 4.38\ N/mm^2$

11. Design shear stress

$v = V/bd_{effn} \qquad = 30.01/(250 \times 182)$

$= 0.66\ N/mm^2 \qquad < v_{all} = 4.38$

12. Design concrete shear stress v_c as per BS 8110-1, Cl. 3.4.5.4 and Table 3.8

$v_c = 1.29\ N/mm^2$

$0.5v_c = 0.645\ N/mm^2$

$v_c + 0.4 = 1.69\ N/mm^2$ Accordingly;

$0.5v_c < v < v_c + 0.4$ Provide minimum shear link r/f

13. Minimum shear link requirement as per BS 8110-1 Cl. 3.4.5.3 and Table 3.7

$A_{sv,min} = 0.4b_v s_v/0.95f_{yv} \qquad = 0.4 \times 250 \times 150/(0.95 \times 250)$

$= 63.16\ mm^2 \qquad A_{sv,min} < A_{sv\ prov} = 78.54\ mm^2$

As such, shear links, R10 @ 150 will be satisfactorily used to counter shear induced by soil nails at the CS5 area of Hakgala Landslide Project.

Design for bearing failure of the nail-facing connection

Although this check is usually carried out for flat slabs and not beams, GEO & HKIE recommends this check for proper nailhead plate sizing in Grillage Structures.

14. Additional parameters for design

Base plate width	b_p	$= 200\ mm$
Base plate perimeter	u_o	$= b_p \times 4$
		$= 800\ mm$
Base plate thickness	t_p	$= 12\ mm$
Depth of beam below the base plate	d_p	$= d - C$
		$= 250 - 30$
		$= 220\ mm$
Effective depth at nailhead	d_{effp}	$= d_p - C_b - 0.5D$
		$= 220 - 40 - 0.5 \times 12$
		$= 174\ mm$

15. Maximum design shear capacity

$v_{\max\ all} = 0.8\sqrt{fcu}\ \ , or\ 5\ N/mm^2\ if\ less$ *BS 8110-1, 3.7.7.2*

$= 0.8 \times \sqrt{30}$

$= 4.38\ N/mm^2$

16. Shear force transferred from nail

$V = T_0 \times \sin(\alpha + \beta)$ $= 116.27 \times \sin\ 66 + 15$

$= 114.84\ kN$

$V_{eff} = 1.15 \times V$ *BS 8110-1, Cl. 3.7.7.2*

$= 132.06\ kN$

17. Calculate design shear stress at bearing plate

$$v_{\max} = \frac{V_{eff}}{u_o d_{effp}}$$ *BS 8110-1, Cl. 3.7.7.2*

$$= \frac{132.06}{800 \times 174}$$

$= 0.95\ N/mm^2$ $< v_{\max(all)} = 4.38\ N/mm^2$

<u>Design for punching shear failure</u>

18. Calculate force from the soil at a nailhead

$A_{1.5} = \min\left(\left(b_p + 3d_{effp}\right)^2, b^2\right)$

$= 0.25^2$

$= 0.063\ m^2$ *$A_{1.5}$ is capped by smaller beam crossing area*

$F_{sh} = w \times A_{1.5}$

$= 27.41 \times 0.063$

$= 1.71\ kN$

In FHWA (1998) method the area covered by the grout column may be subtracted from the loaded area. However, this only makes a small increase in shearing force. As such, this step is omitted in this proposed approach.

19. Calculate resultant force resisted by facing

$$F_{res} = V_{eff} - F_{sh} = 132.06 - 1.71 = 130.35\ kN$$

20. Calculate design shear at 1.5d from loaded area

$$u_{1.5} = Perimeter\ 1.5d\ from\ plate = \min(4b, b_p + 8 \times 1.5 \times d_{effp}) = 4 \times 250 = 1000\ mm$$

$$v_{1.5} = \frac{F_{res}}{u_{1.5} \times d_{effn}} = \frac{130.35}{1000 \times 174} = 0.75\ N/mm^2$$

Where;

$$< v_c = 1.29\ N/mm^2$$

As such, a plate of 200 mm x 200 mm will be satisfactorily used to counter bearing and punching induced by soil nails at the CS5 area of Hakgala Landslide Project.

Although, the plate size is not critical and smaller sizes could be used as per this calculation, in practice, using a 200 mm x 200 mm will allow proper fabrication of reinforcement at the nailhead.

5.3.2 Design of grid beam for the case history for different nailing arrangements

The same method proposed in Section 5.2 for $S_V = S_H = 2\ m$ could be used to assess the nailhead loads at different nail spacings. Accordingly, the practical minimum structural requirements for each case have been obtained and tabulated in Table 5-2 for comparison. The nailhead values for the calculations were obtained from the calculation presented in Section 3.3.1.

Table 5-2. Summary of possible Grid Beam designs for alternative nail spacings

Vertical spacing (S_V) (m)	Horizontal spacing (S_H) (m)	T_{oULT} (kN)	Beam Size (mm)	Main reinforcement	Shear reinforcement
2.5	2.5	153.76	250x250	4Y16	R10@150mm
2	**2**	**116.27**	250x250	**4Y12**	R10@150mm
1.5	1.5	69.86	200x200	4Y10	R10@200mm
2.5	2	136.69	250x250	4Y16	R10@150mm
1.5	**2**	**115.23**	250x250	**4Y12**	R10@150mm

Similar to the Shotcrete facings, the maximum nail spacing seems to govern the main reinforcement requirement of the facing. This can be identified from the 2 m x 2 m arrangement and the 1.5 m x 2 m nail arrangement. As such, it is highly recommended to use $S_V = S_H$ arrangement and adhering to the recommended spacing values as given in Section 2.3 is recommended. There would be no advantage in a spacing reduction in only one direction in the facing design.

5.3.3 Grid Beam facing design for the case history using alternative methods

Alternative design approaches presented in Section 5.1 have been used to obtain the nailhead resistance values for the same Grid Beam section proposed in the case history Section 5.3.1. The difference between the outcomes of the methods used can be understood from this calculation. The design parameters and abbreviations used are the same as Section 5.3.1 unless otherwise stated.

Designing for flexure using FHWA (1998) method

Flexural capacity at midspan & nailhead:

Equation (5.7);

Capacity at the midspan (m_{vm}) as well as the nailhead (m_{vn}) is calculated separately,

$$m_{vn} = A_s F_y \left(d_{effn} - \frac{A_s F_y}{1.7 f_c' b} \right) = 226.19 \times 460 \left(182 - \frac{226.19 \times 460}{1.7 \times 30 \times 250} \right)$$

$$m_{vm} = A_s F_y \left(d_{effm} - \frac{A_s F_y}{1.7 f_c' b} \right) = 226.19 \times 460 \left(204 - \frac{226.19 \times 460}{1.7 \times 30 \times 250} \right)$$

$$m_{vn} = m_{hn} = 18.09\ kNm$$

$$m_{vm} = m_{hm} = 20.38\ kNm$$

Flexural resistance of the facing:

The flexural resistance of the grid beam shall be the minimum of calculated vertical and horizontal resistance values. However, in this case, where $S_V = S_H$, both horizontal and vertical values are the same. Equation (5.5);

$$R_{FF} = C_F \times \left(m_m + m_n\right) \times \frac{8\left(S_V + S_H\right)}{S_V^2}$$

$$= 1 \times \left(18.09 + 20.38\right) \times \frac{8\left(2 + 2\right)}{2^2} \qquad C_F = 1; \textit{Permanent facing}$$

$$= \mathbf{307.72\ kN}$$

Resistance factor for bending/flexure in the facing ϕ_F= 0.9 as per Section 2.4.6 As such;

$$R_{FF,ult} = 0.9 \times 307.72$$

$$= \mathbf{276.95\ kN}$$

As per the FHWA (1998) method, the ultimate flexural resistance is adequate to cater to the induced ultimate nailhead load perpendicular to the facing, $T_{0,eff}$.

$$T_{0,eff} = T_0 \times \sin(\delta + \beta)$$

$$= 116.27 \times \sin(15 + 66)$$

$$= 113.81\ kN \leq R_{FF,ult} = 276.95\ kN$$

Design for flexure using CIRIA (2005) method

Flexural capacity at nailhead (m_{vn})

Equation (5.11);

$$z = \left(1 - \frac{1.1 f_y A_s}{f_{cu} b d_{effn}}\right) d_{effn} = \left(1 - \frac{1.1 \times 460 \times 226.19}{30 \times 250 \times 182}\right) \times 182$$

$$= 167.74\ mm$$

$$0.95d = 0.95 \times 182 = 172.90\ mm$$

$$z \leq 0.95d$$

Equation (5.10);

$$m = \left(0.87 f_y\right) A_s z = \left(0.87 \times 460\right) \times 226.19 \times 167.74$$

$$m_{vn} = m_{hn} \quad = 15.09\ kN.m$$

Flexural capacity at nailhead (m_{vm}):

Equation (5.11);

$$z \quad = \left(1 - \frac{1.1 f_{\mathrm{y}} A_{\mathrm{s}}}{f_{\mathrm{cu}} b d_{effm}}\right) d_{effm} \quad = \left(1 - \frac{1.1 \times 460 \times 226.19}{30 \times 250 \times 204}\right) \times 204$$

$$= 188.74\ mm$$

$$0.95d \quad = 0.95 \times 204 \quad = 193.80\ mm$$

$$z \quad \leq 0.95d$$

Equation (5.10);

$$m \quad = (0.87 f_{\mathrm{y}}) A_{\mathrm{s}} z \quad = 0.87 \times 460 \times 226.19 \times 188.74$$

$$m_{vm} = m_{hm} \quad = 17.09\ kNm$$

Flexure resistance of the facing:

Equation (5.8);

$$w \quad = \min \begin{bmatrix} \dfrac{m_{vn}}{(0.08 \times S_V^2)} & or & \dfrac{m_{vm}}{(0.07 \times S_V^2)} \\ \dfrac{m_{hn}}{(0.08 \times S_H^2)} & or & \dfrac{m_{hm}}{(0.07 \times S_H^2)} \end{bmatrix}$$

$$w \quad = \min \begin{bmatrix} \dfrac{15.09}{(0.08 \times 2^2)} & or & \dfrac{17.09}{(0.07 \times 2^2)} \\ \dfrac{15.09}{(0.08 \times 2^2)} & or & \dfrac{17.09}{(0.07 \times 2^2)} \end{bmatrix}$$

$$= \min \begin{bmatrix} 47.17 & or & 61.04 \\ 47.17 & or & 61.04 \end{bmatrix} \quad = 47.17\ kN/m$$

Equation (5.9);

$$R_{FF} \quad = w \times \ S_H + S_V \quad = 1 \times 47.17 \times \ 2 + 2$$

$$= \mathbf{188.67}\ \boldsymbol{kN}$$

The obtained R_{FF} value is the ultimate value as partial factors are embedded in the expressions. Therefore, no modifications required;

$$R_{FF,ult} \quad = R_{FF} \quad = \mathbf{188.67}\ \boldsymbol{kN}$$

As per the CIRIA (2005) method, the ultimate flexural resistance is adequate to cater to the induced ultimate nailhead load perpendicular to the facing of $\mathbf{113.81}\ \boldsymbol{kN}$.

Conclusion of the outcomes of the results obtained from the methods considered

Given in Figure 5-4 is a comparison of the flexural resistance outputs, R_{ff} of the above grid beam system when installed on soil nail systems with different spacings. Additionally, this chart also provides the nailhead loads, T_0 for this slope under different soil nail spacings. These R_{ff} and T_0 values could be compared against each other at a given spacing value to assess the adequacy of the facing design.

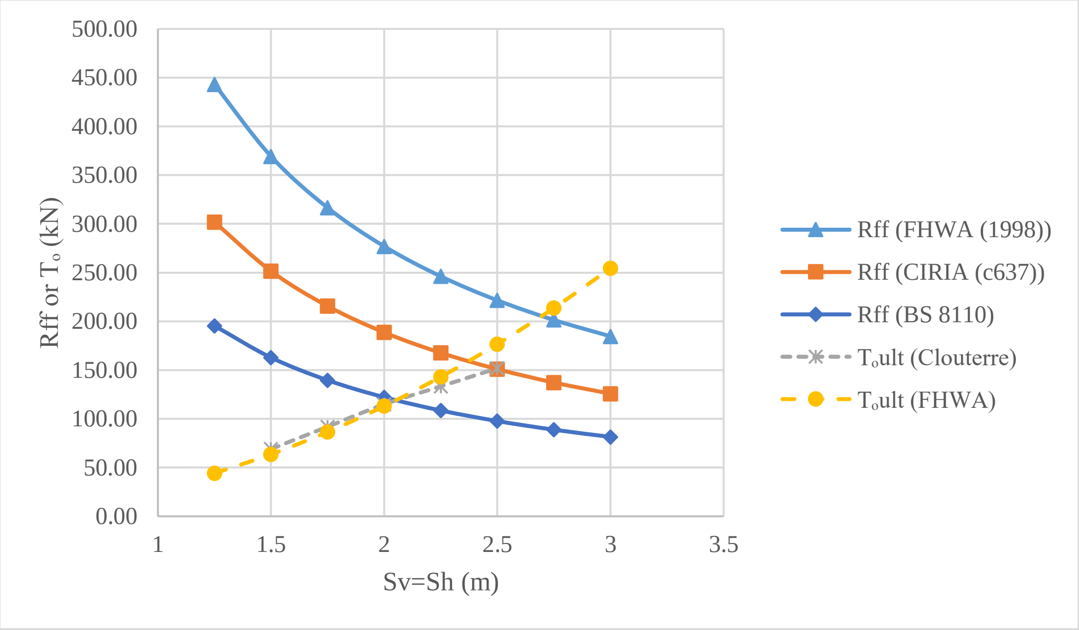


Figure 5-4. Flexural resistance, $T_o$ult vs Nail Spacing for the Grid Beam considered for CS5 of Hakgala landslide mitigation

A trend of variation of flexural resistance with spacing similar to the Shotcrete facings can be seen here as well. However, in this case, the results seem to have deviated away from each other by a considerable amount. As discussed previously, the FHWA and CIRIA methods both consider the Grid Beam as a continuous beam element across the supports (or nailheads), whereas, the proposed BS 8110 method carries out design assuming it as a non-continuous beam. As suggested by the GEO and HKIE (Geotechnical Engineering Office, Hong Kong Institution of Engineers, 2011), the results have been the most critical in the analysis done following a simply supported idealization.

Other than flexure, the most critical failure mode was found to be the line shear of the beam element. However, other than the GEO & HKIE (2011) method, the other major sources of literature do not provide design guides for line shear. They only provide design steps for checking punching shear around the nailhead for hard facings. This is because all of them have been targeted towards Shotcrete facing designs and not beams. In Shotcrete, as discussed, the punching shear becomes critical, and in Grid Beams line shear will become critical other than flexure. As such, the shear resistance results obtained from the BS 8110 method cannot be compared with the other two methods.

Back calculated results of the shear resistance of the Grid Beam, R_{fp} considered for the case study done using the BS 8110 based method have been presented in Figure 5-5. Additionally, this chart also provides the nailhead loads, T_0 for different soil nail spacings. These R_{fp} and T_0 values could be compared against each other to assess the adequacy of the facing design at a given soil nail spacing. The results show that there is only a minor variation of resistance with nail spacing.

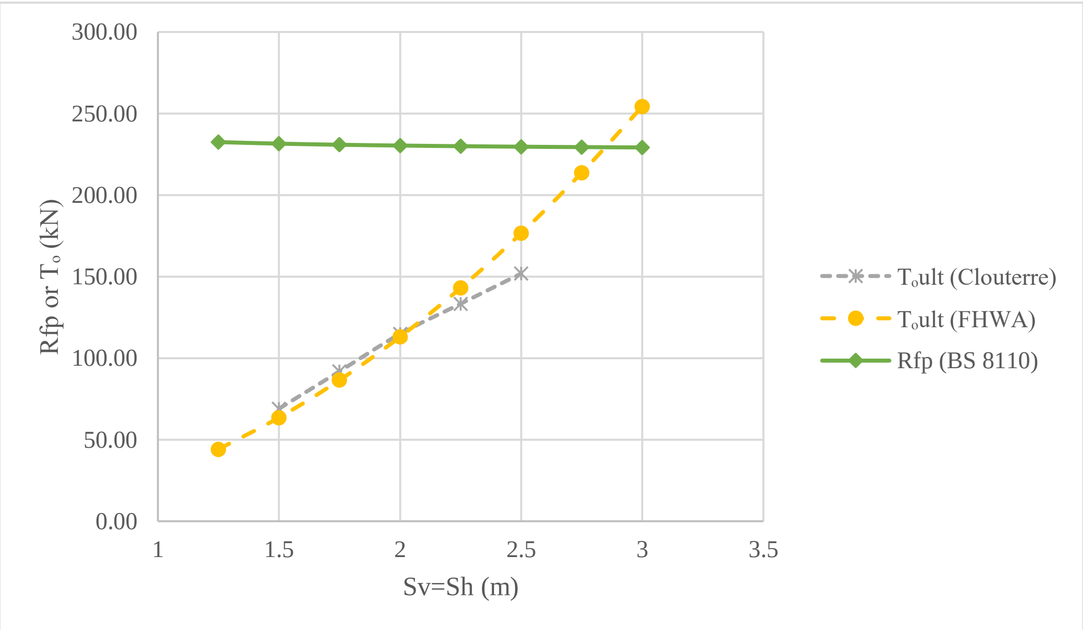


Figure 5-5. Shear resistance, $T_o$ult vs Nail Spacing for the Grid Beam considered for CS5 of Hakgala landslide mitigation

Further analysis reveals an interesting pattern of flexural resistance against nail spacing. The results of the analysis carried out using both the FHWA and CIRIA approaches using the same grid beam system have been illustrated in Figure 5-6 and Figure 5-7.

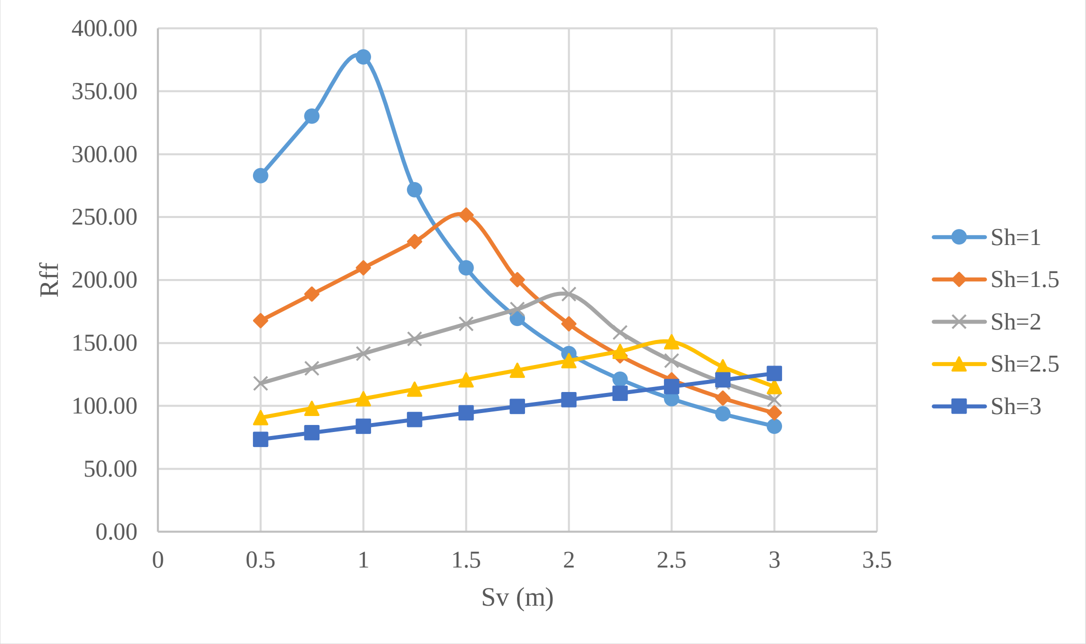


Figure 5-6. Flexural resistance vs Sv [CIRIA] for the Grid beam section used in the case study

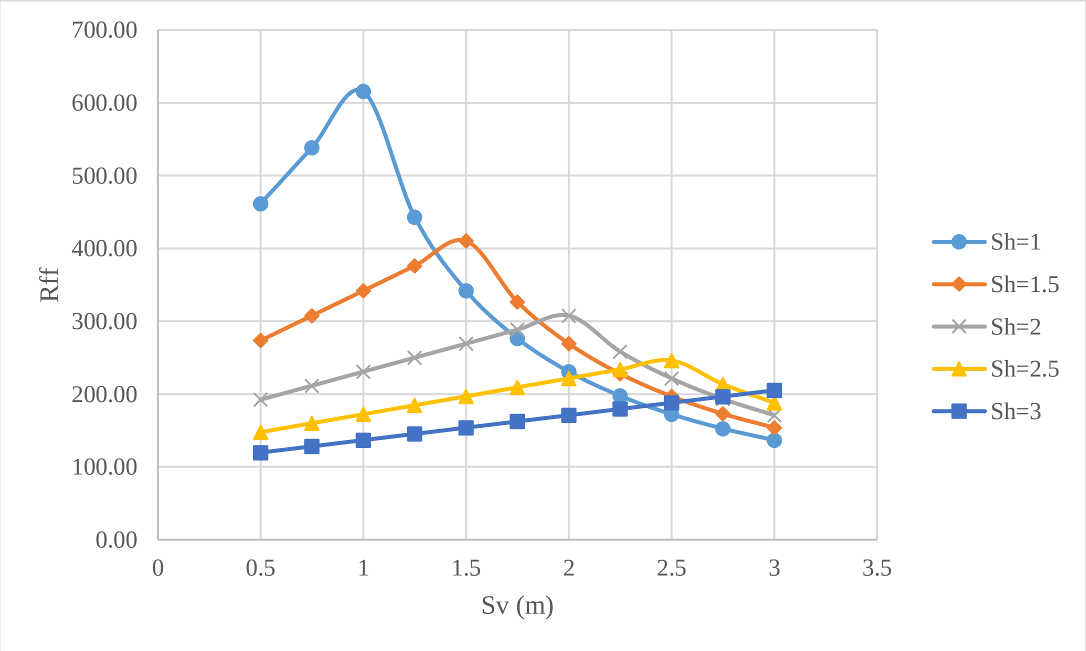


Figure 5-7. Flexural resistance vs Sv [FHWA] for the Grid beam section used in the case study

Both the charts in Figure 5-6 and Figure 5-7 indicate that the maximum flexural resistance is obtained when the horizontal spacing is equal to the vertical spacing. Therefore, for a given grid beam system, keeping the nail spacing ratio close to unity is always advisable to gain the maximum level of assurance over the safety of the facing system.

Additionally, the above outcome can also be observed in the results presented in Table 5-3 reported by FHWA (2015) to estimate the flexural resistance of the nailhead in Shotcrete facings. From this table, it is seen that the Rff value is increasing with the nail spacing ratio for a given system. As such, for both Grid Beam and Shotcrete facings, maintaining a nail spacing ratio of unity is recommended.

Table 5-3. Values of Flexural Resistance, Rff of shotcrete facings (after FHWA 2015)

Facing thickness, h_i	Nail Spacing Ratio(1)	ρ_{tot} = 0.5 % (2)	ρ_{tot} = 1.0 % (2)	ρ_{tot} = 2.0 % (2)
(in.)	(ft/ft)	R_{FF} (kip) (3) (5)	R_{FF} (kip) (4) (5)	R_{FF} (kip) (4) (5)
4	0.67	12	24	48
4	1	18	36	71
6	0.67	20	40	81
6	1	30	60	120
8	0.67	24	48	95
8	1	36	71	143

(1) Nail spacing ratio = smaller of S_V/S_H or S_H/S_V.

(2) $\rho_{tot} = \rho_n + \rho_m$, ρ_n and ρ_m are the nail head and mid-span reinforcement ratios, respectively, in either direction.

(3) The above values are valid for Grade 60 steel. Multiply the values above by 1.24 for Grade 75 steel.

(4) Divide R_{FF} by 2 for final facing thickness $h_i = 4\ in$. Divide R_{FF} by 2 1.5 for final facing thickness $h_i = 6\ in$. For $h_i = 8\ in.$, use same R_{FF}.

CHAPTER 6. DESIGN OF FLEXIBLE STRUCTURAL FACINGS

This chapter utilizes the methods presented in Chapter 2 with regards to flexible facing designs and presents a method for obtaining the unfavorable forces on flexible mesh facings. Additionally, the method of assessing the adequacy of a given flexible net will be presented concerning the guides available in the literature. Also, the methods used in the soil nailing practice in Sri Lanka for installing the flexible meshes will be taken into account and a comprehensive design for a case history as per the findings of this study will be carried out and presented.

6.1 Background

Replacing full face structural facings with flexible structural facing is quite important in a country like Sri Lanka where natural vegetation cover is important to merge a slope rectification work to the surroundings. Vegetation significantly affects the stability of the slope surface by preventing erosion, degradation, and softening of the slope behind the flexible facing. Additionally, unlike hard shotcrete facings, flexible systems do not promote the buildup of pore water pressures behind the facing. This fact is much important because rainfall is the triggering factor in many slope-related disasters in Sri Lanka.

As discussed in 2.7, the flexible facings are used in Sri Lanka along with steel plates at the nailheads (bearing plate method), concrete blocks (pillow method), or grid beams as nailheads. In all cases where a hard facing is used with the flexible facing, the nailhead load discussed in Chapter 3 will not take any part in the flexible facing design. However, in cases where a hard-facing element such as a grid beam or a concrete pillow are not utilized, the flexible facing will carry nailhead loads as well. This chapter predominately discusses obtaining the out of balance force induced on the flexible facing and checking for the punching shear capacity of the facing system considering the former case.

The flexible mesh-facing must resist the out-of-balance force exerted from the soil underneath and transferring it onto the nailheads. This force on the mesh is initiated by a potential failure of a 3D soil mass in between the nailheads. This may also be referred to as a local failure of soil mass between the soil nails. Such mass can be idealized by either a 2-dimensional single wedge or a 2-dimensional two-part-wedge analysis by conservatively ignoring the side friction. In addition to the selection of mesh, proper sizing of the head-plate is equally important when designing flexible facings. The sizing of head plates is also covered in the latter sections.

6.2 Procedure for designing the flexible facing

Preliminary study to determine the requirement of a flexible facing

As discussed in Section 2.7.3, the GEO report No. 175 has presented the maximum vertical spacings between soil nails of a slope that can be left unsupported without using a flexible mesh facing. Accordingly, using the chart given in Figure 2-28, insight could be had on the requirement of a flexible mesh for the slope in consideration.

Two-part wedge failure mode

The HA 68/94 (The Highways Agency, 1994) has presented detailed steps for carrying out a two-part wedge method for slopes as discussed in Section 2.7.2. However, HA 68/94 does not provide details on specific methods to carry out facing load calculations using the two-part wedge mechanism. A brief discussion of this process has been presented in CIRIA c637 (2005) however, a developed analysis method is not presented. Therefore, a mechanism to design flexible meshes based on the fundamentals of the HA 68/94s method has been developed and presented in this section.

Based on the geometry of potential failure of the soil block between the nailheads as shown in Figure 6-1, the inter-wedge boundary could be considered to lie below the slope crest and the base of the top wedge must intersect the slope surface below the crest. Thus, mode 3 presented in HA 64/98 and shown in Figure 6-2 should be used as the failure mechanism in this analysis.

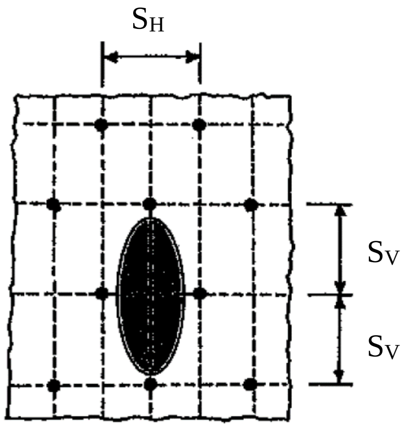


Figure 6-1. Failure geometry of soil in-between the nailheads (Ruegger & Flum, 2000)

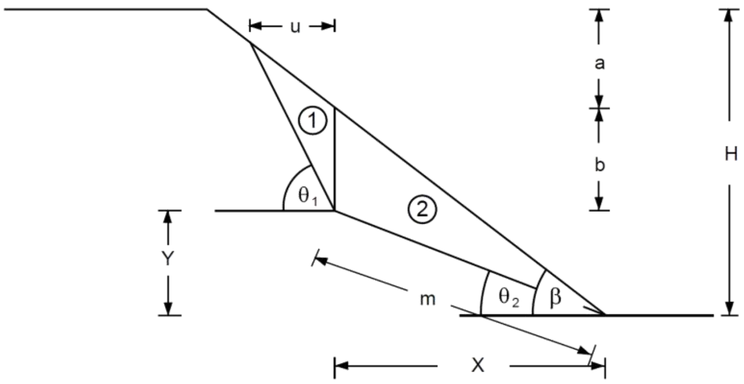


Figure 6-2. Mechanism of two-part-wedge failure considered for obtaining flexible facing load

Having identified the mode of the two-part wedge to be used, the terms in Equation (2.23) could be obtained as follows.

Equation (2.23);

$$T_1 + T_2 = \frac{[W_1(\tan\theta_1 - \tan\phi'_1) + (U_1 \tan\phi'_1 - K_1)/\cos\theta_1]}{(1 + \tan\theta_1 \tan\phi'_1)} + \frac{[W_2(\tan\theta_2 - \tan\phi'_2) + (U_2 \tan\phi'_2 - K_2)/\cos\theta_2]}{(1 + \tan\theta_2 \tan\phi'_2)}$$

Where;

$W_1 = 1/2\,\gamma b\,u$ Where: $b = H - Y - a = X\tan\beta - Y$

$W_2 = 1/2\,\gamma b\,X$ $u = b/\ \tan\theta_1 - \tan\beta$

$U_1 \;\; = 1/2 r_u \gamma bu \sec \theta_1$ $\qquad m \;\; = \sqrt{\{X + Y}$

$U_2 \;\; = 1/2 r_u \gamma b m$ $\qquad a \;\; = H - X \tan \beta$

$K_1 \;\; c_1' \; e + f + w \;\; = c_1' \; g + w$

$K_2 \;\; = c_2' m$

These parameters have been depicted in Figure 2-26. The unknowns can be expressed in terms of the known parameters which are specific to the proposed soil nailing system;

$$H \;\; = VerticalNailSpacing, S_v$$

$$\beta \;\; = Slope\ Angle\ to\ the\ horizontal$$

In addition to the above expressions, the following constraints have also been used in the calculations;

$$Y \;\; = X \tan \theta_2$$

$$0 \;\; \leq X \leq H / \tan \beta$$

$$X \;\; = \frac{H}{\tan \; \beta} \times \frac{\tan \; \beta \;\; - \tan \; \theta_1}{\tan \; \theta_2 \;\; - \tan \; \theta_1}$$

Lastly, all the above equations could be merged, and a computer spreadsheet can be developed on the MS Excel platform to obtain the maximum loading condition as stated above. The basic functionality of the program would be trying different values of θ_1 and $\theta_2$for each soil-friction angle, slope angle, and nail spacing, until the maximum value of out of balance force, T_1+T_2 is obtained via Equation (2.23).

Accordingly, calculations have to be carried out iteratively. The worst groundwater condition at high rainfall intensities is usually assumed and reflected in the analysis by taking r_u=0.5.

Single-wedge failure mode

Additionally, the single-wedge failure mode discussed in 2.7.2 is also used to alternatively obtain the out of balance force on the flexible mesh facing. Equation (2.25) could be used in the analysis as discussed. The terms W, K, and U could be derived straightforwardly due to the simpler geometry. Similar to the two-part wedge, the worst groundwater condition could be assumed taking pore water pressure ratio, r_u=0.5. The calculation is iterative and a computer spreadsheet can be developed on the MS Excel platform to obtain the maximum out of balance force.

As discussed, the larger of the out of balance force values obtained from the two methods will be used in the design of the flexible facing.

Theoretically assessing the bearing resistance capacity of meshes against punching

As discussed in Section 2.7, flexible mesh products are proprietary materials. Therefore, each manufacturer may have different methods for calculating the punching resistance depending on the elements they use for the mesh product. However, it is mandatory to determine the design bearing resistance of a mesh against punching before it is used as a flexible facing.

It should be noted that there are test methods proposed by researchers as well as international standards that can be used to carry out tests on metallic meshes to determine their bearing resistance against punching, shearing, etc. These methods often require complex test setups which is the cause why such testing is not carried out in Sri Lanka at present. However, basic tests such as the tensile capacity of the wire and tensile capacity of the mesh require much simpler test setups and they can be carried out in Sri Lanka as of this date.

Structural mechanics-based calculations could be carried out to deduce the bearing capacity of a mesh against punching using the tensile capacity of the mesh wire.

The following section presents a method to be used to calculate the punching resistance of a rhomboidal metallic mesh product commonly used in slope protection works.

Details on the mesh used

Testing had been carried out on a sample of a proposed metallic mesh at the Department of Material Science and Engineering University of Moratuwa, at the request of the contractor of a slope rectification project. The results of the tests carried out are presented in Table 6-1.

Table 6-1. Tested properties of a proprietary metallic mesh used for slope rectification

Tested Property	Unit	Result	
		Average	Standard deviation
Tensile Strength of Wire	N/mm²	f_m	σ
Opening Size of Mesh	mm	$D_{i,m}$	σ_{in}
Tensile Strength of wire mesh	kN/m	T_m	σ_T

The results obtained from tests could be used to calculate the bearing resistance of the mesh against punching. The physical parameters of the mesh vary from manufacturer to manufacturer. Shown in Figure 6-3 is a general diagram of the mesh considered in this particular case.

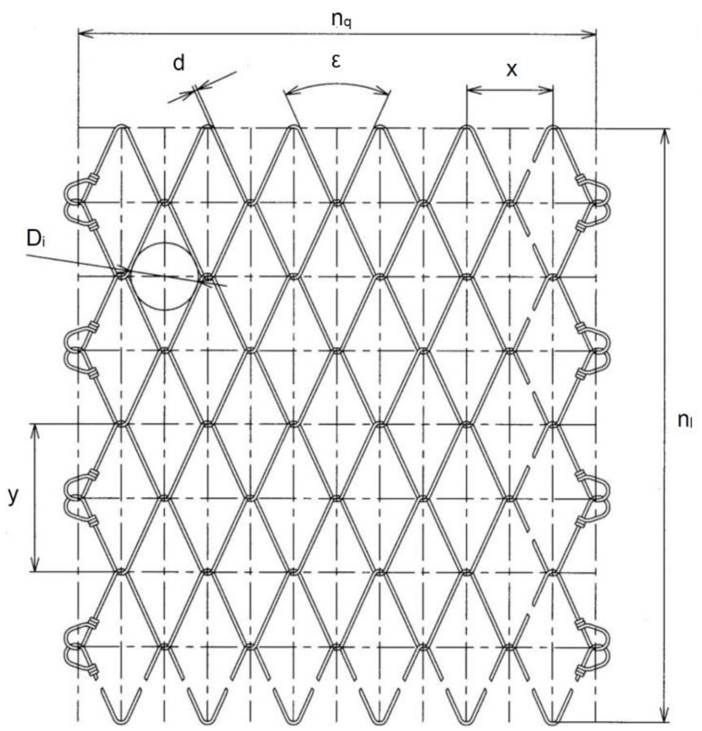


Figure 6-3. Physical parameters of a flexible metallic mesh (Geobrugg product datasheet)

Calculation of the bearing capacity of the steel mesh against punching

Characteristic tensile strength of wires	f_t	$=f_m$-1.64σ	
Allowable tensile strength of wires	f_y	$= \dfrac{f_t}{1.15}$	BS 8110-1:1997
Shear strength of wires	τ_y	$= \dfrac{f_y}{\sqrt{3}}$	Rüegger et al (2001)
Individual Wire Diameter	d		
Shear area of the wire	a_t	$= \pi \times \dfrac{d^2}{4}$	
Shear resistance of a wire	S	$= \tau_y \times a_t$	

Number of rhombuses in the vertical direction $= N_{rv}$

Number of rhombuses in the horizontal direction $= N_{rh}$

Dimensions of bearing plate $= b_{pv} \times b_{ph}$

Number of wires crossing the vertical peripheries of the plate, N_{wv}	$= \left(\dfrac{N_{rv}}{1000} \times b_{pv}\right) \times 2$	
Number of wires crossing the horizontal peripheries of the plate, N_{wh}	$= \left(\dfrac{N_{rh}}{1000} \times b_{ph}\right) \times 2$	
Design punching resistance of the mesh	S_m	$= S \times (N_{wh} + N_{wv})$

From the above calculation, it can be realized that the punching resistance of the mesh varies with the number of contact points around the plate used at the nailhead. Therefore, usage of an adequately sized bearing plate is equally important as using a mesh with adequate strength, wire diameter, and opening size. Manufacturers produce bearing plates with customized shapes and sizes to suit each of their proprietary meshes. These plates have been designed with the aim of achieving the maximum number of crossing points when used with meshes produced by them. Additionally, these bearing plates demonstrate enhanced longitudinal bending resistance than a standard flat plate of the same thickness due to their 3d construction.

Notwithstanding the above, the testing carried out as per standard guidelines such as EAD 230025-00-0106 (European Assessment Document for Flexible Facing Systems) on the metallic meshes have revealed higher puncturing resistance values than the theoretical values calculated using the above manner. This could be due to the many factors other than the shear strength of wires that contribute to the punching shear resistance. As such, in cases where required testing cannot be carried out locally, the designers could use the manufacturer-tested values of puncturing resistance in the designs. However, the products must carry a reliable product certification such as ETA (European Technical Assessment) in order to confidently use the values presented by the mesh manufacturer in the designs.

6.3 Application for a Case History

6.3.1 Flexible facing design for Hakgala landslide mitigation using the presented procedure

The applicability of a Grid Beam facing for the CS5 area of the Hakgala landslide mitigation has been discussed in Section 5.3. As an alternative to the Grid Beam system which is used to transfer the nailhead loads to the slope face, a flexible facing system could be used to sustain the potential failure of the soil mass between the Grid Beam system.

Accordingly, the method proposed in Section 6.2 will be used to determine the applicability of a flexible facing on top of the proposed Grid Beam system. In this exercise, the soil nail arrangement was considered as $S_V = S_H = 2\ m$ similar to the previous previous applications.

Obtaining the punching shear force applied on the flexible facing

Two-wedge failure mode

1. Parameters required

Slope angle	β	$= 66°$
Breadth of Grid Beam	b	$= 250\ mm$
Vertical nail spacings	S_V	$= 2\ m$
	H	$= 2 - 0.25 \times \sin(\beta)$
		$= 1.77\ m$

Horizontal nail spacing	S_H	$= 2\ m$
Angle of soil nail	α	$= 15°$
Density of soil	γ	$= 18\ kN/m^3$
Friction angle of soil	ϕ'	$= 28°$
Cohesion of soil	c'	$= 0$
Pore water pressure ratio	r_u	$= 0.5$

2. Using the equations set out in Section 6.2 and Equation (2.23), with trial and error procedure, obtained the θ_1 and $\theta_2$values that produce the maximum $T_1 + T_2$ value.

The wedge angles that produce the most critical failure mode were;

$\theta_1 = 90°$
$\theta_2 = 14°$

The parameters of Equation (2.23) have been calculated as follows;

$$X = \frac{H}{\tan \beta} \times \frac{\tan \beta - \tan \theta_1}{\tan \theta_2 - \tan \theta_1} = \frac{1.77}{\tan 66} \times \frac{\tan 66 - \tan 90}{\tan 14 - \tan 90}$$

$$= 0.788$$

$$Y = Xtan(\theta_2) = 0.788 \times \tan(14)$$

$$= 0.196$$

$$a = H - X \tan \beta = 1.77 - 0.788 \times \tan(66)$$

$$= 0.00$$

$$b = H - Y - a = X \tan \beta - Y = 1.77 - 0.196 - 0$$

$$= 1.574$$

$$u = b / \tan \theta_1 - \tan \beta = 1.574/(\tan 90 - \tan 66)$$

$$= 0$$

$$m = \sqrt{X + Y} = \sqrt{0.788 + 0.196}$$

$$= 0.81$$

$$W_1 = 1/2\,\gamma b\, u = 0$$

$$W_2 = 1/2\,\gamma b\, X = 11.16\ kN/m$$

$$U_1 = 1/2 r_u \gamma bu \sec \theta_1 = 11.14\ kN/m$$

$$U_2 = 1/2 r_u \gamma b m = 5.75\ kN/m$$

$$K_1 = c_1' \, e + f + w = c_1' \, g + w = 0$$

$$K_2 = c_2' m = 0$$

Accordingly, from Equation (2.23);

$$(T_1 + T_2)_{max} = 11.142 \; kN/m$$

3. Using the maximum value of $T_1 + T_2$ on Equation (2.24), obtain the maximum force on the flexible facing in the direction of the soil nails $(T_1 + T_2)_\delta$

$$\begin{aligned}
\zeta &= [\cos(\theta_1 - \phi_1') / \cos(\theta_1 - \phi_1' + \delta)] \\
&= [\cos(90 - 28)/\cos(90 - 28 + 15)] \\
&= 2.087 \\
(T_1 + T_2)_\delta &= \zeta \cdot (T_1 + T_2)_{max} \\
&= 23.253 \; kN/m
\end{aligned}$$

4. Total punching force induced per nailhead

$$\begin{aligned}
(T_1 + T_2)_{\delta p} &= (T_1 + T_2)_\delta \times (S_H - b) \\
F &= \mathbf{40.692 \; kN}
\end{aligned}$$

<u>Single-wedge failure mode</u>

Use the equation presented in Section 2.7.2. All input parameters were similar to that of the two-part wedge calculation. The calculation carried out using a spreadsheet was iterative and the most critical θ value obtained has been used in this sample calculation.

$$F = \frac{[W(\tan\theta - \tan\phi') + (\mathrm{U}\tan\phi' - \mathrm{K})/\cos\theta]}{(1 + \tan\theta\tan\phi')}$$

$$= \frac{[35.47(\tan 43.65 - \tan 28) + (\mathrm{U}\tan 28 - 0)/\cos 43.65]}{(1 + \tan 43.65 \tan 28)}$$

$$= 15.91 \; kN < 40.692 \; kN$$

As such, the value obtained from the two-wedge mode, $F = \mathbf{40.692 \; kN}$ will be used for the flexible facing design.

Punching shear resistance of the proposed flexible mesh – plate system

The punching resistance of a rhomboidal metallic mesh product proposed for the work could be calculated as follows

1. Details on the mesh used

Testing had been carried out at the Department of Material Science and Engineering University of Moratuwa, for the proposed mesh (TECCO G65/3) for this slope rectification project. Test results of the mesh have been presented in Table 6-2.

Table 6-2. Tested properties of a proprietary metallic mesh used for slope rectification

Tested Property	Unit	Result	
		Average	Standard deviation
Tensile Strength of Wire, f_w	N/mm²	$f_m = 1971.2$	$\sigma = 8.7$
Opening Size of Mesh	mm	$D_{i,m} = 65$	$\sigma_{in} = 1$
Tensile Strength of wire mesh	kN/m	$T_m = 153.4$	$\sigma_T = 2.3$

2. Calculation of the bearing capacity of the steel mesh against punching

Characteristic tensile strength of wires, f_t	$= f_m - 1.64\sigma$	$= 1957\ N/mm^2$	
Allowable tensile strength of wires, f_y	$= \dfrac{f_t}{1.15}$	$= 1701.74\ \mathrm{N/mm^2}$	BS 8110-1:1997
Shear strength of wires, τ_y	$= \dfrac{f_y}{\sqrt{3}}$	$= \dfrac{1701.41}{\sqrt{3}}$ $= 982.31\ \mathrm{N/mm^2}$	Rüegger et al (2001)
Individual Wire Diameter, d	$= 3\ mm$		
Shear area of the wire, a_t	$= \pi \times \dfrac{d^2}{4}$	$= \pi \times \dfrac{3^2}{4}$	$= 7.07\ mm^2$
Shear resistance of a wire, S	$= \tau_y \times a_t$	$= 982.31 \times 7.07$	$= 6.94\ kN$
Number of rhombuses in the vertical direction, N_{rv}	$= 7\ units/m$		

Number of rhombuses in the horizontal direction, N_{rh}	$= 12\ units/m$	
Dimensions of bearing plate	$= 200\ mm \times 200\ mm$	
Number of wires crossing the vertical peripheries of the plate, N_{wv}	$= 2 \times 2$	$= 4\ nos$
Number of wires crossing the horizontal peripheries of the plate, N_{wh}	$= 5 \times 2$	$= 10\ nos$
Design punching resistance of the mesh, S_m	$= S \times (N_{wh} + N_{wv})$	
	$= 6.94 \times (3 + 10)$	$= 90\ kN$

As discussed in 6.2, the testing carried out as per EAD 230025-00-0106 (European Assessment Document for Flexible Facing Systems) using the steel meshes have revealed higher puncturing resistance values than the theoretical values calculated using the above method. Accordingly, the tested strength parameters of the same mesh (TECCO G65/3, wire strength = 1770 N/mm^2, wire diameter = 3 *mm*, opening size = 65 *mm*) can be used to assess its adequacy with the following modifications;

Published punching resistance of the mesh when the plate covers 16 nos of wires	$= 180\ kN$	
Punching resistance of the mesh which covers 14 nos of wires, P_R	$= 180 \times \frac{14}{16}\ kN$	$= 157.5\ kN$
Design value of punching resistance, S_m	$= \frac{157.5}{1.5}$	$= \mathbf{105\ kN}$
Total punching force induced per nailhead, F	$= 40.69\ kN$	$\leq \frac{P_R}{1.5}$

As such, the TECCO G65/3, (wire strength = 1770 N/mm^2, wire diameter = 3 *mm*, opening size = 65 *mm*) is adequate to be used for protection against the local instabilities between the grid beams

It is important to note that if a plate with a shape and a size that achieves the coverage of 16 nos of wires, a mesh of lower strength capacities than the proposed one could be used. This coverage can be achieved by using plates produced by mesh manufacturers with customized shapes and sizes to suit each of their proprietary meshes as discussed in Section 6.2.

However, it seems that a mesh with a lower punching shear capacity than the above mesh could also be adequate for this purpose.

Alternatively, for comparison, another mesh type (3STUTOR Plus 100/2.7, wire strength = 900 N/mm^2, wire diameter = 2.7 *mm*, opening size = 75 *mm*) of which the 200 x 200 mm plate covers 14 wires could also be assessed in this manner;

Published punching resistance of the mesh when the plate covers 16 nos of wires	$= 72\ kN$	
Punching resistance of the mesh which covers 14 nos of wires, P_R	$= 72 \times \frac{14}{16}\ kN$	$= 63\ kN$
Design value of punching resistance, $P_R/1.5$	$= \frac{63}{1.5}$	$= \mathbf{42\ kN}$
Total punching force induced per nailhead, F	$= 40.69\ kN$	$\leq \frac{P_R}{1.5}$

As such, instead of the TECCO G65/3 with higher strength (1770 N/mm^2) material, the 3STUTOR Plus 100/2.7 with a lower strength (900 N/mm^2) could be safely used for the protection of local instabilities in-between the proposed grid beam facing.

6.3.2 Design of flexible facing for different nail spacings and excavation angles

The method used in Section 6.3.1 for obtaining the forces exerted on the flexible facing for Hakgala landslide mitigation could be used to calculate the forces for alternative slope excavation angles. Figure 6-4, presents the results obtained from this analysis.

Further, an analysis has been carried out to observe the variation of out-of-balance force on the mesh with nail spacing. Results have been presented in Figure 6-5. It must be noted that horizontal spacing is taken as equal to vertical spacing in this analysis.

As discussed in Chapter 3, Chapter 4, and Chapter 5 similar to the nailhead load and the nailhead resistance of hard structural facings, the force exerted on flexible facings also increases with nail spacing. However, it could be realized that this out of balance force is increasing exponentially with nail spacing. In addition, Figure 6-4 shows that the slope excavation angle is not having an exponential growth and this could be due to the nature of failure taking a toppling form when the angle increases.

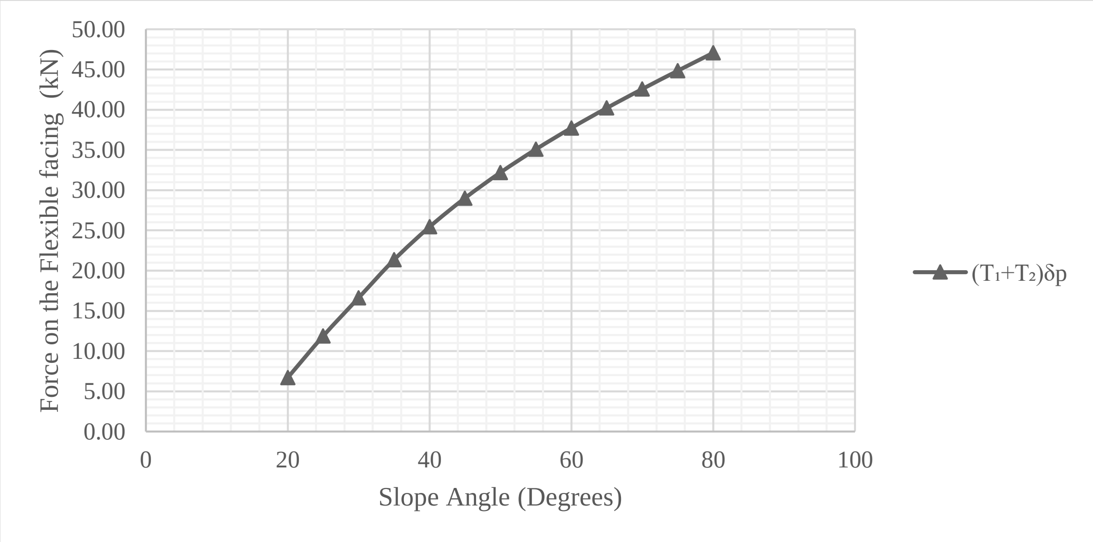


Figure 6-4. Force in the flexible mesh vs Slope angle considered for CS5 of Hakgala landslide mitigation

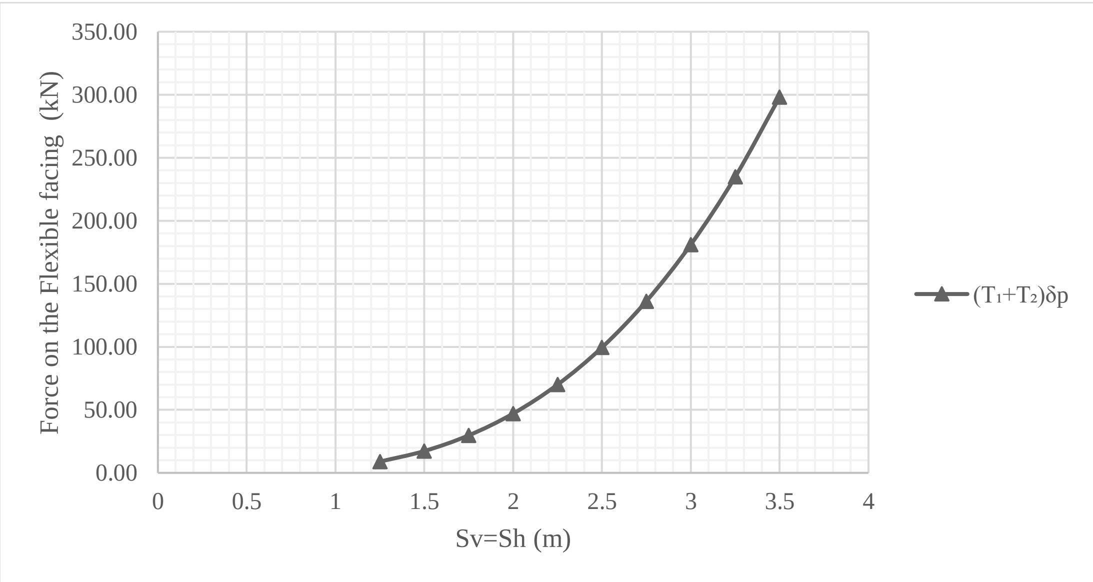


Figure 6-5. Force on the flexible mesh vs Nail spacings considered for CS5 of Hakgala landslide mitigation

6.3.3 The effect of cohesion

It must be noted that the above analysis has been carried out not taking the cohesion into account. This is mainly because, the slopes under the flexible mesh being exposed to rainfall all the time. However, as discussed, a vegetation cover is also grown on slopes through the flexible mesh. This could result in apparent cohesion which may not be inhibited by rainfall. As such, the above analysis could be carried out to observe the effect of cohesion. The variation of out of balance force with cohesion is presented in Figure 6-6.

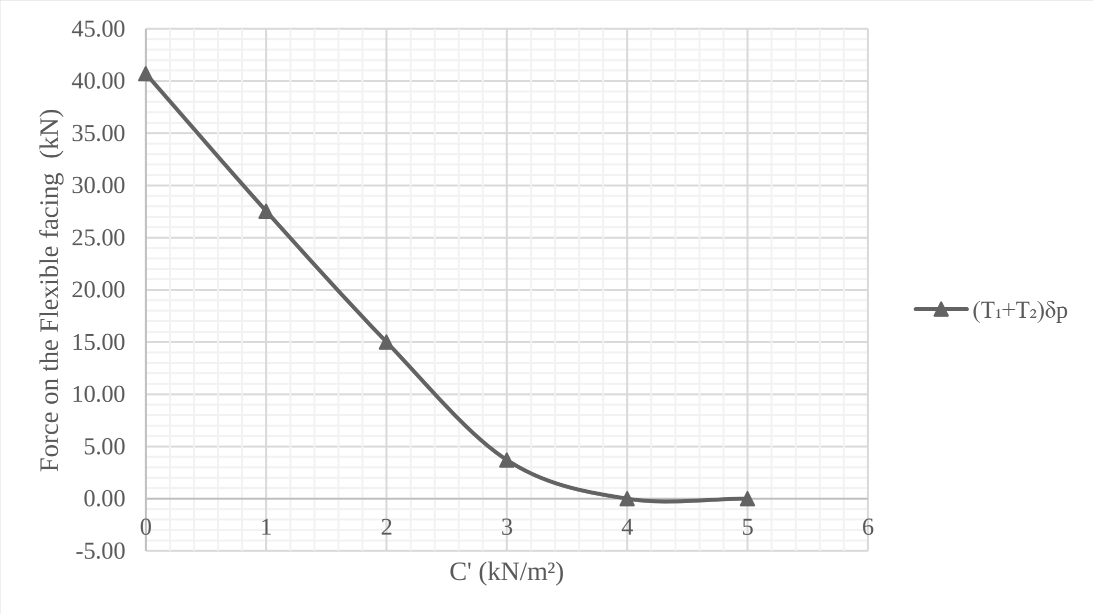


Figure 6-6. Force on the flexible mesh vs Cohesion considered for CS5 of Hakgala landslide mitigation

Despite carrying out analysis without utilizing the cohesion as per the recommendation of literature, it could be noted that a smaller amount of cohesion in the soil could significantly reduce the out-of-balance force on the flexible mesh. The effect of cohesion could be further studied by reviewing the following expressions used in Equation (2.23).

$$K_1 = c_1' u sec(\theta_1) \quad = 1 \times \left(\frac{1.467}{\tan 90 - \tan 66}\right) \times \frac{1}{\cos 90}$$

$$= 1 \times \left(\frac{1.467}{1 - 0} \times \left(\frac{\cos 90}{\sin 90}\right)\right) \times \frac{1}{\cos 90}$$

$$= 1.467$$

$$K_2 \quad = c_2^{'}m \qquad = 1 \times 0.844$$

$$= 0.844$$

Having studied the effect of cohesion on the out of balance force on the flexible facing, it could be realized that although in small amounts, the cohesion available on soil could drastically reduce the structural requirements of the flexible meshes used. It is seen that cohesion value of 4 kN/m^2 could make the out of balance force disappear and one can assure that a mesh facing is not required to sustain local instabilities. This nailhead arrangement where the slope face could be left unsupported has been discussed in Section 2.7.3 as the "Critical vertical distance". The published analysis results of GEO report No. 175 have been shown in Figure 2-28.

As such, the design could be optimized significantly when the cohesion of soil is taken into consideration, in cases where the soil is not predominantly sandy, and the growth of vegetation is successful. However, particularly in cases where the permeability of topsoil is high, relic joints and cracks are present, and the deformation of the soil nail system is high, the presence of cohesion should be thoroughly assessed and justified.

6.3.4 Flexible facing design for the case history using different methods

Method suggested by Ruegger et. al

As discussed in 2.7.4, two mechanisms were proposed by Ruegger et. al, namely one-body and two-body mechanism could also be used to assess the mesh requirement of the case history.

<u>One-body method</u>

Calculations were carried out using equation (2.28) considering the wet condition (streaming effect). Similar to the CIRIA c637 methods, the calculations have been carried out iteratively using a spreadsheet where wedge angle (β) was altered until obtaining the maximum value of out of balance force P.

1. Parameters considered in the calculation

Slope angle α $= 66°$

Breadth of Grid Beam	br	$= 250\ mm$
Vertical nail spacings	S_V	$= 2\ m$
Horizontal nail spacings	S_H	$= 2\ m$
Effective horizontal spacing	a	$= S_H - 0.250$
		$= 1.75\ m$
Heigh of failing body along the slope	$2b$	$= 2/\sin(66)\ - 0.25$
		$= 1.94\ m$
Angle of soil nail	ψ	$= 15°$
Density of soil	γ	$= 18\ kN/m^3$
Design friction angle of soil	ϕ'/γ_ϕ	$= 23.04°$
Design cohesion of soil	c'/γ_c	$= 0$

The original method proposed by Ruegger et. al. utilizes the pretension force on the mesh at the nailhead and considers a reduction in the width of failure mass, a. Since pretension is not done on regular mesh products, this enhancement has been omitted from the calculation. Moreover, a slope parallel force Z is also used by the method proposed by Ruegger. The method of obtaining this value has not been documented conclusively. As such this resistive force has also been omitted in this calculation having a conservative design in mind. However, if conclusive evidence on the value of this slope parallel force exerted on the soil body is available, designers may use them in equation (2.28) as it is.

2. Using the equation (2.28), with trial and error procedure, obtained the β values that produces the maximum P value.

The wedge angles that produce the most critical failure mode were;

$$\beta \quad = 37.66°$$

Accordingly,

Out of balance force $\quad P \quad = 24.97\ kN$

Two-body method

1. Parameters considered in the calculation

All design parameters were identical to that of the One-body method.

Similar to the above the reduction in the width of failure mass, a as well as the resistive force Z have been omitted from the calculation to be on the safer side.

2. Using the equation (2.32), with trial and error procedure, obtained the β and t values that produces the maximum P value.

The wedge angle and the thickness of the soil layer under failure that produces the most critical failure mode were;

$$\beta = 36°$$

$$t = 0.95\ m$$

Accordingly,

Out of balance force P $= 26.17\ kN$

It could be noted that the values obtained from the one-body and two-body mechanisms of this method are more or less in the same range. However, it is clearly seen that the Two-part wedge method proposed by CIRIA c637 has produced a larger value (40.692 kN) than all other methods. This difference in outputs is mainly caused by the differences in the way the two models handle the groundwater conditions.

6.3.5 Conclusion

In addition to the calculations carried out above the structural capacity of the bearing plate should also be assessed. Recommendation on the bearing plate sizes and thicknesses to be used with meshes is also presented in Section 7.3.

The way a flexible mesh interacts with the slope, in reality, involves much more complexity. The forces depend on the interaction with the soil nails, soil, mesh, and relative stiffness. Therefore, for slopes with high-risk levels of facing failure, and where a Grid Beam facing is not used alongside the mesh to cater to the nailhead forces, numeric modeling could be used to obtain the facing loads accurately.

An important fact to be considered is that the soil nail arrangement of a system with isolated nailheads such as Pillows is always kept in a staggered arrangement to maintain the integrity between the soil nails. Because of this difference, the height of the two-part-wedge model (H) of an isolated concrete block (pillow) facing is actually twice the vertical nail spacing. This fact could be identified from the failure diagram presented in Figure 6-1. Because of the presence of grid beam sections across each nail row, the critical slope section of local instability is limited to space in-between the grid beams, or the vertical nail spacing. A calculation for the mesh forces in the case of isolated nailheads is presented in Section 7.3.1.

This gives an edge to the grid beam facing method as a metallic mesh with a lower punching resistance than that of the pillow method can be fixed to the slope to protect against the local instabilities.

Accordingly, for the slope considered for the case history, the out of balance force from local instabilities where each facing type is used with their typical nail spacing could be compared as shown in Table 6-3

Table 6-3. Comparison of punching force requirement of pillow method and grid beam method

Facing Type	Soil nail spacing ($S_V = S_H$) (m)	Height of local failure model (m)	Punching force on the mesh (kN)
Isolated Nailheads	2	4	95.19
Grid Beams	2	2	40.69

CHAPTER 7. DESIGN OF ISOLATED NAILHEADS

7.1 Background

This section discusses the design aspects of the isolated nailheads used as soil nail facings which are often referred to in the local practice as soil nail pillows.

Nailheads must be capable of anchoring the soil nails at the slope surface to withstand the nailhead load discussed in Section 2.9 and Chapter 3. The main function of isolated nailheads is to transfer the load exerted on the nailhead to the slope face. As discussed in Chapter 4 and Chapter 5, the Grid Beams, as well as the Shotcrete facings, serve the same purpose and the nailhead load will be transferred across the entire slope face. As such, the anchoring capacity of hard facing such as Grid beams and full-face Shotcrete is evaluated based on the structural resistance of the facings. However, in the case of isolated nailheads, the load will be transferred to a smaller area of slope and hence the capacity is governed by the bearing capacity of the slope surface against the nail head load applied. This loading arrangement makes the Pillow facing similar in function to a pad footing as discussed in the literature survey.

Methods to calculate the Pillow sizes required for a particular slope arrangement have been presented in HA 68/94 British guide, Hong Kong GEO publications as well as the CIRIA c637 report.

7.2 Procedure for designing Isolated Nailheads

Out of the published methods, the HA 68/94s (The Highways Agency, 1994) lower bound approach could be considered the most used and recommended on most other facing design guides. This method has been further modified in GEO report No. 175 (Shiu & Chang, 2004) with recommendations on obtaining the nailhead load. The GEO report also compares the results of their FLAC finite element analysis with the results of the lower bound calculation where they have observed comparative results. Additionally, the methods followed in Chapter 3 to obtain the nailhead load from SlopeW analysis could be used here as well with some modifications.

Using the lower bound method to obtain the nailhead load

The expressions presented in Figure 2-34 could be used to calculate the nailhead size requirement. The method has been adopted from the guides in HA 68/94 and will not be modified as per the recommendation on GEO report No. 175 (removes the effect of the active zone on the nailhead load). The suggestion of the GEO report, which uses the maximum tensile force of soil nails derives unrealistically larger facing sizes. Therefore, the results have been obtained using the maximum nailhead load as per the studies carried out in Chapter 3. Step by step guide for the calculation is presented in the following section.

Basic parameters for design

1. Acquire the parameters of the soil nailed slope

Slope angle	β
Angle of soil nail	δ
Unit weight of soil	γ
Angle of shearing resistance of soil	ϕ'
Pore water pressure ratio	r_u

2. Calculate parameter η as per expression given in Equation (2.40)

$$\eta = \frac{\gamma(1 - r_u)\tan\beta e^{3\left(\frac{\pi}{4}-\frac{\phi'}{2}+\delta\right)\tan\phi'}}{2\cos\left(\frac{\pi}{4}+\frac{\phi'}{2}\right)\left(1 - \sin\phi'\right)}$$

3. Obtain the nailhead load P_{nail} as per the SlopeW analysis from Chapter 3

$$P_{nail} = Nailhead\ load, T_{0,ULT}$$

4. Then consider a trial nailhead size (a_{req}) and calculate P

$$P = T_{0,ULT}/a_{req}$$

5. Check the trial a values using Equations (2.39)

$$a_{req} \geq \left(\frac{P}{\eta}\right)^{\frac{1}{3}}$$

6. Repeat to obtain the optimum practical a_{req} value is obtained

Using bearing plate in place of the concrete pillow (Bearing plate method)

In some instances, the bearing plate can be used as the member providing the nailhead resistance of the facing. However, in this method, the nailhead plates must be designed for their structural performance. These calculation steps have been discussed in section 2.8.2 and reiterated here.

Pressure acting on the plate, p'	$= \frac{\mathrm{T}_{max}}{\mathrm{A}_{\mathrm{plate}}}$	$= \frac{T_{0,ULT}}{a_{req}^2}$
Section modulus of the plate at the centerline, Z	$= \frac{w - D_h \ \times t^2}{6}$	
Moment exerted at the centerline, M_{zz}	$= \frac{p' \times w^2}{2} \times \left(\frac{w}{4}\right)$	
Maximum stress on the plate, σ	$= \frac{M_{zz}}{Z}$	

Where;

$\mathrm{A}_{\mathrm{plate}}$	Gross area of the plate
w	Width of the plate
t	Thickness of the plate
D_h	Diameter of the hole of the plate

In addition, the bearing plate must also be properly sized to achieve the bearing capacity required to transfer the effective nailhead load onto the soil underneath. In this case, the effective nailhead load transferred to the soil by the bearing plate will be calculated as shown below.

Effective nailhead load transferred to the soil by bearing plate $= T_{0,ULT} - D_r$

Where;

$T_{0,ULT}$	Ultimate nailhead load
D_r	Bearing resistance of the steel mesh against stress in the nail direction

7.3 Application for a Case History

7.3.1 Isolated nailhead design for Hakgala landslide mitigation project using the proposed method

The applicability of a Grid Beam facing as well as a Shotcrete facing for the CS5 area of the Hakgala landslide mitigation has been discussed in Section 5.3 and Section 4.3. Additionally, depending on the erosion characteristics of the topsoil, isolated nailheads can also be used to anchor the soil nails

Accordingly, the method proposed in Section 7.2 is used to determine the applicability of an Isolated Nailhead facing for the CS5 area of Hakgala Landslide mitigation. In this exercise, the soil nail arrangement was considered as $S_V = S_H = 2\ m$ similar to the previous application.

<u>Design calculations of isolated nailhead</u>

1. Acquire the parameters of the soil nailed slope

Slope angle	β	$= 66°$
Angle of soil nail	δ	$= 15°$
Unit weight of soil	γ	$= 18\ kN/m^3$
Angle of shearing resistance of soil	ϕ'	$= 28°$
Pore water pressure ratio	r_u	$= 0.15$

2. η as per expression given in Equation (2.40)

$$\eta = \frac{\gamma(1 - r_u)\tan\beta e^{3\left(\frac{\pi}{4}-\frac{\phi'}{2}+\delta\right)\tan\phi'}}{2\cos\left(\frac{\pi}{4}+\frac{\phi'}{2}\right)\left(1-\sin\phi'\right)} = \frac{18\ \ 1 - 0.15\ \ \tan(66)e^{3\left(\frac{\pi}{4}-\frac{28}{2}+15\right)\tan 28}}{2\cos\left(\frac{\pi}{4}+\frac{28}{2}\right)\ \ 1-\sin 28}$$

$$= 226.31$$

3. Nailhead load P_{nail}

As per Section 3.3.1,

$$P = T_{0,ULT}$$
$$= 116.27\ kN$$

4. Check the adequacy of nailhead size, *a* using Equations (2.39)

$$a_{req} = \left(\frac{P}{\eta}\right)^{\frac{1}{3}} = \left(\frac{116.27}{226.31}\right)^{\frac{1}{3}}$$

$$= 801\ mm \qquad say\ 800\ mm$$

As such, an Isolated Nailhead of 800 mm x 800 mm could be satisfactorily used to counter the expected nailhead load induced by soil nails at the CS5 area of the Hakgala Landslide mitigation. As per the recommendation of GEO report No. 175, the thickness of the element will be taken as 250 mm. From a structural calculation similar to what's done for Shotcrete facings, it can be shown that providing minimum reinforcement is adequate to satisfy the structural integrity of an Isolated Nailhead where the minimum thickness of 250 mm is achieved.

However, due to the unrealistically larger Pillow size, it could be recommended that the Grid Beam solution could be more suited for this location than the isolated nailhead system. This scenario has been already discussed in 2.3 where the GEO report has pointed out using Grillage structures instead of isolated nailheads when the slope angle exceeds 65°.

<u>Use of a bearing plate in place of a concrete pillow</u>

Steel plates could be used in place if concrete pillows given the required structural integrity is satisfied. The difference in the installation is that the pillows are usually embedded in the slope surface while a plate is used to secure the mesh on top of the pillow as shown in Figure 2-32. This plate used on top of the pillow only carries loads of local instabilities transferred by the mesh. However, where a steel plate is used in place of the concrete pillow, the plate has to carry both nailhead load as well as the forces induced by the local instabilities. Moreover, since the mesh is fixed underneath the plate, the mesh carries some portion of the nailhead load. This proportion of punching shear absorbed by the mesh is often tested by manufacturers and presents in product datasheets.

It must be noted that this method could significantly reduce the construction time of soil facing because no casting or excavation for structures is required. As such this method could be considered very effective in instances where the time consumed for construction is extremely critical.

As per the calculation carried out in 6.3.1, the design value of punching shear resistance of the (TECCO G65/3) mesh was obtained. Based on this value, the bearing capacity requirement of the plate could be determined.

Plate size used	$= 250 \times 250\ mm$	
Wires covered by the plate	$= 16\ nos$	
Published punching resistance of the mesh when the plate covers 16 nos of wires	$= 180\ kN$	
Design value of punching resistance of the mesh used, $P_R/1.5$	$= \frac{180}{1.5}\ kN$	$= 120\ kN$
Bearing capacity requirement of plate+soil	$= T_{0,ULT} - 120$	
	$= \mathbf{0\ kN}$	

Therefore, using a plate of 250 x 250 mm on top of TECCO G65/3 mesh in place of an isolated pillow nailhead satisfies the nailhead load resistance requirement.

However, if the mesh capacity was lower than the design nailhead load $T_{0,ULT}$, the lower bound method proposed earlier could be used to determine the plate plan size requirement:

The required thickness and steel yield capacity could be calculated in the following manner.

The dimension of the trial plate:

Thickness	= t	$= 20\ mm$
Yield strength of steel	$= f_y$	$= 275\ N/mm^2$

Calculation:

Pressure acting on the plate, p' $= \frac{T_{0,ULT}}{A_{plate}} = \frac{116.27}{250 \times 250}$

$= 1.86\ N/mm^2$

Section modulus of the plate at the centerline, Z $= \frac{(w - D_h) \times t^2}{6}$ $= \frac{(250 - 32) \times 20^2}{6}$

$= 14466.67\ mm^3$

Moment exerted at the centerline, M_{zz} $= \frac{p' \times w^2}{2} \times \left(\frac{w}{4}\right)$ $= \frac{1.86 \times 250^2}{2} \times \left(\frac{250}{4}\right)$

$= 3633437.5\ kNm$

Maximum stress on the plate, σ $= \frac{M_{zz}}{Z}$ $= \frac{3633437.5}{14466.67}$

$= 251.16\ N/mm^2$

As such, a steel plate of $f_y = 275\ N/mm^2$ thickness 20 mm, and size 250 x 250 mm could be used as an isolated plate nailhead.

However, the use of a heavier plate could cause unnecessary bending forces on the soil nail thread, as such, a proprietory plate produced by mesh manufacturers which has enhanced moment capacities would be more appropriate in these cases.

<u>The use of flexible facing to address local instabilities between isolated nailheads</u>

The calculation of out of balance force could be carried out following the steps shown in 6.3.1 for the flexible facing design for the Grid beam method. However, when the flexible meshed is used with isolated nailheads, the height of the potentially failing soil mass, H value shall be taken as follows:

Slope angle	β	$= 66°$
Isolated nailhead size	a_{req}	$= 800\ mm$
Vertical nail spacings	S_V	$= 2\ m$
	H	$= (2 \times 2) - 0.8 \times \sin(\beta)$
		$= 3.27\ m$

Accordingly, from Equation (2.23);

$$(T_1 + T_2)_{\max} = 79.324\ kN/m$$

Total punching force-induced per nailhead

$$(T_1 + T_2)_{\delta p} = (T_1 + T_2)_{\delta} \times (S_H - a_{req})$$
$$F = 79.32 \times (2 - 0.8)$$
$$= 95.19\ kN$$

This punching shear force exerted on the mesh is a considerable increase compared to the value achieved from the Grid beam system (40.69 kN) although the same nail spacing is maintained in both instances. As such, a flexible mesh and a plate that mobilizes higher punching shear resistance should be used with this Isolated nailhead system.

As calculated in 6.3.1, the mesh of wire strength = 1770 N/mm^2, wire diameter = 3 *mm*, opening size = 65 *mm* (TECCO G65/3) could be considered for this.

Design punching resistance of TECCO G65/3, S_m $= \mathbf{97.5\ kN} > 95.19\ kN = F$

Alternatively, for comparison another mesh type (3STUTOR Plus 100/4.0, wire strength = 900 N/mm^2, wire diameter = 4 *mm*, opening size = 75 *mm*) of which the 200 x 200 mm plate covers 14 wires could also be assessed in this manner;

Published punching resistance of the mesh when the plate covers 16 nos of wires	$= 170\ kN$	
Punching resistance of the mesh which covers 14 nos of wires, P_R	$= 170 \times \frac{14}{16}\ kN$	$= 148.75\ kN$
Design value of punching resistance, $P_R/1.5$	$= \frac{148.75}{1.5}$	$= \mathbf{99.17\ kN}$
Total punching force-induced per nailhead, F	$= 95.19\ kN$	$\leq \frac{P_R}{1.5}$

As such, instead of the TECCO G65/3 with higher strength (1770 N/mm^2) material, the 3STUTOR Plus 100/4.0 with a lower strength (900 N/mm^2) could be safely used for the protection of local instabilities in-between the proposed isolated nailheads (Pillows).

7.3.2 Design of isolated nailheads for different nailing arrangements

The lower bound method used in Section 7.3.1 has been used for obtaining the nailhead sizing for Hakgala landslide mitigation for alternative soil nail spacings. The nailhead forces at each nail spacing were obtained from Table 3-4. Table 7-1 presents the results obtained from this analysis.

Table 7-1. Required size of isolated nailhead vs nail spacing considered for CS5 of Hakgala landslide mitigation

Soil nail spacing (S_V = S_H) (m)	Nailhead Load (kN)	Minimum Pillow size required (mm)
1.5	69.86	676 x 676
2	116.27	800 x 800
2.5	136.69	845 x 845

It can be observed from these results, that the nailhead size is increasing with increasing nail spacing. As such, the nail spacing shall be appropriately kept within the limits recommended in Section 2.3 to prevent having to design heavy uneconomical pillows nailheads.

7.3.3 Design of isolated nailheads for different slope excavation angles

Calculations have been carried out to compare the variation of bearing resistance against the friction angle of the soil when the spacing is kept at $S_V = S_H = 2\ m$. All other parameters in the analysis have been kept the same as the case history. The results are presented in Table 7-2.

Table 7-2. Variation of Bearing Resistance of Nailheads, Lower bound method, considered for CS5 of Hakgala landslide mitigation

Slope angle considered (degrees)	Nailhead Load (kN)	Minimum Pillow size required (mm)
40	24	657
50	57.29	781
60	92.62	810
70	129.41	776
80	169.45	667

It should be noted that lower nailhead sizes are required at shallower slope angles. However, this has resulted from the lower nailhead force at shallower slope angles and not because of a gain in nailhead capacity. Also, it could be seen that the required pillow size decreasing as the slope angle increases regardless of the increased nailhead load where the slope angle exceeds 60°. This is predominantly because the bearing resistance in this calculation step has been obtained considering the overburden soil above the failure wedge at the base of the nailhead which increases significantly at steeper slope angles.

However, the GEO reports recommend using Grid Beams instead of isolated nailheads when the slope angle exceeds 65°. This could be due to the unrealistically large nailhead sizes as well as because of the local instabilities which will be predominant at such steep slope angles as discussed in Chapter 6. As such, the use of Pillows for the slope considered for the case history could not be recommended despite the above calculations.

CHAPTER 8. PERFORMANCE ASSESSMENT OF FACINGS ON FIELD

8.1 Introduction

At the time of conducting the research study, all the different soil nailing facing types discussed herein have been constructed in various slope stabilization sites around Sri Lanka. Throughout the research, an attempt has been made using the published research and design methods, to assess the adequacy of these facing types for the ground conditions in Sri Lanka at high rainfall intensities. Therefore, as a means of triangulating the findings of the research, a post-construction performance assessment was carried out using a set of soil nailing sites in Sri Lanka chosen to cover all the facing types in the discussion.

As per the FHWA, instrumentation, and monitoring should be carried out in “critical or unusual” soil nailed slopes such as those with high surcharge loads (Carlos A. Lazarte, et al., 2015). A similar recommendation is given in Hong Kong Geoguide 7 where it states, soil nail installations on which sustained loads are exerted shall be monitored for at least two rainy seasons after construction (Geotechnical Engineering Office, 2008). Therefore, most of the mitigated slopes in Sri Lanka have not been categorized as critical soil nail installations for which dedicated monitoring is necessary to be used.

As a result, many of the soil nailed slopes in Sri Lanka currently do not have instrumentation or continuous data to monitor the deformations, loads on soil nails, and pore water pressures which could be used to assess the performance of the facing. Therefore, the method of assessing the performance used in this study has been purely based on observation.

8.2 Assessment Criteria

To make the approach more systematic, the field inspection criteria were based on the criteria used in Hong Kong Geoguide 5 for inspecting and maintenance of slopes.

The following criteria are used to assess the performance of the slope in each location:

Slope face in-between facing elements

Items checked	Condition	
Vegetation Cover	Good/Fair/Poor	
Metallic mesh	Good/Fair/Poor	
Signs of nonconformity	**Description**	**Photograph**
Any local failure?	If Yes, details of the observations and implications of the problems	If Yes, a photograph showing each anomaly in detail
Any erosion?		
Scouring or undercutting at facing element?		
Any signs of bulging of the slope?		
Any tension cracks at the slope?		
Any signs of excessive seepage?		
Any other signs of distress?		

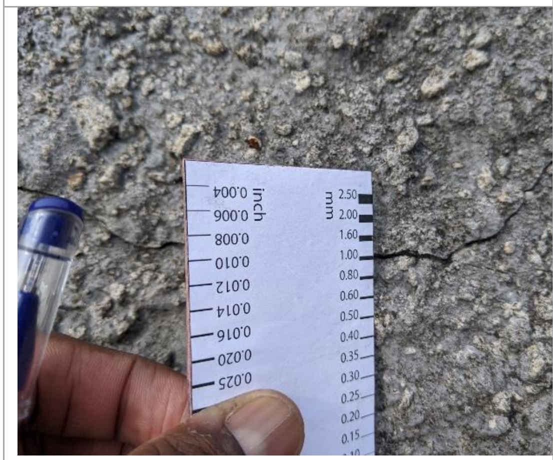
Structural facing elements:

Items checked	Condition	
Grid Beam	Good/Fair/Poor	
Shotcrete facing	Good/Fair/Poor	
Pillows	Good/Fair/Poor	
Signs of nonconformity	**Description**	**Photograph**
Any concrete/ shotcrete cracking?	If Yes, details of the observations and implications of the problems	If Yes, a photograph showing each anomaly in detail
Any face/ beam bulging?		
Any seepage/ leaching through shotcrete or pore water built-up?		
Any other signs of distress?		

8.3 Assessment of facing the condition

8.3.1 Southern Expressway CH 42+500

<table>
<tr><td>Facing Composition</td><td colspan="2">Full Face Shotcrete</td></tr>
<tr><td>Critical Slope Angle</td><td colspan="2">60 degrees</td></tr>
<tr><td>Critical Slope height</td><td colspan="2">15m</td></tr>
<tr><td>Completion Date</td><td colspan="2">17.07.2015</td></tr>
<tr><td>Date of Assessment</td><td colspan="2">06.03.2020</td></tr>
<tr><td colspan="2">At the completion of Construction</td><td>At Assessment:</td></tr>
<tr><td colspan="2"></td><td></td></tr>
<tr><td colspan="2">Items checked</td><td>Condition</td></tr>
<tr><td colspan="2">Shotcrete facing</td><td>Fair</td></tr>
</table>

<table>
<tr><td colspan="2">Signs of nonconformity</td></tr>
<tr><td>Signs of Cracking:
Cracks up to 1mm in width can be found at the toe area of shotcrete which could be due to thermal expansion and lack of reinforcement. Cannot be considered detrimental to the slope stability.</td><td>Seepage/ Leaching:
Clusters of large Calthemite deposits caused by water seepage through minor cracks present in Shotcrete. This indicates pore water pressure built-up. These cracks may have self-healed due to calcium deposits over time.</td></tr>
<tr><td>
</td><td></td></tr>
<tr><td colspan="2">Remarks:</td></tr>
<tr><td colspan="2">In case the leeching is continuous throughout the service life, the facing could get deteriorated over time, corrosion of embedded reinforcement will take place and further cracks could develop.
In the long run, for sites with excess pore water pressure built up, alternative methods to full-face shotcrete which promotes seepage could be recommended. Alternatively, an effective drainage layer including vertical band drains can be installed behind the full-face shotcrete facings in such conditions.</td></tr>
</table>

8.3.2 Kandy Mahiyangana Road A026 - Location 2

Facing Composition	Grid Beams with Metallic Mesh & Full-Face Shotcrete
Critical Slope Angle	60 degrees
Critical Slope height	7m
Completion Date	20.02.2016
Date of Assessment	05.03.2020

At the completion of Construction	At Assessment
	

Items checked	Condition
Grid Beam and Full-face shotcrete	Good
Vegetation Cover:	Good
Metallic Mesh:	Good

Signs of nonconformity	
Seepage in between facing elements: Seepage is observed in the area of Grid Beam, not necessarily from the horizontal drains. The use of grid beams in this area has permitted free seepage reducing the excess pore water pressure built-up in b-between nailheads.	**Seepage/ Leaching through facing:** Minor signs of seepage at the toe of shotcrete facing at RHS. However, no signs of leaching/ water pressure built-up behind shotcrete facing are observed.
	

Remarks:

- Vegetation cover has grown successfully, despite the steep slope angle.
- The selection of Grid Beam for the LHS of the slope has been an exceptional choice considering the amount of seepage observed at the area between nailheads.
- The difference in aesthetic appeal between two facing types (Grid beam & Shotcrete) can be observed

8.3.3 Kandy Mahiyangana Road A026 - Location 3

Facing Composition	Concrete Nailheads (Pillows) with Metallic Mesh
Critical Slope Angle	65 degrees
Critical Slope height	8m
Completion Date	20.02.2016
Date of Assessment	05.03.2020

At the completion of Construction	At Assessment
	

Items checked	Condition
Concrete Nailheads (Pillows):	Good
Vegetation Cover:	Fair
Metallic Mesh:	Good

Signs of nonconformity

Local instabilities

Minor local slope instabilities could be observed at the RHS side of the nailed area. Soil mass in between the nailhead has failed and the debris had slid through the metallic mesh and the slope.

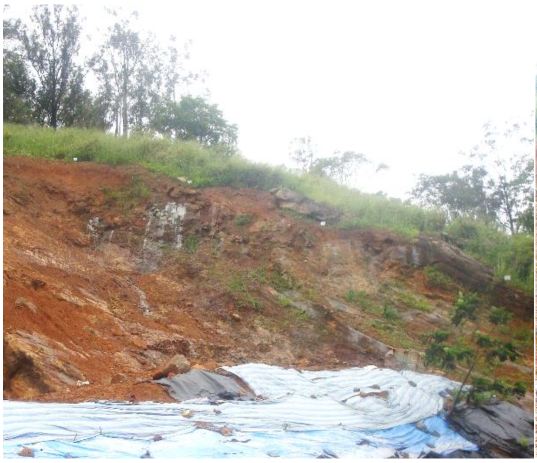
Remarks:

- Vegetation cover has failed to grow as expected on a few steep-sloped spots and, this could also be attributed to the boulder-rich slope surface.

• In the area where the vegetation cover is lacking, local failures or erosion had taken place at the slope surface. This shows the unreliability of pillow facings on instances where the slope surface is steep, jointed, erodible, or embedded with boulders. • The Coir mesh which had been used to support the growth of vegetation has decayed away at the time of assessment.

8.3.4 Kandy Mahiyangana Road A026 - Location 4

Facing Composition	Concrete Nailheads (Pillows) with Metallic Mesh
Critical Slope Angle	70 degrees
Critical Slope height	6m
Completion Date	21.05.2016
Date of Assessment	05.03.2020
At the completion of Construction	At Assessment
	
Items checked	Condition
Concrete Nailheads (Pillows):	Good
Vegetation Cover:	Fair
Metallic Mesh:	Good
Signs of nonconformity	
None	

Remarks:
• The mitigated slope face consists of a weathered rock mixed soil formation.
 
• Despite having weathered rock formations, vegetation has managed to grow successfully using the limited amount of fertile ground available.

8.3.5 Kandy Mahiyangana Road A026 - Location 7

Facing Composition	Full-Face Shotcrete
Critical Slope Angle	70 degrees
Critical Slope height	5m
Completion Date	21.05.2016
Date of Assessment	05.03.2020

At the completion of Construction	At Assessment
	
Items checked	Condition
Full-face shotcrete	Fair

<table>
<tr><th colspan="2">Signs of nonconformity</th></tr>
<tr><td>Signs of Cracking:
At the completion of the construction, cracks had been observed at the LHS slope on the shotcrete face. These were repaired immediately before the handing over.

These cracks could have been formed due to the shrinkage of concrete. However, during the assessment, patches of white deposits were observed particularly in the area where the cracks had been formed previously.

</td><td>Seepage/ Leaching:
Clusters of medium-sized Calthemite deposits caused by water seepage through minor cracks are present in Shotcrete. This indicates pore water pressure built-up behind the facing. These cracks may have self-healed due to calcium deposits over time.

</td></tr>
</table>

Remarks:
• The use of Shotcrete in such a slope where the surcharge area is high and the angle of surcharge water is high could be the causation for the aforementioned issues. • The formed cracks and the water built-up areas may be required to be monitored and repaired if required in the long run.

8.4 Recent developments in Shotcrete facings

As seen in the performance assessment, the full-face shotcrete facings standout from the rest of the greener background creating a disturbing appearance. Shotcrete facings have undergone criticism during the short history of usage in Sri Lanka due to this artificial appearance. However, methods to grow vegetation cover over the shotcrete face are being developed around the world to get over this issue.

Some of such examples can be found in Princess Margarette Hospital in Hong Kong where the full-face shotcrete has been fully covered with green vegetation as shown in Figure 8-1.

Figure 8-1. Vegetation cover on steep shotcrete slope (before and after)

Another example could be found in Hsichi, adjacent to the national highway no. 1 in Taiwan where a technique referred to as SVMT (Shotcrete Vegetation Mulching Technique) has been successfully used. Findings of this experiment have been published in the paper "Effect evaluation of shotcrete vegetation mulching technique applied to steep concrete-face slopes on a highway of Taiwan" (Fan, Huang, Yang, Liao, & Liao, 2011). The results of this experiment are shown in Figure 8-2.

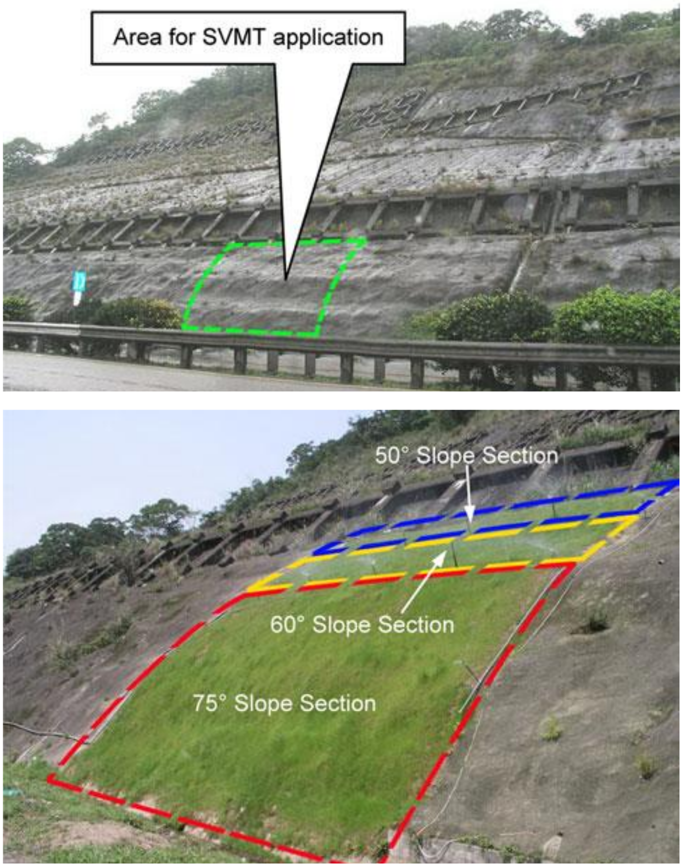


Figure 8-2. Shotcrete Vegetation Mulching Technique - Taiwan (After Fan et. al.)

CHAPTER 9. COMPARISON OF COSTS OF DIFFERENT FACING TYPES

9.1 Cost factors of soil nail system constructions

Soil nailing includes several components which include drilling, installing, grouting of soil nails as well as construction and installation of facing elements. The cost factors concerning soil nailing have not been properly recorded thus far in Sri Lankan practice and not available on any published schedule of rates. A breakdown of the cost factors inside soil nailing components is herewith presented with quantities set out, based on the construction experience in the local practice;

Soil Nail Installation

32mm dia. soil nails (12m or less length) inserted into 125mm dia. borehole with grouting and coupling.

Considering a slope segment of 13 m high and 8 m long. 28 Nos. of 12 m long nails (336m nail length);

Cost Factor	Unit	Qty
DRILLING (3 nos. /day)		
Labour		
Skilled Special Grade (Driller)	Day	12.60
Skilled	Day	18.90
Unskilled	Day	25.20
Machinery		
Drilling machine	Hr	57.60
Compressor 750 CFM	Hr	50.40
Lifting Machine	Hr	36.00
NAIL INSERTING		
Labour		
Skilled	Day	12.00
Unskilled	Day	15.00
Material		
Nail 32mm including wastage	ton	2.122
Centralizers	nr.	340.00
Galvanizing	ton	2.122
Thread Cutting (4")	nr.	30.00

Machinery		
Compressor 750 CFM	Hr	16.00
Lifting Machine	Hr	4.00
Material transport and handling		Item
GROUTING (28nrs/day)		
Labour		
Skilled	Day	16.00
Unskilled	Day	24.00
Material		
Cement including wastage (10%)	bag	148.00
Admixtures including wastage (1 pkt per cwt)	bag	148.00
Water	ltr	4,440.00
Concrete fill	Allow	
Machinery		
Grout Mixer	Hr	16.00
Grout Pumping Machine	Hr	32.00
Lifting Machine	Hr	4.00
Water Bowser (6000 ltr)	Day	2.00
Other		
Small Tools	%	3%
Material transport and handling	Item	As necessary

Facing construction

Full face shotcrete

100mm thick shotcreting with steel netting and dowels.

Considering a 13 m high, 7 m long slope segment with an angle 66º to the horizontal. (100 Sq.m shotcrete area);

Cost Factor	Unit	Qty
MATERIAL		
Cement	Bags	121.00
Sand	m3	4.10
Chips	m3	11.78
Quarry Dust	m3	3.69
water for shotcreting and dowels	Ltr	2,236.52
2"x2" GI net	m2	138.00
Y12 steel bars for dowels	kg	85.96
Binding wire	kg	3.50
MACHINERY & PLANTS FOR INSTALLATION		
Air Compressor-750	Hrs	24.00
Shotcrete gunning machine	Hrs	24.00
Generator 50kv	Hrs	24.00
Mobile Crane 25Ton - Laying steel net R/F work	Hrs	30.00
Mobile Crane 25Ton - Shotcreting	Hrs	10.00
LABOUR FOR FABRICATING, CASTING & PLACING IN POSITION		
Skilled Labour - for surface preparation	Hrs	24.00
Unskilled Labour - for surface preparation	Hrs	40.00
Skilled Labour - for preparation and laying steel net	Hrs	160.00
Unskilled Labour - for preparation and laying steel net	Hrs	240.00
Skilled Labour - for shotcreting	Hrs	48.00
Unskilled Labour - for shotcreting	Hrs	240.00
Unskilled Labour - for curing	Hrs	24.00

Isolated nailheads (Concrete Pillows)

Construction of 600 mm x 600 mm x 250 mm Concrete Pillow including excavation, formwork, reinforcement bars, fixing of MS bearing plate, nut and washers inside the head, in-situ concrete C30/20 and plate, nut, washer to be used to fix Coated Metallic Wire Mesh to the nail head.

Considering a 13 m high 4 m long slope segment, (212 nos. of Pillows);

Cost Factor	Unit	Qty
MATERIAL		
Cement	50 kg bag	54.00
Aggregate 19 mm (less piling) including transport	Cu.m.	7.64
Sand including transport	Cu.m.	5.09
Allow for water	item	
T16 "U" bar	kg	1,390.01
Binding Wire	kg	12.51
Plywood (15mm thick)	Sq.m	34.00
MACHINERY & PLANTS FOR INSTALLATION.		
Concrete Mixer	day	3.00
Poker Vibrator	day	3.00
150mm x 150mm x 12 mm Head plate	Nr	425.00
Galvanized Nut	Nr	425.00
LABOUR FOR FABRICATING, CASTING & PLACING IN POSITION		
Skilled Labour-A	day	56.90
Semi-Skilled Labour	day	14.00
Unskilled Labour	day	90.24
TOTAL COST OF LABOUR		
MISC COST COMPONENTS		
Add for tools	%	3.00%

<u>Grid beams</u>

Grid beam 250mm x 250mm – concrete C 30/20 including slope preparation, excavation, formwork, RF, and dowels.

Considering a 13 m high 9.5 m long slope segment, (136 m of Grid Beams (8.5 Cu.m))

Cost Factor	Unit	Qty
<u>EARTHWORKS</u>		
Excavation for Structures (incl. all costs)	Cu.m	8.50
<u>CONCRETING</u>		
Mixing and Laying G30 concrete using a concrete mixer (incl. all costs)	Cu.m	8.50
Allow for FW; if required (incl. all costs)	Sq.m	34.00
<u>REINFORCEMENT</u>		
12mm dia and 6mm dia reinforcement (incl. all costs)	kg	640.02
16mm Grouted dowels (incl. all costs)	m	136.00

It can be seen by observing the cost factors, soil nailing operations require some specialized machinery such as drilling machines, grout mixers, and grout pumps. For the facing construction, shotcreting operations require specialized shotcrete pumping machinery as well as well-trained gunmen to properly carryout shotcreting operations. Whereas, other facing types such as grid beams and pillows are similar to any other concrete casting operation. These may have the root factors for the increased cost for Shotcrete in general compared to other facing systems which will be further discussed in Section 9.2.

9.2 Costs of facing construction

This comparison was carried out using the market prices extracted from documents used in the bidding process in recent slope mitigation projects initiated in Sri Lanka. As such the values could be considered most recent.

The projects consist of all major facing modes Pillows, Grid Beams, Full-Shotcrete, and bearing plate method discussed in this research. Unit rate data were aggregated from 6 individual bidders before taking a percentile value of the rates to obtain a reasonable value filtering out the extreme inputs. Cost data aggregated have been presented in Appendix A. The cost of facing construction per unit area of the slope was calculated for comparison purposes as follows.

9.2.1 Cost of isolated nailhead construction

Using the costs obtained for Pillow nailhead construction, supplying and fixing the steel wire mesh, construction of the boundary beam, supplying and laying the coir mesh, and planting and maintain the vegetation cover, the overall cost per unit area of the slope face has been calculated hereunder.

	Item	**Unit**	**Qty**	**Percentile 50 value from Contractors rates (LKR)**	**Total cost (LKR)**
1.1	Concrete Pillow method including excavation, in-situ concrete C30/20 with all accessories used to fix Wire Mesh to the nail head.	Nos	232	9,720.00	2,255,040.00
1.2	Steel wire mesh (wire strength 900 N/mm^2, diameter 4.0 mm) including all necessary accessories	m^2	2,660	8,400.00	22,344,000.00
1.3	Concrete C30/20 boundary beams including slope preparation, excavation, formwork, RF and dowels	m	230	4,200.00	966,000.00
1.4	Coir mesh	m^2	2,660	585.00	1,556,100.00
1.5	Planting/Seeding on slope, stabilized surface	m^2	2,542	1,365.00	3,469,802.70
	The overall cost per m^2 of the slope segment				**11,500.35**

9.2.2 Cost of grid beam construction

Similarly, the cost for grid beam construction per unit area of the slope was calculated for comparison purposes as follows.

	Item	Unit	Qty	Percentile 50 value from Contractors rates (LKR)	Total cost (LKR)
2.1	Steel wire mesh (wire strength 900 N/mm^2, diameter 3.4mm) including all necessary accessories	m^2	1,267	5,600.00	7,095,200.00
2.2	Grid beam – concrete C 30/20 grid beams with nailheads including slope preparation, excavation, formwork, RF, and dowels	m	1,373	5,535.00	7,599,555.00
2.3	Coir mesh	m^2	1,267	382.50	484,627.50
2.4	Planting/Seeding on slope stabilized surface	m^2	924	1,036.00	957,005.00
	The overall cost per m^2 of the slope segment				**12,735.90**

9.2.3 Cost of full-face shotcrete construction

Costing for full-face shotcrete is usually done separating the facing as two components for practicality. The strip of facing inside with which the rebar runs connecting the nailheads (nailhead connecting beam) is measured separately in linear meters whereas the remaining facing area surrounded by the nailhead-connecting beams is measured in square meters. Using the costs obtained for full-face shotcrete construction, and the nailhead connecting beam construction, the overall cost per unit area of the slope face has been calculated hereunder.

	Item	Unit	Qty	Percentile 50 value from Contractors rates (LKR)	Total cost (LKR)
3.1	Shotcreting (wet mix) with concrete Grade 30	m^2	912	13,500.00	12,312,000.00
3.2	150mm wide, 100mm thick nail heads connecting beams with nail heads in shotcrete face	m	757	3,780.00	2,861,460.00
	The overall cost per m^2 of the slope segment				**14,795.44**

9.2.4 Cost of bearing plate with high tensile wire facing

The method discussed in Section 7.3.1 consists of a proprietary mesh product which ships with bearing plates produced by the mesh manufacturer and all accessories required to fix the mesh on the slope surface and to the soil nails. However, the vegetating of the slope shall be done using a coir mesh medium similar to the Grid beam and Pillow methods.

	Item	Unit	Qty	Percentile 50 value from Contractors rates (LKR)	Total cost (LKR)
4.1	Steel wire mesh (wire strength 1770 N/mm^2, diameter 3 mm) including bearing plates, nuts, cables and all necessary accessories	m^2	1,267	18,100.00	22,932,700.00
4.2	Coir mesh	m^2	1,267	585.00	741,195.00
4.3	Planting/Seeding on slope stabilized surface	m^2	1,267	1,365.00	1,729,455.00
	The overall cost per m^2 of the slope segment				**20,050.00**

9.2.5 Conclusion

Accordingly, the comparable costs per unit area are as follows.

Table 9-1. Cost per unit area of each facing type

Item	Cost per unit Area (LKR)
Isolated Nailheads (Pillows) with high tensile steel netting with vegetation of the slope in-between the nailheads	**11,500.35**
Grid beams with high tensile netting with vegetation of the slope in-between the grid beams	**12,735.90**
Full face Shotcrete	**14,795.44**
Bearing plate with high tensile wire facing	**20,050.00**

As per the status of the market at the time of writing this paper, shotcrete concrete is the costliest option whereas the pillow method stands as the lowest cost method.

It must be noted that the estimated values of the tensile mesh used for the pillow method were higher than that of the grid beam method. This is due to the higher punching shear capacity requirement of the mesh used in the pillow method as discussed previously in Section 7.3.1. However, the contractors often quote the same price for both meshes leaving the question of the contractor's awareness of the technical terms specified for the projects. As such, alternative prices from the bid prices have been used for the tensile mesh item in this comparison.

The two methods pillow and grid beams have the least price difference considering the price differences of all 4 methods. Full face shotcrete is considerably expensive compared to the pillow and grid beam methods, whereas the bearing plate method which utilizes high tensile wire meshes becomes the most expensive facing construction method of all methods considered.

CHAPTER 10. DISCUSSION AND CONCLUSION

Choosing the most appropriate soil nail facing for a given mitigation work from a large variety of available facing types and designing them to meet the requirements of a given project is an extremely important task. Guidelines as well as studies carried out around the world assist the designers to choose the most appropriate facing type as well as designing them to perfection. This chapter summarizes the outcomes of the studies and analysis carried out on the different facing types presented in the preceding chapters in the thesis.

As pointed out in Section 2.3 - General Rules for Selecting the Facing Type, hard structural facings are preferred as the criticality of slope segments increases. This is because, out of all facing types, the hard-structural facings provide the most versatile solution when it comes to strength and reliability. The design methods reviewed and proposed in Chapter 4 and Chapter 5 enable the designers to carry out the structural designs of these facings without any complications and ambiguity because the procedures have been set out following the familiar BS 8110 standard code of practice. Additionally, having studied these design methods, it could be realized how Grid Beams and Full Face Shotcrete facings offer the most freedom in achieving a higher value of nailhead resistance as well as the advantage of more effectively connecting the nail heads to enhances the ability of the soil nails to work as an integrated system. This could be further illustrated by comparing their designs with that of Isolated Nail Heads and Flexible Structural Facings or combinations of both. As discussed in Chapter 7, the resistance of isolated nail heads (Pillows) is predominated by the bearing capacity of the slope surface underneath or the strength of flexible mesh in the case of Plate Nailheads. Also, the connection between the nail heads in the case of pillows and plate nail heads does allow for the independent moment of individual nail heads. This affects the versatility of the whole soil nail system at the global level. Therefore, the designer must opt for a hard-structural facing as the first choice at least in the case of critical slopes of angle steeper than 65°.

Moreover, comparing the Full-face shotcrete against the Grid beam method, the Full-face shotcrete proves to be superior in terms of the overall performance. This is particularly due to the ability of shotcrete facings to resist both nailhead loads as well as local instabilities without having to resort to other components such as a flexible mesh facing. In the case of Grid beams, a flexible mesh facing is essential to prevent local instabilities on critical slopes where the nail spacing surpasses a certain upper limit as discussed in Section 2.7.

A full-face shotcrete facing may be the preferred choice on slopes where the surface soil is showing a more sandy, blocky, or crumbly nature. These slopes are prone to erosion and collapsible in the event of water flowing on the slope. The confinement provided by shotcrete throughout the slope face is extremely effective in such circumstances.

However, the designer must be mindful that the cost of construction of Shotcrete is relatively high compared to other facing methods as per the cost analysis presented in Chapter 9.

The other aspect the designer must be mindful of when choosing facing type is the groundwater condition inside of the slope under consideration. In a tropical country like Sri Lanka where most of the slope failures are triggered by high-intensity rainfall, the slope under consideration is likely to experience high pore water pressure buildup during periods of heavy rain. This matter was evident from some of the Calthemite deposits observed on the shotcrete slopes as discussed in the performance assessment of facings in Chapter 8. As such, the designer must be cautious in opting to go with full-face shotcrete, because the shotcrete facings do not effectively promote pore water pressure dissipation unless an effective drainage mechanism is provided behind the shotcrete facing. The soil nail reference manual by FHWA (Carlos A. Lazarte, et al., 2015) prescribes effective drainage solutions that can be deployed behind shotcrete facings. These can be in the form of short drains looking after the region immediately behind the shotcrete facing and long sub-surface drains extending 10-15m facilitating the maintenance of a low groundwater table even during the periods of heavy rain.

However, on the other hand, shotcrete facings may be a less appealing solution to most people because of their appearance. In areas where flora and fauna are predominantly covering all-natural slope segments, shotcrete facing on a mitigated slope stands out unattractively and stays like that for a long while. Techniques of introducing a finishing grass layer on steep shotcrete facing have also developed and put into practice as discussed in Section 4.3.3.

An alternative solution for these situations could be the use of grid beam and isolated nail head systems with which, planting, sodding, or hydroseeding could be carried out to blend the mitigation measures with the rest of the natural landscape. However, in cases where nailhead loads are larger and the out-of-balance force from local instabilities is high, the regular flexible mesh systems become ineffective. In such instances, high tensile metallic mesh products could be used. The advantage of these high tensile mesh products is that the nailhead could be limited to a "Bearing Plate (metal)" and the required bearing plate size is small compared to the size of nail heads used in regular low strength mesh systems as illustrated via calculations in Section 7.3.1. This, in turn, provides the ability to carry out the construction of facings faster and the slope preparation effort could be minimum as there will be no casted concrete structures in the facing system.

Moreover, when grid beams, isolated nailheads, or bearing plate nailhead systems are used along with flexible mesh systems, planting, seeding, and sodding must be essentially carried out to prevent erosion as well as to reduce the potential of local instabilities to some extent. In general, very steep slopes, sandy soils, slopes with embedded boulders, or slopes covered by structures (bridge abutments) do not support the growth of these plants. As such, using flexible facings on such slopes could give rise to erosion problems which affect the performance of the soil nail system in the long run.

Having followed these general guidelines and using the detailed methods of facing designs discussed in each chapter, designers could design soil nail facing systems economically and aesthetically pleasing manner while effectively serving their purpose.

REFERENCES

Bowles, J. E. (2011). *Foundation analysis and design.* McGraw-Hill Education - Europe.

AASHTO. (2002). *Standard Specifications for Highway Bridges* (17th ed.). Washington, D.C: American Association of State Highway and Transportation Officials.

BSI. (1999). *Strengthened/ reinforced soils and other fills : BS 8006:1995.* BSI.

BSI. (2010). *Execution of special geotechnical works — Soil nailing (BS EN 14490).* EUROPEAN COMMITTEE FOR STANDARDIZATION.

Buildings Department, Hong Kong. (2013). *Code of Practice for Structural Use of Concrete.*

Byrne, R., Cotton, D., Porterfield, J., Wolshlag, C., & Ueblacker, G. (1998). *Manual for design & construction monitoring of soil nail walls.* Washington DC: U.S. Department of transportation, Federal Highway Administration.

Cala, M., Flum, D., Rüegger, R., Roduner, A., & Wartmann, S. (2020). *TECCO® Slope Stabilization System and RUVOLUM® Dimensioning Method.* Romanshorn, Switzerland: AGH UNIVERSITY OF SCIENCE AND TECHNOLOGY.

Carlos A. Lazarte, P. P., Helen Robinson, P., Jesús E. Gómez, P. P., Andrew Baxter, P. P., Allen Cadden, P., & Ryan Berg, P. (2015). *GEOTECHNICAL ENGINEERING CIRCULAR NO. 7, SOIL NAIL WALLS - REFERENCE MANUAL.* Washington DC: FHWA.

Carlos A. Lazarte, P. P., Victor Elias, P., R. David Espinoza, P. P., & Paul J. Sabatini, P. P. (2003). *GEOTECHNICAL ENGINEERING CIRCULAR NO. 7, SOIL NAIL WALLS - REFERENCE MANUAL.* Washington, D.C: FHWA.

Fan, J.-C., Huang, C.-L., Yang, C.-H., Liao, K.-W., & Liao, W.-W. (2011). *Effect evaluation of shotcrete vegetation mulching technique applied to steep concrete-face slopes on a highway of Taiwan.* Springer-Verlag.

French National Project Clouterre. (1991). *RECOMMANDATIONS CLOUTERRE 1993.* Washington, DC.: FHWA.

Geotechnical Engineering Office . (2013). *Design Illustrations on the Use of Soil Nails to Upgrade Loose Fill Slopes.* The Hong Kong Institution of Engineers.

Geotechnical Engineering Office. (2008). *GUIDE TO SOIL NAIL DESIGN AND CONSTRUCTION, Geoguide 7.* Homantin, Kowloon, Hong Kong: The Government of the Hong Kong, Special Administrative Region.

Geotechnical Engineering Office, Hong Kong Institution of Engineers. (2011). *Design of Soil Nails for Upgrading Loose Fill Slopes.* GEO-HKIE.

Geotechnical Engineering Office, Hong Kong Institution of Engineers. (2013). *Design Illustrations on the Use of Soil Nails to Upgrade Loose Fill Slopes.* GEO-HKIE.

Giacchetti, G., Grimod, A., & Cheer, D. (2013). Soil Nailing with Flexible Structural Facing: Design and Experiences. *The Second World Landslide Forum* (pp. 655-660). Berlin: Springer.

Laguna, L. M., Perera, J., Chaminade, R. l., Hoyos, A., Carpio, J., & Velasco, a. J. (2013). *High Capacity Flexible Systems for Slope Stabilization in La Gomera (Spain).* Berlin: Springer.

Laguna, L. M., Perera, J., Chaminade, R., Hoyos, A., Carpio, J., & Velasco, J. (2013). High Capacity Flexible Systems for Slope Stabilization in La Gomera (Spain). *Landslide Science and Practice* (pp. 639-645). Berlin: Springer.

Margottini, C., Canuti, P., & Sassa, K. (2013). Landslide Science and Practice. *Proceedings of the Second World Landslide Forum, Volume 6*.

Ng, F., Lau, M., Shum, K., & Cheung, W. (2007). *REVIEW OF SELECTED LANDSLIDES INVOLVING SOIL-NAILED SLOPES.* HONG KONG: GEOTECHNICAL ENGINEERING OFFICE.

Phear, A., Dew, C., Ozsoy, B., Wharmby, N. J., Judge, J., & Barley, A. D. (2005). *Soil nailing – best practice guidance C637.* London: CIRIA.

Ruegger, R., & Flum, D. (2000). Slope stabilization with high-performance steel wire meshes in combination with nails and anchors. Seoul: Korean Geotechnical Society.

Seible, F. (1996). *STRUCTURAL RESPONSE ASSESSMENT OF SOIL NAIL WALL FACINGS.* San Diego, La Jolla, California: Division of Structural Engineering, Division of Structural Engineering.

Shiu, Y., & Chang, G. (2004). *SOIL NAIL HEAD REVIEW.* Hong Kong: Geotechnical Engineering Office.

Tei, K., Tayor, R., & Milligan, G. (1998). Centrifuge Model Tests of Nailed Soil Slopes. *Soils and Foundations,, 32*(2), 165-177.

The Highways Agency. (1994). *HA 68/94 DESIGN METHODS FOR THE REINFORCEMENT OF HIGHWAY SLOPES BY REINFORCED SOIL AND SOIL NAILING TECHNIQUES.* THE HIGHWAYS AGENCY.

APPENDIX A COST INPUT FROM CONTRACTORS

Pillow Method

	Item	Unit	Qty	Contractor's Rates								
				Contractor 1	Contractor 2	Contractor 3	Contractor 4	Contractor 5	Contractor 6	Contractor 7	Contractor 8	Contractor 9
1.1	Concrete Pillow method including excavation, in-situ concrete C30/20 with all accessories used to fix Wire Mesh to the nail head.	Nos	232	9,000.00	9,507.50	9,653.00	13,743.00	7,280.00	9,720.00	22,248.00	16,800.00	14,200.00
1.2	Steel wire mesh (wire strength 900 N/mm^2, diameter 4.0 mm) including all necessary accessories	m^2	2,660	5,200.00	7,990.00	7,020.00	6,437.00	7,280.00	8,640.00	7,342.00	10,080.00	8,320.00

1.3	Concrete C30/20 boundary beams including slope preparation, excavation, formwork, RF and dowels	m	230	4,200.00	2,936.25	4,446.00	6,212.00	2,500.00	3,780.00	5,006.00	4,200.00	9,050.00
1.4	Coir mesh	m^2	2,660	300.00	382.50	585.00	795.00	318.50	734.40	650.00	420.00	680.00
1.5	Planting/Seedi ng on slope, stabilized surface	m^2	2,542	1,400.00	300.00	985.00	2,173.00	1,365.00	2,160.00	1,300.00	420.00	1,540.00

Grid Beam Method

	Item	Unit	Qty	Contractor's Rates								
				Contractor 1	Contractor 2	Contractor 3	Contractor 4	Contractor 5	Contractor 6	Contractor 7	Contractor 8	Contractor 9
1.1	Steel wire mesh (wire strength 900 N/mm^2, diameter 3.4mm) including all necessary accessories	Nos	1,267	5,200.00	7,990.00	7,020.00	6,437.00	7,280.00	8,640.00	7,342.00	10,080.00	8,320.00
1.2	Grid beam – concrete C 30/20 grid beams with nail heads including slope preparation, excavation, formwork, RF and dowels	m^2	1,373	4,100.00	7,000.00	4,886.25	5,535.00	4,500.00	6,180.00	6,602.00	5,085.00	10,050.00
1.3	Coir mesh	m	1,267	350.00	300.00	382.50	615.00	450.00	247.00	2,173.00	360.00	450.00
1.4	Planting/Seeding on slope stabilized surface	m^2	924	800.00	1,400.00	300.00	1,036.00	1,800.00	1,187.00	689.00	360.00	1,600.00