# Design Illustrations on the Use of Soil Nails to Upgrade Loose Fill Slopes

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**and**
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**(Geotechnical Division)**

**November 2013**

## Disclaimer

This report presents design examples with reference to the guidelines given in "Geoguide 7: Guide to Soil Nail Design and Construction" promulgated by the Geotechnical Engineering Office of the Civil Engineering and Development Department (GEO/CEDD), and new design recommendations suggested in the report entitled "Design of Soil Nails for Upgrading Loose Fill Slopes" jointly published by the GEO/CEDD and the Hong Kong Institution of Engineers, Geotechnical Division (GEO-HKIE, 2011).

The Hong Kong Institution of Engineers and GEO/CEDD do not accept any responsibility for the use or misuse of this report. A designer using the recommendations contained in this report shall retain total responsibility for the adequacy of the design.

# Abstract

Based on the findings from a study initiated by the Geotechnical Engineering Office of the Civil Engineering and Development Department (GEO/CEDD), new design guidelines which supplement the original design guidance for the use of soil nails to upgrade loose fill slopes have been recommended and are summarised in a report jointly published by the GEO/CEDD, and The Hong Kong Institution of Engineers, Geotechnical Division (GEO-HKIE, 2011). The new guidelines recommend the use of a hybrid nail arrangement comprising soil nails at two different orientations. A design procedure for delineating loose fill at great depths which can be assumed to be non-liquefiable is also recommended.

To demonstrate the implementation of the new recommendations given in the GEO-HKIE (2011) Report, GEO-HKIE prepared three worked examples pursuant to the new recommendations, which are presented in this report. The first example demonstrates the procedure for adopting the hybrid nail arrangement, which is a novelty to local practitioners. The second example assumes that steeply inclined nails have to be adopted due to site constraints. The design therefore includes the provision of an embedded concrete footing which aims to reduce slope deformation. The effectiveness of the nail arrangement is verified by numerical analysis. The last example considers a fill slope with loose fill of a considerable depth, and illustrates the procedure to delineate the non-liquefiable loose fill zone by limit equilibrium method. Recommendations in respect of the selection of design parameters are also presented in the worked examples. The draft version of this report was issued in April 2013, and circulated for comments among government departments and practitioners via The Hong Kong Institution of Engineers, Geotechnical Division (HKIE-GD). These comments and the responses are attached in Appendix B. The salient points of these comments and responses are summarised in Section 5. The worked examples serve to illustrate the principles of the design guidelines only, and the design parameters, assumptions and considerations adopted in the worked examples are not meant to be adopted for general design purposes. Designers should use design parameters appropriate to their project at hand.

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# 1    Introduction

## 1.1    Background

In 2009, the Geotechnical Engineering Office of the Civil Engineering and Development Department (GEO/CEDD) commissioned AECOM Asia Company Limited to undertake a study to review the practice of designing soil nails to upgrade loose fill slopes and explore possible improvements in the design methodology and enhancements in robustness and detailing.    Based on the findings of the study, a report – hereafter referred to as the GEO-HKIE Report – was jointly published by the GEO/CEDD and the Hong Kong Institution of Engineers, Geotechnical Division (GEO-HKIE, 2011) to present the findings of the study and recommendations to supplement the original design guidance given in Geoguide 7 (GEO, 2008) and the report published by The Hong Kong Institution of Engineers, Geotechnical Division (HKIE-GD) in 2003 (HKIE, 2003).

To illustrate the design procedure pursuant to the new design recommendations, GEO-HKIE prepared three worked examples, each of which demonstrates a situation whereby the new design recommendations could be applied to improve the robustness of the design. The first worked example considers a simple case that adopts the recommended hybrid nail arrangement which is a novelty to local practitioners.    The second worked example considers a case in which steeply inclined nails are used due to site constraints.    Based on the new design recommendations, a toe embedment is provided to reduce the likely slope deformation caused by the use of steeply inclined nails.    The use of numerical analysis to assess the induced slope deformation is illustrated.    The third example considers a loose fill slope with a significant depth of fill.    The procedure to delineate the zone of loose fill which is unlikely to undergo static liquefaction and the design calculations adopting the recommended hybrid soil nail arrangement are provided.

This report presents the detailed design calculations for the three design examples as described above.    All the design steps including calculations of the required stabilising surface pressure, determination of the orientations and embedded lengths of the soil nails, and the structural design of the grillage structure are demonstrated.    The general design procedure is first summarised in Section 1.2.    The design assumptions and considerations for the three worked examples are presented in Sections 2 to 4.    Apart from the key aspects highlighted above, other recommendations in respect of the selection of design parameters and detailing of the soil nail arrangement and grillage system are also presented in the worked examples by reference to the GEO-HKIE Report.    The draft version of this report was issued in April 2013, and circulated for comments among government departments and practitioners via HKIE-GD. These comments and the responses are attached in Appendix B.    The salient points of these comments and responses are summarised in Section 5.    It should be noted that the worked examples serve to illustrate the principles of the design guidelines only, and the design parameters, assumptions and considerations adopted in the worked examples are not meant to be adopted for general design purposes.    Design parameters, assumptions and considerations should be established on a case-by-case basis to suit the actual site conditions.

**1.2     General Design Procedure**

The general design procedure consists of the following steps:

(a) Determine if slope upgrading works are required in respect of static liquefaction (Section 3.2 of GEO-HKIE (2011)) - the stability of the loose fill slope is examined under drained conditions using soil parameters corresponding to the onset of liquefaction ($\phi'_{mob}$). These parameters can be determined from site-specific laboratory testing. In the absence of site-specific laboratory test results, prescribed lower bound design parameters of $c' = 0$ kPa, $\phi'_{mob} = 26°$ can be used (Section 5.1 of GEO-HKIE (2011)). Upgrading works are required if the required factor of safety specified in the Geotechnical Manual for Slopes (GCO, 1984) cannot be achieved. For deep fill, liquefiable and non-liquefiable zones can be delineated using $\phi'_{mob}$ following the procedure described in Section 5.3 of GEO-HKIE (2011).

(b) Determine the earth pressure to be sustained by the soil nails – if upgrading works are found to be necessary based on the assessment in part (a), stability analyses using limit equilibrium method should be carried out to determine the earth pressure exerted on the grillage structure which is to be sustained by the soil nails. In the limit equilibrium analyses, the loose fill is assumed to have reached its steady state undrained shear strength ($c_{ss}$) due to static liquefaction, which is determined from site-specific laboratory testing. In the absence of such information, a prescribed lower bound of $c_{ss} = 0.2 \times p'_{peak}$ may be assumed (Section 5.1 of GEO-HKIE (2011)), where $p'_{peak}$ is the mean effective stress at peak shear strength. The distribution of the earth pressure can be assumed to be triangular in shape and can be modelled by a series of equivalent concentrated forces acting in the normal direction of the slope surface. It is important that the stabilising pressure specified should not lead to an increase in shear strength of the fill in slope stability calculations. The use of equivalent concentrated forces, rather than a distributed surface pressure, will avoid the increase in the undrained shear strength of the loose fill in some commercial computer programs (Section 5.2 of GEO-HKIE (2011)). A constant basal shear resistance acting along the base of the grillage may be assumed in the limit equilibrium analysis. The magnitude of the earth pressure to be sustained by the soil nails should be determined for a minimum factor of safety of 1.1 as suggested in the HKIE-GD Report (HKIE, 2003).

(c) Design the soil nails – the earth pressure determined in part (b) controls the size and length of the soil nails to be constructed.

For enhanced robustness, a hybrid nail arrangement should be adopted (Section 5.4 of GEO-HKIE (2011)). The soil nails at the upper portion of the slope can be designed to be sub-horizontal (e.g. with an inclination of about 20° to the horizontal), whilst the remaining nails are assumed to be perpendicular to the slope surface. According to Section 5.4 of GEO-HKIE (2011), the ratio of sub-horizontal nails to steeply inclined nails can be estimated by considering force equilibrium of the grillage facing. The number of sub-horizontal nails should be approximately 40% to 50% of total number of soil nails to ensure sufficient sub-horizontal nails are present to counter sliding failure. The size and length of the nails are then determined by considering three internal failure modes, namely tensile failure of steel bar, bond failure between grout and steel reinforcement, and shear failure of the adjacent ground of the soil nail. The corresponding factors of safety are given in HKIE (2003) and Geoguide 7 (GEO, 2008). If hybrid nail arrangement is not used, numerical analysis should be carried out to study the robustness and effectiveness of the soil nail design.

(d) Confirm the adequacy of the design under drained conditions – the stability of the loose fill slope under drained conditions is examined assuming that the designed soil nail forces are mobilised. The design is considered adequate if the required factor of safety used in part (a) can be achieved.

(e) Design the grillage structure – the size of the grillage beams is governed by the structural forces that the grillage has to sustain and the minimum coverage area of the grillage on the slope surface such that the risk of uncontrolled liquefaction due to squeezing out of the liquefied loose fill through the grillage opening is reasonably low. The grillage beams are designed to sustain the combined action of the earth pressure generated from the liquefied loose fill and the forces mobilised in the soil nails. The structural design can be performed according to Code of Practice for Structural Use of Concrete (BD, 2013). For slopes not higher than 25 m, a minimum grillage coverage of 50% of the slope surface is considered acceptable (Sections 2.1 and 5.6 of GEO-HKIE (2011)). A minimum grillage embedment of 0.3 m is considered adequate (Section 5.6 of GEO-HKIE (2011)).

(f) Design the soil nail heads – the soil nail heads should be designed against bearing and punching shear failure around the loaded area based on the guidelines set out in Code of Practice for Structural Use of Concrete (BD, 2013). The minimum size of the steel plate should be determined such that the bearing pressure, generated from the nail force, can be

sustained by the concrete shear resistance. In addition, the shear reinforcement within the soil nail heads should be designed to sustain the maximum shear stress generated in the concrete by the nail force.

## 2 Worked Example 1 – Design for Hybrid Nail Arrangement

### 2.1 Design Scenario and Assumptions

The first worked example is prepared to demonstrate the design procedure of soil nails in a hybrid nail arrangement. A 10 m high loose fill slope inclined at 33.7° to the horizontal (i.e. 1:1.5) with a 3 m uniform loose fill layer is assumed. The loose fill layer is underlain by Completely Decomposed Granite (CDG). A design perched water table (DPWT) is assumed to be located at 1.0 m above the loose fill/CDG interface. The geometry and soil profiles of the slope are shown in Figure 2.1.

The design soil parameters used in this worked example are assumed to have been determined from site-specific laboratory testing according to the recommendation in Section 5.1 of GEO-HKIE Report to avoid the use of overly conservative lower bound design parameters. In other words, the prescribed lower bound soil parameters are not used in this worked example. For onset of liquefaction, the design parameters of the saturated loose fill are $c' = 0$ kPa and $\phi'_{mob} = 28°$. Upon liquefaction, the steady state shear strength of loose fill is assumed to be $c_{ss} = 0.267\sigma'_v$ (i.e. $c_{ss}/p'_{peak} = 0.4$ assuming $K_0 = 0.5$). The bulk density of the loose fill and CDG is assumed to be 1.8 Mg/m$^3$ and hence the unit weights of both soils are equal to 17.7 kN/m$^3$. The design soil parameters are tabulated in Table 2.1.
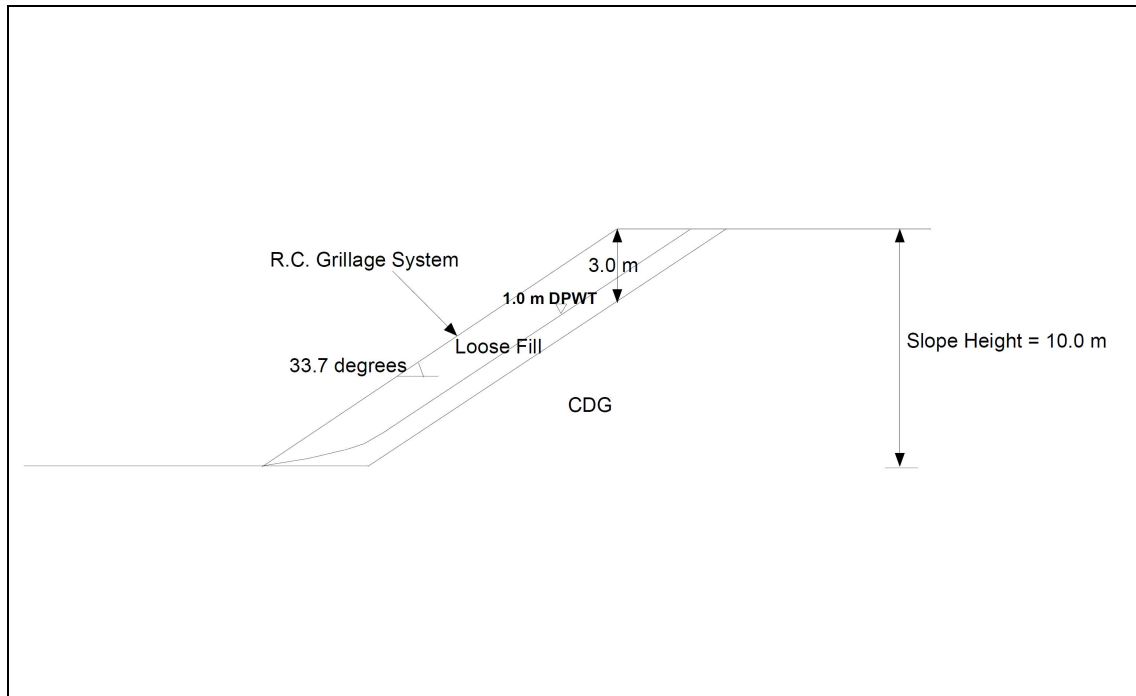


**Figure 2.1   Slope Geometry of Worked Example 1**

**Table 2.1   Design Soil Parameters in Worked Example 1**

| Soil Type | Parameter | Value |
|---|---|---|
| Loose Fill (Onset of Liquefaction) | Bulk Unit Weight, $\gamma$ (kN/m$^3$) | 17.7 |
| | Cohesion, $c'$ (kPa) | 0 |
| | Friction Angle, $\phi'_{mob}$ ($^o$) | 28 |
| Loose Fill (After Liquefaction) | Bulk Unit Weight, $\gamma$ (kN/m$^3$) | 17.7 |
| | Undrained Shear Strength, $c_{ss}$ (kPa) | $0.267\sigma'_v$ |
| | Friction Angle, $\phi'$ ($^o$) | 0 |
| CDG | Bulk Unit Weight, $\gamma$ (kN/m$^3$) | 17.7 |
| | Cohesion, $c'$ (kPa) | 5 |
| | Friction Angle, $\phi'$ ($^o$) | 35 |

## 2.2   Stability Assessment of the Loose Fill Slope without Soil Nails

To determine if upgrading works are required to maintain slope stability against static liquefaction, stability assessment is carried out using limit equilibrium method.   Commercial computer program SLOPE/W has been used in this worked example.   The loose fill is assumed to be saturated by rainfall infiltration and its mobilised shear strength is represented by the design parameters corresponding to the onset of liquefaction (i.e. $c' = 0$ kPa and $\phi'_{mob} = 28°$).   As shown in Figure 2.2, the minimum factor of safety obtained in the first worked example is 0.772.   Assuming that the required factor of safety is 1.4, the assessment suggests that upgrading works are required to upgrade the slope against liquefaction failure.

## 2.3   Determination of Earth Pressure to be Sustained by Soil Nails

The earth pressure exerted on the grillage is to be resisted by soil nails.   The magnitude of this pressure can be determined by limit equilibrium method.   This has been carried out by SLOPE/W as shown in Figure 2.3.   The magnitude of a set of concentrated point loads distributed in a triangular shape representing the earth pressure is to be determined. The shear strength of liquefied loose fill is represented by the "strength as a function of overburden" model in SLOPE/W in which the undrained shear strength of the loose fill is calculated by the user-specified "undrained shear strength-to-vertical effective stress" ($c_u/\sigma'_v$) ratio; this ratio is equal to 0.267 in this worked example.   A piezometric line is assigned at 1 m above the interface between the loose fill and the CDG to simulate a 1 m perched water table.   Suction effect above the piezometric line is ignored to avoid an unintended increase in undrained shear strength in SLOPE/W.   A lower bound basal shear resistance of 3 kPa is assumed, and is modelled by an evenly distributed traction in the SLOPE/W analysis.   The design objective is to identify the minimum earth pressure that the grillage has to sustain in order to achieve a factor of safety of 1.1 (Section 5.2 of GEO-HKIE (2011)).   The

calculation results indicate that the minimum earth pressure to be sustained by the grillage, hence the soil nails, is 66 kPa (Figure 2.3).
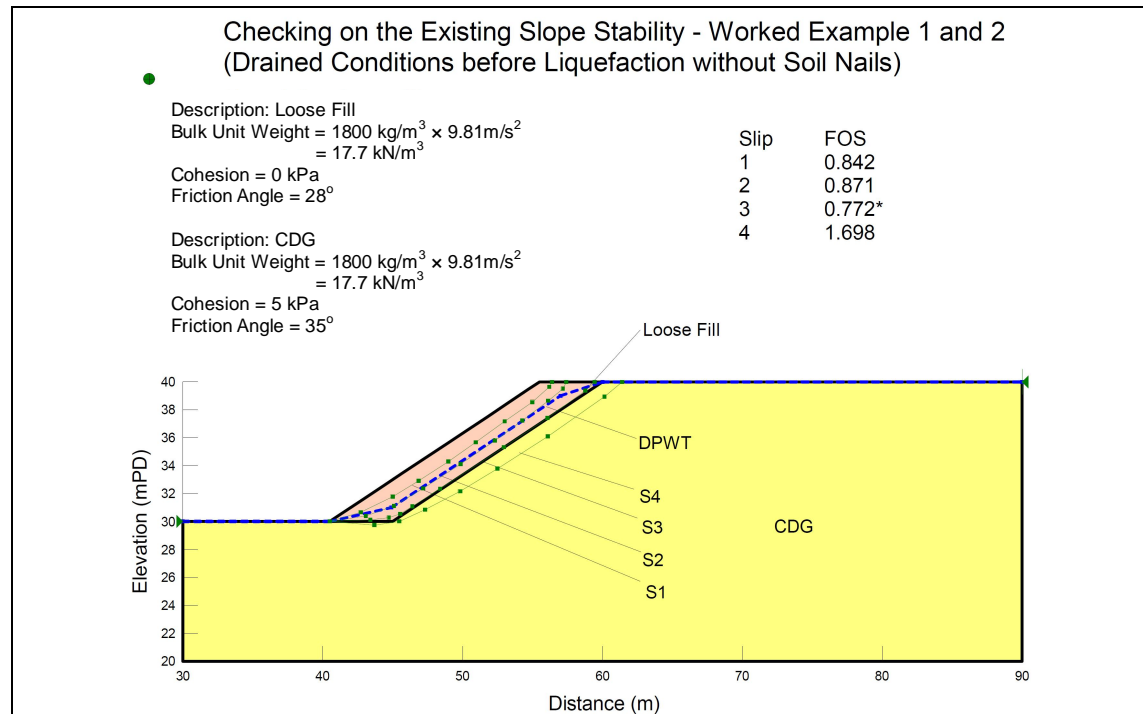


**Figure 2.2   Stability Assessment of the Loose Fill Slope without Soil Nails in Worked Example 1**
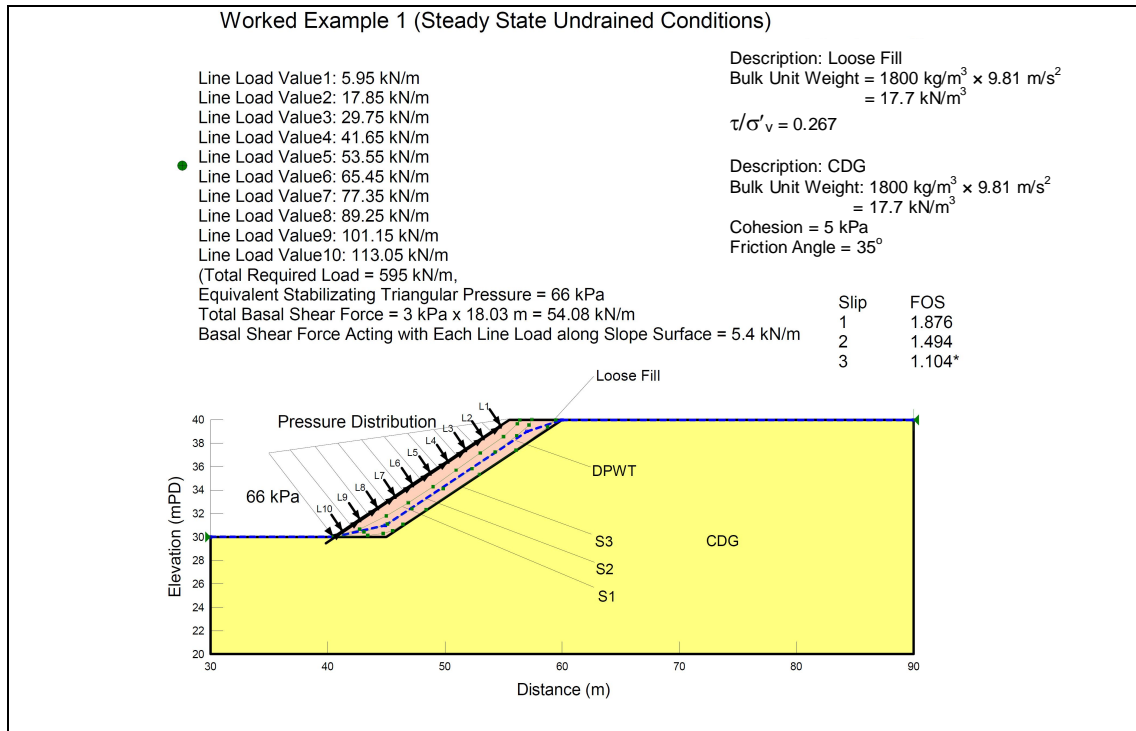
**Figure 2.3    Determination of Earth Pressure Exerted on Grillage Facing upon Liquefaction in Worked Example 1**

## 2.4    Design of Soil Nails

A hybrid soil nail arrangement with two nail orientations, namely three rows of sub-horizontal nails at 20° to horizontal and four rows of nails at 56° to the horizontal (i.e. perpendicular to slope surface), is adopted in this worked example.   The number of sub-horizontal nails is 43% of the total number of nails, which satisfies the initial guidance (Section 5.4 of GEO-HKIE (2011)).   The adequacy of this ratio is to be confirmed during the design grillage structure (presented in Section 2.6).   The soil nails are assumed to have a spacing of 1.5 m (centre-to-centre) in both the vertical and horizontal directions.   The calculations of individual tensile forces to be supported by each row of soil nails are presented in Appendix A.1.1.

The minimum diameter and bond length of the soil nails have been determined by considering three internal failure modes, namely tensile failure of the steel bar, bond failure between grout and steel reinforcement and shear failure of adjacent ground of the soil nail. The partial factor for tensile failure of the steel bar has been taken as 1.15 in accordance with Section 7.2 of HKIE (2003), which recommends that the soil nails axial tension capacity is based on the ultimate capacity as specified in BS8110.   This is consistent with the Code of Practice for Structural Use of Concrete (BD, 2013).   The factors of safety against pullout failure at grout-reinforcement interface and pullout failure at soil-grout interface are taken as 2.0 and 1.5 respectively, according to Table 5.6 of Geoguide 7 (GEO, 2008).   It is assumed that the steel bars used in permanent installation are provided with Class 2 corrosion protection measures.   Detailed calculations of the size and length of the soil nails are presented in Appendix A.1.2.   The final nail arrangement is shown in Figure 2.4.
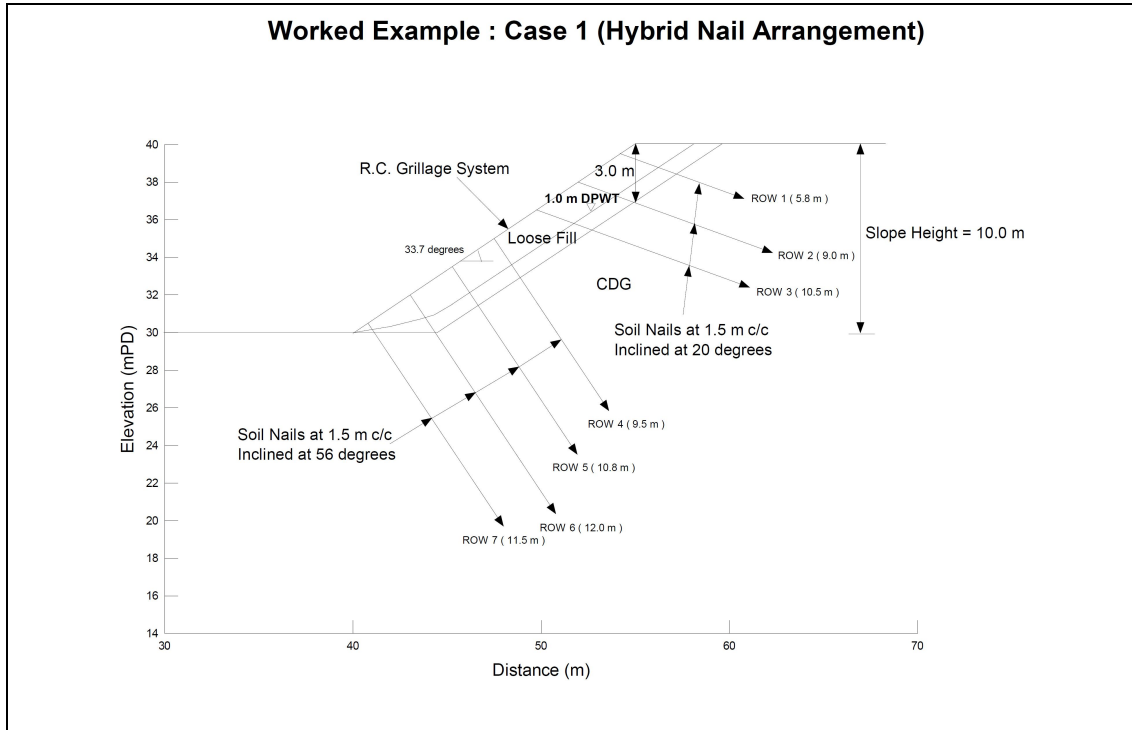
**Figure 2.4   Soil Nail Arrangement of Worked Example 1**

## 2.5   Stability Assessment of the Loose Fill Slope with Soil Nails under Drained Conditions

After individual nail forces are determined (as shown in Appendix A.1.1), stability of the nailed slope under drained conditions needs to be verified.   This can be assessed by limit equilibrium method assuming that the loose fill is saturated by rainfall infiltration and that the presence of the soil nails avoids liquefaction from triggering.   Therefore, in this assessment, the loose fill is represented by the design parameters at the onset of liquefaction (i.e. $c' =$ 0 kPa and $\phi'_{mob} = 28°$).   In this worked example, this assessment has been carried out using SLOPE/W in which the design nail forces at the intended nail orientations are specified as shown in Figure 2.5.   The calculated minimum factor of safety is above 1.4, which is the assumed minimum factor of safety as described in Section 2.2.   The design is therefore considered adequate.
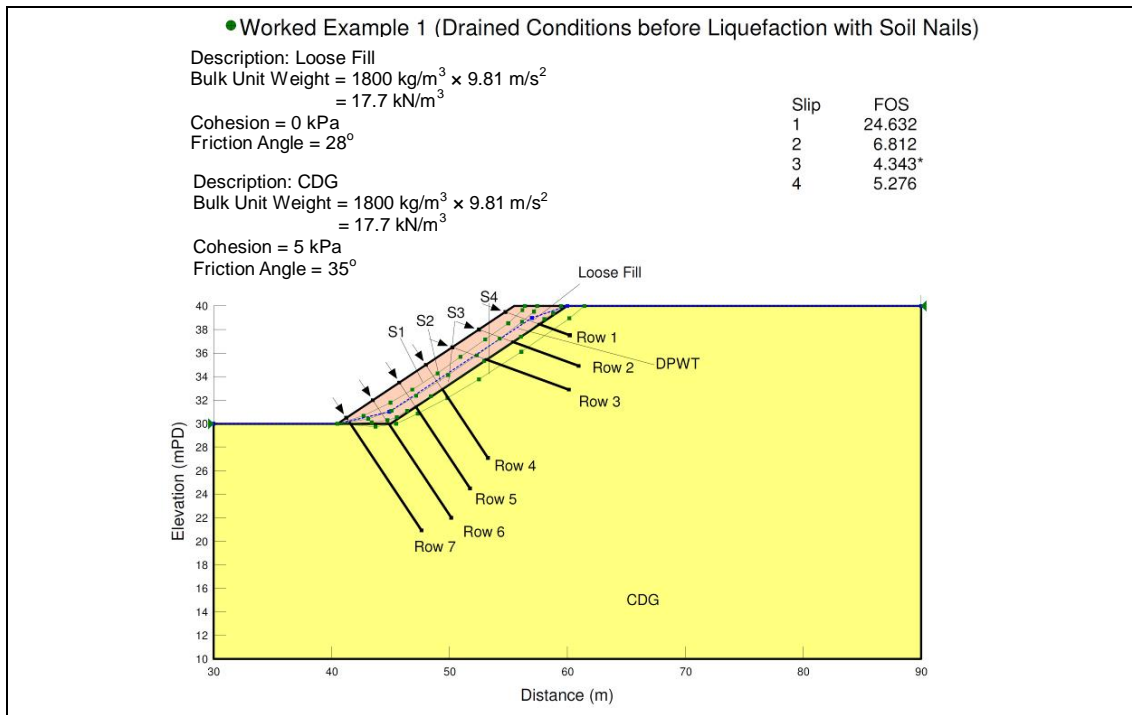
**Figure 2.5    Stability Assessment of the Loose Fill Slope with Soil Nails under Drained Conditions in Worked Example 1**

## 2.6    Design of the Grillage Structure and Soil Nail Heads

To minimise the risk associated with uncontrolled liquefaction failure due to squeezing out of liquefied loose fill through the grillage opening, a minimum grillage coverage of 50% of the slope surface is adopted (Sections 2.1 and 5.6 of GEO-HKIE (2011)).    The width of all the grillage beams is assumed to be 600 mm.    This is equivalent to a surface coverage of 53% which satisfies the design recommendation (calculations are presented in Appendix A.1.3).    The grillage beams are assumed to have a depth of 300 mm embedded into the slope surface.    The minimum embedment of 300 mm as suggested in Section 5.6 of the GEO-HKIE Report is satisfied.

The structural design of the grillage beams is carried out in accordance with the Code of Practice for Structural Use of Concrete (BD, 2013).    The assumption that the loose fill is fully liquefied and has reached the steady state undrained shear strength corresponds to the ultimate limit state.    A partial factor of 1.0 is therefore adopted for the design of the grillage subjected to the loading generated from the earth pressure upon liquefaction.    Each segment of the grillage beams is assumed to be simply supported and is subjected to the action of the earth pressure originated from the liquefied loose fill.    Since the earth pressure distribution is assumed to be triangular in shape, the most critical location is normally near the slope toe where the earth pressure is largest.    In this worked example, the most critical location is the segment between the sixth and seventh rows of soil nails due to the larger unsupported span length.    The reinforcement details are determined by checking the bending moment and shear capacity of the reinforced grillage beams.    It should be emphasised that with the hybrid

nail arrangement, a net tensile force is expected to be induced in the grillage structure due to the upward component of the tensile forces in the sub-horizontal nails. An independent assessment of the structural capacity under combined tension and bending moment is necessary. Figure 2.6 shows the axial force-bending moment interaction diagram for the segment of the grillage beam under consideration. The design is considered adequate as the applied tensile force and bending moment fall inside the interaction diagram. Detailed design calculations of the grillage structure are presented in Appendix A.1.4.

Since the soil nail heads are connected to the grillage structure, the guidance for soil-nail head design stipulated in Section 5.6.4 of Geoguide 7 (GEO, 2008) does not apply. The minimum size of bearing steel plate is determined by the shear resistance of the concrete. In this example, the steel plate adopted has a plan area of 150 mm × 150 mm, and is assumed to be embedded at 250 mm above the base of the grillage beams. Since the grillage beams have a width of 600 mm, the effective size of the soil nail heads can be taken as 600 mm × 600 mm. The shear reinforcement details within the soil nail heads are determined by considering punching shear failure of the loaded area in the concrete. Detailed calculations for the soil nail heads are shown in Appendix A.1.5.
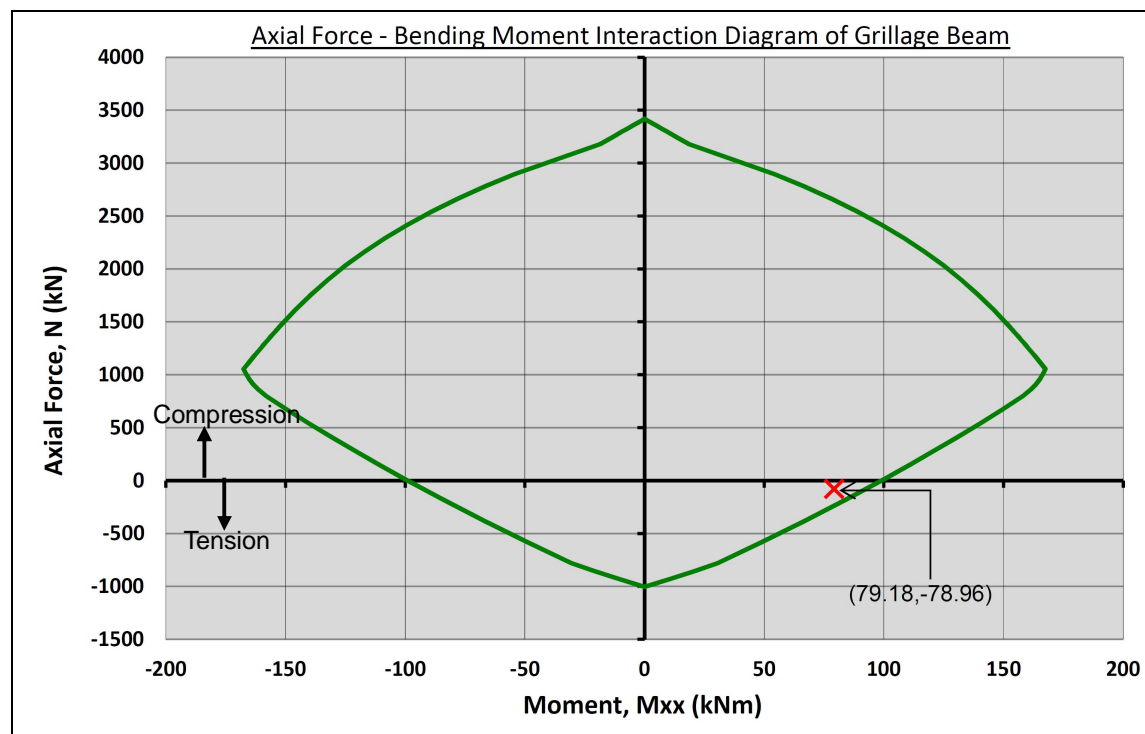


**Figure 2.6     Axial Force - Bending Moment Interaction Diagram of Grillage Beam**

### 3 Worked Example 2 – Design for Steeply Inclined Nails

### 3.1 Design Scenario and Assumptions

The second worked example is to illustrate a design scheme which adopts steeply inclined soil nails at a single orientation with an embedded concrete footing (or toe wall) at the toe of a loose fill slope.   The use of hybrid nail arrangement is strongly recommended to enhance the robustness of the system.   This example is only relevant for special circumstances whereby the designer may be obliged to design soil nails that are steeply inclined due to site constraints.   The use of steeply inclined nails in a loose fill slope may cause excessive movement due to the limited structural stiffness of the steeply inclined nails along the potential sliding direction.   The role of the embedded concrete footing is to reduce slope movement upon fill liquefaction (Section 5.5 of GEO-HKIE (2011)).   Since the proposed nail arrangement deviates from the recommended hybrid arrangement, an assessment of the effectiveness and robustness of the proposed nail arrangement using numerical analysis technique is required (Section 5.4 of GEO-HKIE (2011)).   This is illustrated in this worked example using a commercial finite element program PLAXIS2D.

The slope geometry and soil profiles assumed in the second worked example are the same as those in worked example 1, and are presented in Figure 3.1.   The assumed design soil parameters are tabulated in Table 3.1.   A lower steady state undrained shear strength $c_{ss} = 0.2\sigma'_v$ kPa is assumed, which is again assumed to be derived directly from site-specific laboratory testing.   Other design soil parameters are the same as those adopted in worked example 1.
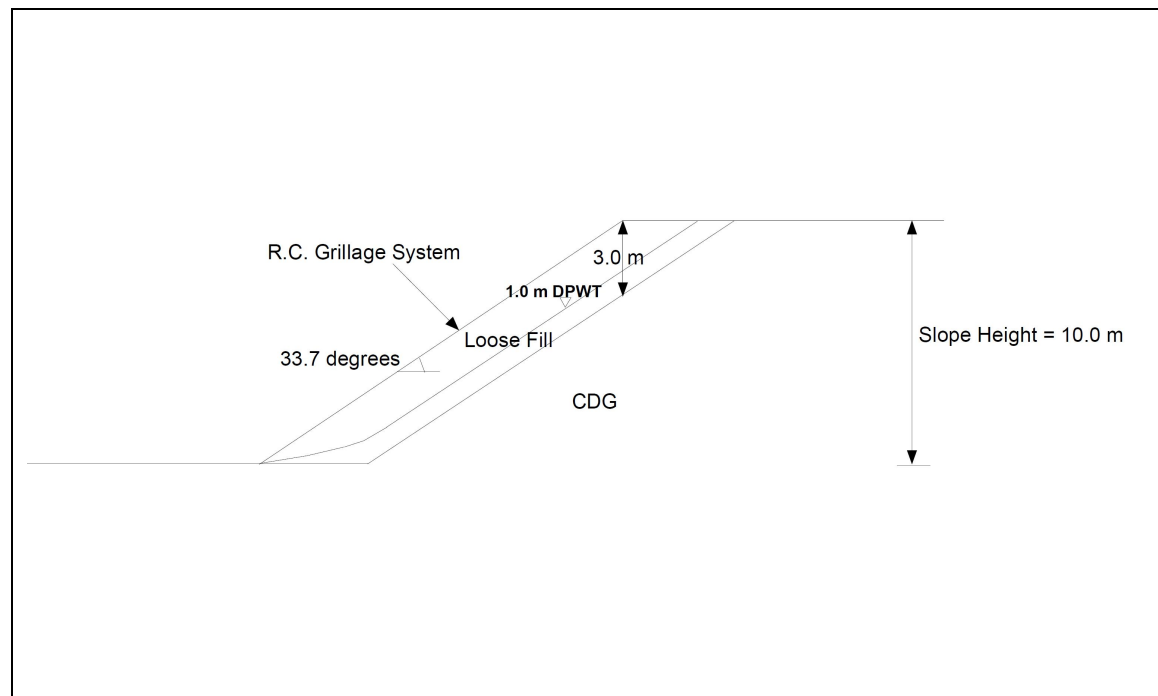


**Figure 3.1     Slope Geometry of Worked Example 2**

**Table 3.1   Design Soil Parameters in Worked Example 2**

| Soil Type | Parameter | Value |
|---|---|---|
| Loose Fill (Onset of Liquefaction) | Bulk Unit Weight, $\gamma$ (kN/m$^3$) | 17.7 |
| | Cohesion, $c'$ (kPa) | 0 |
| | Friction Angle, $\phi'_{mob}$ ($^o$) | 28 |
| Loose Fill (After Liquefaction) | Bulk Unit Weight, $\gamma$ (kN/m$^3$) | 17.7 |
| | Undrained Shear Strength, $c_{ss}$ (kPa) | $0.2\sigma'_v$ |
| | Friction Angle, $\phi'$ ($^o$) | 0 |
| CDG | Bulk Unit Weight, $\gamma$ (kN/m$^3$) | 17.7 |
| | Cohesion, $c'$ (kPa) | 5 |
| | Friction Angle, $\phi'$ ($^o$) | 35 |

## 3.2   Stability Assessment of the Loose Fill Slope without Soil Nails

Since the geometry of worked example 2 is the same as that in worked example 1 and that the friction angles at onset of liquefaction of the two cases are also the same (i.e. $\phi'_{mob}$ = 28°), the assessment of the initial stability without soil nails in worked example 2 is identical to that in worked example 1 and is presented in Figure 2.2.   The conclusion of the assessment is that the minimum factor of safety is 0.772, which suggests that upgrading works are required.

## 3.3   Determination of Earth Pressure to be Sustained by Soil Nails

The methodology to determine the earth pressure exerted on the slope facing is the same as that described in Section 2.3.   Since a lower steady state undrained shear strength of $0.2\sigma'_v$ is assumed in this example, a higher earth pressure is expected.   As shown in Figure 3.2, the design earth pressure is 87 kPa for a minimum factor of safety of 1.1.
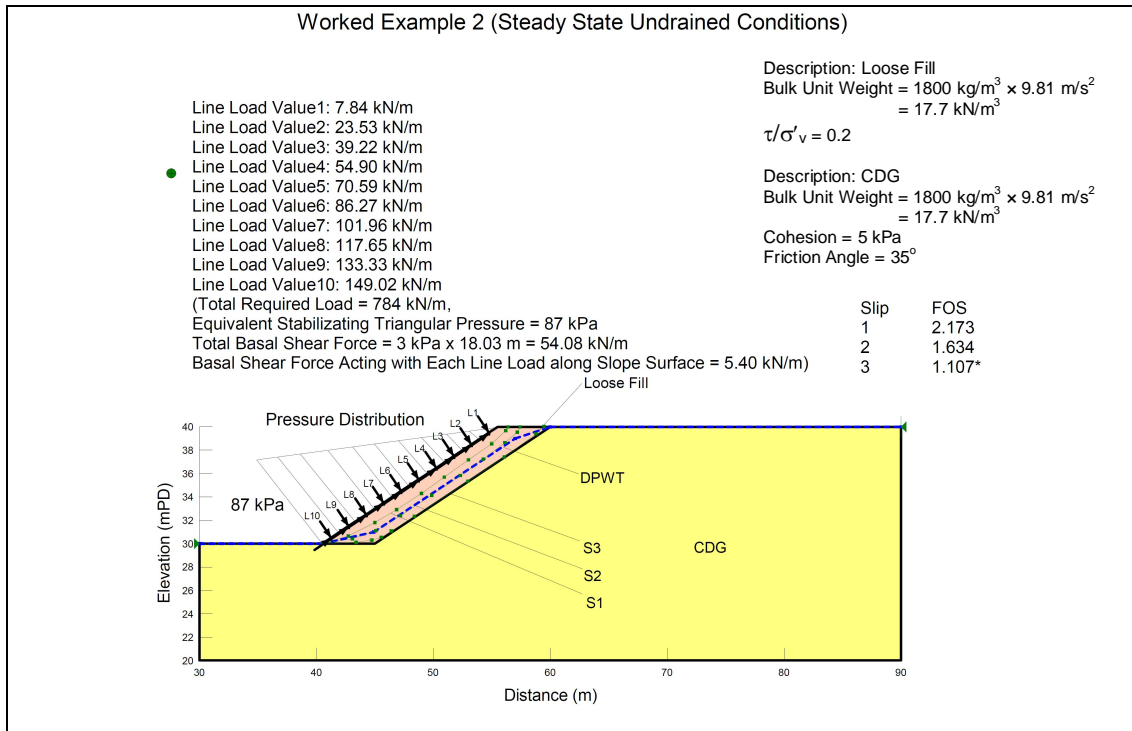
**Figure 3.2    Determination of Earth Pressure Exerted on Grillage Facing upon Liquefaction in Worked Example 2**

## 3.4   Design of Soil Nails

This example assumes that the use of sub-horizontal nails is restricted due to site constraints and steeply inclined soil nails at the same orientation have to be adopted.   In theory, the inclination of the soil nails could be determined by considering force equilibrium of the grillage structure taking into account the grillage weight, the design nail forces and the basal shear resistance.   However, a low basal shear resistance would imply that the inferred nail orientation would be close to perpendicular to the slope surface.   For example, the influence on the nail orientation is only 1° if a basal resistance of 3 kPa is taken into account for the assumed geometry and design nail forces in worked example 2.   Moreover, the provision of an embedded concrete footing introduces an unknown reaction force which depends on the mobilised passive soil resistance.   This renders the force equilibrium condition of the grillage statically indeterminate.   To illustrate the design procedure, the nail orientation is assumed to be perpendicular to the slope surface (i.e. 56° to the horizontal).

The soil nails are assumed to have a spacing of 1.5 m (centre-to-centre) in both the vertical and horizontal directions.   The calculations of individual nail forces are presented in Appendix A.2.1.   The design principle for determination of the diameters and the bond lengths of the soil nails are the same as that described in Section 2.4.   Detailed calculations are presented in Appendix A.2.2.   The nail arrangement is shown in Figure 3.3.
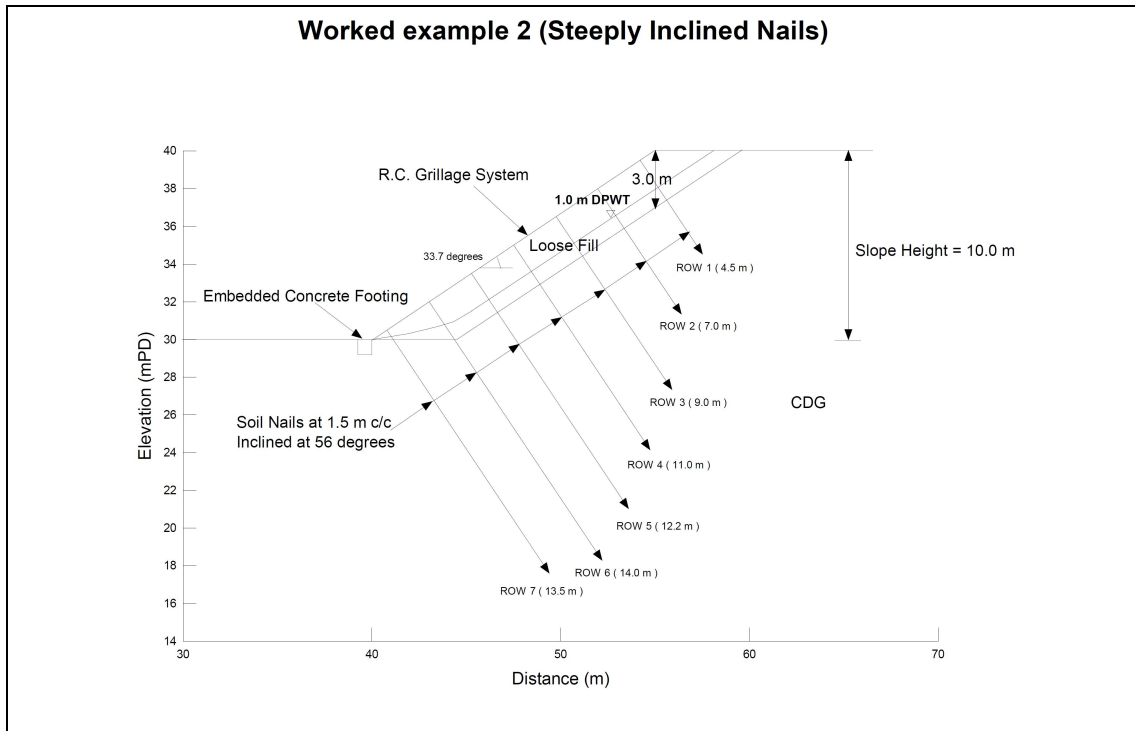
**Figure 3.3 – Soil Nail Arrangement of Worked Example 2**

## 3.5 Stability Assessment of the Loose Fill Slope with Soil Nails under Drained Conditions

Upon designing the stabilising force to be provided by each row of soil nails, an assessment on the overall stability under drained conditions should be carried out. The analysis is conducted using SLOPE/W, and the results are shown in Figure 3.4. The minimum calculated factor of safety is 4.181, which satisfies the assumed design requirement of a minimum factor of safety of 1.4.
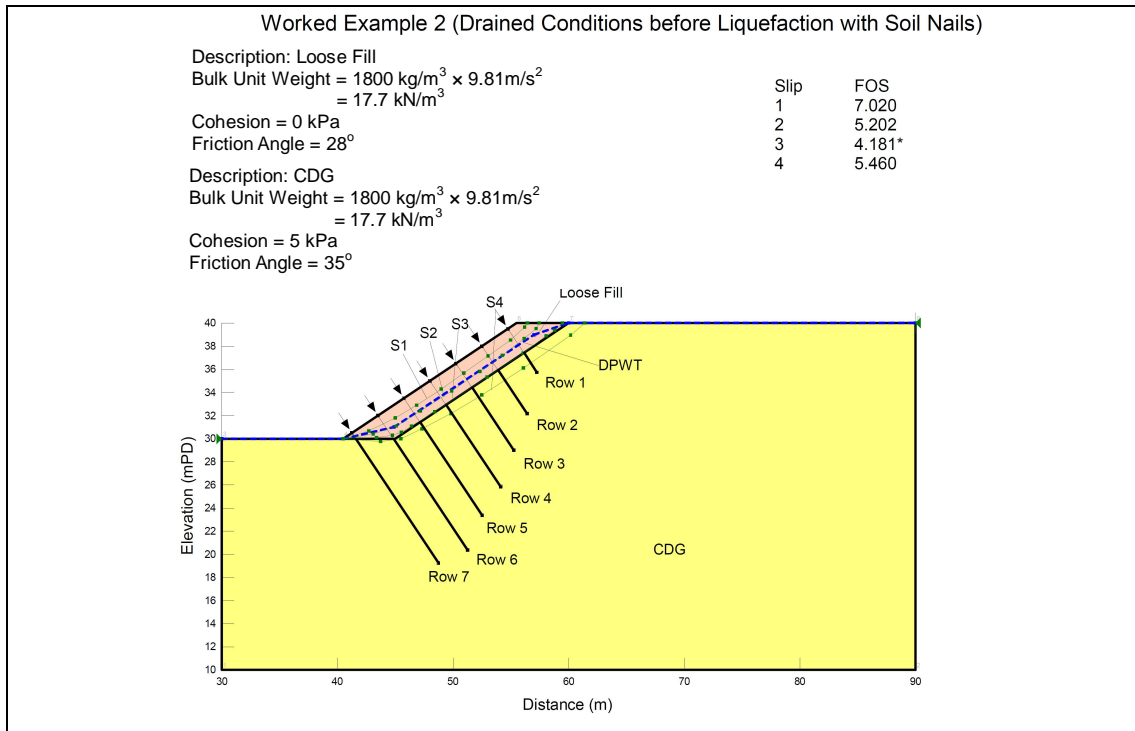
**Figure 3.4    Stability Assessment of the Loose Fill Slope with Soil Nails under Drained Conditions in Worked Example 2**

## 3.6  Design of Grillage Structure and Soil Nail Heads

Despite that larger nail forces are required due to the lower steady state undrained shear strength, the size of the grillage structure is the same as that adopted in worked example 1.   In other words, the grillage beams have a width of 600 mm and a depth of 300 mm embedded into the slope surface.   The calculations of minimum coverage area are presented in Appendix A.2.3.

The structural design procedure for the grillage facing and soil nail heads are similar to that adopted in worked example 1.   The calculations are presented in Appendix A.2.4 and A.2.5 for the grillage beams and soil nail heads respectively.

To minimise the overall slope movement, an embedded concrete footing is adopted. A nominal embedment depth of 1 m has been adopted for the embedded footing.   The predicted performance of the nailed slope, with and without the embedded footing is assessed by numerical analysis and is presented in Section 3.7.   The numerical analysis can also provide the shear force and bending moment induced in the concrete footing for structural design, which can be carried out in accordance with the Code of Practice for Structural Use of Concrete (BD, 2013).   The design structural forces for the embedded concrete footing are shown in Appendix A.2.5.

### 3.7 Slope Deformation Assessment by Numerical Analysis

To demonstrate the robustness of the proposed nail arrangement which involves the use of steeply inclined nails augmented by an embedded concrete footing, a finite element analysis has been performed using two-dimensional finite element program PLAXIS2D (version 9.0.2). The model geometry is shown in Figure 3.5, which is identical to that presented in Figure 3.1. The slope geometry and nail orientation are essentially the same as the case considered in Section 4.2.1 of the GEO-HKIE (2011). According to GEO-HKIE (2011), larger structural movement may be incurred under interface liquefaction compared to full liquefaction if steeply inclined nails are adopted only. Therefore, the interface liquefaction scenario is considered more critical as far as slope deformation is concerned. Numerical model is used to study the effectiveness of the soil nail design against the interface liquefaction. For design of slopes of other geometry, numerical study for the full liquefaction scenario should also be undertaken. In the numerical model, interface liquefaction is modelled by the sudden reduction of shear resistance in a 0.5 m thick of loose fill at a depth of 3 m under undrained conditions.
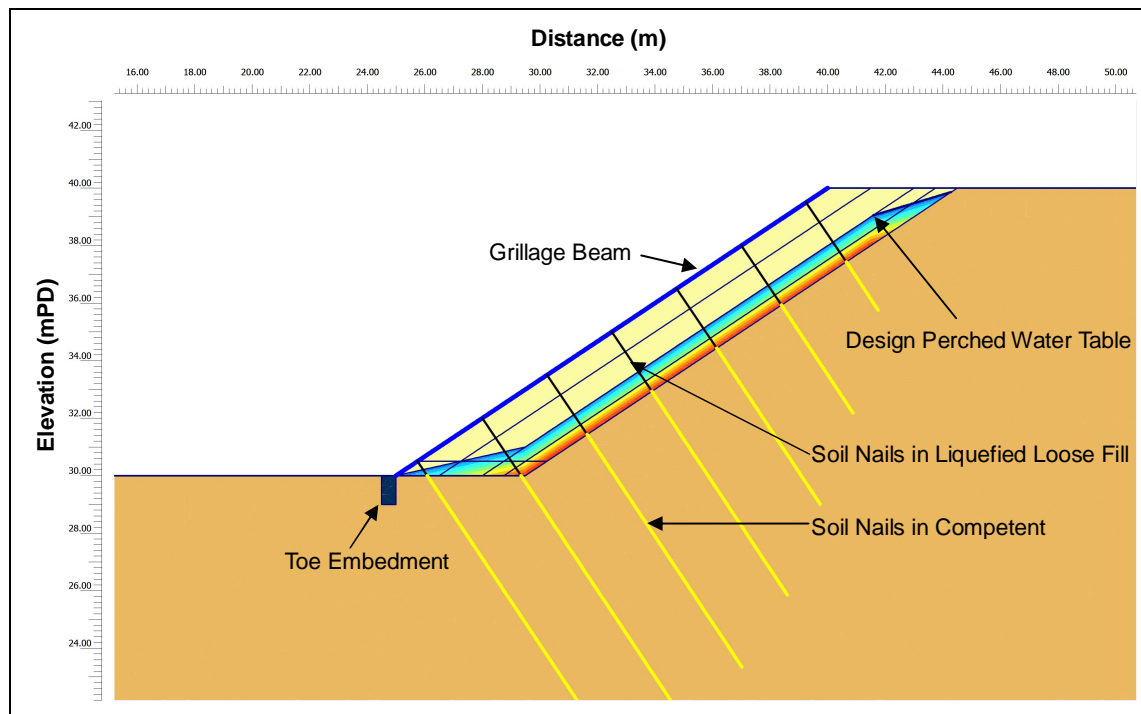


**Figure 3.5 Model Geometry and Initial Pore Water Pressure Distribution in the Numerical Model**

In the numerical model, both loose fill and CDG are modelled as an elastic-perfectly plastic soil model with a Mohr-Coulomb failure criterion. The model parameters for soils are tabulated in Table 3.2. For the loose fill, the Young's modulus ($E$) is assumed to be 5 MPa and a Poisson's ratio ($v$) of 0.3 is assumed. Two sets of strength parameters have been used to represent the soil states before and after liquefaction. Before liquefaction, effective strength parameters of $c' = 5$ kPa and $\phi' = 35°$ have been assigned to establish the initial stresses (see Figure 3.6). Upon liquefaction, an undrained model is used to mimic the

loss of shear resistance under constant volume conditions.   The effective stiffness parameters (i.e. $E$ and $\nu$) remain the same, and the corresponding undrained values are calculated in PLAXIS2D automatically.   The steady state undrained shear strength is a function of initial effective vertical stress (i.e. $c_{ss} = 0.2\sigma'_v$).   The bottom 0.5 m of loose fill layer is assigned an undrained shear strength calculated from the effective vertical stress at the mid-depth of the layer as shown in Figure 3.7.   The loose fill on top of the liquefied layer is represented by soil parameters at the onset of liquefaction (i.e. $c' = 0$ kPa and $\phi' = 28°$).   The 1 m toe embedment is modelled as an elastic material with a Young's modulus ($E$) of 22 GPa and a Poisson's ratio ($\nu$) of 0.2.

**Table 3.2   Model Parameters for Soils in the Numerical Analysis**

| Soil Type | Model Type | Parameter | Input Value |
|---|---|---|---|
| Loose Fill (Before Liquefaction) | Elastic-Perfectly Plastic Model with Mohr-Coulomb failure Criterion (Drained) | Bulk Unit Weight, $\gamma$ (kN/m³) | 17.7 |
| | | Cohesion, $c'$ (kPa) | 5 |
| | | Friction Angle, $\phi'$ (°) | 35 |
| | | Young's Modulus, $E$ (kPa) | 5,000 |
| | | Poisson's Ratio, $\nu$ | 0.3 |
| Loose Fill (After Liquefaction) | Elastic-Perfectly Plastic Model with Mohr-Coulomb Failure Criterion (Undrained) | Bulk Unit Weight, $\gamma$ (kN/m³) | 17.7 |
| | | Undrained Shear Strength, $c_{ss}$ (kPa) | 3.0 to 8.3 |
| | | *Young's Modulus, $E$ (kPa) | 5,000 |
| | | *Poisson's Ratio, $\nu$ | 0.3 |
| Loose Fill (Non-liquefied) | Elastic-Perfectly Plastic Model with Mohr-Coulomb Failure Criterion (Drained) | Bulk Unit Weight, $\gamma$ (kN/m³) | 17.7 |
| | | Cohesion, $c'$ (kPa) | 0 |
| | | Friction Angle, $\phi'$ (°) | 28 |
| | | Young's Modulus, $E$ (kPa) | 5,000 |
| CDG | Elastic-Perfectly Plastic Model with Mohr-Coulomb Failure Criterion (Drained) | Bulk Unit Weight, $\gamma$ (kN/m³) | 17.7 |
| | | Cohesion, $c'$ (kPa) | 5 |
| | | Friction Angle, $\phi'$ (°) | 35 |
| | | Young's Modulus, $E$ (kPa) | 25,000 |
| | | Poisson's Ratio, $\nu$ | 0.3 |
| Toe Embedment | Elastic Model | Bulk Unit Weight, $\gamma$ (kN/m³) | 24 |
| | | Young's Modulus, $E$ (kPa) | $2.2 \times 10^7$ |
| | | Poisson's Ratio, $\nu$ | 0.2 |

*Notes: These are drained parameters.   The corresponding undrained parameters are automatically calculated in PLAXIS2D.
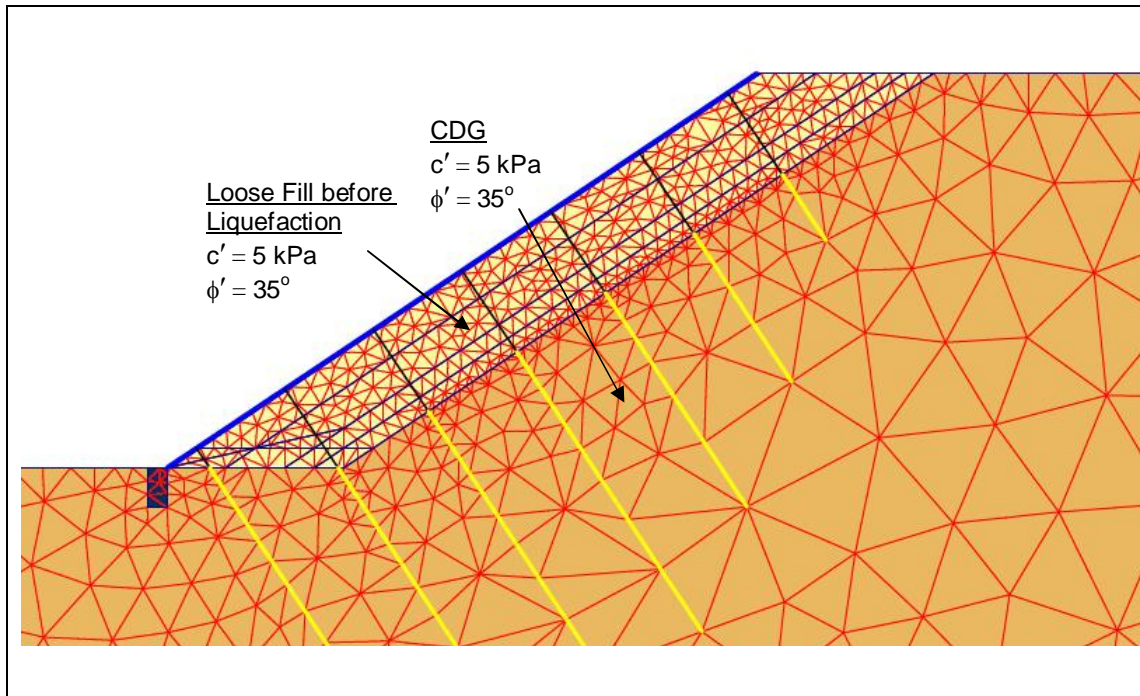
**Figure 3.6    Shear Strength Parameters of the Loose Fill before Interface Liquefaction**
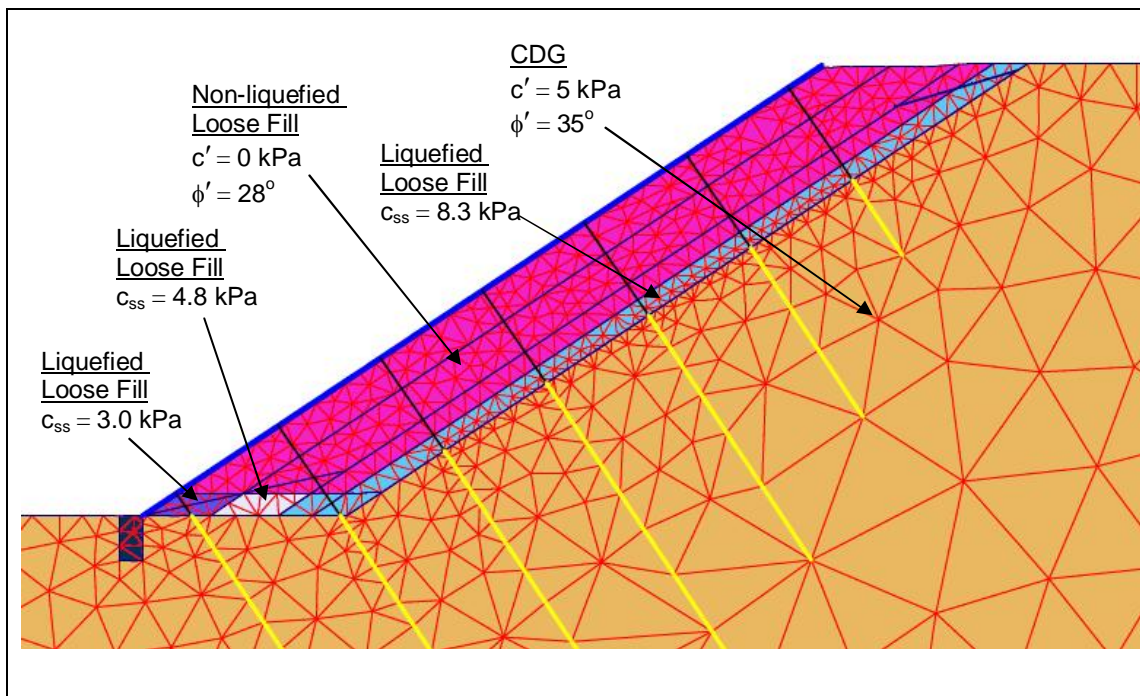


**Figure 3.7    Undrained Shear Strength of the Loose Fill after Interface Liquefaction**

The soil nails are modelled as a combination of node-to-node anchor element and geogrid element as illustrated in Figure 3.5. The geogrid element, which considers interaction between the structural element and the surrounding soil, is used to model the bond length of the soil nail. On the other hand, the node-to-node anchors are spring elements which ignore the interaction with the surrounding soil along its length. They are appropriate for modelling the portion of the soil nail within the loose fill due to possible "flow" behaviour of the liquefied loose fill around the soil nails (Appendix B.2 of GEO-HKIE (2011)). The maximum mobilised tensile force in each soil nail is limited according to the tensile strength of the steel bar. In order to model the soil nails in a plane strain model, the axial stiffness ($EA$) is scaled down by the soil nail spacing of 1.5 m. The grillage facing is modelled as a plate element characterised by its axial stiffness ($EA$) and bending stiffness ($EI$). The structural parameters adopted in the analysis are tabulated in Table 3.3.

The purpose of conducting the numerical analysis is to demonstrate the effectiveness of the proposed nail arrangement. Therefore the focus of the assessment should be placed on whether sufficient nail forces could be mobilised under the assumed loading conditions (i.e. interface liquefaction) and the associated slope deformation. Comparison of design nail forces and mobilised nail forces calculated from the numerical analyses is presented in Figure 3.8. It shows that the mobilised nail forces in interface liquefaction are less than the forces assumed for the full liquefaction. However, since the calculated deformation is limited, the design is considered acceptable. The acceptance criteria for slope deformation should be determined on a site-specific basis taking into account the tolerable movements of any sensitive receivers in close proximity to the slope. The predicted deformation pattern of worked example 2 with and without the toe embedment is presented in Figure 3.9. The input parameters and numerical results of the analysis are presented in Appendix A.2.6.

**Table 3.3   Model Parameters for Soil Nails and Grillage Facing in the Numerical Analysis**

| Structural Type | Model Type | Parameter | Input Value |
|---|---|---|---|
| Soil Nail | Node-to-node Anchor and Geogrid | Axial Stiffness, $EA$ (kN/m) | T16: 26,808<br>T20: 41,867<br>T25: 65,467<br>T32: $1.07 \times 10^5$<br>T40: $1.67 \times 10^5$ |
| | | Maximum Compressive Force, $F_{max, comp}$ (kN/m) | $1.0 \times 10^{15}$ |
| | | Maximum Tensile Force, $F_{max, tens}$ (kN/m) | T16: 61.7<br>T20: 96.3<br>T25: 150.5<br>T32: 246.7<br>T40: 385.4 |
| Grillage Facing | Plate | Axial Stiffness, $EA$ (kN/m) | $2.7 \times 10^6$ |
| | | Bending Stiffness, $EI$ (kNm$^2$/m) | 19,980 |
| | | Weight, $w$ (kN/m$^2$) | 3.5 |
| | | Poisson's Ratio, $v$ | 0.2 |
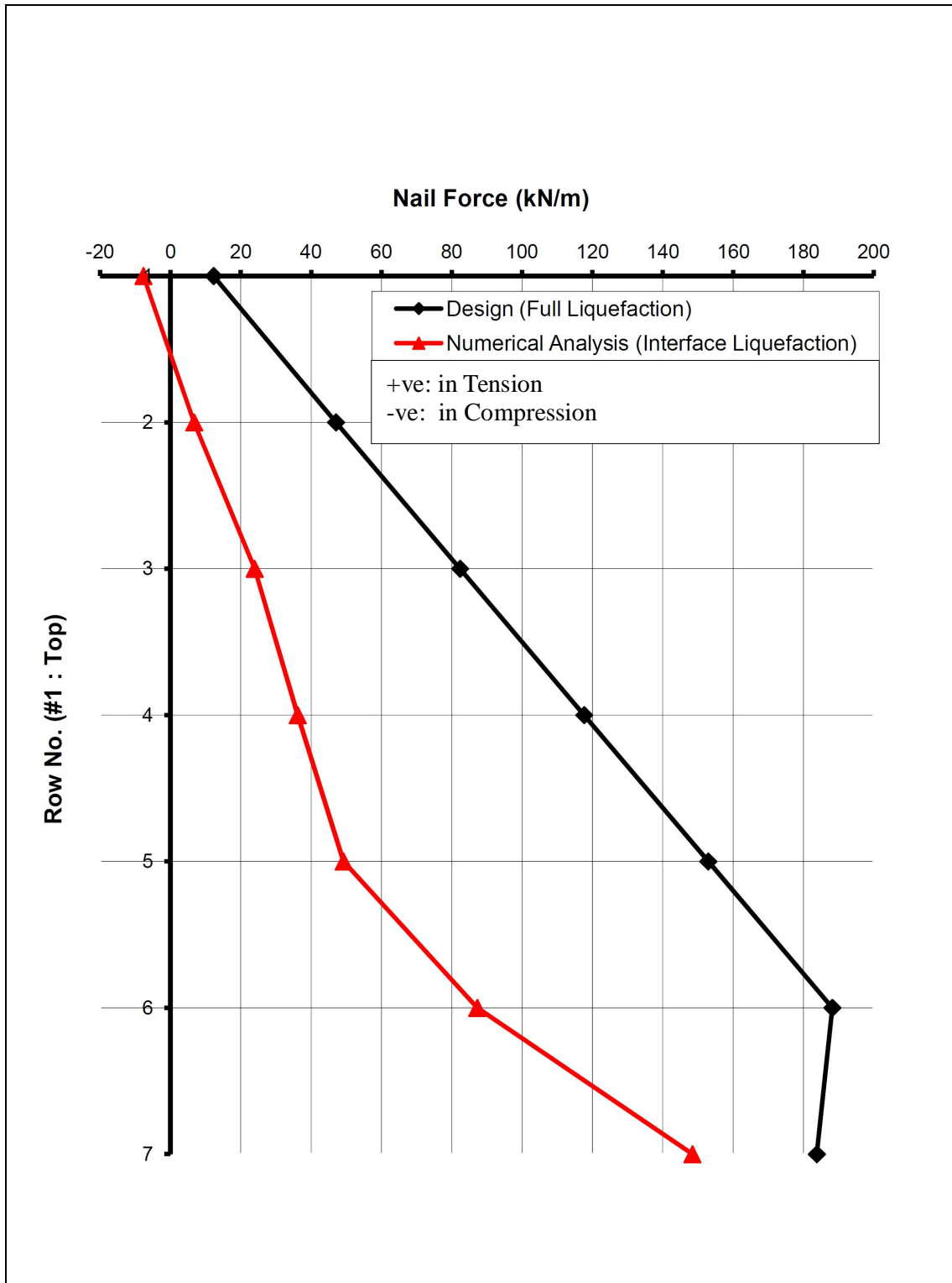
**Figure 3.8   Comparison of Design Nail Forces and Mobilised Nail Forces upon Interface Liquefaction**

Max. Slope Deformation = 108 mm
(a)With Toe Embedment

Max. Slope Deformation = 129 mm
(b)Without Toe Embedment
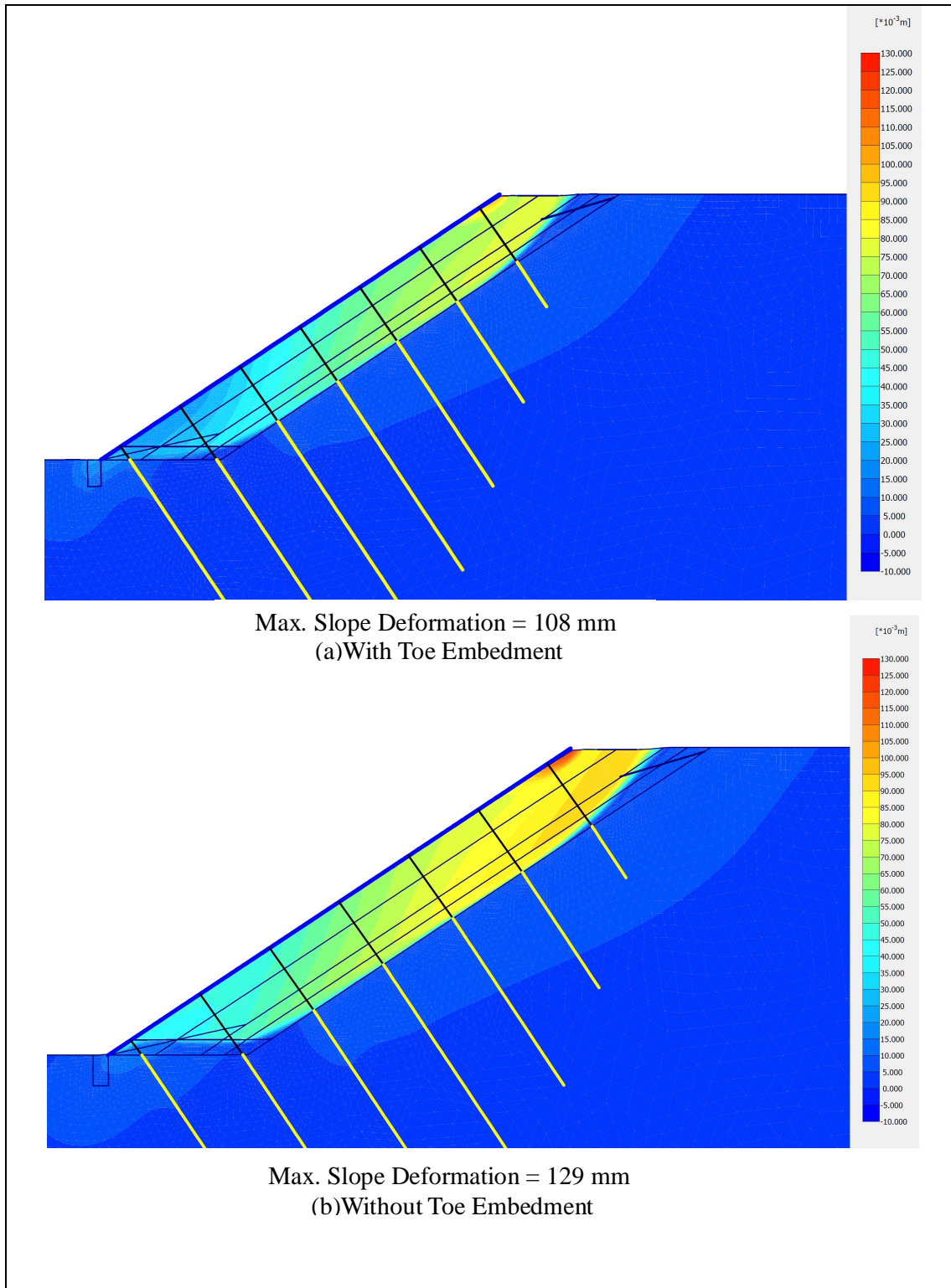
**Figure 3.9   Slope Deformation Predicted by the Numerical Analysis**

### 4 Worked Example 3 – Design for Deep Fill Profile

#### 4.1 Design Scenario and Assumptions

Loose fill at great depths is less likely to undergo static liquefaction. In Section 5.3 of the GEO-HKIE Report, a procedure is proposed to delineate the zone of loose fill that can be assumed to be "non-liquefiable". During stability assessment, the shear resistance of the non-liquefiable fill can be represented by the friction angle at the onset of liquefaction. This would reduce the predicted earth pressure exerted on the grillage facing and hence the required soil nail forces, although the bond strength in this zone has to be ignored.

In the third worked example, a loose fill slope with a deep fill layer is considered to illustrate the design procedure. The design problem considers a 10 m high loose fill slope at an inclination of 33.7° (i.e. 1:1.5) to the horizontal (see Figure 4.1). The loose fill layer has a maximum depth of 6 m deep near the crest of the slope. A design groundwater table (DGWT) at about one-third of the slope height is assumed together with a design perched water table (DPWT) at 1 m above the loose fill / CDG interface. The design soil parameters are tabulated in Table 4.1, which are basically the same as those assumed in worked example 1. For the zone of loose fill which is identified to be non-liquefiable (discussed in Section 4.2), the shear strength parameters are the same as those at the onset of liquefaction for stability assessment purposes.

**Table 4.1   Design Soil Parameters in Worked Example 3**

| Soil Type | Parameter | Value |
|---|---|---|
| Loose Fill (Onset of Liquefaction or Non-liquefiable fill) | Bulk Unit Weight, $\gamma$ (kN/m$^3$) | 17.7 |
| | Cohesion, $c'$ (kPa) | 0 |
| | Friction Angle, $\phi'_{mob}$ ($^o$) | 28 |
| Loose fill (After Liquefaction) | Bulk Unit Weight, $\gamma$ (kN/m$^3$) | 17.7 |
| | Undrained Shear Strength, $c_{ss}$ (kPa) | $0.267\sigma'_v$ |
| | Friction Angle, $\phi'$ ($^o$) | 0 |
| CDG | Bulk Unit Weight, $\gamma$ (kN/m$^3$) | 17.7 |
| | Cohesion, $c'$ (kPa) | 5 |
| | Friction Angle, $\phi'$ ($^o$) | 35 |

**Figure 4.1    Slope Geometry of Worked Example 3**

## 4.2    Delineation of Non-liquefiable Loose Fill Zone

Limit equilibrium method is used to delineate the zone of loose fill which can be assumed to be non-liquefiable (Section 5.3 of GEO-HKIE (2011)).   The calculations have been carried out by SLOPE/W as shown in Figure 4.2.   In the analyses, the entire loose fill zone is represented by the drained friction angle corresponding to the onset of liquefaction (i.e. $\phi'_{mob} = 28°$).   A number of planar "slip surfaces" drawn from the slope toe at different inclinations are specified.   It should be noted that these planar "slip surfaces" must fall entirely within the fill layer.   The procedure is not applicable if the assumed slip surface intercepts the soil layer underneath the fill.   Assuming that a minimum factor of safety of 1.4 is required for this slope, the zone of non-liquefiable fill would be that bounded above by the slip surface which gives a factor of safety of 1.4 (i.e. slip surface 5 in Figure 4.2).   Since the other slip surfaces above give a factor of safety below 1.4, upgrading works are required.

## 4.3    Determination of Earth Pressure to be Sustained by Soil Nails

Once the zone of non-liquefiable fill has been identified, the design procedure that follows is similar to that illustrated in worked example 1.   The earth pressure exerted on the grillage facing is determined by limit equilibrium analysis using the design soil parameters listed in Table 4.1.   Steady state undrained shear strength is assumed in the liquefiable fill only as presented in Figure 4.3.
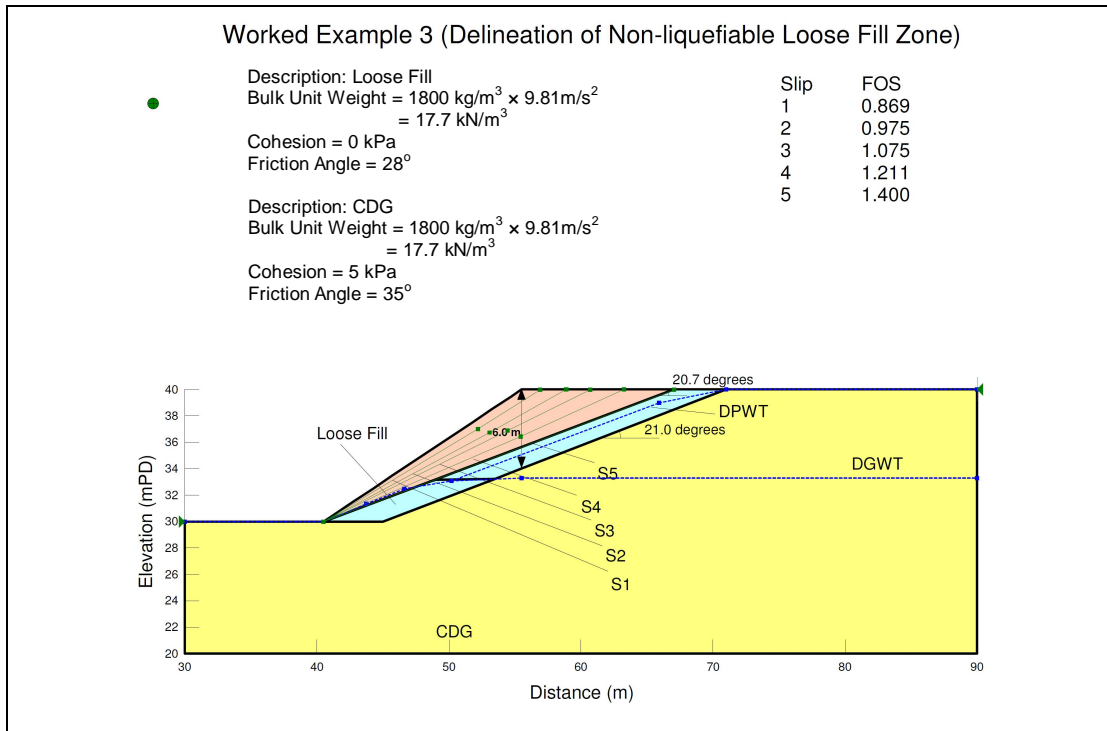
**Figure 4.2    Delineation of Non-liquefiable Loose Fill Zone**
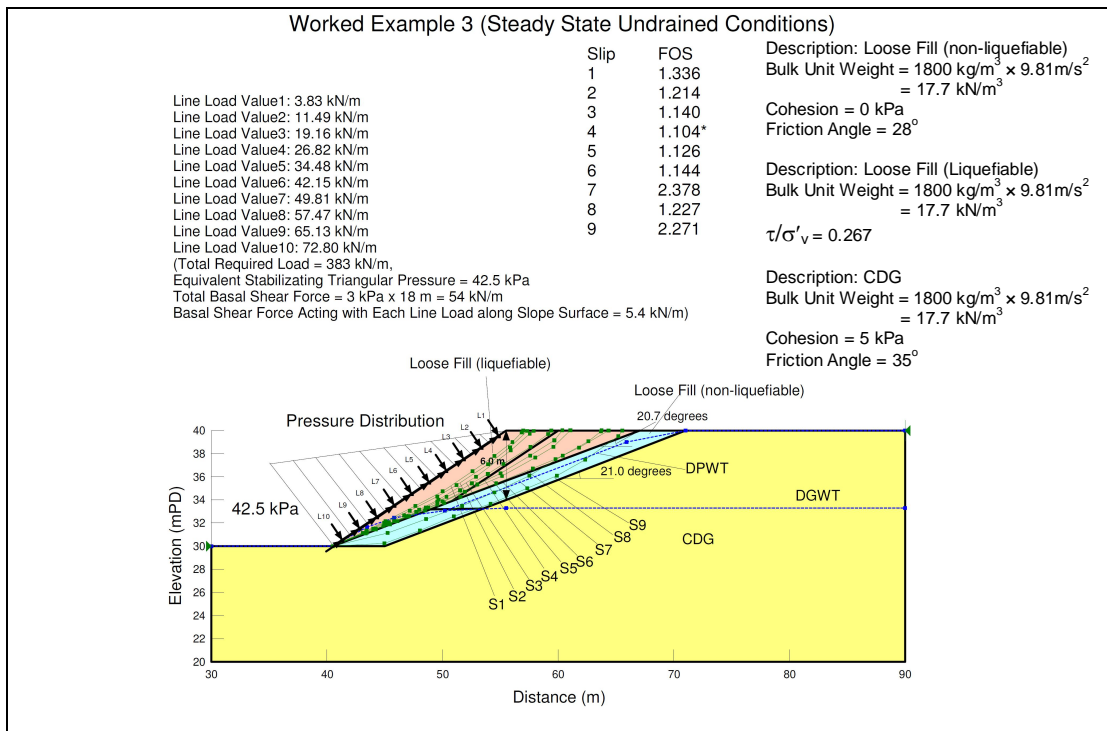


**Figure 4.3    Determination of Earth Pressure Exerted on Grillage Facing upon Liquefaction in Worked Example 3**

**4.4   Design of Soil Nails**

A hybrid nail arrangement is adopted in this worked example.   Similar to the design scheme adopted in worked example 1, the top three rows of soil nails are assumed to be sub-horizontal (i.e. inclined at 20° to the horizontal), whilst the remaining four rows of soil nails are inclined at 56° to the horizontal.   This implies that the number of sub-horizontal nails is 43% of the total number of nails, which satisfies the initial guidance (Section 5.4 of GEO-HKIE (2011)).   The adequacy of this ratio is to be confirmed during the design of the grillage structure (presented in Section 4.6).

The soil nails are assumed to have a spacing of 1.5 m (centre-to-centre) in both the vertical and horizontal directions.   The calculations of individual tensile forces to be supported by each row of soil nails are presented in Appendix A.3.1.   The calculations of diameters and bond lengths of the soil nails are presented in Appendix A.3.2.   It should be emphasised that the bond strength along the soil nails in the non-liquefiable zone has to be ignored (Section 5.3 of GEO-HKIE (2011)).   The final nail arrangement is presented in Figure 4.4.
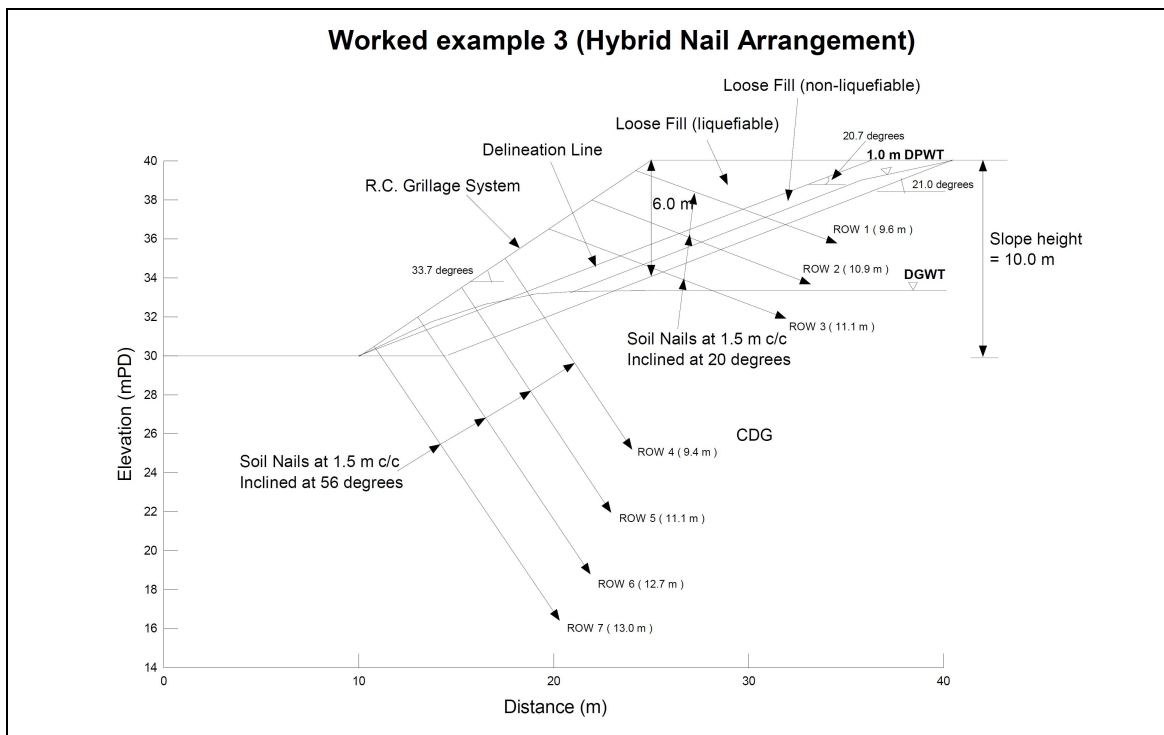


**Figure 4.4 – Soil Nail Arrangement of Worked Example 3**

## 4.5 Stability Assessment of the Loose Fill Slope with Soil Nails under Drained Conditions

The stability of the nailed loose fill slope is assessed again after individual nail forces are determined by limit equilibrium method as shown in Figure 4.5.
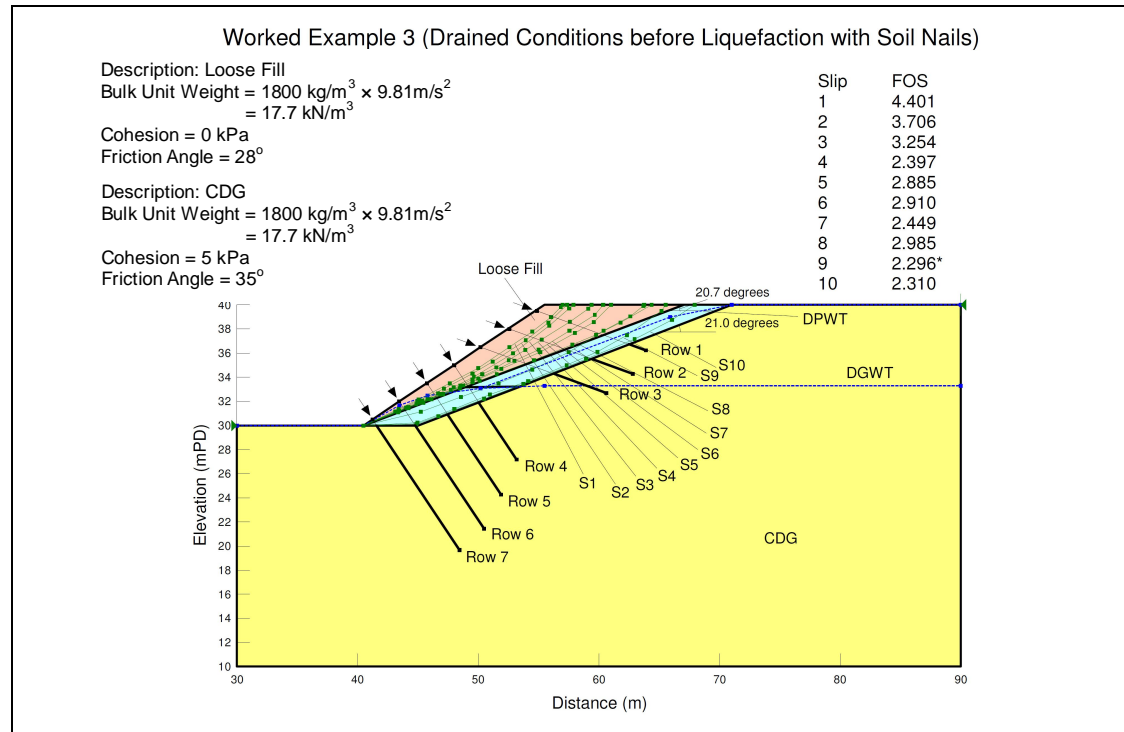


**Figure 4.5    Stability Assessment of the Loose Fill Slope with Soil Nails under Drained Conditions in Worked Example 3**

## 4.6  Design of Grillage Structure and Soil Nail Heads

The design of the grillage facing is similar to that adopted in worked example 1 which consists of grillage beams with a width of 600 mm and a thickness of 300 mm.   The calculations of the coverage area are presented in Appendix A.3.3.   The structural design calculations of the grillage beams are presented in Appendix A.3.4, whilst the design calculations for the soil nail heads are given in Appendix A.3.5.   To confirm that the number of sub-horizontal nails is sufficient to support the weight of the grillage, force equilibrium of the grillage structure is considered.   The calculations are presented in Appendix A.3.6.

## 5  Discussion

The draft version of this report issued in April 2013 was circulated for comments among government departments and practitioners via HKIE-GD. The comments received have been incorporated in the final version of this report issued in October 2013. This Section highlights a few salient points which deserve special attention from designers. The comments and responses are attached in Appendix B.

### 5.1  Limit Equilibrium Analyses

An important step in the design procedure is to determine the required stabilising pressure on the slope facing using limit equilibrium method. The magnitude of a set of concentrated point loads distributed in a triangular shape representing the earth pressure can be determined. In theory, the triangular earth pressure should be modelled using as many point loads as possible. However, if too many point loads are used, the trial-and-error procedure to identify the required stabilising pressure would become impracticable. A simple case of a 10 m high slope has been considered to illustrate that 10 nos. of point loads are sufficient to limit the discrepancy to within 3% (see Comments and Responses on Page 89). For other design conditions, designers should make their own engineering judgement to determine the minimum number of point loads to represent the earth pressure. In addition, when the "Grid and Radius Method" is employed in slope stability analysis, the designer should bear in mind that the slip surface should embrace all the point loads exerted on the slope surface. If the critical slip surface does not embrace all the point loads on the slope surface, omission of some point loads may lead to underestimation of the loading condition.

### 5.2  Irregular Slope Profile

The purpose of this study is to demonstrate the implementation of the new recommendations given in the GEO-HKIE (2011) Report, therefore the slope geometry considered in the three worked examples is simplified. In reality, the slope profile may sometimes be irregular. In this case, the designer should use the actual slope profile to determine the earth pressure exerted on the facing structure. The earth pressure exerted on the facing structure should be assumed to be acting in the normal direction of the slope surface since very small shear stress can be mobilised along the interface between the facing and the loose fill. The small shear stress has already been taken into consideration by the assumed basal shear force. Therefore the earth pressure must be normal to the slope surface. When applying the stabilising point loads, one should consider if the proposed soil nail arrangement would be able to provide the assumed stabilising effects. For example the downward force exerted on the slope berm is unrealistic as no nail force would be mobilised in this direction, and stabilisation force applied on slope berm should be neglected for design purposes.

### 5.3  Delineation of Non-liquefiable Loose Fill Zone

The thickness of loose fill may vary. For the case where the thickness of the loose fill increases with height, similar to the one illustrated in Fig. 4.2, a linear slip surface can be

used to delineate the non-liquefiable zone. For the case where the thickness of loose fill decreases towards the slope crest, the non-liquefiable zone can be identified by assuming linear or bi-linear slip surfaces provided that these slip surfaces fall entirely within the loose fill zone (see Comments and Responses on Page 91). The liquefiable zone refers to the zone above the slip surface which has a minimum FOS of 1.4.

## 5.4 Structural Design of Grillage Structure

In the three worked examples presented in this report, structural design of the grillage was carried out by considering a segment of the grillage near the slope toe. For simplicity, the grillage beam is assumed to be simply supported, and is subjected to a uniformly distributed load. In theory, a continuous beam representing the entire facing structure should be used to determine the structural forces when a triangular pressure is exerted on the facing. A parametric study based on the soil nail arrangement of worked example 2 has been conducted to examine the possible discrepancies produced by the two different approaches described above (i.e., simply supported with uniformly distributed load and continuous beam with a triangular pressure). It is found that the calculated maximum bending moment is in general larger in the case of a simply supported beam with a uniformly distributed load for the conditions considered. The shear forces calculated from the two approaches are comparable in the worked example, with a percentage difference in the order of 1% in general. It can be concluded that simplifying the grillage as segments of simply supported beams is generally acceptable. However, designers should verify this approach on a case-by-case basis.

## 6 Conclusions

Three worked examples have been prepared and presented to illustrate the recommended procedure for designing soil nails to upgrade loose fill slopes. The design scenarios considered in these worked examples are relatively simple as the key objective is to demonstrate the use of hybrid nail arrangement which is recommended for enhanced system robustness. The adopted hybrid nail arrangement has been demonstrated through numerical modelling techniques to be sufficiently robust. For other proposed nail arrangements, e.g. similar to that adopted in worked example 2, designers are required to demonstrate the robustness of the proposed scheme. Particular focus should be placed on the nail force mobilisation mechanisms and the associated mobilised deformation.

# 7 References

BD (2013). *Code of Practice for Structural Use of Concrete.* Buildings Department, Hong Kong, 187 p.

GCO (1984). *Geotechnical Manual for Slopes. (2ⁿᵈ Edition).* Geotechnical Control Office, Civil Engineering Services Department, Hong Kong, 295 p.

GEO (1993). *Guide to Retaining Wall Design (Geoguide 1). Second Edition.* Geotechnical Engineering Office, Civil Engineering Department, Hong Kong, 258 p.

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GEO-HKIE (2011). *Design of Soil Nails for Upgrading Loose Fill Slopes.* Geotechnical Engineering Office, Civil Engineering and Development Department, Hong Kong and The Hong Kong Institution of Engineers (Geotechnical Division), 96 p.

HKIE (2003). *Soil Nails in Loose Fill Slopes. A Preliminary Study (Final Report).* The Hong Kong Institution of Engineers (Geotechnical Division), 88 p.

Appendix A

Detailed Design Calculations of Worked Examples

# Contents

## A.1 Detailed Design Calculation of Worked Example 1

## A.1.1 Calculations of Individual Nail Forces

Grillage structure

$L_s =$ **18.03** m

$H =$ **10.00** m

$\theta =$ **33.7** Degrees

**Slope Geometry**
Slope angle = **33.7** Degrees
Fill slope height = **10** m

**Material Properties**
Fill strength: $c_{ss}/p'_{peak} =$ **0.4**
assumed $K_0 =$ **0.5**
$c_{ss} =$ **0.267** $\sigma'_v$
Note : (i) Min. FoS = **1.1** for liquefaction

**Stabilizing Pressure**
Normal stabilizing pressure u = **66** kPa (see SLOPE/W analysis in Figure 2.3)
Basal shear resistance $\tau$ = **3** kPa *Minimum basal shear resistance has been assumed
Basal shear force $F_b$ = **54.08** kN/m *Shear force is applied on the whole slope surface

**Soil Nails Configuration**
Horizontal spacing = **1.5** m
Vertical spacing = **1.5** m
Spacing between nails on the inclined slab = **2.70** m
Total number of rows = **7**

$F'_{n1}$, $F'_{n2}$, $F'_{n3}$, $F'_{n4}$, $F'_{n5}$, $F'_{n6}$, $F'_{n7}$

$u_1$, $u_2$, $u_3$, $u_4$, $u_5$, $u_6$, $u_7$

$S_{v1}$, $S_{v2}$, $S_{v3}$, $S_{v4}$, $S_{v5}$, $S_{v6}$, $S_{v7}$

$L_{s7}$

Slope surface

$F'_{si}$, $\alpha$, $F_i$, $\theta$, $F'_{ni}$
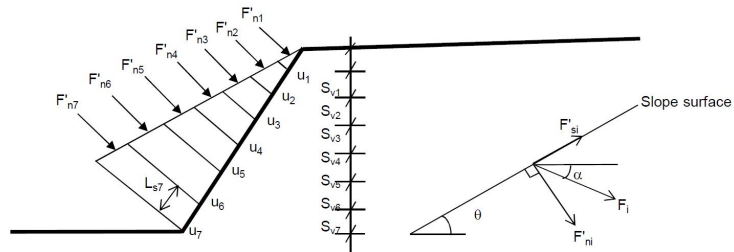
| | Vertical distance from crest (m) | $\alpha$ (°) | $S_{vi}$ (m) | $\Sigma S_{vi}$ from crest (m) | $u_i$ (kPa) | $L_{si}$ (m) | $F'_{ni}$ (kN/m) | $F_i$ (kN/m) | $F'_{si}$ (kN/m) | Individual nail force (kN) |
|---|---|---|---|---|---|---|---|---|---|---|
| Row 1 | 0.50 | 20.00 | 1.25 | 1.25 | 8.25 | 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 |

## A.1.2  Soil Nail Design Calculations

**Soil Parameters**

Type 1,  **LOOSE FILL**          $c' =$  **0.267** $\sigma'_v$          $\gamma =$  **17.7** kN/m³          $\phi' =$  **0.0** °
Type 2,  **CDG**          $c' =$  **5.0** kPa          $\gamma =$  **17.7** kN/m³          $\phi' =$  **35.0** °

Drillhole diameter (D) =          **0.15** m
Unit weight of water, $\gamma_w$ =          **9.81** kN/m³
Horizontal spacing =          **1.50** m
Sacrificial thickness =          **2.00** mm



DGWT

Type 1 material

Type 2 material

Depth to mid-point of the effective bond length for each type of materials

$L_a$

$L_{es}$

Type 3 material

$L_a$ : Length of soil nail in active zone
$L_{es}$ : Effective bond length

Slip surface
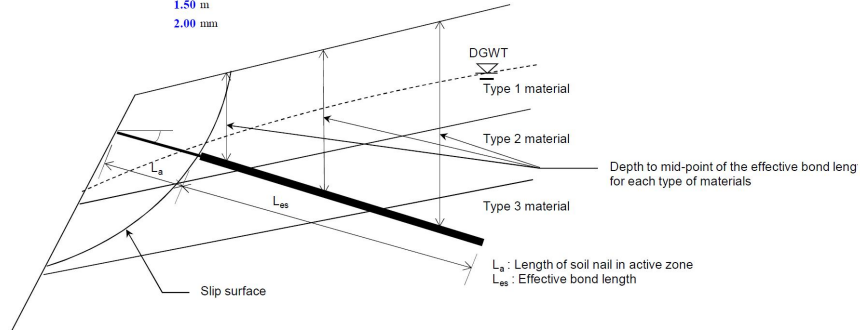
**A.       Tension Failure of the Steel Bar**

Characteristic yield strength, $f_y$  =          **460** MPa
Partial factor for tensile failure of steel bar, $F_T$ =          **1.15**

| Row No. | Level (mPD) | Bar Length (m) | Bar Size | Horizontal Spacing (m) | Force Required per unit width (kN/m) | Force Required $T_r$ (kN) | Allowable Tensile Force $T_T$ (kN) | $T_T / T_r$ |
|---------|-------------|----------------|----------|------------------------|--------------------------------------|---------------------------|------------------------------------|-------------|
| 1 | 39.50 | 5.80 | T16 | 1.50 | 11.54 | 17.30 | 45.24 | 2.61 |
| 2 | 38.00 | 9.00 | T20 | 1.50 | 44.30 | 66.44 | 80.42 | 1.21 |
| 3 | 36.50 | 10.50 | T25 | 1.50 | 77.52 | 116.28 | 138.54 | 1.19 |
| 4 | 35.00 | 9.50 | T25 | 1.50 | 89.24 | 133.86 | 138.54 | 1.04 |
| 5 | 33.50 | 10.80 | T32 | 1.50 | 116.01 | 174.02 | 246.30 | 1.42 |
| 6 | 32.00 | 12.00 | T32 | 1.50 | 142.78 | 214.17 | 246.30 | 1.15 |
| 7 | 30.50 | 11.50 | T32 | 1.50 | 139.44 | 209.15 | 246.30 | 1.18 |

Minimum $T_T / T_r$ =          **1.04**                    **O.K.**

**B.** **Bond Failure between Grout and Steel Reinforcement**

Design cube strength of cement grout at 28 days $f_{cu}$ =     **30** MPa

For type 2 deformed bar, $\beta$ =     **0.5**

FOS against pullout failure at grout-reinforcement interface ($F_{GR}$) =     **2.0**

Allowable bond stress = Ultimate bond stress / $F_{GR}$ = $[\beta(f_{cu})^{1/2}] / F_{GR}$ =     2738.62/2 = **1369.31** $kN/m^2$

Allowable pullout resistance along grout-reinforcement interface = $[\beta(f_{cu})^{1/2}] * \pi * (d-2s) * L_{eg} / F_{GR}$

**Resisting Zone - for soil nail design**

| Row No. | Resisting Zone | | | | | | |
|---|---|---|---|---|---|---|---|
| | Effective Bond Length in Each Layer | | Depth to Mid-point of the Effective Bond Length (m) | | | | |
| | LOOSE FILL | CDG | LOOSE FILL | | CDG | | |
| | | | LOOSE FILL | WATER | LOOSE FILL | CDG | WATER |
| 1 | 0.00 | 2.80 | 0.00 | 0.00 | 0.50 | 1.50 | 0.00 |
| 2 | 0.00 | 6.00 | 0.00 | 0.00 | 1.00 | 3.00 | 0.00 |
| 3 | 0.00 | 7.50 | 0.00 | 0.00 | 2.10 | 3.70 | 0.00 |
| 4 | 0.00 | 7.00 | 0.00 | 0.00 | 3.00 | 4.20 | 0.00 |
| 5 | 0.00 | 8.30 | 0.00 | 0.00 | 3.00 | 5.10 | 0.00 |
| 6 | 0.00 | 9.50 | 0.00 | 0.00 | 3.00 | 5.80 | 0.00 |
| 7 | 0.00 | 11.00 | 0.00 | 0.00 | 3.00 | 4.50 | 0.00 |

| Row No. | Level (mPD) | Bar Length (m) | Bar Size | Horizontal Spacing (m) | $L_{eg}$ (m) | $L_a$ (m) | Force Required per unit width (kN/m) | Force Required $T_r$ (kN) | Allowable Pullout Resistance $T_{GR}$ (kN) | $T_{GR}/T_r$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 39.50 | 5.80 | T16 | 1.50 | 2.80 | 3.00 | 11.54 | 17.30 | 144.54 | 8.35 |
| 2 | 38.00 | 9.00 | T20 | 1.50 | 6.00 | 3.00 | 44.30 | 66.44 | 412.97 | 6.22 |
| 3 | 36.50 | 10.50 | T25 | 1.50 | 7.50 | 3.00 | 77.52 | 116.28 | 677.53 | 5.83 |
| 4 | 35.00 | 9.50 | T25 | 1.50 | 7.00 | 2.50 | 89.24 | 133.86 | 632.37 | 4.72 |
| 5 | 33.50 | 10.80 | T32 | 1.50 | 8.30 | 2.50 | 116.01 | 174.02 | 999.74 | 5.75 |
| 6 | 32.00 | 12.00 | T32 | 1.50 | 9.50 | 2.50 | 142.78 | 214.17 | 1144.28 | 5.34 |
| 7 | 30.50 | 11.50 | T32 | 1.50 | 11.00 | 0.50 | 139.44 | 209.15 | 1324.96 | 6.33 |

Minimum of $T_{GR} / T_r$ =     **4.72**     > 1     **O.K.**

**C.** **Shear Failure of Adjacent Ground**

Allowable pullout resistance provided by the soil-grout bond length in the passive zone:  $T_{SG} = (\pi D c' + 2 D \sigma_v' \tan\phi) * L_{es} / F_{SG}$

The rock-grout bond strength for weathred rock mass of PW90/100 or better=  **0.35** MPa
FOS against pullout failure at soil-grout interface ($F_{SG}$) =  **1.5**

| LOOSE FILL | = | ( | 0.126 | + | 0.000 $\sigma_v'$ ) * | $L_{es}/F_{SG}$ |

| CDG | = | ( | 2.356 | + | 0.210 $\sigma_v'$ ) * | $L_{es}/F_{SG}$ |

**Soil Nail Schedule Table**

| Row No. | Level (mPD) | Bar Length (m) | Bar Size | Horizontal Spacing (m) | $L_{es}$ (m) | $L_a$ (m) | Force Required per unit width (kN/m) | Force Required $T_r$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | 39.50 | 5.80 | T16 | 1.50 | 2.80 | 3.00 | 11.54 | 17.30 |
| 2 | 38.00 | 9.00 | T20 | 1.50 | 6.00 | 3.00 | 44.30 | 66.44 |
| 3 | 36.50 | 10.50 | T25 | 1.50 | 7.50 | 3.00 | 77.52 | 116.28 |
| 4 | 35.00 | 9.50 | T25 | 1.50 | 7.00 | 2.50 | 89.24 | 133.86 |
| 5 | 33.50 | 10.80 | T32 | 1.50 | 8.30 | 2.50 | 116.01 | 174.02 |
| 6 | 32.00 | 12.00 | T32 | 1.50 | 9.50 | 2.50 | 142.78 | 214.17 |
| 7 | 30.50 | 11.50 | T32 | 1.50 | 11.00 | 0.50 | 139.44 | 209.15 |

| Row No. | Effective Vertical Stress $\sigma_v'$ (kPa) | | Allowable Pullout Resistance in Soil $T_{SG\_soil}$ (kN) | | Embedded Rock Length | Allowable Pullout Resistance in Rock | Allowable Pullout Resistance | |
|---|---|---|---|---|---|---|---|---|
| | LOOSE FILL | CDG | LOOSE FILL | CDG | L (m) | $T_{SG\ rock}$ (kN) | $T_{SG}$ (kN) | $T_{SG} / T_r$ |
| 1 | N/A | 35.40 | 0.00 | 18.28 | 0.00 | 0.00 | 18.28 | 1.06 |
| 2 | N/A | 70.80 | 0.00 | 68.91 | 0.00 | 0.00 | 68.91 | 1.04 |
| 3 | N/A | 102.66 | 0.00 | 119.60 | 0.00 | 0.00 | 119.60 | 1.03 |
| 4 | N/A | 127.44 | 0.00 | 135.92 | 0.00 | 0.00 | 135.92 | 1.02 |
| 5 | N/A | 143.37 | 0.00 | 179.68 | 0.00 | 0.00 | 179.68 | 1.03 |
| 6 | N/A | 155.76 | 0.00 | 222.14 | 0.00 | 0.00 | 222.14 | 1.04 |
| 7 | N/A | 132.75 | 0.00 | 221.77 | 0.00 | 0.00 | 221.77 | 1.06 |

Minimum of $T_{SG}/T_r$ =  **1.02**  > 1  **O.K.**

## A.1.3  Percentage Coverage of Grillage Facing

Width of grillage beam = **600** mm
Thickness of grillage beam = **300** mm
Embedment of grillage beam = **300** mm

0.30 m

2.70 m

0.30 m          1.50 m
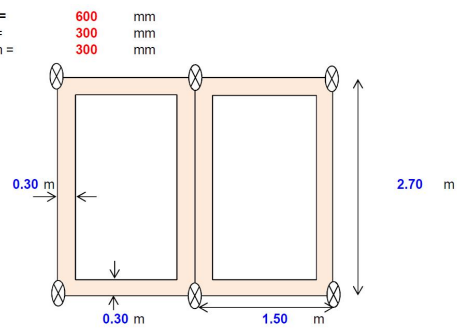
% Coverage of Slope Surface = **53%**          **O.K. (i.e. % Coverage >50%)**

## A.1.4  Structural Design Calculations of Grillage Facing

<u>Section Properties of Concrete Grillage Beam</u>
Width of grillage beam, b=                                       **600** mm
Depth of grillage beam, h=                                      **300** mm

Minimum cover of grillage beam, C=                         **50** mm
Concrete strength, $f_{cu}$=                                        **30** N/mm²

Characteristic strength of steel bar, $f_y$=                  **460** N/mm²

**Plan View**
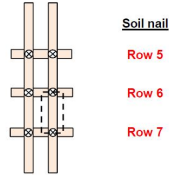
**Soil nail**

**Row 5**

**Row 6**

**Row 7**

<u>Proposed reinforcement</u>
No. of steel bar =                                                     **4**
Diameter of steel bar =                                          **20** mm

Area of steel, $A_s$ =                                              **1257** mm²
Diameter of stirrup, $\phi_s$ =                                    **12** mm
Effective depth, d =                                              **228** mm

**(a) Determine Design Bending Moment and Shear in the Grillage System (Longitudinal Span)**
**(i) Design of Bending Reinforcement (Longitudinal Span)**

Design moment  $M = u' * L^2 * S_h / 8$
$\quad\quad$ u' =                              **57.75** kPa
$\quad\quad$ M =                              **79.18** kNm

Minimum reinforcement = 0.13%bh =             **234** mm²        **O.K.**

$\quad\quad$ K =          $M / (bd^2 * f_{cu})$ =          **0.08** O.K. (i.e. <0.156 - No compression steel is required)
$\quad$ Lever arm, z =    $(0.5+ \sqrt{(0.25-K/0.9)}*d$ =      **204** mm

Therefore, $A_s$ (Required) = $M / (0.87 * f_y * z)$ =          **970** mm²      < $A_s$ (provided) =                    **1257** mm²        **O.K.**

**(ii) Design of Shear Reinforcement (Longitudinal Span)**

Design shear $V = u' * L * S_h / 2$ =                          **117.12** kN

Design shear stress, $\nu$ = V/bd =                           **0.86** N/mm²

According to Table 6.3 in Code of Practice of Structural use of Concrete 2013
$\quad\quad\quad\quad\quad\quad$ $\nu_c = \dfrac{0.79 * (100A_s/bd)^{1/3} * (400/d)^{1/4} * (f_{cu}/25)^{1/3}}{\gamma_m}$          where $\gamma_m$ = **1.25**

$\quad\quad\quad\quad$ =          **0.75** N/mm²

<u>Detail of Shear Link:</u>
No. of steel bar =                                                  **2**
Diameter of steel bar =                                          **12** mm
Spacing of links along the member =                        **200** mm
Area of steel, $A_{sv}$ =                                            **226** mm²

According to Table 6.2 in Code of Practice of Structural use of Concrete 2013
$\quad\quad$ $0.5\nu_c < \nu < \nu_c + 0.4$ =          **0.38**        <          **0.86**        <          **1.15**

Area of shear reinforcement, $A_{sv} \geq (0.4*b_v*S_v) / 0.87f_{yv}$ =          **221** mm²        **O.K.**          where $f_{yv}$ = **250** N/mm²

**(iii) Checking on Axial Tensile Force (Longitudinal Span)**

Axial tensile force generated =                  **78.96** kN          **in tension**
Design bending moment =                        **79.18** kNm

According to Figure 6.1 and Cl 6.1.2.4(a) in Code of Practice of Structural use of Concrete 2013
The axial tensile force generated with respect to the bending moment are within the allowable capacity of the beam as shown in the chart below

Axial Force - Bending Moment Interaction Diagram of Grillage Beam

Axial Force, N (kN)

Compression

Tension

(79.18,-78.96)

Moment, Mxx (kNm)

Section Properties of Concrete Grillage Beam
**(b) Determine Design Bending Moment and Shear in the Grillage System (Transverse Span)**

| | | |
|---|---|---|
| **Plan View** | | **Soil nail** |
| | | **Row 5** |
| | | **Row 6** |
| | | **Row 7** |
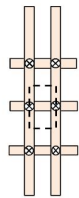
Section Properties of Concrete Grillage Beam
Proposed reinforcement
No. of steel bar = **2**
Diameter of steel bar = **20** mm

Area of steel, $A_s$ = **628** mm$^2$
Dia. of stirrup, $\phi_s$ = **12** mm
Effective depth, d = **228** mm

**(i) Design of Bending Reinforcement (Transverse Span)**

Since **Row 6** requires the greatest nail force, use resultant pressure at **Row 6** to determine the design bending moment.  Hence resultant pressure,
Design Moment  $M = u' * L * S_h^2 / 8$

u" = **52.80** kPa
M = **40.16** kNm

Minimum reinforcement = 0.13%bh = **234** mm$^2$          O.K.

According to Cl 6.1.2.4 (c) in Code of Practice of Structural Use of Concrete 2013

K = $M / (bd^2 * f_{cu})$ = **0.04**     O.K. (i.e. <0.156 - No compression steel is required)
Lever arm, z = $(0.5+ \sqrt{(0.25-K/0.9)}*d$ = **217** mm

Therefore, $A_s$ (Required) = $M / (0.87 * fy * z)$ =   **463** mm$^2$   < $A_s$ (provided) =   **628** mm$^2$   O.K.

**(ii) Design of Shear Reinforcement (Transverse Span)**

Design shear $V = u' * L * S_h / 2$ =   **107.08** kN

Design shear stress, $\nu$ = V/bd =   **0.78** N/mm$^2$

According to Table 6.3 in Code of Practice of Structural use of Concrete 2013

Design concrete shear strength, $v_c = \dfrac{0.79 * (100A_s/bd)^{1/3} * (400/d)^{1/4} * (f_{cu}/25)^{1/3}}{\gamma_m}$   where $\gamma_m$ = **1.25**

= **0.60** N/mm$^2$

Detail of Shear Link:
No. of steel bar = **2**
Diameter of steel bar = **12** mm
Spacing of links along the member = **200** mm
Area of steel, $A_{sv}$ = **226** mm$^2$

According to Table 6.2 in Code of Practice of Structural use of Concrete 2013

$0.5v_c<v<v_c+0.4$ =   **0.30**   <   **0.78**   <   **1.00**

Area of shear reinforcement, $A_{sv} \geq (0.4*b_v*S_v) / 0.87f_{yv}$ =   **220.69** mm$^2$   O.K.   where $f_{yv}$ = **250** N/mm$^2$

## A.1.5  Structural Design Calculations of Soil Nail Heads

Section Properties of Concrete Grillage Beam
**(c) Design of Connection between Soil Nail & Grillage Beam**

**(i) Bearing Failure of the Base Plate**

According to Cl 6.1.5.7 (b) in Code of Practice of Structural use of Concrete 2013

Maximum design shear stress, $v_{max}$ =     $V/u_o d$

Maximum soil nail load, V = **214.17** kN

Width of base plate, $b_1$ = **150** mm

Perimeter of base plate, $u_o$ = 4 * 150mm =     **600** mm

Depth below steel plate, $d_1$ = **180** mm

Concrete cover, C= **50** mm

Bar reinforcement, $\phi$ = **20** mm

Effective depth of nail head, $d_e$ = **100** mm

$v_{max}$ = **3.57** N/mm$^2$    $<0.8*\sqrt{f_{cu}}$,    **4.38** N/mm$^2$    **O.K.**

**(ii) Punching Failure at the Nail Head**
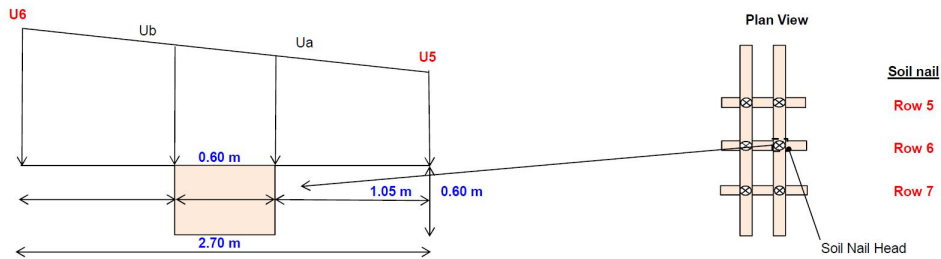


Proposed reinforcement
No. of steel bar = **3**
Diameter of steel bar = **20** mm

Area of steel, $A_s$ = **942** mm$^2$
Width of nail head, b = **600** mm
Effective depth of nail head, $d_e$ = **100** mm

According to Cl 6.1.5.7 (c) in Code of Practice of Structural use of Concrete 2013

Punching force, $F_{nail}$ = **214.17** kN

$U_a$= **51.70** kPa

$U_b$= **53.90** kPa

Force from soil, $F_{soil}$ = $(U_a+U_b)$ / 2 * Area of soil nail head

= **19.01** kN

Hence, $F_{resultant}$ = $F_{nail}$ - $F_{soil}$

= **195.16** kN

Outer perimeter 1.5d from loaded area, u = **1800** mm

Hence, v = V/ud = **1.08** N/mm$^2$

According to Table 6.3 & Cl 6.1.5.7 (d) in Code of Practice of Structural use of Concrete 2013

Design concrete shear strength, $v_c$ = $\dfrac{0.79 * (100 A_s / bd)^{1/3} * (400/d)^{1/4} * (f_{cu}/25)^{1/3}}{\gamma_m}$    where $\gamma_m$ = **1.25**

= **1.10** N/mm$^2$    > v =    **1.08** N/mm$^2$    **O.K.**

## A.1.6 Force Equilibrium Assessment of Grillage Facing

$L_s$ = **18.03** m

$F_b$

$F_w$'

$F_w$

θ = **33.7 Degrees**

Embedded Grillage Beam

**300** mm

Cross-section of grillage beam
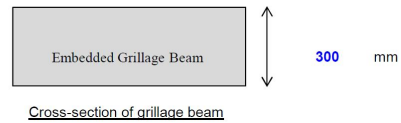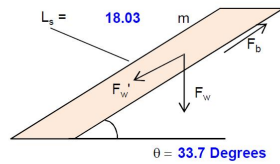
Driving Force
Weight of grillage structure, $F_g$ = **69.20** kN/m
Weight of backfill inside the opening, $F_s$ = **44.69** kN/m
Total Weight of grillage structure and backfill, $F_w$ = **113.89** kN/m
Force component along potential sliding direction, $F'_w$ = **63.18** kN/m

Resisting Force
Available resisting force along the sliding direction from sub-horizontal nails, $\Sigma F'_{si}$= **78.96** kN/m
Resisting force due to liquefied loose fill, $F_b$ = **54.08** kN/m
Therefore, $\Sigma F'_{si}$+ $F'_b$ = **133.05** kN/m   / $F'_w$ = **63.18** kN/m  =  2.11   O.K. (>1)

Note: The evaluation of force equilibrium along the potential sliding direction ensures that there is sufficient resisting force to avoid excessive slope deformation due to the weight of the grillage structure when the underlying loose fill liquefies.

[Refer to GEO-HKIE (2011) - Section 5.4]

## A.2 Detailed Design Calculation of Worked Example 2

## A.2.1 Calculations of Individual Nail Forces
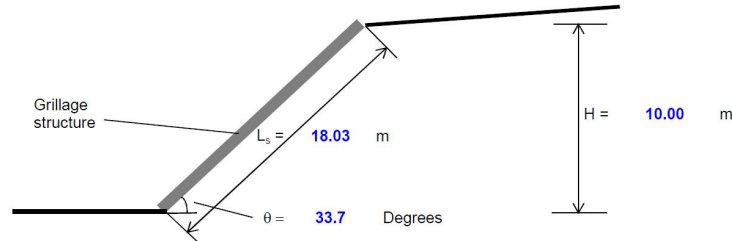


**Slope Geometry**
Slope angle = 33.7 Degrees
Fill slope height = 10 m

**Material Properties**
Fill strength: $c_{ss}/p'_{peak}$ = 0.3
assumed $K_0$ = 0.5
$c_{ss}$ = 0.200 $\sigma'_v$
i\ Min. FoS = 1.1 for liquefaction

**Stabilizing Pressure**
Normal stabilizing pressure u = 87.0 kPa (see SLOPE/W analysis in Figure 3.2)
Basal shear resistance τ = 3 kPa *Minimum basal shear resistance has been assumed
Basal shear force $F_b$ = 54.08 kN/m *Shear force is applied on the whole slope surface

**Soil Nails Configuration**
Horizontal spacing = 1.5 m
Vertical spacing = 1.5 m
Spacing between nails on the inclined slab = 2.70 m
Total number of rows = 7
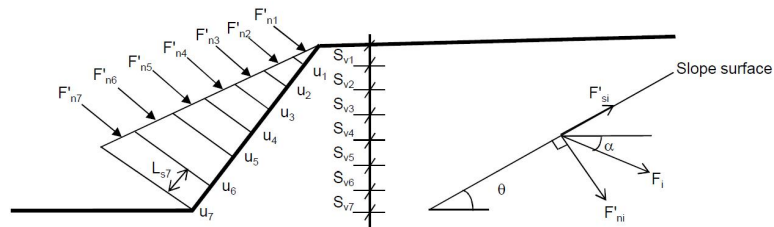
| | Vertical distance from crest (m) | α (°) | $S_{vi}$ (m) | $\Sigma S_{vi}$ from crest (m) | $u_i$ (kPa) | $L_{si}$ (m) | $F'_{ni}$ (kN/m) | $F_i$ (kN/m) | $F'_{si}$ (kN/m) | Individual nail force (kN) |
|---|---|---|---|---|---|---|---|---|---|---|
| Row 1 | 0.50 | 56 | 1.25 | 1.25 | 10.88 | 2.25 | 12.25 | 12.25 | 0.00 | 18.38 |
| Row 2 | 2.00 | 56 | 1.50 | 2.75 | 23.93 | 2.70 | 47.05 | 47.05 | 0.00 | 70.58 |
| Row 3 | 3.50 | 56 | 1.50 | 4.25 | 36.98 | 2.70 | 82.34 | 82.34 | 0.00 | 123.51 |
| Row 4 | 5.00 | 56 | 1.50 | 5.75 | 50.03 | 2.70 | 117.63 | 117.63 | 0.00 | 176.45 |
| Row 5 | 6.50 | 56 | 1.50 | 7.25 | 63.08 | 2.70 | 152.92 | 152.92 | 0.00 | 229.38 |
| Row 6 | 8.00 | 56 | 1.50 | 8.75 | 76.13 | 2.70 | 188.21 | 188.21 | 0.00 | 282.31 |
| Row 7 | 9.50 | 56 | 1.25 | 10.00 | 87.00 | 2.25 | 183.80 | 183.80 | 0.00 | 275.70 |
| | | | | | | Total= | 784.21 | 784.21 | 0.00 | 1176.31 |
| | | | | | | Max. value= | | 188.21 | | 282.31 |

## A.2.2  Soil Nail Design Calculations

**Soil Parameters**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Type 1, | **LOOSE FILL** | $c' =$ | **0.20** $\sigma'_v$ | $\gamma =$ | **17.7** kN/m³ | $\phi' =$ | **0.0** ° |
| Type 2, | **CDG** | $c' =$ | **5.0** kPa | $\gamma =$ | **17.7** kN/m³ | $\phi' =$ | **35.0** ° |

Drillhole diameter (D) = **0.15** m
Unit weight of water, $\gamma_w$ = **9.81** kN/m³
Horizontal spacing = **1.50** m
Sacrificial thickness = **2.00** mm

DGWT
Type 1 material
Type 2 material
Depth to mid-point of the effective bond length for each type of materials
Type 3 material
$L_a$
$L_{es}$
$L_a$ : Length of soil nail in active zone
$L_{es}$ : Effective bond length
Slip surface
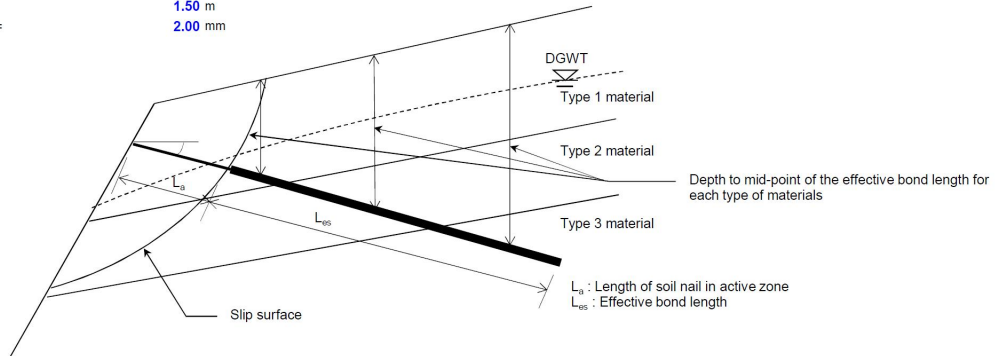
**A.      Tension Failure of the Steel Bar**

Characteristic yield strength, $f_y$ =                                    **460** MPa
Partial factor for tensile failure of steel bar, $F_T$ =          **1.15**

| Row No. | Level (mPD) | Bar Length (m) | Bar Size | Horizontal Spacing (m) | Force Required per unit width (kN/m) | Force Required $T_r$ (kN) | Allowable Tensile Force $T_T$ (kN) | $T_T / T_r$ |
|---|---|---|---|---|---|---|---|---|
| 1 | 39.50 | 4.50 | T16 | 1.50 | 12.25 | 18.38 | 45.24 | 2.46 |
| 2 | 38.00 | 7.00 | T20 | 1.50 | 47.05 | 70.58 | 80.42 | 1.14 |
| 3 | 36.50 | 9.00 | T25 | 1.50 | 82.34 | 123.51 | 138.54 | 1.12 |
| 4 | 35.00 | 11.00 | T32 | 1.50 | 117.63 | 176.45 | 246.30 | 1.40 |
| 5 | 33.50 | 12.20 | T32 | 1.50 | 152.92 | 229.38 | 246.30 | 1.07 |
| 6 | 32.00 | 14.00 | T40 | 1.50 | 188.21 | 282.31 | 407.15 | 1.44 |
| 7 | 30.50 | 13.50 | T40 | 1.50 | 183.80 | 275.70 | 407.15 | 1.48 |

Minimum $T_T / T_r$ =          **1.07**                > 1                **O.K.**

**B.** **Bond Failure between Grout and Steel Reinforcement**

Design cube strength of cement grout at 28 days $f_{cu}$ =                                          **30** MPa

For type 2 deformed bar, $\beta$ =                                          **0.5**

FOS against pullout failure at grout-reinforcement interface ($F_{GR}$) =                                          **2.0**

Allowable bond stress = Ultimate bond stress / $F_{GR}$ = $[\beta (f_{cu})^{1/2}] / F_{GR}$ =                 2738.62/2 =   **1369.31** kN/m$^2$

Allowable pullout resistance along grout-reinforcement interface = $[\beta (f_{cu})^{1/2}] * \pi * (d-2s) * L_{eg} / F_{GR}$

**Resisting Zone - for soil nail design**

| Row No. | Resisting Zone | | | | | | |
|---|---|---|---|---|---|---|---|
| | Effective Bond Length in Each Layer | | Depth to Mid-point of the Effective Bond Length (m) | | | | |
| | LOOSE FILL | CDG | LOOSE FILL | | CDG | | |
| | | | LOOSE FILL | WATER | LOOSE FILL | CDG | WATER |
| 1 | 0.00 | 2.00 | 0.00 | 0.00 | 2.00 | 1.50 | 0.00 |
| 2 | 0.00 | 4.50 | 0.00 | 0.00 | 3.00 | 3.00 | 0.00 |
| 3 | 0.00 | 6.50 | 0.00 | 0.00 | 3.00 | 4.20 | 0.00 |
| 4 | 0.00 | 8.50 | 0.00 | 0.00 | 3.00 | 5.20 | 0.00 |
| 5 | 0.00 | 9.70 | 0.00 | 0.00 | 3.00 | 6.20 | 0.00 |
| 6 | 0.00 | 11.50 | 0.00 | 0.00 | 3.00 | 7.00 | 0.00 |
| 7 | 0.00 | 12.80 | 0.00 | 0.00 | 3.00 | 5.50 | 0.00 |

| Row No. | Level (mPD) | Bar Length (m) | Bar Size | Horizontal Spacing (m) | $L_{eg}$ (m) | $L_a$ (m) | Force Required per unit width (kN/m) | Force Required $T_r$ (kN) | Allowable Pullout Resistance $T_{GR}$ (kN) | $T_{GR}/T_r$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 39.50 | 4.50 | T16 | 1.50 | 2.00 | 2.50 | 12.25 | 18.38 | 103.24 | 5.62 |
| 2 | 38.00 | 7.00 | T20 | 1.50 | 4.50 | 2.50 | 47.05 | 70.58 | 309.73 | 4.39 |
| 3 | 36.50 | 9.00 | T25 | 1.50 | 6.50 | 2.50 | 82.34 | 123.51 | 587.20 | 4.75 |
| 4 | 35.00 | 11.00 | T32 | 1.50 | 8.50 | 2.50 | 117.63 | 176.45 | 1023.83 | 5.80 |
| 5 | 33.50 | 12.20 | T32 | 1.50 | 9.70 | 2.50 | 152.92 | 229.38 | 1168.37 | 5.09 |
| 6 | 32.00 | 14.00 | T40 | 1.50 | 11.50 | 2.50 | 188.21 | 282.31 | 1780.95 | 6.31 |
| 7 | 30.50 | 13.50 | T40 | 1.50 | 12.80 | 0.70 | 183.80 | 275.70 | 1982.27 | 7.19 |

Minimum of $T_{GR} / T_r$ =         **4.39**         > 1         **O.K.**

**C.** **Shear Failure of Adjacent Ground**

Allowable pullout resistance provided by the soil-grout bond length in the passive zone:   $T_{SG} = (\pi\ D\ c' + 2\ D\ \sigma_v'\ \tan f) * L_{es}\ /F_{SG}$

The rock-grout bond strength for weathred rock mass of PW90/100 or better=   **0.35** MPa
FOS against pullout failure at soil-grout interface $(F_{SG})$ =   **1.5**

**LOOSE FILL**   =   (   0.094   +   0.000 $\sigma_v'$ )*   $L_{es}\ /F_{SG}$

**CDG**   =   (   2.356   +   0.210 $\sigma_v'$ )*   $L_{es}\ /F_{SG}$

<u>Soil Nail Schedule Table</u>

| Row No. | Level (mPD) | Bar Length (m) | Bar Size | Horizontal Spacing (m) | $L_{es}$ (m) | $L_a$ (m) | Force Required per unit width (kN/m) | Force Required $T_r$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | 39.50 | 4.50 | T16 | 1.50 | 2.00 | 2.50 | 12.25 | 18.38 |
| 2 | 38.00 | 7.00 | T20 | 1.50 | 4.50 | 2.50 | 47.05 | 70.58 |
| 3 | 36.50 | 9.00 | T25 | 1.50 | 6.50 | 2.50 | 82.34 | 123.51 |
| 4 | 35.00 | 11.00 | T32 | 1.50 | 8.50 | 2.50 | 117.63 | 176.45 |
| 5 | 33.50 | 12.20 | T32 | 1.50 | 9.70 | 2.50 | 152.92 | 229.38 |
| 6 | 32.00 | 14.00 | T40 | 1.50 | 11.50 | 2.50 | 188.21 | 282.31 |
| 7 | 30.50 | 13.50 | T40 | 1.50 | 12.80 | 0.70 | 183.80 | 275.70 |

| Row No. | Effective Vertical Stress $\sigma_v'$ (kPa) | | Allowable Pullout Resistance in Soil $T_{SG\_soil}$ (kN) | | Embedded Rock Length L (m) | Allowable Pullout Resistance in Rock $T_{SG\_rock}$ (kN) | Allowable Pullout Resistance $T_{SG}$ (kN) | $T_{SG}\ /\ T_r$ |
|---|---|---|---|---|---|---|---|---|
| | LOOSE FILL | CDG | LOOSE FILL | CDG | | | | |
| 1 | N/A | 61.95 | 0.00 | 20.49 | **0.00** | 0.00 | 20.49 | **1.11** |
| 2 | N/A | 106.20 | 0.00 | 73.99 | **0.00** | 0.00 | 73.99 | **1.05** |
| 3 | N/A | 127.44 | 0.00 | 126.21 | **0.00** | 0.00 | 126.21 | **1.02** |
| 4 | N/A | 145.14 | 0.00 | 186.12 | **0.00** | 0.00 | 186.12 | **1.05** |
| 5 | N/A | 162.84 | 0.00 | 236.44 | **0.00** | 0.00 | 236.44 | **1.03** |
| 6 | N/A | 177.00 | 0.00 | 303.12 | **0.00** | 0.00 | 303.12 | **1.07** |
| 7 | N/A | 150.45 | 0.00 | 289.79 | **0.00** | 0.00 | 289.79 | **1.05** |

Minimum of $T_{SG}/T_r$ =   **1.02**   > 1   **O.K.**

## A.2.3 Percentage Coverage of Grillage Facing

Width of grillage beam =          600      mm
Thickness of grillage beam =      300      mm
Embedment of grillage beam =      300      mm

0.30 m

2.70    m

0.30 m          1.50      m
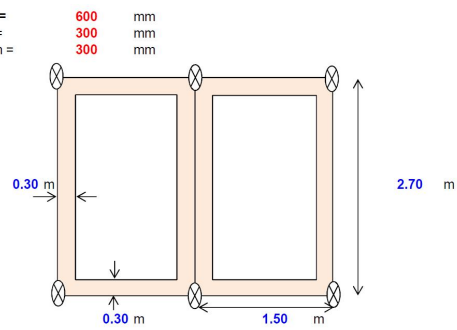
% Coverage of Slope Surface =          53%          O.K. (i.e. % Coverage >50%)

## A.2.4  Structural Design Calculations of Grillage Facing

Section Properties of Concrete Grillage Beam

Width of grillage beam,  b= **600** mm

Depth of grillage beam, h= **300** mm

Minimum cover of grillage beam, C= **50** mm

Concrete strength, $f_{cu}$= **30** N/mm$^2$

Characteristic strength of steel bar, $f_y$= **460** N/mm$^2$

**Plan View**
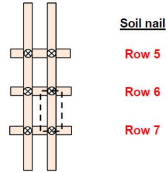
**Soil nail**

**Row 5**

**Row 6**

**Row 7**

Proposed reinforcement

No. of steel bar = **4**

Diameter of steel bar = **25** mm

Area of steel, $A_s$ = **1963** mm$^2$

Diameter of stirrup, $\phi_s$ = **12** mm

Effective depth, d = **226** mm

**(a) Determine Design Bending Moment and Shear in the Grillage System (Longitudinal Span)**
**(i) Design of Bending Reinforcement (Longitudinal Span)**

Design moment  M = u' * L$^2$ * Sh / 8

u' = **76.13** kPa

M = **104.37** kNm

Minimum reinforcement = 0.13%bh = **234** mm$^2$       **O.K.**

According to Cl. 6.1.2.4 (c) in Code of Practice of Structural Use of Concrete 2013

K = M / (bd$^2$ * $f_{cu}$) = **0.11** **O.K. (i.e. <0.156 - No compression steel is required)**

Lever arm, z = (0.5+ $\sqrt{}$(0.25-K/0.9)*d = **192** mm

Therefore, $A_s$ (Required) = M / (0.87 * $f_y$ * z) = **1359** mm$^2$       < $A_s$ (provided) = **1963** mm$^2$     **O.K.**

**(ii) Design of Shear Reinforcement (Longitudinal Span)**

Design shear V = u' * L * $S_h$ / 2 = **154.39** kN

Design shear stress, $\nu$ = V/bd = **1.14** N/mm$^2$

According to Table 6.3 in Code of Practice of Structural use of Concrete 2013

Design concrete shear strength ($v_c$)    $\nu_c$ = $\dfrac{0.79 * (100A_s/bd)^{1/3} * (400/d)^{1/4} * (f_{cu}/25)^{1/3}}{\gamma_m}$       where $\gamma_m$ = **1.25**

= **0.878** N/mm$^2$

Detail of Shear Link:

No. of steel bar = **2**

Diameter of steel bar = **12** mm

Spacing of links along the member = **200** mm

Area of steel, $A_{sv}$ = **226** mm$^2$

According to Table 6.2 in Code of Practice of Structural use of Concrete 2013

0.5$v_c$<v<$v_c$+0.4= **0.44**     <     **1.14**     <     **1.28**

Area of shear reinforcement, $A_{sv} \geq$ (0.4*$b_v$*$S_v$) / 0.87$f_{yv}$ = **220.69** mm$^2$       **O.K.**       where $f_{yv}$ = **250** N/mm$^2$

**(iii) Checking on Axial Tensile Force (Longitudinal Span)**

Axial tensile force generated = **0.00** kN

Design bending moment = **104.37** kNm

According to Figure 6.1 and Cl 6.1.2.4(a) in Code of Practice of Structural use of Concrete 2013

The axial tensile force generated with respect to the bending moment are within the allowable capacity of the beam as shown in the chart below



Axial Force - Bending Moment Interaction Diagram of Grillage Beam

Section Properties of Concrete Grillage Beam
**(b) Determine Design Bending Moment and Shear in the Grillage System (Transverse Span)**

<div style="text-align:center">

**Plan View**         **Soil nail**

**Row 5**

**Row 6**

**Row 7**

</div>
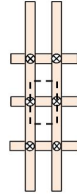
Section Properties of Concrete Grillage Beam
Proposed reinforcement

No. of steel bar = **4**
Diameter of steel bar = **20** mm

Area of steel, $A_s$ = **1257** mm$^2$
Dia. of stirrup, $\phi_s$ = **12** mm
Effective depth, d = **228** mm

**(i) Design of Bending Reinforcement (Transverse Span)**

Since **Row 6** requires the greatest nail force, use resultant pressure at **Row 6** to determine the design bending moment.  Hence resultant pressure,
Design Moment  $M = u'' * L * S_h^2 / 8$

$u''$ = **69.60** kPa
$M$ = **52.93** kNm

Minimum reinforcement = 0.13%bh = **234** mm$^2$     **O.K.**

According to Cl 6.1.2.4 (c) in Code of Practice of Structural Use of Concrete 2013
$K = M / (bd^2 * f_{cu}) =$ **0.06**   **O.K. (i.e. <0.156 - No compression steel is required)**
Lever arm, $z = (0.5+ \sqrt{(0.25-K/0.9)}*d =$ **213** mm

Therefore, $A_s$ (Required) $= M / (0.87 * f_y * z) =$   **622**  mm$^2$   $< A_s$ (provided) =    **1257** mm$^2$   **O.K.**

**(ii) Design of Shear Reinforcement (Transverse Span)**

Design shear $V = u' * L * S_h / 2 =$   **141.16** kN

Design shear stress, $\nu$ = V/bd =   **1.03** N/mm$^2$

According to Table 6.3 in Code of Practice of Structural use of Concrete 2013
Design concrete shear strength, $v_c = \dfrac{0.79 * (100A_s/bd)^{1/3} * (400/d)^{1/4} * (f_{cu}/25)^{1/3}}{\gamma_m}$    where $\gamma_m$ = **1.25**

= **0.75** N/mm$^2$

Detail of Shear Link:
No. of steel bar = **2**
Diameter of steel bar = **12** mm
Spacing of links along the member = **200** mm
Area of steel, $A_{sv}$ = **226** mm$^2$

According to Table 6.2 in Code of Practice of Structural use of Concrete 2013
$0.5v_c<v<v_c+0.4 =$   **0.38**  **<**  **1.03**  **<**  **1.15**

Area of shear reinforcement, $A_{sv} \geq (0.4*b_v*S_v) / 0.87f_{yv} =$   **220.69** mm$^2$   **O.K.**   where $f_{yv}$ = **250** N/mm$^2$

## A.2.5 Structural Design Calculations of Soil Nail Heads

Section Properties of Concrete Grillage Beam
**(c) Design of Connection between Soil Nail & Slab**

**(i) Bearing Failure of the Base Plate**

According to Cl 6.1.5.7 (b) in Code of Practice of Structural use of Concrete 2013

| | | |
|---|---|---|
| Maximum design shear stress, $v_{max}$ = | $V/u_o d$ | |
| Maximum soil nail load, V = | **282.31** kN | |
| Width of base plate, $b_1$ = | **150** mm | |
| Perimeter of base plate, $u_o$ = 4 * 150mm = | **600** mm | |
| Depth below steel plate, $d_1$ = | **200** mm | |
| Concrete cover, C= | **50** mm | |
| Bar reinforcement, $\phi$ = | **25** mm | |
| Effective depth of nail head, $d_e$ = | **113** mm | |

$v_{max}$ = **4.18** N/mm²    <0.8*√ $f_{cu}$,    **4.38** N/mm²    **O.K.**

**(ii) Punching Failure at the Nail Head**

U6

Ub     Ua

U5

0.60 m

1.05 m   0.60 m

2.70 m

**Plan View**

**Soil nail**

**Row 5**

**Row 6**

**Row 7**

Soil Nail Head
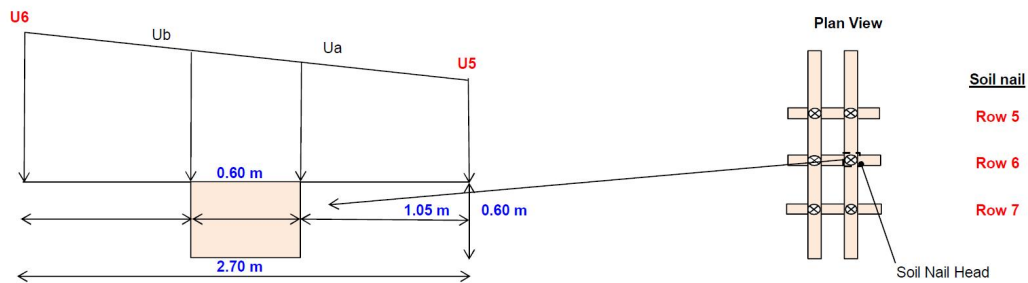
Proposed reinforcement
No. of steel bar =     **3**
Diameter of steel bar =     **25** mm

Area of steel, $A_s$ =     **1473** mm²
Width of nail head, b =     **600** mm
Effective depth of nail head, $d_e$ =     **113** mm

According to Cl 6.1.5.7 (c) in Code of Practice of Structural use of Concrete 2013

| | |
|---|---|
| Punching force, $F_{nail}$ = | **282.31** kN |
| $U_a$= | **68.15** kPa |
| $U_b$= | **71.05** kPa |
| Force from soil, $F_{soil}$ = $(U_a+U_b)$ / 2 * Area of soil nail head | |
| = | **25.06** kN |
| Hence, $F_{resultant}$ = $F_{nail}$ - $F_{soil}$ | |
| = | **257.26** kN |
| Outer perimeter 1.5d from loaded area, u = | **1950** mm |
| Hence, v = V/ud = | **1.17** N/mm² |

According to Table 6.3 & Cl 6.1.5.7 (d) in Code of Practice of Structural use of Concrete 2013

Design concrete shear strength, $v_c = \dfrac{0.79 * (100A_s/bd)^{1/3} * (400/d)^{1/4} * (f_{cu}/25)^{1/3}}{\gamma_m}$    where $\gamma_m$ =    **1.25**

= **1.20** N/mm²    > v =    **1.17** N/mm²    **O.K.**

**(d) Reinforcement Design of Toe Embedment**

Structural Forces Induced in Toe Embedment
Design Shear Force     **128.3** kN/m     Taken from Numerical
Design Bending Moment     **12.2** kNm/m     Analysis

Note:
Structural design of the toe embedment can be carried out according to the guidelines in
"Code of Practice for Structural Use of Concrete 2013"

## A.2.6   Numerical Analysis for Slope Performance Assessment

### A.2.6.1   Model Geometry
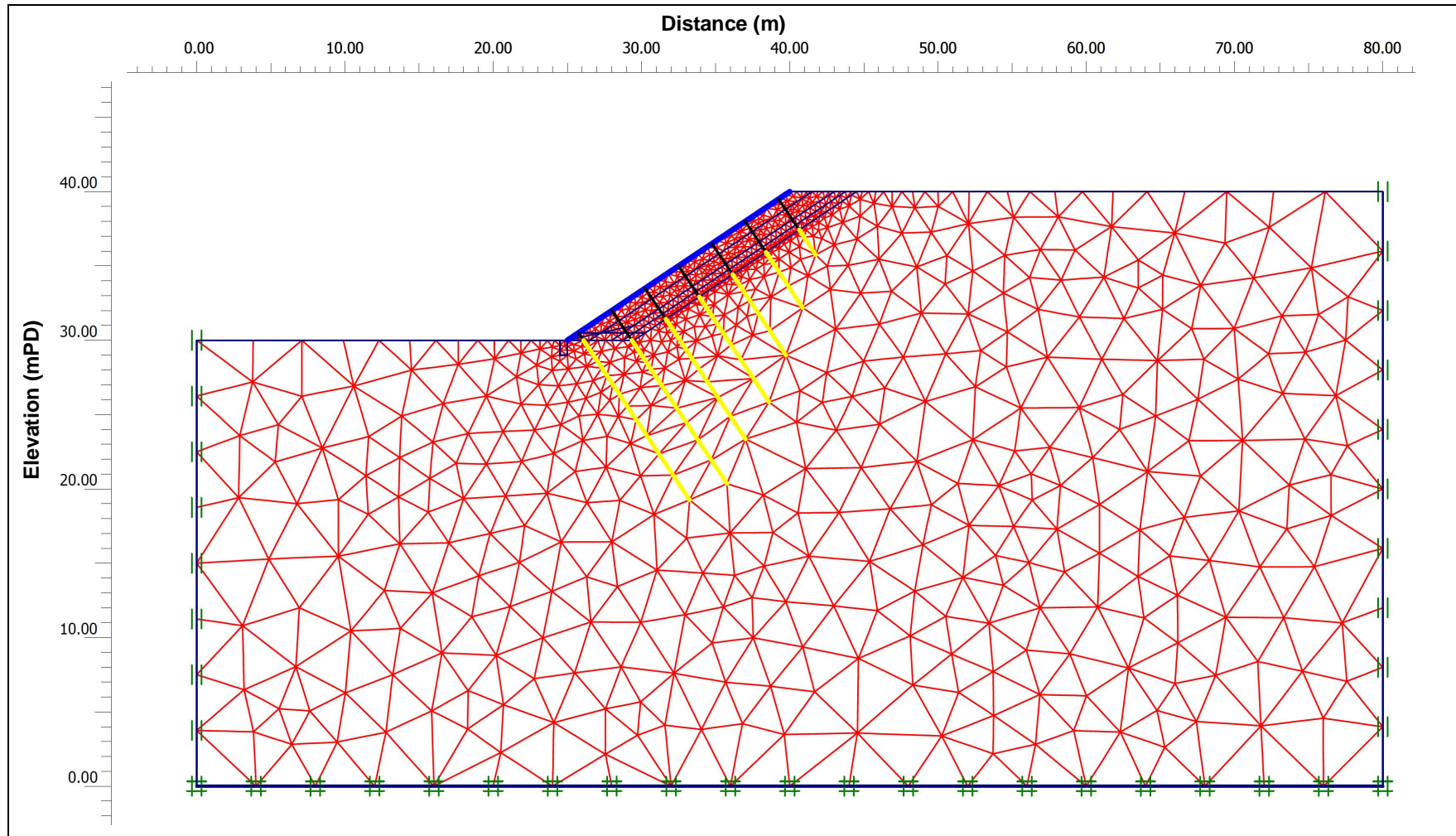
## A.2.6.2  Model Parameters (Soil)

| ID | Name | Type | $\gamma_{sat}$ [kN/m$^3$] | $\nu$ [ - ] | $E_{ref}$ [kN/m$^2$] | $c_{ref}$ [kN/m$^2$] | $\varphi$ [ ° ] | $\psi$ [ ° ] |
|---|---|---|---|---|---|---|---|---|
| 1 | CDG | Drained | 17.7 | 0.30 | 25000.0 | 5.0 | 35.0 | 0.0 |
| 2 | Loose Fill (Before Liquefaction) | Drained | 17.7 | 0.30 | 5000.0 | 5.0 | 35.0 | 0.0 |
| 3 | Liquefied Fill 1m | UnDrained | 17.7 | 0.30 | 5000.0 | 3.0 | 0.0 | 0.0 |
| 4 | Liquefied Fill 2m | UnDrained | 17.7 | 0.30 | 5000.0 | 4.8 | 0.0 | 0.0 |
| 6 | Liquefied Fill (bottom seam) | UnDrained | 17.7 | 0.30 | 5000.0 | 8.3 | 0.0 | 0.0 |
| 7 | Non-liquefied Fill | Drained | 17.7 | 0.30 | 5000.0 | 0.1 | 28.0 | 0.0 |

Glossary of Symbols:

$\gamma_{sat}$:  Bulk Unit Weight – Saturated (kN/m$^3$)

$\nu$:  Poisson's Ratio

$E_{ref}$:  Young's Modulus (kN/m$^2$)

$c_{ref}$:  Cohesion (kN/m$^2$)

$\phi$:  Friction Angle (°)

$\psi$:  Dilatancy Angle (°)

## A.2.6.3    Model Parameters (Grillage Facing)

| ID | Name | Type | EA [kN/m] | EI [kNm$^2$/m] | w [kN/m$^2$] | $\nu$ [ - ] | $M_p$ [kNm/m] | $N_p$ [kN/m] |
|---|---|---|---|---|---|---|---|---|
| 1 | Facing | Elastic | 2.664E6 | 19980.0 | 3.5 | 0.20 | 1E15 | 1E15 |

Glossary of Symbols:
EA :        Axial Stiffness (kN/m)
EI:         Flexural Rigidity (kNm$^2$/m)
w:          Weight (kN/m$^2$)
$\nu$:          Poisson's Ratio
$M_p$:         Maximum Bending Moment (kNm/m)
$N_p$:         Maximum Axial Force (kN/m)

**A.2.6.4   Model Parameters (Soil Nails)**

## Free Length of Soil Nails

| ID | Name | EA [kN/m] | $|F_{max,comp}|$ [kN/m] | $|F_{max,tens}|$ [kN/m] |
|----|------|-----------|---------------|---------------|
| 1 | T20 | 41867.0 | 6.6667E14 | 96.3 |
| 2 | T25 | 65467.0 | 6.6667E14 | 150.5 |
| 3 | T32 | 1.0733E5 | 6.6667E14 | 246.7 |
| 4 | T40 | 1.6733E5 | 6.6667E14 | 385.4 |
| 5 | T16 | 26808.0 | 6.6667E14 | 61.7 |

Glossary of Symbols:
EA :        Axial Stiffness (kN/m)
$F_{max,comp}$:   Maximum Compressive Force (kN/m)
$F_{max,tens}$:   Maximum Tensile Force (kN/m)

## Bond Length of Soil Nails

| ID | Name | Type | EA [kN/m] | $N_p$ [kN/m] |
|----|------|------|-----------|---------|
| 1 | T20 | Plastic | 4.19E04 | 96.30 |
| 2 | T25 | Plastic | 6.54E04 | 151.00 |
| 3 | T32 | Plastic | 1.07E05 | 247.00 |
| 4 | T40 | Plastic | 1.68E05 | 385.00 |
| 5 | T16 | Plastic | 2.68E04 | 61.70 |

Glossary of Symbols:
EA :        Axial Stiffness (kN/m)
$N_p$:        Maximum Axial Tension Force (kN/m)

## A.2.6.5   Model Parameters (Toe Embedment)

| ID | Name | Type | $\gamma_{sat}$ [kN/m$^3$] | $\nu$ [ - ] | $E_{ref}$ [kN/m$^2$] |
|---|---|---|---|---|---|
| 5 | Toe embedment | Non-porous | 24.0 | 0.20 | 2.2E7 |

Glossary of Symbols:

$\gamma_{sat}$:  Bulk Unit Weight – Saturated (kN/m$^3$)

$\nu$:  Poisson's Ratio

$E_{ref}$:  Young's Modulus (kN/m$^2$)

**A.2.6.6    Material Sets before Liquefaction**
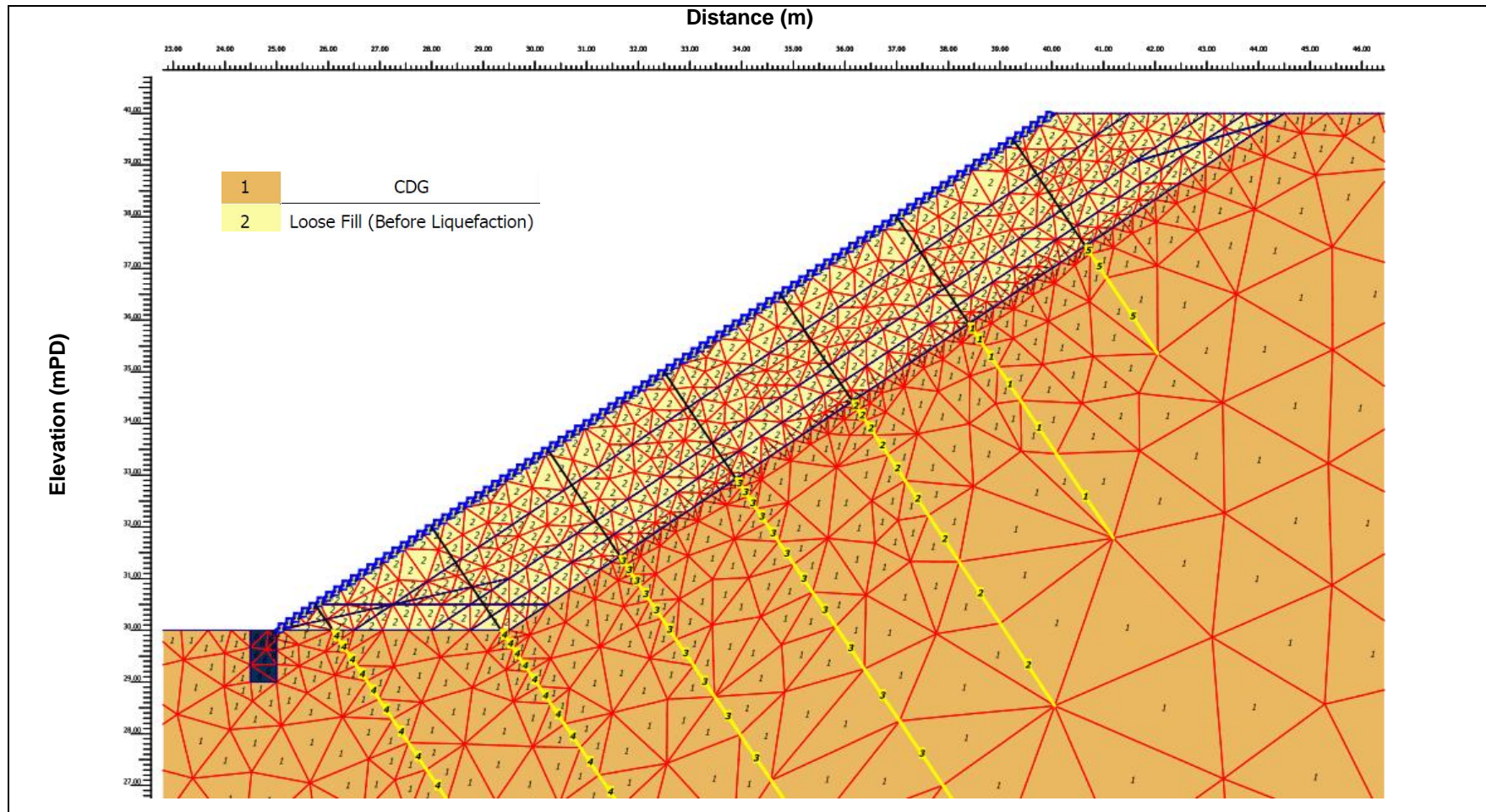
**A.2.6.7   Pore Water Pressure Distribution before Liquefaction**
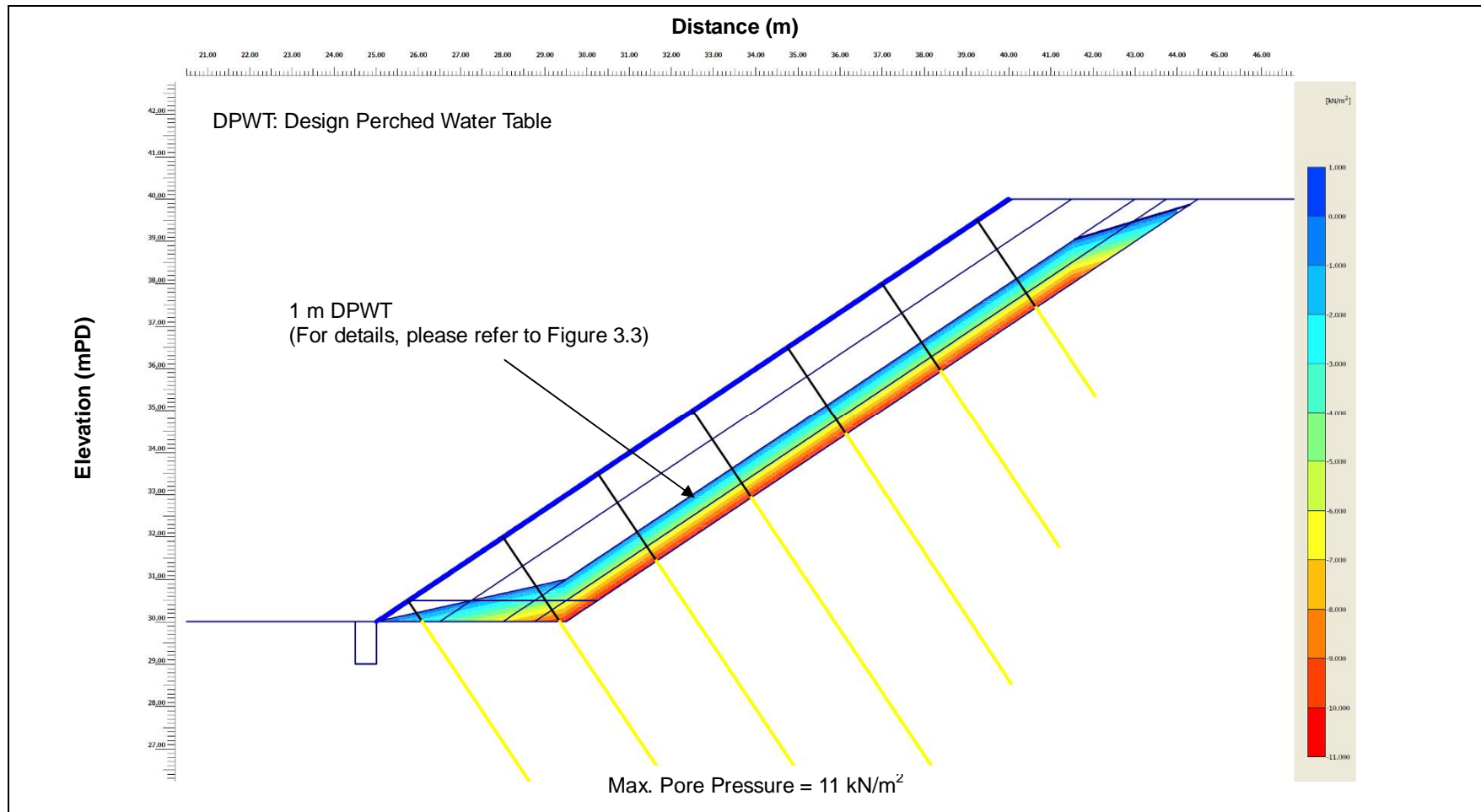
## A.2.6.8 Material Sets after Liquefaction
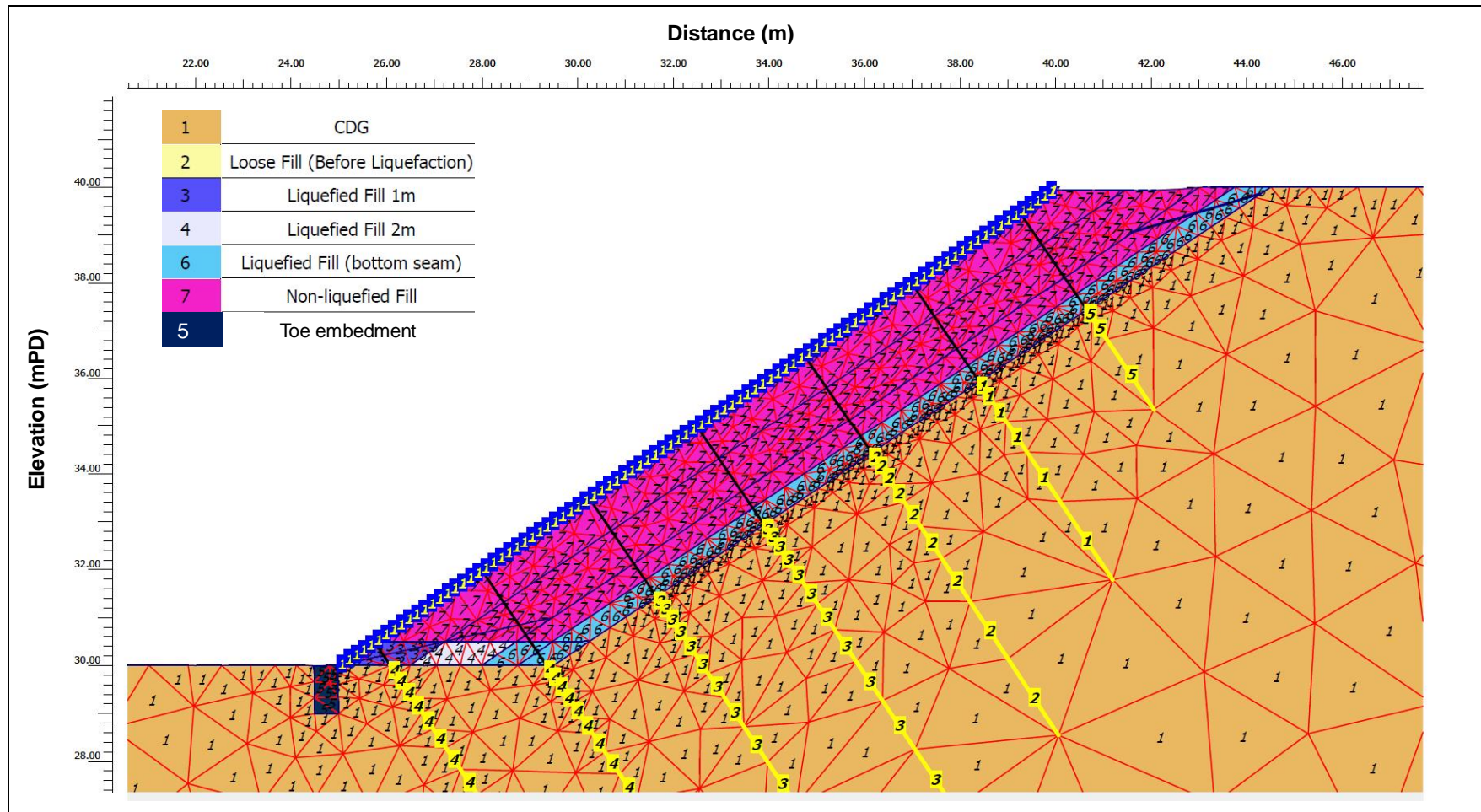
**A.2.6.9 Pore Water Pressure Distribution after Liquefaction**



Max. Excess Pore Pressure = 80 kN/m$^2$
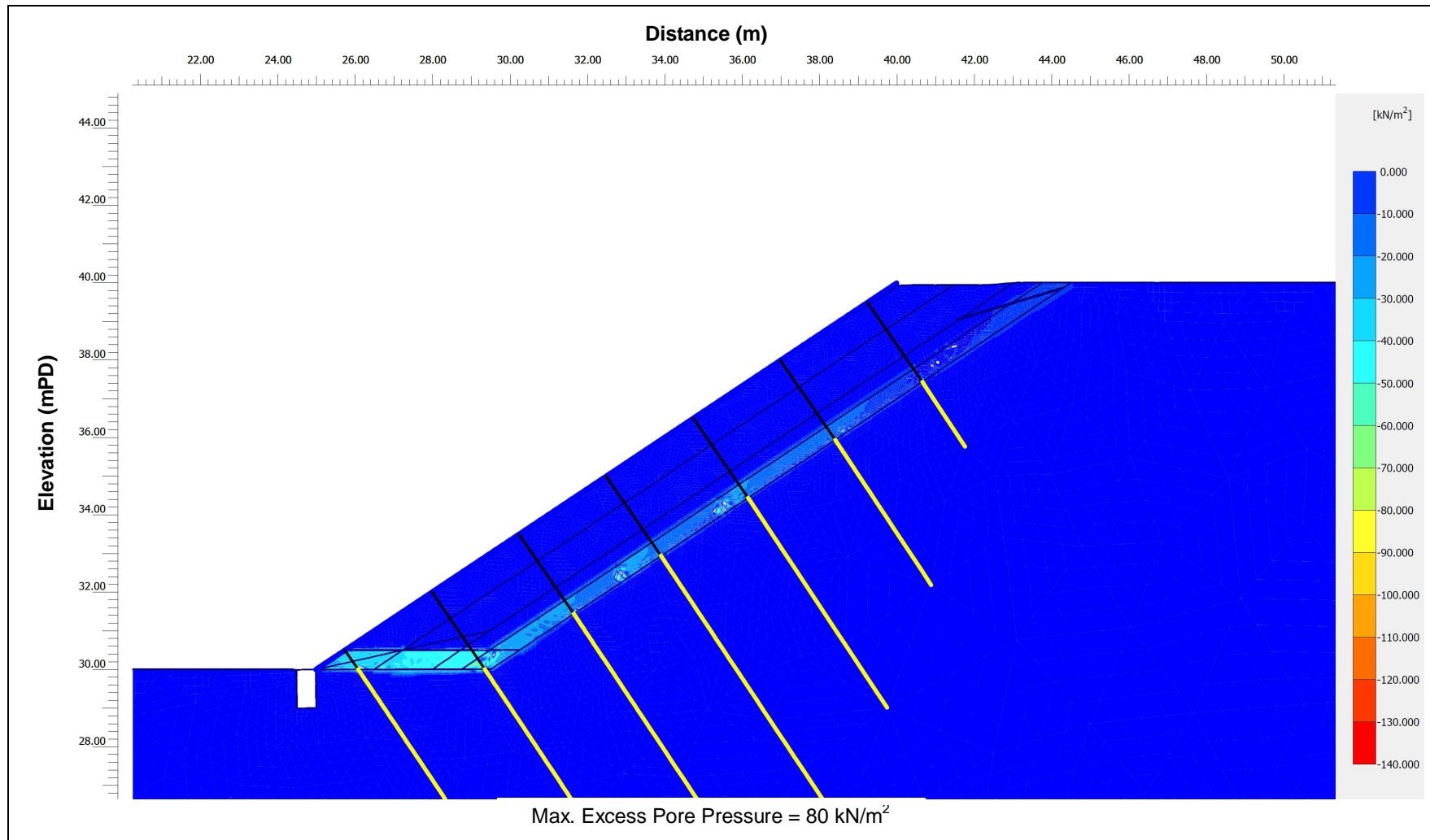
**A.2.6.10    Slope Deformation after Liquefaction**



Max. Slope Deformation = 108 mm
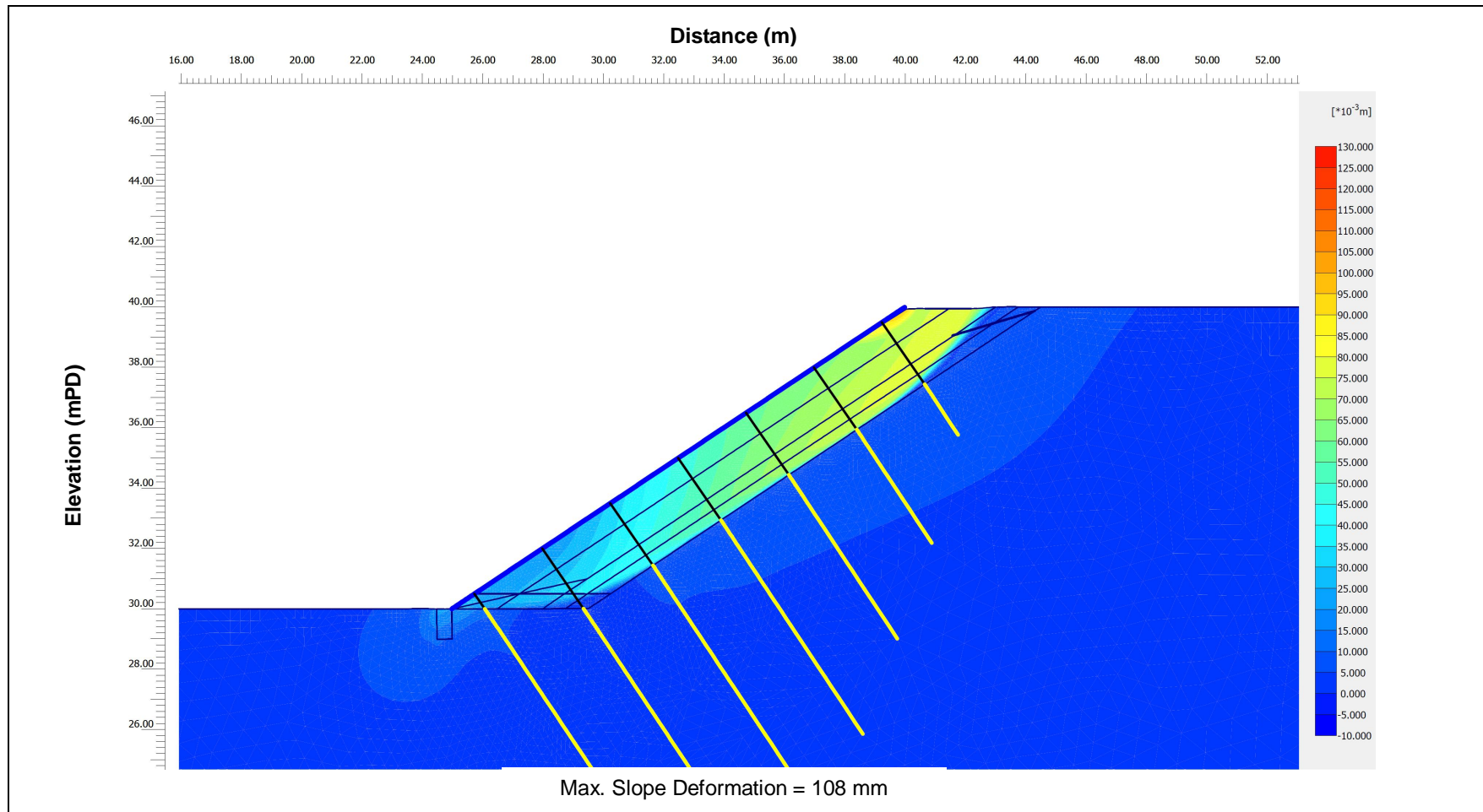
## A.3 Detailed Design Calculation of Worked Example 3

## A.3.1 Calculations of Individual Nail Forces
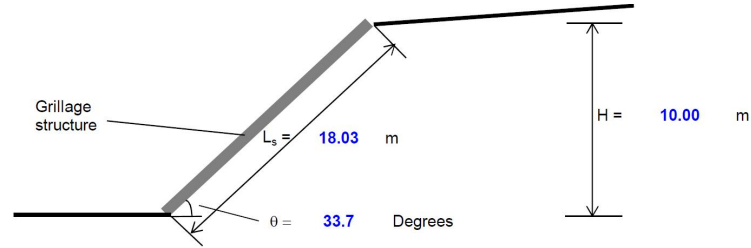


**Slope Geometry**
Slope angle = 33.7 Degrees
Fill slope height = 10 m

**Material Properties**
Fill strength: $c_{ss}/p'_{peak}$= 0.4
assumed $K_0$ = 0.5
$c_{ss}$ = 0.267 $\sigma'_v$
Note : (i) Min. FoS = 1.1 for liquefaction

**Stabilizing Pressure**
Normal stabilizing pressure u = 42.5 kPa (see SLOPE/W analysis in Figure 4.3)
Basal shear resistance $\tau$ = 3 kPa *Minimum basal shear resistance has been assumed
Basal shear force $F_b$ = 54.08 kN/m *Shear force is applied on the whole slope surface

**Soil Nails Configuration**
Horizontal spacing = 1.5 m
Vertical spacing = 1.5 m
Spacing between nails on the inclined slab = 2.70 m
Total number of rows = 7
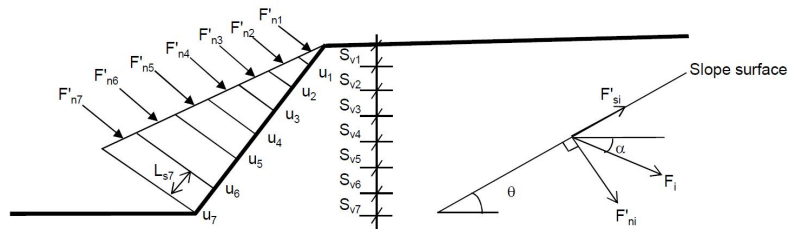
| | Vertical distance from crest (m) | $\alpha$ (°) | $S_{vi}$ (m) | $\Sigma S_{vi}$ from crest (m) | $u_i$ (kPa) | $L_{si}$ (m) | $F'_{ni}$ (kN/m) | $F_i$ (kN/m) | $F'_{si}$ (kN/m) | Individual nail force (kN) |
|---|---|---|---|---|---|---|---|---|---|---|
| Row 1 | 0.50 | 20.00 | 1.25 | 1.25 | 5.31 | 2.25 | 5.99 | 7.43 | 4.40 | 11.14 |
| Row 2 | 2.00 | 20.00 | 1.50 | 2.75 | 11.69 | 2.70 | 22.99 | 28.52 | 16.89 | 42.79 |
| Row 3 | 3.50 | 20.00 | 1.50 | 4.25 | 18.06 | 2.70 | 40.22 | 49.92 | 29.56 | 74.88 |
| Row 4 | 5.00 | 56.00 | 1.50 | 5.75 | 24.44 | 2.70 | 57.46 | 57.46 | 0.00 | 86.20 |
| Row 5 | 6.50 | 56.00 | 1.50 | 7.25 | 30.81 | 2.70 | 74.70 | 74.70 | 0.00 | 112.06 |
| Row 6 | 8.00 | 56.00 | 1.50 | 8.75 | 37.19 | 2.70 | 91.94 | 91.94 | 0.00 | 137.91 |
| Row 7 | 9.50 | 56.00 | 1.25 | 10.00 | 42.50 | 2.25 | 89.79 | 89.79 | 0.00 | 134.68 |
| | | | | | | Total= | 383.09 | 399.77 | 50.85 | 599.65 |
| | | | | | | Max. value= | | 91.94 | | 137.91 |

## A.3.2  Soil Nail Design Calculations

**Soil Parameters**

Type 1,  **Fill (liquefiable)**    $c' =$ **0.267** $\sigma'_v$    $\gamma =$ **17.7** kN/m³    $\phi' =$ **0.0** °
Type 2,  **CDG**    $c' =$ **5.0** kPa    $\gamma =$ **17.7** kN/m³    $\phi' =$ **35.0** °
Type 3,  **Fill (non-liquefiable)**    $c' =$ **0.0** kPa    $\gamma =$ **17.7** kN/m⁴    $\phi' =$ **28.0** °
Drillhole diameter (D) =    **0.15** m
Unit weight of water, $\gamma_w$ =    **9.81** kN/m³
Horizontal spacing =    **1.50** m
Sacrificial thickness =    **2.00** mm



$L_a$ : Length of soil nail in active zone
$L_{es}$ : Effective bond length
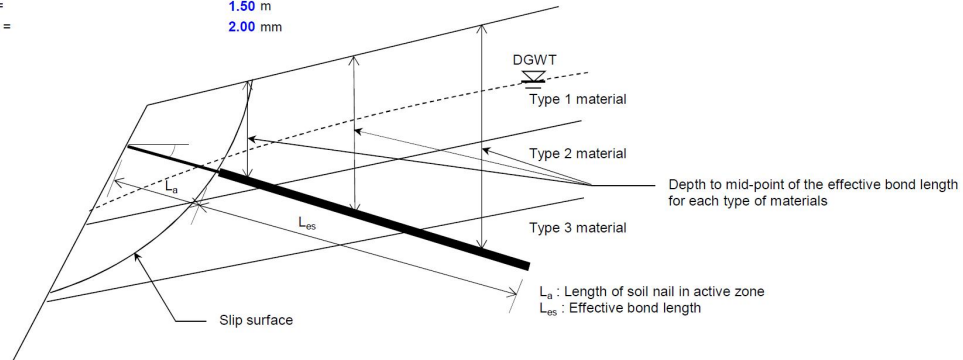
**A.  Tension Failure of the Steel Bar**

Characteristic yield strength, $f_y$ =    **460** MPa
Partial factor for tensile failure of steel bar, $F_T$ =    **1.15**

| Row No. | Level (mPD) | Bar Length (m) | Bar Size | Horizontal Spacing (m) | Force Required per unit width (kN/m) | Force Required $T_r$ (kN) | Allowable Tensile Force $T_T$ (kN) | $T_T / T_r$ |
|---|---|---|---|---|---|---|---|---|
| 1 | 39.5 | 9.6 | T16 | 1.50 | 7.43 | 11.14 | 45.24 | 4.06 |
| 2 | 38.0 | 10.9 | T16 | 1.50 | 28.52 | 42.79 | 45.24 | 1.06 |
| 3 | 36.5 | 11.1 | T20 | 1.50 | 49.92 | 74.88 | 80.42 | 1.07 |
| 4 | 35.0 | 9.4 | T25 | 1.50 | 57.46 | 86.20 | 138.54 | 1.61 |
| 5 | 33.5 | 11.1 | T25 | 1.50 | 74.70 | 112.06 | 138.54 | 1.24 |
| 6 | 32.0 | 12.7 | T25 | 1.50 | 91.94 | 137.91 | 138.54 | 1.00 |
| 7 | 30.5 | 13.0 | T32 | 1.50 | 89.79 | 134.68 | 246.30 | 1.83 |

Minimum $T_T / T_r$ =    **1.00**    $\geqq$ 1    **O.K.**

**B.  Bond Failure between Grout and Steel Reinforcement**

Design cube strength of cement grout at 28 days $f_{cu}$ =                                    **30** MPa
For type 2 deformed bar, $\beta$ =                                                                          **0.5**
FOS against pullout failure at grout-reinforcement interface ($F_{GR}$) =            **2.0**

Allowable bond stress = Ultimate bond stress / $F_{GR}$ = [ $\beta \, (f_{cu})^{1/2}$ ] / $F_{GR}$ =        2738.62/2 =   **1369.31** kN/m$^2$

Allowable pullout resistance along grout-reinforcement interface = [ $\beta \, (f_{cu})^{1/2}$ ] * $\pi$ * (d-2s) * $L_{eg}$ / $F_{GR}$

**Resisting Zone - for soil nail design**

| Row No. | Resisting Zone | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Effective Bond Length in Each Layer (m) | | | Depth to Mid-point of the Effective Bond Length (m) | | | | | | | |
| | | | | Fill (liquefiable) | | Fill (non-liquefiable) | | | CDG | | |
| | Fill (non-liquefiable) | Fill (liquefiable) | CDG | Fill (liquefiable) | WATER | Fill (non-liquefiable) | WATER | Fill (non-liquefiable) | Fill (liquefiable) | CDG | WATER |
| 1 | 0.00 | 0.00 | 1.50 | 0.00 | 0.00 | 0.00 | 0.00 | 1.50 | 1.20 | 0.30 | 0.00 |
| 2 | 0.00 | 0.00 | 3.60 | 0.00 | 0.00 | 0.00 | 0.00 | 1.50 | 2.20 | 1.20 | 0.00 |
| 3 | 0.00 | 0.00 | 4.60 | 0.00 | 0.00 | 0.00 | 0.00 | 1.50 | 3.20 | 1.40 | 0.00 |
| 4 | 0.00 | 0.00 | 5.70 | 0.00 | 0.00 | 0.00 | 0.00 | 1.50 | 3.20 | 3.00 | 3.60 |
| 5 | 0.00 | 0.00 | 8.00 | 0.00 | 0.00 | 0.00 | 0.00 | 1.50 | 2.50 | 4.20 | 5.40 |
| 6 | 0.00 | 0.00 | 10.30 | 0.00 | 0.00 | 0.00 | 0.00 | 1.50 | 2.10 | 5.30 | 7.00 |
| 7 | 0.00 | 0.00 | 12.40 | 0.00 | 0.00 | 0.00 | 0.00 | 1.60 | 1.30 | 5.00 | 7.30 |

| Row No. | Level (mPD) | Bar Length (m) | Bar Size | Horizontal Spacing (m) | $L_{eg}$ (m) | $L_a$ (m) | Force Required per unit width (kN/m) | Force Required $T_r$ (kN) | Allowable Pullout Resistance $T_{GR}$ (kN) | $T_{GR}/T_r$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 39.5 | 9.6 | T16 | 1.5 | 1.50 | 8.10 | 7.43 | 11.14 | 77.43 | 6.95 |
| 2 | 38.0 | 10.9 | T16 | 1.5 | 3.60 | 7.30 | 28.52 | 42.79 | 185.84 | 4.34 |
| 3 | 36.5 | 11.1 | T20 | 1.5 | 4.60 | 6.50 | 49.92 | 74.88 | 316.61 | 4.23 |
| 4 | 35.0 | 9.4 | T25 | 1.5 | 5.70 | 3.70 | 57.46 | 86.20 | 514.93 | 5.97 |
| 5 | 33.5 | 11.1 | T25 | 1.5 | 8.00 | 3.10 | 74.70 | 112.06 | 722.70 | 6.45 |
| 6 | 32.0 | 12.7 | T25 | 1.5 | 10.30 | 2.40 | 91.94 | 137.91 | 930.48 | 6.75 |
| 7 | 30.5 | 13.0 | T32 | 1.5 | 12.40 | 0.60 | 89.79 | 134.68 | 1493.59 | 11.09 |

Minimum of $T_{GR}$ / $T_r$ =     **4.23**       > 1          **O.K.**

**C.** **Shear Failure of Adjacent Ground**

Allowable pullout resistance provided by the soil-grout bond length in the passive zone: $T_{SG} = (\pi\, D\, c' + 2\, D\, \sigma_v'\tan\phi) * L_{es}/F_{SG}$

The rock-grout bond strength for weathred rock mass of PW90/100 or better= **0.35** MPa
FOS against pullout failure at soil-grout interface ($F_{SG}$) = **1.5**

| Fill (liquefiable) | = | ( | 0.126 | + | 0.000 $\sigma_v'$ ) * | $L_{es}/F_{SG}$ |

| CDG | = | ( | 2.356 | + | 0.210 $\sigma_v'$ ) * | $L_{es}/F_{SG}$ |

**Soil Nail Schedule Table**

| Row No. | Level (mPD) | Bar Length (m) | Bar Size | Horizontal Spacing (m) | $L_{es}$ (m) | $L_a$ (m) | Force Required per unit width (kN/m) | Force Required $T_r$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | 39.50 | 9.60 | T16 | 1.5 | 1.50 | 8.10 | 7.43 | 11.14 |
| 2 | 38.00 | 10.90 | T16 | 1.5 | 3.60 | 7.30 | 28.52 | 42.79 |
| 3 | 36.50 | 11.10 | T20 | 1.5 | 4.60 | 6.50 | 49.92 | 74.88 |
| 4 | 35.00 | 9.40 | T25 | 1.5 | 5.70 | 3.70 | 57.46 | 86.20 |
| 5 | 33.50 | 11.10 | T25 | 1.5 | 8.00 | 3.10 | 74.70 | 112.06 |
| 6 | 32.00 | 12.70 | T25 | 1.5 | 10.30 | 2.40 | 91.94 | 137.91 |
| 7 | 30.50 | 13.00 | T32 | 1.5 | 12.40 | 0.60 | 89.79 | 134.68 |

| Row No. | Effective Vertical Stress $\sigma_v'$ (kPa) | | Allowable Pullout Resistance in Soil $T_{SG\_soil}$ (kN) | | Embedded Rock Length | Allowable Pullout Resistance in Rock | Allowable Pullout Resistance | |
|---|---|---|---|---|---|---|---|---|
| | Fill (liquefiable & non-liquefiable) | CDG | Fill (liquefiable & non-liquefiable) | CDG | L (m) | $T_{SG\_rock}$ (kN) | $T_{SG}$ (kN) | $T_{SG}/T_r$ |
| 1 | N/A | 53.10 | 0.00 | 13.51 | 0.00 | 0.00 | 13.51 | 1.21 |
| 2 | N/A | 86.73 | 0.00 | 49.38 | 0.00 | 0.00 | 49.38 | 1.15 |
| 3 | N/A | 107.97 | 0.00 | 76.78 | 0.00 | 0.00 | 76.78 | 1.03 |
| 4 | N/A | 100.97 | 0.00 | 89.55 | 0.00 | 0.00 | 89.55 | 1.04 |
| 5 | N/A | 92.17 | 0.00 | 115.82 | 0.00 | 0.00 | 115.82 | 1.03 |
| 6 | N/A | 88.86 | 0.00 | 144.35 | 0.00 | 0.00 | 144.35 | 1.05 |
| 7 | N/A | 68.22 | 0.00 | 137.94 | 0.00 | 0.00 | 137.94 | 1.02 |

Minimum of $T_{SG}/T_r$ = **1.02** > 1 **O.K.**

## A.3.3  Percentage Coverage of Grillage Facing

Width of grillage beam =        **600**      mm
Thickness of grillage beam =    **300**      mm
Embedment of grillage beam =    **300**      mm

**0.30** m

**2.70**    m

**0.30** m          **1.50**      m
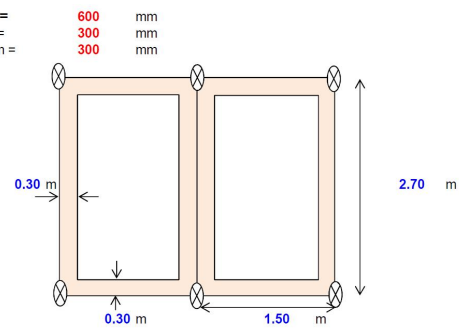
% Coverage of Slope Surface =        **53%**          **O.K. (i.e. % Coverage >50%)**

## A.3.4 Structural Design Calculations of Grillage Facing

Section Properties of Concrete Grillage Beam
Width of grillage beam, b=     600 mm
Depth of grillage beam, h=     300 mm

Minimum cover of grillage beam, C=     50 mm
Concrete strength, $f_{cu}$=     30 N/mm$^2$

Characteristic strength of steel bar, $f_y$=     460 N/mm$^2$

**Plan View**

Soil nail

Row 5

Row 6

Row 7

Proposed reinforcement
No. of steel bar =     4
Diameter of steel bar =     16 mm
Area of steel, $A_s$ =     804 mm$^2$
Diameter of stirrup, $\phi_s$ =     12 mm
Effective depth, d =     230 mm

**(a) Determine Design Bending Moment and Shear in the Grillage System (Longitudinal Span)**
**(i) Design of Bending Reinforcement (Longitudinal Span)**

Design moment $M = u' * L^2 * Sh / 8$

u' =     37.19 kPa
M =     50.99 kNm

Minimum reinforcement = 0.13%bh =     234 mm$^2$     **O.K.**

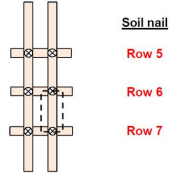According to Cl. 6.1.2.4 (c) in Code of Practice of Structural Use of Concrete 2013
K =   $M / (bd^2 * f_{cu})$ =     0.05 **O.K. (i.e. <0.156 - No compression steel is required)**
Lever arm, z =   $(0.5+ \sqrt{(0.25-K/0.9)}*d$ =     215 mm

Therefore, $A_s$ (Required) = $M / (0.87 * f_y * z)$ =     592 mm$^2$   < $A_s$ (provided) =     804 mm$^2$     **O.K.**

**(ii) Design of Shear Reinforcement (Longitudinal Span)**

Design shear $V = u' * L * Sh / 2$ =     75.42 kN

Design shear stress, $\nu$ = V/bd =     0.55 N/mm$^2$

According to Table 6.3 in Code of Practice of Structural use of Concrete 2013
Design concrete shear strength ($v_c$)     $\nu_c = \dfrac{0.79 * (100A_s/bd)^{1/3} * (400/d)^{1/4} * (f_{cu}/25)^{1/3}}{\gamma_m}$     where $\gamma_m$ = 1.25

=     0.64 N/mm$^2$

Detail of Shear Link:
No. of steel bar =     2
Diameter of steel bar =     12 mm
Spacing of links along the member =     200 mm
Area of steel, $A_{sv}$ =     226 mm$^2$

According to Table 6.2 in Code of Practice of Structural use of Concrete 2013
$0.5v_c<v<v_c+0.4$     0.32    <    0.55    <    1.04

Area of shear reinforcement, $A_{sv} \geq (0.4*b_v*S_v) / 0.87f_{yv}$ =     220.69 mm$^2$     **O.K.**     where $f_{yv}$ = 250 N/mm$^2$

**(iii) Checking on Axial Tensile Force (Longitudinal Span)**

Axial tensile force generated =     50.85 kN     **in tension**
Design bending moment =     50.99 kNm

According to Figure 6.1 and Cl 6.1.2.4(a) in Code of Practice of Structural use of Concrete 2013
The axial tensile force generated with respect to the bending moment are within the allowable capacity of the beam as shown in the chart below

Axial Force - Bending Moment Interaction Diagram of Grillage Beam

Axial Force, N (kN)

Compression

Tension

(50.99,-50.85)

Moment, Mxx (kNm)

Section Properties of Concrete Grillage Beam
**(b) Determine Design Bending Moment and Shear in the Grillage System (Transverse Span)**

<div align="center">

**Plan View**          **Soil nail**

**Row 5**

**Row 6**

**Row 7**

</div>
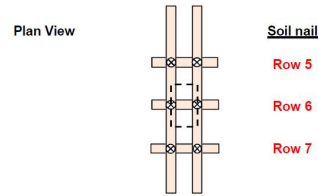
Section Properties of Concrete Grillage Beam
Proposed reinforcement
No. of steel bar =                                    **2**
Diameter of steel bar =                              **16** mm

Area of steel, $A_s$ =                               **402** mm$^2$
Dia. of stirrup, $\phi_s$ =                          **12** mm
Effective depth, d =                                 **230** mm

**(i) Design of Bending Reinforcement (Transverse Span)**

Since **Row 6** requires the greatest nail force, use resultant pressure at **Row 6** to determine the design bending moment.  Hence resultant pressure,
Design Moment  M = u' * L * $S_h^2$ / 8
                              u" =          **34.00** kPa
                              M =           **25.86** kNm

Minimum reinforcement = 0.13%bh =                          **234** mm$^2$          <span style="background:yellow">**O.K.**</span>
<span style="background:cyan">According to Cl 6.1.2.4 (c) in Code of Practice of Structural Use of Concrete 2013</span>
                    K =       M / (bd$^2$ * $f_{cu}$) =          **0.03**     <span style="background:yellow">**O.K. (i.e. <0.156 - No compression steel is required)**</span>
              Lever arm, z = (0.5+ $\sqrt{}$(0.25-K/0.9)*d =          **223** mm

Therefore, $A_s$ (Required) = M / (0.87 * fy * z) =          **290**     mm$^2$      < $A_s$ (provided) =                    **402** mm$^2$     <span style="background:yellow">**O.K.**</span>

**(ii) Design of Shear Reinforcement (Transverse Span)**

Design shear V = u' * L * $S_h$ / 2 =                          **68.96** kN

Design shear stress, $\nu$ = V/bd =                          **0.50** N/mm$^2$

<span style="background:cyan">According to Table 6.3 in Code of Practice of Structural use of Concrete 2013</span>
                    Design concrete shear strength, $v_c$ = $\frac{0.79 * (100A_s/bd)^{1/3} * (400/d)^{1/4} * (f_{cu}/25)^{1/3}}{\gamma_m}$                    where $\gamma_m$ = **1.25**

                                  =     **0.51** N/mm$^2$
Detail of Shear Link:
No. of steel bar =                          **2**
Diameter of steel bar =                     **12** mm
Spacing of links along the member =         **200** mm
Area of steel, $A_{sv}$ =                   **226** mm$^2$

<span style="background:cyan">According to Table 6.2 in Code of Practice of Structural use of Concrete 2013</span>
                    0.5$v_c$<v<$v_c$+0.4=          **0.256**          <          **0.500**          <          **0.911**

Area of shear reinforcement, $A_{sv} \geq$ (0.4*$b_v$*$S_v$) / 0.87$f_{yv}$ =          **220.69** mm$^2$          <span style="background:yellow">**O.K.**</span>                    where $f_{yv}$ = **250** N/mm$^2$

## A.3.5 Structural Design Calculations of Soil Nail Heads
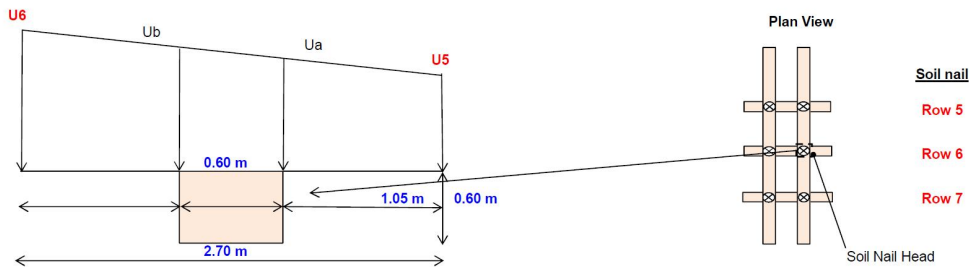
Section Properties of Concrete Grillage Beam
**(c) Design of Connection between Soil Nail & Slab**

**(i) Bearing Failure of the Base Plate**

According to Cl 6.1.5.7 (b) in Code of Practice of Structural use of Concrete 2013

| | | |
|---|---|---|
| Maximum design shear stress, $v_{max}$ = | $V/u_o d$ | |
| Maximum soil nail load, V = | **137.91** kN | |
| Width of base plate, $b_1$ = | **150** mm | |
| Perimeter of base plate, $u_o$ = 4 * 150mm = | **600** mm | |
| Depth below steel plate, $d_1$ = | **180** mm | |
| Concrete cover, C= | **50** mm | |
| Bar reinforcement, $\phi$ = | **16** mm | |
| Effective depth of nail head, $d_e$ = | **106** mm | |

$v_{max}$ = **2.17** N/mm$^2$  $<0.8*\sqrt{f_{cu}}$, **4.38** N/mm$^2$  **O.K.**

**(i) Punching Failure at the Nail Head**



Plan View

Soil nail

Row 5

Row 6

Row 7

Soil Nail Head

Proposed reinforcement
No. of steel bar = **3**
Diameter of steel bar = **16** mm

Area of steel, $A_s$ = **603** mm$^2$
Width of nail head, b = **600** mm
Effective depth of nail head, $d_e$ = **106** mm

According to Cl 6.1.5.7 (c) in Code of Practice of Structural use of Concrete 2013

| | |
|---|---|
| Punching force, $F_{nail}$ = | **137.91** kN |
| $U_a$= | **33.29** kPa |
| $U_b$= | **34.71** kPa |
| Force from soil, $F_{soil}$ = $(U_a+U_b)$ / 2 * Area of soil nail head | |
| = | **12.24** kN |
| Hence, $F_{resultant}$ = $F_{nail}$ - $F_{soil}$ | |
| = | **125.67** kN |
| Outer perimeter 1.5d from loaded area, u = | **1872** mm |
| Hence, v = V/ud = | **0.63** N/mm$^2$ |

According to Table 6.3 & Cl 6.1.5.7 (d) in Code of Practice of Structural use of Concrete 2013

Design concrete shear strength, $v_c = \dfrac{0.79 * (100A_s/bd)^{1/3} * (400/d)^{1/4} * (f_{cu}/25)^{1/3}}{\gamma_m}$  where $\gamma_m$ = **1.25**

= **0.92** N/mm$^2$  > v = **0.63** N/mm$^2$  **O.K.**

## A.3.6  Force Equilibrium Assessment of Grillage Facing
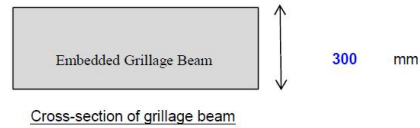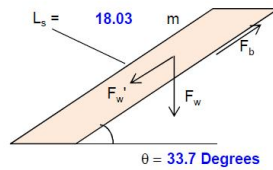


$L_s = $ **18.03**  m

$F_b$

$F_w'$   $F_w$

$\theta = $ **33.7 Degrees**

Embedded Grillage Beam    **300**  mm

Cross-section of grillage beam

Driving Force
Weight of grillage structure, $F_g = $   **69.20**   kN/m
Weight of backfill inside the openning, $F_s = $   **44.69**   kN/m
Total Weight of grillage structure and backfill, $F_w = $   **113.89**   kN/m
Force component along potential sliding direction, $F'_w = $   **63.18**   kN/m

Resisting Force
Available resisting force along the sliding direction from sub-horizontal nails, $\Sigma F'_{si} = $   **50.85**   kN/m
Resisting force due to liquefied loose fill, $F_b = $   **54.08**   kN/m
Therefore, $\Sigma F'_{si} + F'_b = $   **104.93**   kN/m   $/ F'_w = $   **63.18**   kN/m   $= $   1.66   O.K. (>1)

Note: The evaluation of force equilibrium along the potential sliding direction ensures that there is sufficient resisting force to avoid excessive slope deformation due to the weight of the grillage structure when the underlying loose fill liquefies.

[Refer to GEO-HKIE (2011) - Section 5.4]

Appendix B

Responses to Comments on Draft Report issued in April 2013

**Design Illustrations on the Use of Soil Nails to Upgrade Loose Fill Slopes**
**Responses to Comments**

| Comments | Responses |
|---|---|
| GEO/CEDD | |
| 1. Section 1.2(c) (P.10)<br>According to Section 5.4 of GEO-HKIE Report, the sub-horizontal nails are provided to counter sliding failure due to interface liquefaction. The sentence "The number of sub-horizontal nails is considered appropriate if it is sufficient to support the weight of the grillage upon fill liquefaction" is misleading. | 1. According to Section 5.4 of GEO-HKIE (2011), the ratio of sub-horizontal nails to steeply inclined nails can be estimated by considering force equilibrium of the grillage facing. The number of sub-horizontal nails should be approximately 40% to 50% of the total number of soil nails to ensure sufficient sub-horizontal nails are present to counter sliding failure.<br><br>To avoid misunderstanding, the sentence "The number of sub-horizontal nails is considered appropriate if it is sufficient to support the weight of the grillage upon fill liquefaction" has been replaced by the above sentences. |
| 2. Section 3.6 (P.23)<br>The design calculation for the embedded concrete footing including the required reinforcement should be given in worked example 2. | 2. The purpose of these worked examples is to illustrate the new design recommendations highlighted in GEO-HKIE (2011). The structural design of the embedded footing can be carried out according to the guidelines in "Code of Practice for Structural Use of Concrete 2004 (Second Edition)" as for typical reinforced concrete structures. This is explained in Section 3.6 as follows:<br><br>"The numerical analysis can also provide the shear force and bending moment induced in the concrete footing for structural design which can be carried out in accordance with the Code of Practice for Structural Use of Concrete (BD, 2004). The design structural forces for worked example 2 are shown in Appendix A.2.5." |

| Comments | Responses |
|---|---|
| 3. Section 3.7 (P.23-P.29) <br> Only interface liquefaction scenario has been considered in worked example 2. In fact, both full liquefaction and interface liquefaction failure modes should be checked in order to demonstrate the robustness of the proposed nail arrangement. The mobilized nail forces upon full liquefaction should also be plotted in Figure 3.9 for comparison. | 3. Worked example 2 is based on the same slope geometry and nail orientation adopted in Section 4.2.1 of GEO-HKIE (2011). As discussed in Section 4.2.1 of GEO-HKIE (2011), larger structural movement may be incurred under interface liquefaction compared to full liquefaction if steeply inclined nails are adopted only. Therefore, the interface liquefaction scenario was considered more critical as far as slope deformation is concerned for worked example 2. This explanation has added to Section 3.7. |
| 4. In Figures 2.3 & 3.2, there are failure slips (no. 4 of each figure) across the CDG stratum and the "upgraded" FOS of these slips are indicated (i.e. FOS of 4.889 and 6.963 in Figures 2.3 and 3.2 respectively). Given the potential failure mode for landslips across the CDG stratum is different from liquefaction of loose fill concerned in these worked examples and those nails orientated perpendicular to the slope surface are hard to mobilise in case the CDG fails, please consider whether it is appropriate to present the "upgraded" FOS of the slips across the CDG stratum in these figures. | 4. The failure slip no. 4 in Figure 2.3 & 3.2 has been deleted. |
| 5. Some characters in the Figures (e.g. Figure 4.5) may be too small to read if the pages are printed in A4 paper. | 5. Noted. Characters in figures have been enlarged. |
| 6. It is suggested to demonstrate the derivation of the css/s'v ratio from triaxial test results in one of the worked examples. | 6. The purpose of these worked examples is to illustrate the new design recommendations highlighted in GEO-HKIE (2011). For the derivation of the $c_{ss}/p'_{peak}$ ratio, reference can be made to HKIE (2003). |
| 7. It is suggested to include the reinforcement details/configuration of grillage beams, soil nail heads | 7. It is not the intention of this report to illustrate the structural detailing of grillage beams, soil nail heads and embedded concrete footings. |

| Comments | Responses |
|---|---|
| and embedded concrete footings in the worked examples. | |
| 8. Worked Example 2: It is noted that "an embedded concrete footing of 1m depth is adopted to minimize the overall slope movement". It is suggested to include the detailed design of the concrete footing and the connection details between the concrete footing and the grillage beam in the worked example. | 8. The structural design of the embedded footing can be carried out according to relevant structural design standards such as "Code of Practice for Structural Use of Concrete 2013". This is explained in Section 3.6 as follows:<br><br>"The numerical analysis can also provide the shear force and bending moment induced in the concrete footing for structural design which can be carried out in accordance with relevant structural design standards. The design structural forces for worked example 2 are shown in Appendix A.2.5." |
| 9. In Section 1.2, subsection (a), Please clarify whether this step is referring to Section 3.2 of GEO-HKIE (2011). If so, please state it in Section 1.2 for clarity. | 9. Section 1.2 (a) refers to Section 3.2 of GEO-HKIE (2011). This section has been revised as follows for clarity:<br><br>Determine if slope upgrading works are required in respect of static liquefaction (Section 3.2 of GEO-HKIE (2011)) – the stability of the loose fill slope is examined under drained conditions using soil parameters corresponding to the onset of liquefaction ($\phi'_{mob}$). |
| 10. In Section 2.4, it is mentioned that the partial factor for tensile failure of the steel bar has been taken as 1.15 in accordance with Section 7.2 of HKIE (2003). However, in Table 5.6 of Geoguide 7 (2008), a factor of 1.5 is recommended. Some justifications for adopting a smaller FOS should be given. | 10. Section 5.8.1 of Geoguide 7 states that the use of soil nails in loose fill slopes should in-principle follow the recommendations given in Study Report on Soil Nails in Loose Fill prepared by the HKIE (HKIE, 2003), whilst Section 7.2 of the HKIE (2003) report recommends that the soil nail axial tension capacity should base on the ultimate capacity (not characteristic yield strength) as defined in accordance with BS 8110. Therefore the approach of using partial factor of 1.15 is considered appropriate. This justification is given in Section 2.4 of the report. |

| Comments | Responses |
|---|---|
| 11. Also in Section 2.4, line 8 of paragraph 2, please clarify whether the term "ground-reinforcement interface" should be read as "grout-reinforcement interface. | 11. The term in line 8 of paragraph 2 in Section 2.4 should read as "grout-reinforcement interface". This has been revised. |
| 12. In A.1.1, "Row 1" is missing in the soil-nail table. | 12. Noted. Appendix A.1.1 has been revised. |
| 13. In Section 2.4, the assumption of the corrosion class, e.g. Class 2, should be stated for clarity as this would affect the design strength of the soil-nail. | 13. This section has been revised as follows for clarity: "It is assumed that the steel bars used in permanent installation are provided with Class 2 corrosion protection measures." |
| 14. In P.28, there is no noticeable difference between the two figures. Please enlarge the critical zone and/or assign movement contour for better illustration. | 14. Figures in P.28 have been revised for better illustration. |
| 15. The step detailed in Section 4.2 should be a general design procedure for fill slope which has deep loose fill. As such, a step for delineating the non-liquefiable loose fill zone should be added in Section 1.2 for completeness. | 15. The following sentence has been added in Section 1.2 (a) as follows: "For deep fill, liquefiable and non-liquefiable zones can be delineated using $\phi'_{mob}$ following the procedure described in Section 5.3 of GEO-HKIE (2011)." |
| 16. In A.2.6.10, there is no indication of whether the deformation diagram is under the situation where there is a toe wall or not. | 16. Noted. The figure is enlarged and a label has been added in the figure to indicate the location of embedded concrete footing. |
| 17. The report on the hybrid arrangement (GEO-HKIE, 2011) recommends use of numerical analysis to check effectiveness of the system if the nail arrangement "deviates significantly" from the illustrative example given in the report. The illustrative examples given in this report are more or less the same as that shown in the | 17. The purpose of the worked examples is to illustrate the new design recommendations highlighted in GEO-HKIE (2011). It is not the intention of this report to cover all possible scenarios with regard to site constraints. Nonetheless, the use of steeply inclined nail arrangement in worked example 2 reflects a common problem encountered in Hong Kong whereby sub-horizontal nails cannot be installed due to site constraint. |

| Comments | Responses |
|---|---|
| 2011 report. They could not illustrate cases where nail arrangement has to be varied because of site condition (e.g. a private lot close to slope crest constraints the use or length of horizontal nails etc) in shallow/deep fills or if the height is much higher than 10 m, say 20 m, etc. | |
| 18. Page 10, 1.2(b)-To better tally with the recommendations given in GEO-HKIE (2011) and to clearly spell out the objective of the treatment in Slope/W, suggest to insert on line 15 the sentence marked in red and to amend the follow-up sentence as shown: The distribution of the earth pressure can be assumed to be triangular in shape and can be modelled by a series of equivalent concentrated forces acting in the normal direction of the slope surface. It is important that the stabilising pressure specified should not lead to an increase in shear strength of the fill in slope stability calculations. The use of equivalent concentrated forces, rather than a distributed surface pressure, will avoid the increase in the undrained shear strength of the loose fill in Slope/W program (Section 5.2 of GEO-HKIE (2011)). | 18. Section 1.2 (b) has been revised as follows: "It is important that the stabilising pressure specified should not lead to an increase in shear strength of the fill in slope stability calculations. The use of equivalent concentrated forces, rather than a distributed surface pressure, will avoid the increase in the undrained shear strength of the loose fill in some commercial computer programs (Section 5.2 of GEO-HKIE (2011))." |
| 19. Page 11, 1.2(e) - It is currently suggested in the last sentence that "A minimum grillage embedment of 0.3 m, which can be provided by back-filling, is considered adequate (Section 5.6 of GEO-HKIE (2011))." It is not apparent in GEO-HKIE (2011) that the 0.3 m depth grillage embedment can be provided by back-filling. Additional surcharge to loose fill may cause further settlement of the fill body. This recommendation should be further reviewed. | 19. We agree that GEO-HKIE (2011) does not explicitly state that the 0.3 m depth grillage embedment can be provided by back-filling. Therefore the last sentence in Section 1.2 (e) has been revised as follows: "A minimum grillage embedment of 0.3 m is considered adequate (Section 5.6 of GEO-HKIE (2011))." The settlement involved should be minimal. |

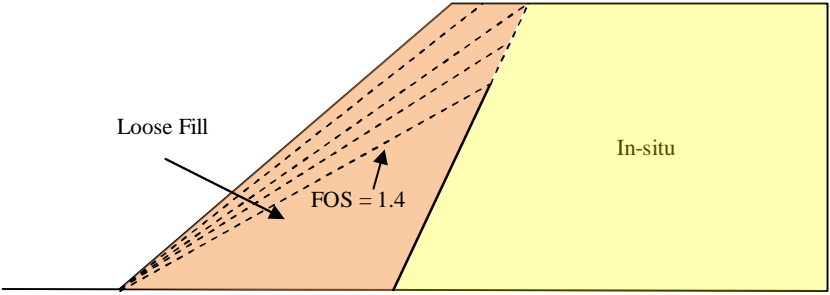| Comments | Responses |
|---|---|
| 20. Page 27, Table 3.3 - A very large compressive strength is being specified for the soil nails in the Plaxis model. Why should the compressive strength of the model nails be excessively greater than its tensile strength? | 20. The purpose of using a large maximum compressive capacity in the soil nail is to allow development of compressive force, if any, in the nail. In theory the calculated compressive force should be checked against the buckling capacity of the soil nail. However in this worked example, the calculated compressive forces are very small. Explicit checking is not required. |
| 21. Page 31, Figure 4.1 - Should the line delineating the critical 'planar slip surface' at 21° be labelled? Please review the profile of the DGWT that intercept the two ground strata (loose fill and CDG), as the two strata are likely to have a huge contrast in permeability. | 21. The delineating line in Figure 4.1 has been labelled. The main groundwater table is governed by the local hydrogeological conditions. In this worked example, the adopted design groundwater table (DGWT) is an assumption to illustrate the design procedure for a main groundwater table as compared to a perched water table. A design perched water table has also been added to reflect the contrast on soil permeability at the loose fill / CDG interface. |
| 22. A1.4, A2.4 and A3.4 Design of Shear Reinforcement for Grillage System. The design shear force has been taken as 0.5 of the UDL x beam span (assuming a simply supported beam?). Please note that the maximum shear force for a continuous beam can be 0.625 of the UDL x beam span, which is 25% higher than that for a simply supported beam. | 22. In theory, structural forces should be calculated assuming a continuous beam subjected to a triangular pressure. For simplicity, the grillage segment with the longest span is chosen for structural design and the loading is simplified as a uniformly distributed load (i.e. u = 76 kPa between rows 6 & 7 of soil nail if the maximum earth pressure is 87 kPa as shown in Worked Example 2). $\quad$ Therefore, calculations for the simply supported assumption yield the following results: Bending Moment, $M = u \times$ nail spacing $\times L^2 / 8 = 104.37$ kNm Shear Force, $V = u \times$ nail spacing $\times L / 2 = 154.39$ kN $\quad$ A simple structural analysis has been performed for a continuous beam and the results show that the maximum shear force induced in a simply supported beam (i.e. 154.39 kN) and that in a continuous beam (i.e. 156.00 |

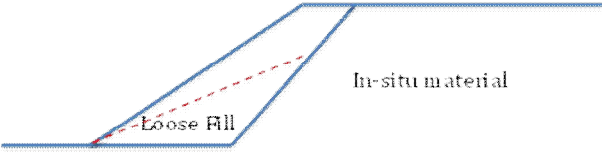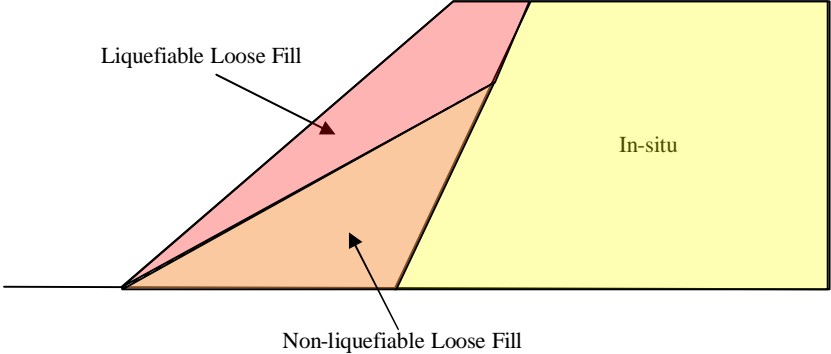| Comments | Responses |
|---|---|
| | kN) are comparable, with a difference of 1% (see Appendix B.1.1 on Page 93).<br><br>To examine if the discrepancy significantly changes with loading conditions, a series of further structural analyses have been performed for a continuous beam with different magnitudes of triangular pressure varying from 50 kPa to 1000 kPa. The calculated shear forces are compared with those obtained from simply supported assumptions. The results show that the maximum shear force in a simply supported beam and that in a continuous beam are comparable, with a percentage difference in the order of 1% in the worked example concerned (see Appendix B.1.2-1.3 on Pages 96-97). Designers are reminded to carry out necessary verification (see Section 5.4) |
| CGE/SM, LandsD<br>1. Section 1.2(c) seems to suggest that if the ratio of sub-horizontal nails to steeply inclined nails is approx. 40 - 50%, then the design should be adequate. In fact, it is desirable that for any given ratio, the basal resistance at the base of the grillage should be checked to ensure it is within the permissible value. Please advise if such a check has been performed.<br><br>2. Section 3.1 states that the purpose of the embedded concrete footing is to reduce slope movement. However, numerical analysis indicates that slope deformation seems to be reduced by 25mm (from 115mm to 90 mm). Please advise if the reduction achieved is adequate, and what other measures could be adopted if more substantial reduction is contemplated. | 1. At stated in Section 1.2 (c), the adequacy of the ratio of sub-horizontal nail to total number of nail should be checked by considering the overall force equilibrium on the grillage structure. The calculations for such checking as shown in Appendices A.1.6 and A.3.6 have assumed the maximum permissible basal resistance which can be mobilised at the base of the grillage.<br><br>2. The adequacy of the resulting slope movement achieved by incorporating the embedded concrete footing is case dependent. As discussed in Section 3.7, the acceptance criteria for slope deformation should be determined on a site-specific basis taking into account the tolerable movements of any sensitive receivers in close proximity to the slope. This worked example only serves to demonstrate the deformation assessment by the numerical analysis. To achieve a more substantial reduction, a stronger structure can be provided. |

| Comments | Responses |
|---|---|
| 3. There does not appear to be any analysis of the effect of the settlement of the grillage under its own weight on the integrity of the soil nail system, e.g. due to differential settlement induced at the nail/grillage connection. Please advise if such a check has been performed. | 3. Additional bending moment could be resulted where differential settlement occurs. However, since a uniform dimension of grillage beam is usually adopted, differential settlement is not likely. Thus, the suggested checking is considered not necessary in general. However, where certain conditions arise whereby differential settlements could occur, designers should duly consider the effects of differential settlements in the design. |
| CGE/HD<br>1. Section 2.1 Design Scenario and Assumptions On p.12 lines 7-10 – We note from Worked Example 1 that the design soil parameters are assumed to be determined from the site-specific laboratory testing. It would be desirable to provide relevant guidance / examples on how to interpret site-specific laboratory testing results. | 1. The purpose of these worked examples is to illustrate the new design recommendations highlighted in GEO-HKIE (2011). For the derivation of the $c_{ss}/p'_{peak}$ ratio, reference can be made to HKIE (2003). |
| 2. Section 2.4 Design of Soil Nails<br>On p.15 lines 1-3 – It is noted that three rows of sub-horizontal nails at 20$^o$ to horizontal and four rows of nails at 56$^o$ to the horizontal are adopted in Worked Example 1. If the ratio of sub-horizontal nails to steeply inclined nails complies with the recommendation in Section 5.4 of the GEO-HKIE Report, could the alternative nail patterns (i.e. nail inclinations deviated from the above) be adopted in the design without the need to conduct numerical analyses? | 2. According to Section 5.4 of GEO-HKIE (2011), if an alternative nail arrangement which deviates significantly from that illustrated in Figure 4.2c is intended to suit actual site conditions, the designer should demonstrate the effectiveness and robustness of the proposed nail arrangement using numerical analyses. If a hybrid nail arrangement is adopted following Section 5.4 of GEO-HKIE (2011), numerical analysis to demonstrate the robustness of the proposed scheme is considered not necessary. However, it should be noted that the required number of sub-horizontal nails and their inclinations are governed by the slope geometry, soil properties, and groundwater conditions. This should be checked by considering the force equilibrium of the facing as described in Section 5.4 of GEO-HKIE (2011). |
| 3. Section 3.1 Design Scenario and Assumptions<br>On p.19 lines 8-9 – It is mentioned that the roles of the embedded concrete footing is to reduce slope movement | 3. The purpose of these worked examples is to illustrate the new design recommendations highlighted in GEO-HKIE (2011). The structural design of the embedded footing can be carried out according to relevant |

| Comments | Responses |
|---|---|
| upon fill liquefaction. It would be useful to include a worked example to illustrate the footing design based on the results of numerical analyses described in Section 3.7. | structural design standards such as "Code of Practice for Structural Use of Concrete 2013". This is explained in Section 3.6 as follows:<br><br>"The numerical analysis can also provide the shear force and bending moment induced in the concrete footing for the structural design which can be carried out in accordance with relevant structural design standards. The design structural forces for worked example 2 are shown in Appendix A.2.5." |
| 4. Appendix A Detailed Design Calculation of Worked Examples<br>The following editorial errors are noted:<br>i) On p.66, the title of Worked Example A.3 "A.3 Detailed Design Calculation of Worked Example 3" should be inserted.<br>ii) The page numbers for Worked Example A.3 are missing. | 4. Noted. The corresponding sections have been revised. |
| Ir Leslie Swann of Jacobs (HKIE member)<br><br>1. Section 1.2 (b) – Modelling of stabilizing pressure by equivalent concentrated forces<br><br>It seems that modelling of the stabilizing earth pressure to be sustained by the soil nails by using a set of discrete point loads can lead to have great variance of FOS if the "Grid and Radius Method" in Slope/W is employed in the limit equilibrium analysis.<br><br>We have conducted a preliminary study using a slope geometry and design parameters similar to Figure 2.1 and Table 2.1 of the above Report respectively, as | 1. One possible reason for the large variance in FOS when the "Grid and Radius Method" is employed is that the critical slip surface may not intercept the slope surface at the slope toe and therefore have omitted the contribution from some of the point loads near the toe. The figures provided are not sufficiently clear to identify the location where the critical slip surface intercepts the slope surface. To illustrate the potential problems associated with the use of the "Grid and Radius Method", a set of Slope/W analyses with the same assumptions adopted by the reviewer have been conducted. The four analyses considered different numbers of point loads (i.e., 10, 20, 30, and 60) and the FOS was calculated using the "Grid and Radius Method". |

| Comments | Responses |
|---|---|
| presented in the enclosed Figs 1 to 4 (see Appendix B.2 on Pages 98 to 101). In this model, we have calculated the FOS based on different numbers of point loads (10, 20, 30 and 60). The results indicate that the FOS can range from 0.991 to 1.122 with the same equivalent stabilizing triangular pressure of 60kPa when comparing the same slip surface.<br><br>We have also compared the case of 10 nos. of point load as shown in Figs 1 and 5 (see Appendix B.2 on Pages 98 to 102), the Umax had increased from 60kPa to 72kPa if FOS of 1.1 to be required. As a result, the length of the soil nail would increase about 10% as presented in Figure 6 (see Appendix B.2 on Page 103). As such, it is advisable to provide more guidance or recommendation on the adoption of number of point to be used in the stability analysis in order to provide a more cost-effectiveness design. | The results of the analysis with 60 point loads are presented in Appendix B.3 on Page 106. It can be seen that four no. of point loads have been omitted in the calculations as the critical slip surface intercepts the slope surface above these points. The omitted stabilising force is 76.66 kN/m and the resulting FOS is 1.176. The results of the other cases are summarised in the table shown below.<br><br>The results suggest that FOS decreases as the magnitude of total omitted point loads increases. As the assumed distribution of the earth pressure is triangular, the omitted point loads near the slope toe will have larger effect on FOS than that of other locations. Although the omission of point loads errs on the conservative side, it does not reflect the reality that the facing structure is monolithic.<br><br>A better way to investigate the effect of numbers of point loads on FOS is to specify a slip surface which embraces all the point loads exerted on the slope surface. Following the same geometry and soil parameters in Figure 2.1 and Table 2.1 in the Report - "Design Illustrations on the Use of Soil Nails to Upgrade Loose Fill Slopes", 10-, 20-, 30-, and 60-point loads cases have been compared. The resulting FOSs are 1.102, 1.105, 1.078, and 1.08, respectively. The small variance is considered acceptable.<br><br>In theory, the triangular earth pressure should be modelled using as many |

| Case | No. of omitted point loads | Total omitted point loads | FOS |
|---|---|---|---|
| 60 point loads | 4 | 76.66 kN/m | 1.176 |
| 30 point loads | 2 | 76.66 kN/m | 1.175 |
| 20 point loads | 1 | 57.99 kN/m | 1.246 |
| 10 point loads | 1 | 113 kN/m | 1.056 |

| Comments | Responses |
|---|---|
| | point loads as possible.   However, if many point loads are used, the trial-and-error procedure to identify the required stabilising pressure would become impractical.   In the case considered, it has been demonstrated that 10 nos. of point loads are sufficient to limit the error to within 3%. For other design conditions, designers should make their own engineering judgement to determine the minimum number of point loads to represent the earth pressure. |
| 2. Section 1.2 (b) – Modelling of stabilizing pressure by equivalent concentrated forces<br><br>It is mentioned that the distribution of the earth pressure can be modelled by a series of equivalent concentrated forces acting in the normal direction of the slope surface. In fact, the slope profile is usually non-uniform or comprises of several batters and designers may opt for adopting a single average slope angle to calculate the direction of the earth pressure for all concentrated forces. A comparison was done (see Figs 7 and 8 in Appendix B.2 on Pages 104 and 105) to review the effect of the orientation of the point load to the calculation of the FOS. It was found that adopting the average slope angle could lead to significant variance in the FOS calculation (Slip No. #3).   It may worth mentioning this issue in the report as a reminder to the practitioners when conducting modelling. | 2. The earth pressure exerted on the facing structure is assumed to be acting in the normal direction of the slope surface since very small shear stress can be mobilised along the interface between the facing and the loose fill. This small shear stress has already been taken into consideration by the assumed basal shear force.   Therefore the earth pressure must be normal to the slope surface.   When applying the stabilising point loads, one should consider if the proposed soil nail arrangement would be able to provide the assumed stabilising effects. For example the downward force exerted on the slope berm is unrealistic as no nail force would be mobilised in this direction, and stabilisation force exerted on slope berm should be neglected for design purpose. |
| 3. Section 2.6 – Design of the Grillage Structure and Soil Nail Heads<br><br>According to Section 2.6 and the Structural Design Calculations of Grillage Facing for the examples, it is | 3. The suggestion to model the continuous grillage beam as "simply supported" is for simplicity.   It is acceptable to analyze the entire grillage beam as a continuous structure to come up with an optimized design. |

| Comments | Responses |
|---|---|
| noted that each segment of the grillage beams is assumed to be simply supported. With reference to Section 7.3 in Soil Nails in Loose Fill Slopes – A Preliminary Study (HKIE, 2003), a continuous structural grillage beam / cover is suggested for the development of required nail force. It is doubt that the continuous grillage beam is assumed to be "simply supported" while the deflection behaviour between continuous beam and a simply supported beam is different. As a result, the maximum bending moment for a simply support beam is relatively larger than continuous beam under the same loading condition and may cause over-conservatism.<br><br>4. Section 4.2 – Delineation of Non-liquefiable Loose Fill Zone<br><br>The document recommends that a planar slip surface must fall entirely within the fill layer to delineate the non-liquefiable layer. However, it is not uncommon to encounter a slope having increasing thickness of fill with decreasing height (as shown below). In such case, a planar slip surface with a Factor of Safety of 1.4 is unlikely to be obtained without encroaching into the underlying material. As such, a full liquefiable layer will have to be assumed. As such, it is advisable to provide more guidance on the delineation of non-liquefiable layer in this occasion. | 4. For the case considered, the non-liquefiable zone can be identified by assuming linear or bi-linear slip surfaces which fall entirely within the loose fill zone as depicted in the figure below. The liquefiable zone refers to the zone above the slip surface which has a minimum FOS of 1.4 (i.e. slip surface S8 in Appendix B.4 on Page 107) as illustrated in the following figures.<br><br> |

| Comments | Responses |
|---|---|
| In-situ material<br><br>Loose Fill | Liquefiable Loose Fill<br><br>In-situ<br><br>Non-liquefiable Loose Fill |

**B.1 – Structural Analysis of Continuous Beam (Pressure is taken from worked example 2)**
**B.1.1 Example of the Structural Analysis (Maximum Earth Pressure = 87 kPa)**



Load Cases:
— 1 P Title of case 1

Pressure induced in grillage in worked example 2 at this position = 76 kPa

spacing of soil nails = 1.5 m

Hence,
76 kPa x 1.5 m = 114 kN/m

Properties of Grillage:
E = 22 GPa
Width = 0.6 m
Thickness = 0.3 m

130
124
104
84.8
65.2
45.7
26.1
5.53

Soil Nail Row 7 (Bottom)
Soil Nail Row 6
Soil Nail Row 5
Soil Nail Row 4
Soil nail Row 3
Soil Nail Row 2
Soil Nail Row 1 (Top)
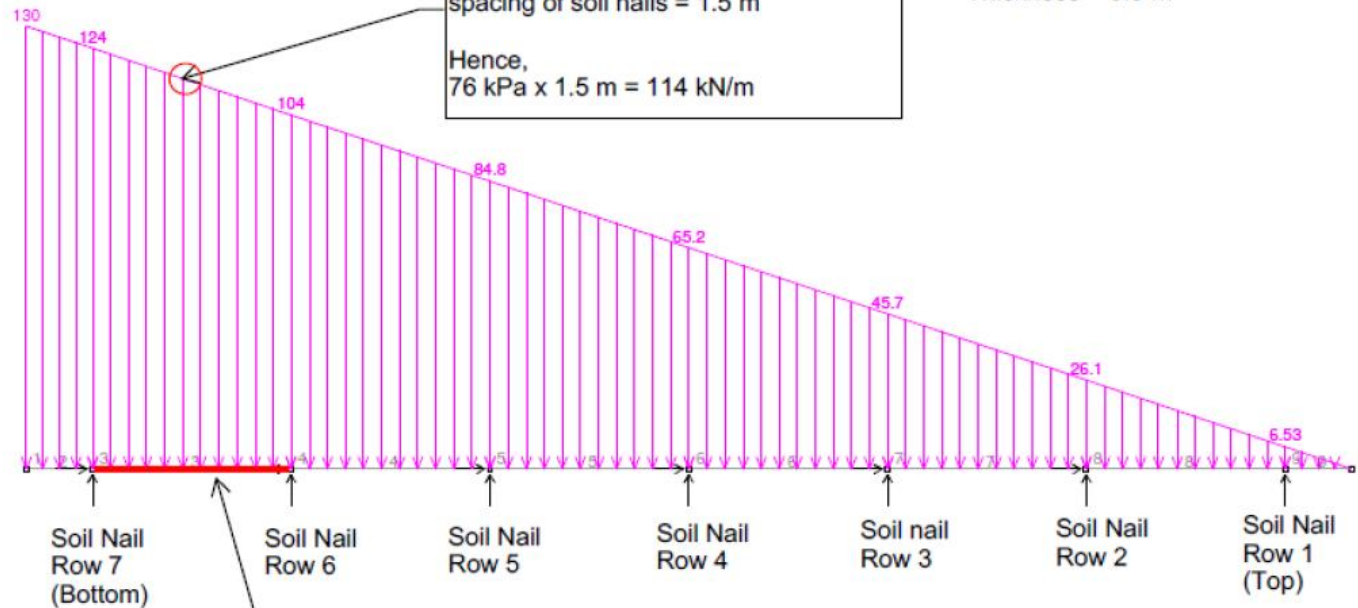
Design approach in the worked examples:
The grillage segment with the longest span and maximum loading condition of a uniformly distributed load is chosen and assumed to be simply supported for structural design.

Y
Z — X
theta: 270  phi: 0

Continuous Beam - Loading (87kN/m2x1.5m=130kN/m)

Load Cases:
—— 1 P Title of case 1

<u>Simply Supported Beam Assumption:</u>
Maximum Bending Moment = 104.37 kNm

<u>Continuous Beam Assumption:</u>
Maximum Bending Moment = 69.70 kNm

Maximum
Bending Moment

69.7  69.7

52  52

49.8  49.8

40.1  40.1

27.4  27.4

16.7  16.7

0.881  0.881

26.5

23.4

16.8

10.6

6.42

43.2

Y

Z→ X
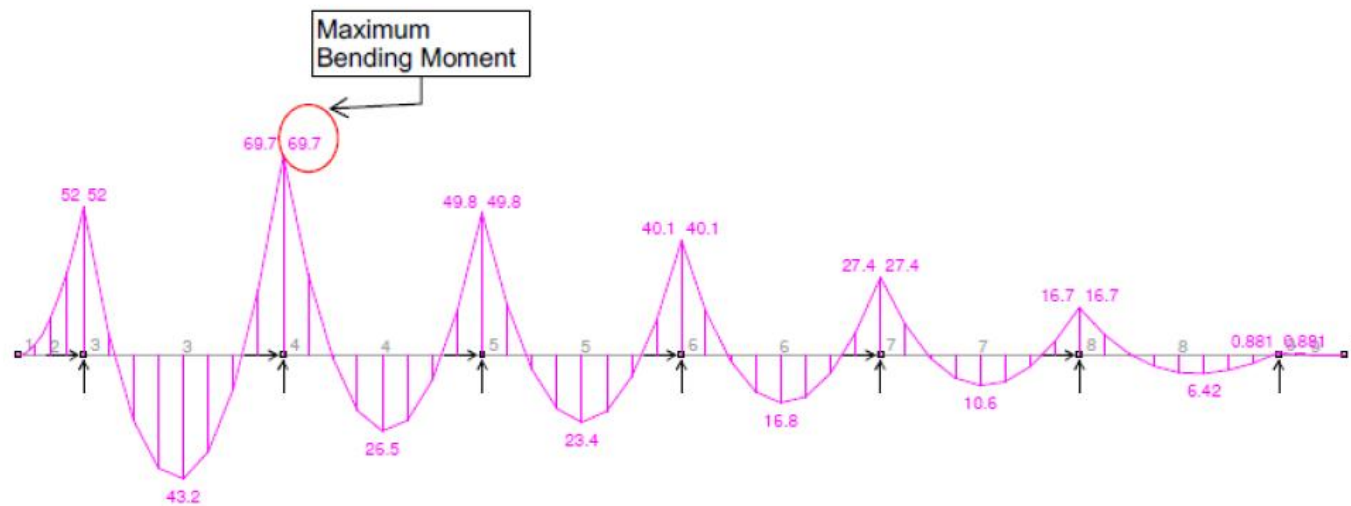
theta: 270  phi: 0

Continuous Beam - Bending Moment

Bending Moment, Mz

Load Cases:
— 1 P Title of case 1

Simply Supported Beam Assumption:
Maximum Shear Force = 154.39 kN

Continuous Beam Assumption:
Maximum Shear Force = 156.00 kN



156

Maximum Shear Force

115

116

93.3

65.8

40.1

11.8

1 2 3 3 4 4 5 5 6 6 7 7 8 8 9 9 2

2.94

152

139

109

84

56.8

32.3
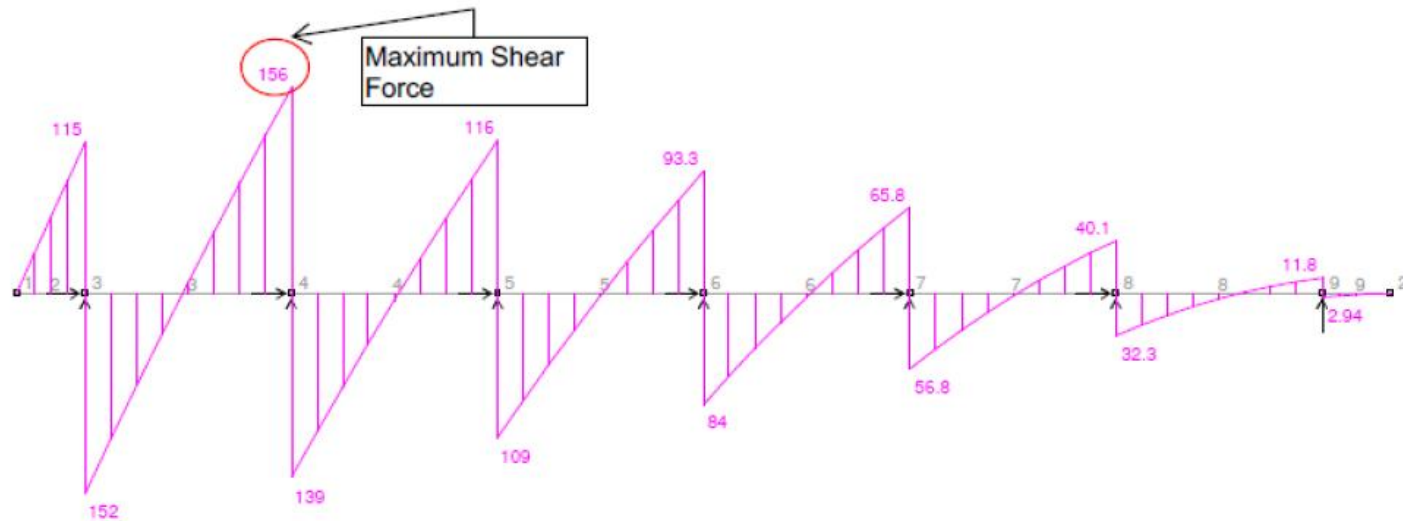
Y
Z — X
theta: 270  phi: 0

Continuous Beam - Shear force

Shear Force, Fy

**B.1.2 Summary of Analyses**

| Max. Earth Pressure (kPa) | Maximum Shear force (kN) | | % difference |
|---|---|---|---|
| | Simply Supported Beam Assumption | Continuous Beam Assumption | |
| 50 | 88.7 | 89.9 | 1.30% |
| 87 | 154.4 | 156.0 | 1.03% |
| 100 | 177.5 | 180.0 | 1.41% |
| 150 | 266.2 | 270.0 | 1.41% |
| 200 | 354.9 | 359.0 | 1.14% |
| 350 | 621.1 | 629.0 | 1.25% |
| 500 | 887.3 | 898.0 | 1.19% |
| 1000 | 1774.6 | 1797.0 | 1.25% |

**B.1.3 Results of Analyses**

## B.2 – Figures Attached in Comments made by Ir Leslie Swann

Earth pressure divided into 10 number of point loads

File Name: Trial_006D.gsz



Name: FILL (Undrained)
Model: S=f(overburden)
Unit Weight: 17.7 kN/m³
Tau/Sigma Ratio: 0.268
Minimum Strength: 0
Piezometric Line: 2

Name: CDG
Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 5 kPa
Phi: 35 °
Piezometric Line: 1

L1 Value: 5.41 kN/m
L2 Value: 16.22 kN/m
L3 Value: 27.03 kN/m
L4 Value: 37.85 kN/m
L5 Value: 48.66 kN/m
L6 Value: 59.48 kN/m
L7 Value: 70.29 kN/m
L8 Value: 81.1 kN/m
L9 Value: 91.92 kN/m
L10 Value: 102.73 kN/m

Total Required Load = 540.69 kN/m
Equivalent Stabilizing Triangular Pressure = 60 kPa
Total Basal Shear Force = 3kPa x 18.02m = 54.07 kN/m
Basal Shear Force Acting with Each Point Load = 5.41 kN/m

**Fig. 1**

Earth pressure divided into 20 number of point loads

Name: FILL (Undrained)
Model: S=f(overburden)
Unit Weight: 17.7 kN/m³
Tau/Sigma Ratio: 0.268
Minimum Strength: 0
Piezometric Line: 2

Name: CDG
Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 5 kPa
Phi: 35 °
Piezometric Line: 1

L1 Value: 1.35 kN/m
L2 Value: 4.06 kN/m
L3 Value: 6.76 kN/m
L4 Value: 9.46 kN/m
L5 Value: 12.17 kN/m
L6 Value: 14.87 kN/m
L7 Value: 17.57 kN/m
L8 Value: 20.28 kN/m
L9 Value: 22.98 kN/m
L10 Value: 25.68 kN/m
L11 Value: 28.39 kN/m
L12 Value: 31.09 kN/m
L13 Value: 33.79 kN/m
L14 Value: 36.5 kN/m
L15 Value: 39.2 kN/m
L16 Value: 41.9 kN/m
L17 Value: 44.61 kN/m
L18 Value: 47.31 kN/m
L19 Value: 50.01 kN/m
L20 Value: 52.72 kN/m

Total Required Load = 540.69 kN/m
Equivalent Stabilizing Triangular Pressure = 60 kPa
Total Basal Shear Force = 3kPa x 18.02m = 54.07 kN/m
Basal Shear Force Acting with Each Point Load = 2.70 kN/m

**Fig. 2**

Earth pressure divided into 30 number of point loads

File Name: Trial_006F.gsz

Name: FILL (Undrained)
Model: S=f(overburden)
Unit Weight: 17.7 kN/m³
Tau/Sigma Ratio: 0.268
Minimum Strength: 0
Piezometric Line: 2

Name: CDG
Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 5 kPa
Phi: 35 °
Piezometric Line: 1

| | |
|---|---|
| L1 Value: 0.6 kN/m | L16 Value: 18.62 kN/m |
| L2 Value: 1.8 kN/m | L17 Value: 19.83 kN/m |
| L3 Value: 3 kN/m | L18 Value: 21.03 kN/m |
| L4 Value: 4.21 kN/m | L19 Value: 22.23 kN/m |
| L5 Value: 5.41 kN/m | L20 Value: 23.43 kN/m |
| L6 Value: 6.61 kN/m | L21 Value: 24.63 kN/m |
| L7 Value: 7.81 kN/m | L22 Value: 25.83 kN/m |
| L8 Value: 9.01 kN/m | L23 Value: 27.03 kN/m |
| L9 Value: 10.21 kN/m | L24 Value: 28.24 kN/m |
| L10 Value: 11.41 kN/m | L25 Value: 29.44 kN/m |
| L11 Value: 12.62 kN/m | L26 Value: 30.64 kN/m |
| L12 Value: 13.82 kN/m | L27 Value: 31.84 kN/m |
| L13 Value: 15.02 kN/m | L28 Value: 33.04 kN/m |
| L14 Value: 16.22 kN/m | L29 Value: 34.24 kN/m |
| L15 Value: 17.42 kN/m | L30 Value: 35.45 kN/m |

Total Required Load = 540.69 kN/m
Equivalent Stabilizing Triangular Pressure = 60 kPa
Total Basal Shear Force = 3kPa x 18.02m = 54.07 kN/m
Basal Shear Force Acting with Each Point Load = 1.80 kN/m

**Fig. 3**

## Earth Pressure divided into 60 number of point loads

File Name: Trial_006G.gsz



Name: FILL (Undrained)
Model: S=f(overburden)
Unit Weight: 17.7 kN/m³
Tau/Sigma Ratio: 0.268
Minimum Strength: 0
Piezometric Line: 2

Name: CDG
Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 5 kPa
Phi: 35 °
Piezometric Line: 1

| | |
|---|---|
| L1 Value: 0.15 kN/m | L31 Value: 9.16 kN/m |
| L2 Value: 0.45 kN/m | L32 Value: 9.46 kN/m |
| L3 Value: 0.75 kN/m | L33 Value: 9.76 kN/m |
| L4 Value: 1.05 kN/m | L34 Value: 10.06 kN/m |
| L5 Value: 1.35 kN/m | L35 Value: 10.36 kN/m |
| L6 Value: 1.65 kN/m | L36 Value: 10.66 kN/m |
| L7 Value: 1.95 kN/m | L37 Value: 10.96 kN/m |
| L8 Value: 2.25 kN/m | L38 Value: 11.26 kN/m |
| L9 Value: 2.55 kN/m | L39 Value: 11.56 kN/m |
| L10 Value: 2.85 kN/m | L40 Value: 11.87 kN/m |
| L11 Value: 3.15 kN/m | L41 Value: 12.17 kN/m |
| L12 Value: 3.45 kN/m | L42 Value: 12.47 kN/m |
| L13 Value: 3.75 kN/m | L43 Value: 12.77 kN/m |
| L14 Value: 4.06 kN/m | L44 Value: 13.07 kN/m |
| L15 Value: 4.36 kN/m | L45 Value: 13.37 kN/m |
| | |
| L16 Value: 4.66 kN/m | L46 Value: 13.67 kN/m |
| L17 Value: 4.96 kN/m | L47 Value: 13.97 kN/m |
| L18 Value: 5.26 kN/m | L48 Value: 14.27 kN/m |
| L19 Value: 5.56 kN/m | L49 Value: 14.57 kN/m |
| L20 Value: 5.86 kN/m | L50 Value: 14.87 kN/m |
| L21 Value: 6.16 kN/m | L51 Value: 15.17 kN/m |
| L22 Value: 6.46 kN/m | L52 Value: 15.47 kN/m |
| L23 Value: 6.76 kN/m | L53 Value: 15.77 kN/m |
| L24 Value: 7.06 kN/m | L54 Value: 16.07 kN/m |
| L25 Value: 7.36 kN/m | L55 Value: 16.37 kN/m |
| L26 Value: 7.66 kN/m | L56 Value: 16.67 kN/m |
| L27 Value: 7.96 kN/m | L57 Value: 16.97 kN/m |
| L28 Value: 8.26 kN/m | L58 Value: 17.27 kN/m |
| L29 Value: 8.56 kN/m | L59 Value: 17.57 kN/m |
| L30 Value: 8.86 kN/m | L60 Value: 17.87 kN/m |

Total Required Load = 540.69 kN/m
Equivalent Stabilizing Triangular Pressure = 60 kPa
Total Basal Shear Force = 3kPa x 18.02m = 54.07 kN/m
Basal Shear Force Acting with Each Point Load = 0.90 kN/m

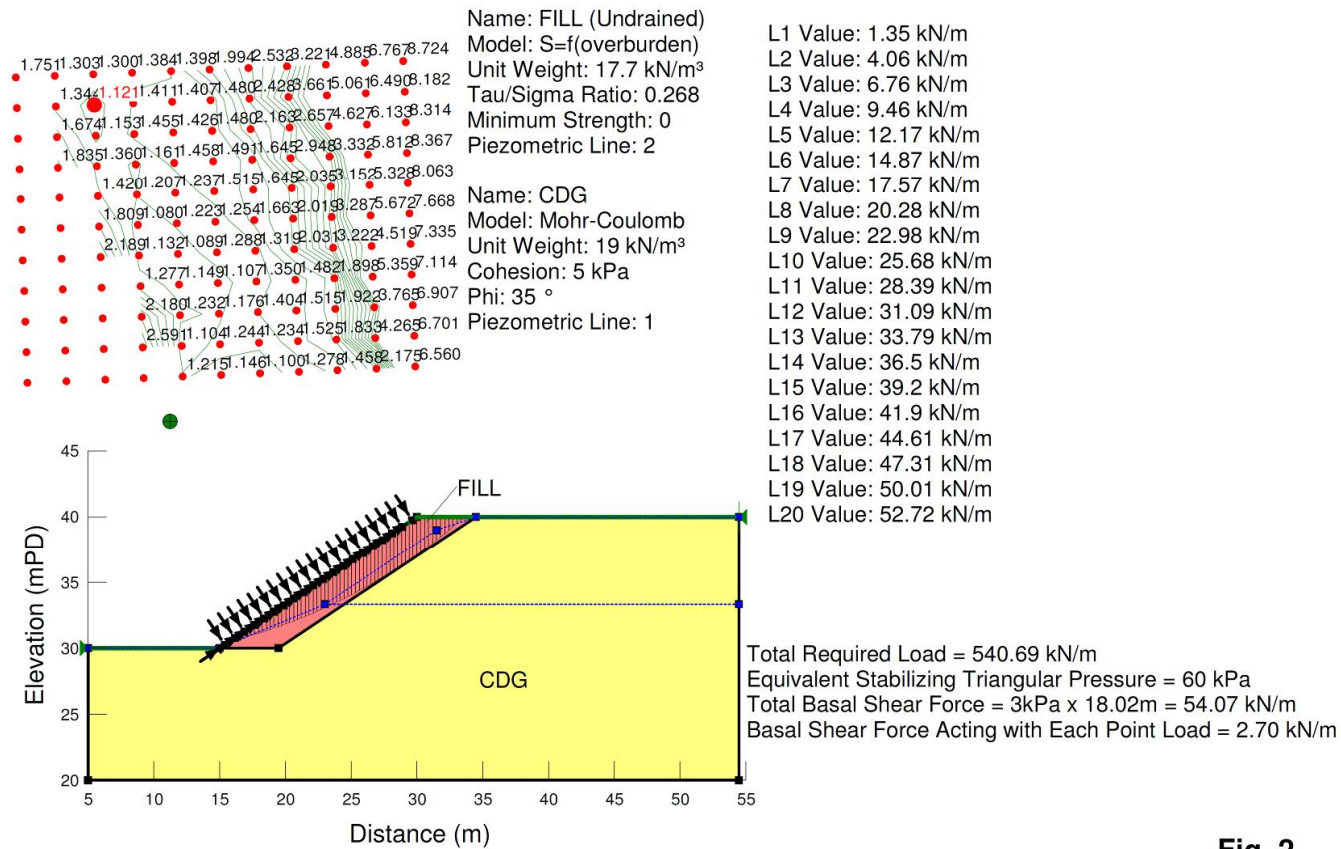**Fig. 4**

Earth pressure divided into 10 number of point loads

File Name: Trial_006Da.gsz



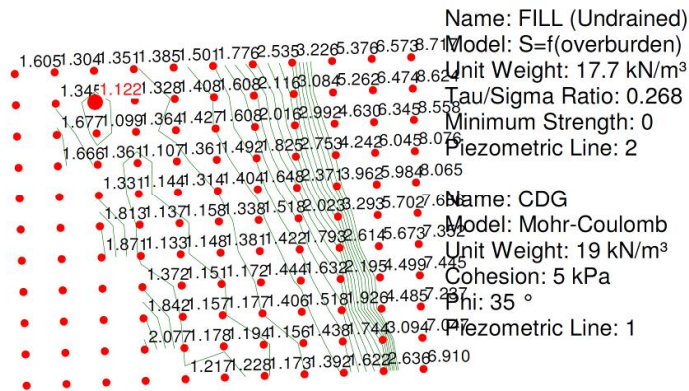Name: FILL (Undrained)
Model: S=f(overburden)
Unit Weight: 17.7 kN/m³
Tau/Sigma Ratio: 0.268
Minimum Strength: 0
Piezometric Line: 2

Name: CDG
Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 5 kPa
Phi: 35 °
Piezometric Line: 1

L1 Value: 6.49 kN/m
L2 Value: 19.46 kN/m
L3 Value: 32.44 kN/m
L4 Value: 45.42 kN/m
L5 Value: 58.39 kN/m
L6 Value: 71.37 kN/m
L7 Value: 84.35 kN/m
L8 Value: 97.32 kN/m
L9 Value: 110.3 kN/m
L10 Value: 123.28 kN/m

Total Required Load = 648.83 kN/m
Equivalent Stabilizing Triangular Pressure = 72 kPa
Total Basal Shear Force = 3kPa x 18.02m = 54.07 kN/m
Basal Shear Force Acting with Each Point Load = 5.41 kN/m

**Fig. 5**

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Fig. 6

Case 1: Orientation of Point Loads Normal to Slope Surface

File Name: Trial_008C.gsz

Name: FILL (Undrained)
Model: S=f(overburden)
Unit Weight: 17.7 kN/m³
Tau/Sigma Ratio: 0.268
Minimum Strength: 0
Piezometric Line: 2

Name: CDG
Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 5 kPa
Phi: 35 °
Piezometric Line: 1

| Slip No. | F.O.S. |
|---|---|
| 1 | 1.393 |
| 2 | 1.378 |
| 3 | 1.258 |

1.258

L1 Value: 1.66 kN/m
L2 Value: 4.99 kN/m
L3 Value: 8.31 kN/m
L4 Value: 11.64 kN/m
L5 Value: 14.96 kN/m
L6 Value: 18.29 kN/m
L7 Value: 21.61 kN/m
L8 Value: 24.94 kN/m
L9 Value: 28.26 kN/m
L10 Value: 31.59 kN/m
L11 Value: 34.91 kN/m
L12 Value: 38.24 kN/m
L13 Value: 41.56 kN/m
L14 Value: 44.89 kN/m
L15 Value: 48.21 kN/m
L16 Value: 51.54 kN/m

L17 Value: 54.86 kN/m
L18 Value: 50.54 kN/m
L19 Value: 53.06 kN/m
L20 Value: 55.59 kN/m
L21 Value: 58.12 kN/m
L22 Value: 60.64 kN/m
L23 Value: 63.17 kN/m
L24 Value: 65.7 kN/m
L25 Value: 68.22 kN/m
L26 Value: 70.75 kN/m
L27 Value: 73.28 kN/m
L28 Value: 75.8 kN/m
L29 Value: 78.33 kN/m
L30 Value: 80.86 kN/m
L31 Value: 83.38 kN/m
L32 Value: 85.91 kN/m



Total Required Load = 1503.79 kN/m
Equivalent Stabilizing Triangular Pressure = 100 kPa
Total Basal Shear Force = 3kPa x 30.08m = 90.23 kN/m

**Fig. 7**

Case 2: Orientation of Point Loads Normal to Average Slope Angle (30.26 deg)     File Name: Trial_008B.gsz

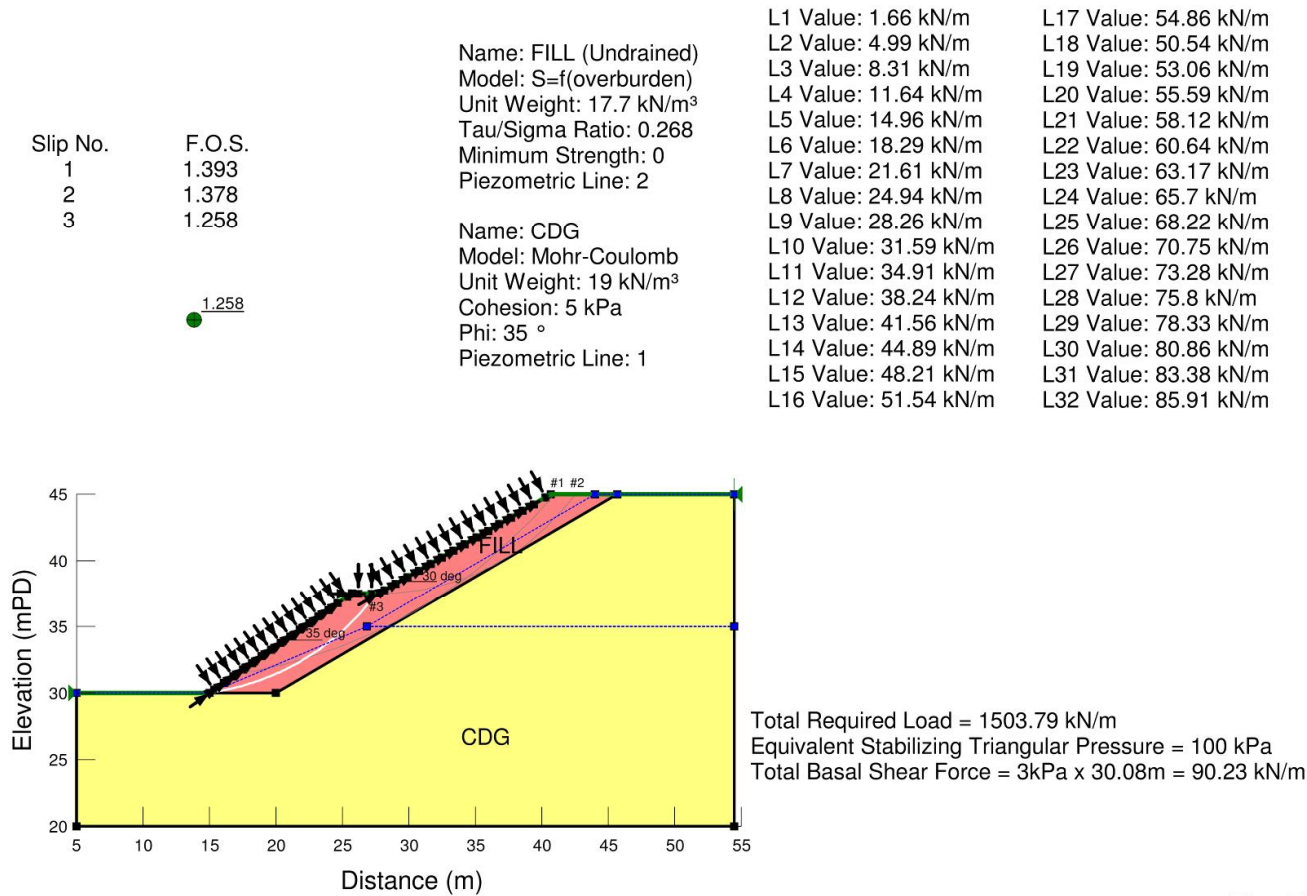| | |
|---|---|
| L1 Value: 1.66 kN/m | L17 Value: 54.86 kN/m |
| L2 Value: 4.99 kN/m | L18 Value: 50.54 kN/m |
| L3 Value: 8.31 kN/m | L19 Value: 53.06 kN/m |
| L4 Value: 11.64 kN/m | L20 Value: 55.59 kN/m |
| L5 Value: 14.96 kN/m | L21 Value: 58.12 kN/m |
| L6 Value: 18.29 kN/m | L22 Value: 60.64 kN/m |
| L7 Value: 21.61 kN/m | L23 Value: 63.17 kN/m |
| L8 Value: 24.94 kN/m | L24 Value: 65.7 kN/m |
| L9 Value: 28.26 kN/m | L25 Value: 68.22 kN/m |
| L10 Value: 31.59 kN/m | L26 Value: 70.75 kN/m |
| L11 Value: 34.91 kN/m | L27 Value: 73.28 kN/m |
| L12 Value: 38.24 kN/m | L28 Value: 75.8 kN/m |
| L13 Value: 41.56 kN/m | L29 Value: 78.33 kN/m |
| L14 Value: 44.89 kN/m | L30 Value: 80.86 kN/m |
| L15 Value: 48.21 kN/m | L31 Value: 83.38 kN/m |
| L16 Value: 51.54 kN/m | L32 Value: 85.91 kN/m |

Name: FILL (Undrained)
Model: S=f(overburden)
Unit Weight: 17.7 kN/m³
Tau/Sigma Ratio: 0.268
Minimum Strength: 0
Piezometric Line: 2

Name: CDG
Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 5 kPa
Phi: 35 °
Piezometric Line: 1

| Slip No. | F.O.S. |
|---|---|
| 1 | 1.351 |
| 2 | 1.398 |
| 3 | 0.912 |

0.912



Total Required Load = 1503.79 kN/m
Equivalent Stabilizing Triangular Pressure = 100 kPa
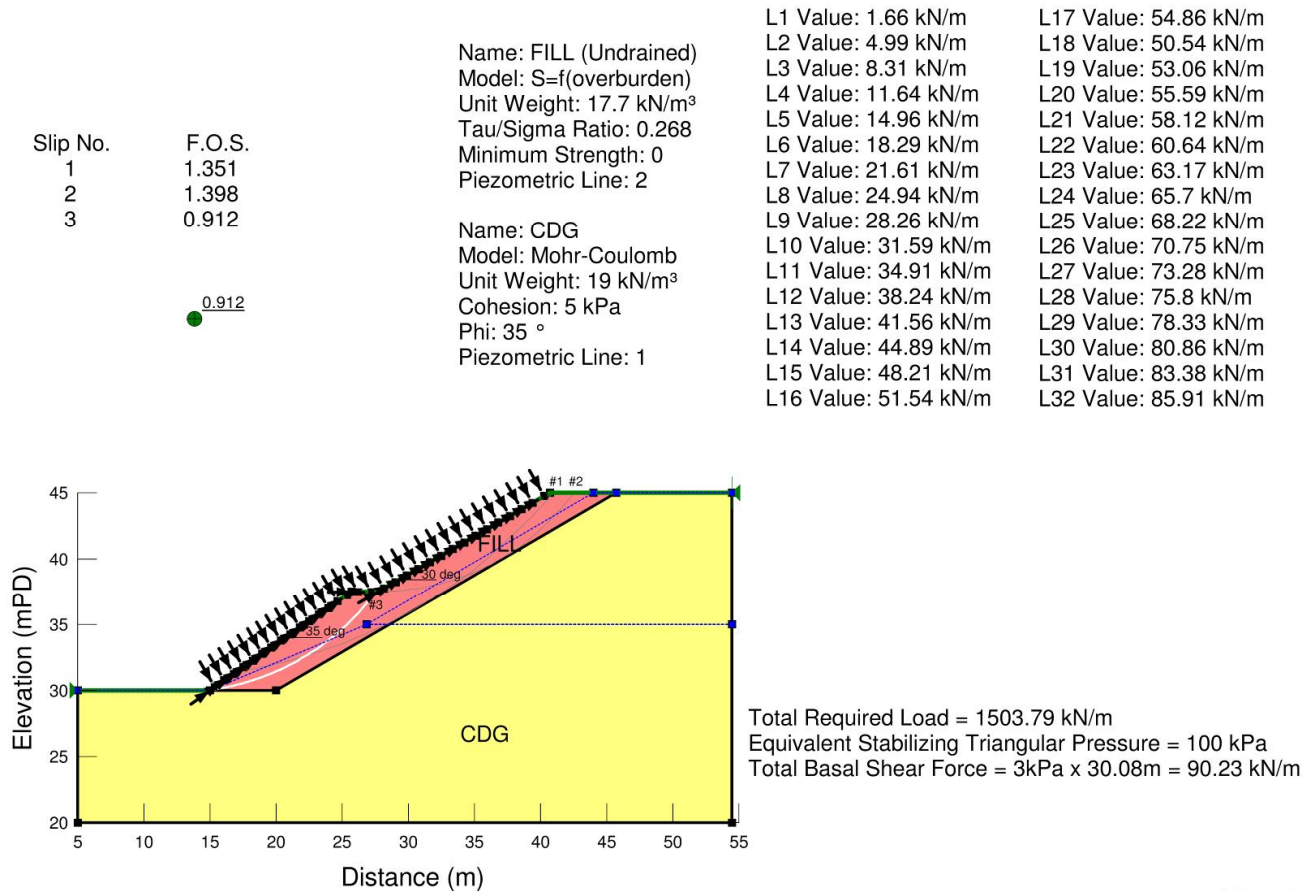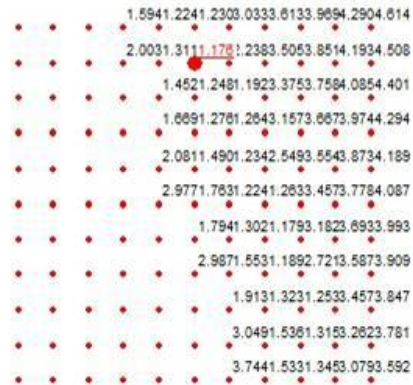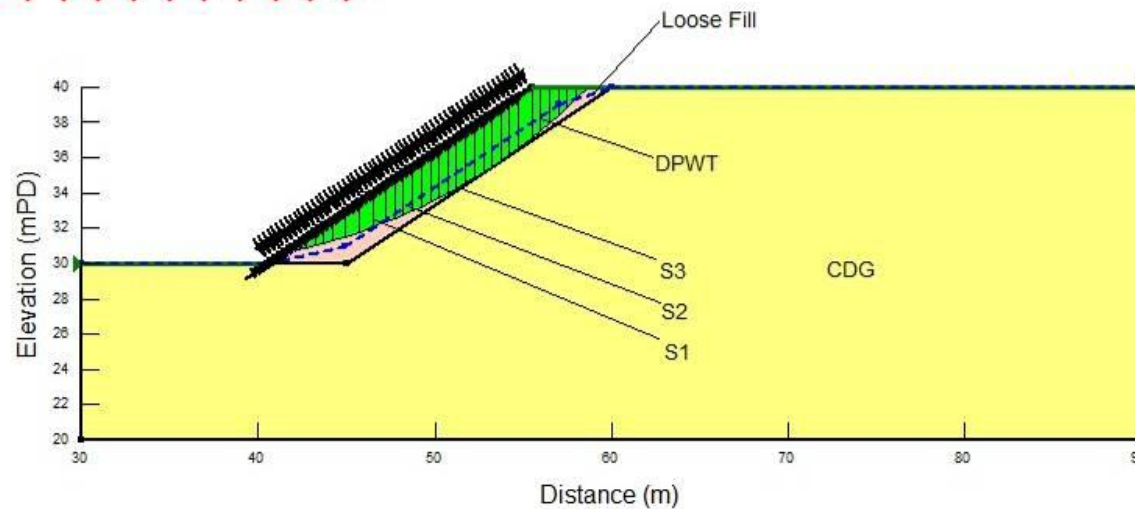Total Basal Shear Force = 3kPa x 30.08m = 90.23 kN/m

**Fig. 8**

**B.3 Slope Stability Analyses of Case of 60 Point Load Assumption with Grid and Radius Slip Surface**

## Worked Example 1 (Steady State Undrained Conditions)



Description: Loose fill
Wt: 1800 kg/cu.m x 9.81 m/sq.s
= 17.7 kN/cu.m
Tau/Sigma Ratio:0.267

Description: CDG
Wt: 1800 kg/cu.m x 9.81 m/sq.s
= 17.7 kN/cu.m
Cohesion: 5 kPa
Phi: 35 degrees

**B.4 The Use of Bi-linear Slip Surfaces to Delineate the Non-liquefiable Zone**

## Worked example (drained condition)

Description: Loose fill
Wt: 1800kg/cu.m x 9.81m/sq.s
   = 17.7kN/cu.m
Cohesion: 0kPa
Phi: 28 degrees

Description: CDG
Wt: 1800kg/cu.m x 9.81m/sq.s
   = 17.7kN/cu.m
Cohesion: 5kPa
Phi: 35 degrees

| Slip | FOS |
|------|-------|
| 1 | 0.874 |
| 2 | 0.978 |
| 3 | 1.074 |
| 4 | 1.195 |
| 5 | 1.272 |
| 6 | 1.324 |
| 7 | 1.369 |
| 8 | 1.401 |