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ARTICLE

Influence of soil nail orientations on stabilizing mechanisms of loose fill slopes

C.Y. Cheuk, K.K.S. Ho, and A.Y.T. Lam

Abstract: Soil nailing has been used to upgrade substandard loose fill slopes in Hong Kong. Due to the possibility of static liquefaction failure, a typical design arrangement comprises a structural slope facing anchored by a grid of soil nails bonded into the in situ ground. Numerical analyses have been conducted to examine the influence of soil nail orientations on the behaviour of the ground nail–facing system. The results suggest that the use of steeply inclined nails throughout the entire slope could avoid global instability, but could lead to significant slope movement especially when sliding failure prevails, for instance, due to interface liquefaction. The numerical analyses also demonstrate that if only subhorizontal nails are used, the earth pressure exerted on the slope facing may cause uplift failure of the slope cover. To overcome the shortcomings of using soil nails at a single orientation, a hybrid nail arrangement comprising nails at two different orientations is proposed. The numerical analyses illustrate that the hybrid nail arrangement would limit slope movement and enhance the robustness of the system.

Key words: soil nail, loose fill, slopes, static liquefaction, strain-softening, stability.

Résumé : Les clous de sol ont été utilisés pour améliorer les pentes de sols pulvérulents de qualité inférieure à Hong Kong. En raison de la possibilité de rupture en liquéfaction statique, le concept typique comprend une face de pente structurale ancrée par une grille de clous de sol liée dans le sol in situ. Des analyses numériques ont été réalisées pour examiner l'influence de l'orientation du clou de sol sur le comportement du système sol-clou-face. Les résultats suggèrent que l'utilisation de clous très inclinés dans la pente entière pourrait permettre d'éviter l'instabilité globale, mais pourrait entraîner des mouvements de pente significatifs particulièrement lors de rupture en glissement, par exemple, lors de liquéfaction à l'interface. Les analyses numériques ont aussi démontré que si des clous semi-horizontaux sont utilisés, la pression des terres exercée sur la face de la pente peut causer une rupture en soulèvement du recouvrement de la pente. Pour contourner les limitations lors de l'utilisation de clous de sol dans une seule orientation, un arrangement hybride de clous, comprenant des clous dans deux orientations, est proposée. Les analyses numériques illustrent que l'arrangement hybride de clous limiterait le mouvement de la pente et augmenterait la robustesse du système. [Traduit par la Rédaction]

Mots-clés : clou de sol, sol pulvérulent, pentes, liquéfaction statique, amollissement, stabilité.

Introduction

Loose fill slopes in Hong Kong

Prior to the establishment of the Geotechnical Control Office (renamed as Geotechnical Engineering Office in 1991) in 1977, many old loose fill slopes in Hong Kong were formed by end-tipping without proper compaction. The fill material, derived mainly from completely decomposed granitic or volcanic rocks, was deposited in layers with a low relative (or dry) density. Saturated granular material is contractive under shearing and may exhibit significant strain-softening upon shearing under undrained conditions. The sudden reduction of shear stress is termed undrained instability and is associated with the onset of flow liquefaction according to Murthy et al. (2007). As this type of flow liquefaction is triggered by static loading, the term "static liquefaction" has been used to describe the behaviour (e.g., Skopek et al. 1994; Yamamuro and Lade 1998). The static liquefaction behaviour of sands and silty sands has been widely studied, e.g., Lade (1992); Sasitharan et al. (1993); Pitman et al. (1994), Yamamuro and Lade (1997, 1998). Similar behaviour was observed in loosely compacted decomposed granite and volcanics (e.g., Law et al. 1997; Ng and Chiu 2003; Ng et al. 2004).

Static liquefaction of loose fill slopes has resulted in landslides with dire consequences in Hong Kong (Government of Hong Kong 1977). Wong et al. (1997) conducted a review of past rain-induced failures of loose fill slopes in Hong Kong and suggested that there are three major types of failure modes; namely, sliding, static liquefaction, and washout failure. The conventional method to upgrade substandard loose fill slopes in Hong Kong consists of excavating the top 3 m of the loose fill and re-compacting the excavated fill material or new filling material to an adequate standard, together with the provision of a drainage blanket at the base of the compacted fill. This method has proved to be effective in reducing the landslide risk associated with the three possible failure modes. Nonetheless, the method can be hazardous because heavy machinery is normally required to operate on slopes with a steep temporary cutting during construction in many heavily populated areas in Hong Kong. There are also environmental issues as tree felling is often necessary to enable excavation and re-compaction on the slopes.

Given the constraints of the 3 m re-compaction method, alternative schemes were explored to upgrade old loose fill slopes in Hong Kong. Soil nailing was identified as a potential solution capitalizing on the experience gained from its usage in upgrading

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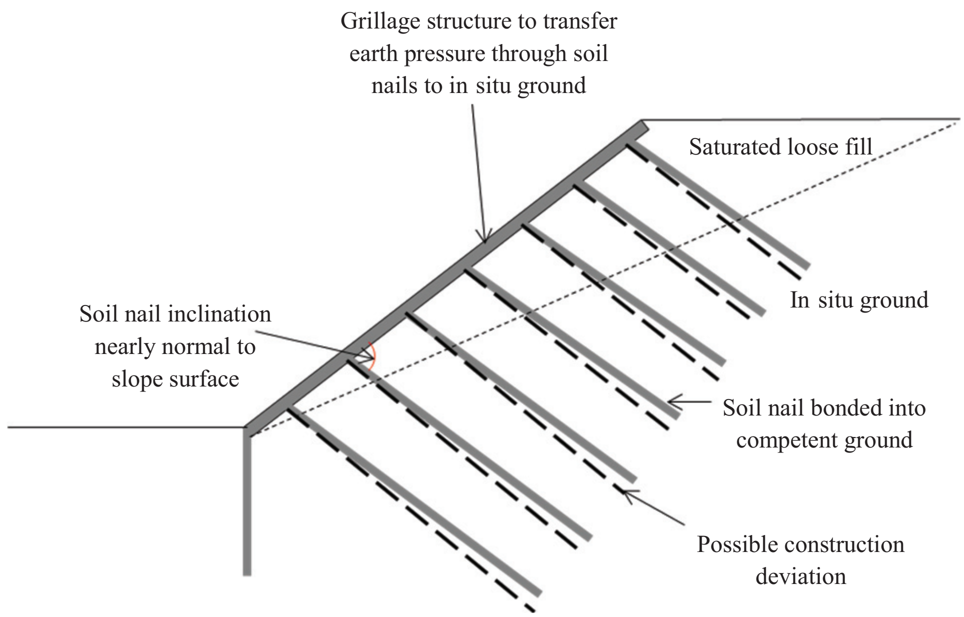
Fig. 1. Typical soil nail design in loose fill slopes in Hong Kong.



many existing cut slopes in Hong Kong. However, the suitability of using soil nails in loose fill, which is vulnerable to undrained strain-softening, had been controversial and generated many technical debates. In response to these concerns, research work has been conducted to shed light on the behaviour of loosely compacted decomposed rock and its interaction with soil nails (e.g., Cheuk et al. 2005). Based on the available research findings, HKIE (2003) suggested that the use of soil nails to upgrade loose-fill slopes is feasible and recommended a design methodology that is described in the following section. GEO (2003) provided further guidance on designing soil nails in loose fill slopes.

Soil nail design approach

The design method recommended in HKIE (2003) suggests that for design purposes it may be assumed that the loose fill has been subjected to sufficient straining and reached the critical state by the time nail forces are mobilized. With this assumption, the loose fill can be characterized by its critical-state undrained shear strength and it is not necessary to examine the rate of nail force mobilization vis-à-vis the rate of strain-softening.

Figure 1 shows the typical design arrangement adopted in Hong Kong. A key component of the design is the structural facing that connects all the soil nail heads together at the slope surface. When the loose fill liquefies, the earth pressure generated from the liquefied fill is resisted by the facing structure and is transferred to the in situ ground underneath the fill through the soil nails. The earth pressure is assumed to be zero at the slope crest and increases linearly towards the slope toe (i.e., triangular distribution). The continuous slope facing or grillage structure, anchored by soil nails at regular spacing, is similar to an anchored structure resisting earth pressure normal, or nearly normal, to the slope face. As a result, soil nails are constructed almost perpendicular to the slope surface at a relatively steep angle. Structural supports in the form of vertical nails are usually provided at the slope toe to absorb any unbalanced forces arising from possible construction deviation in the alignment of the soil nails.

To resist the earth pressure generated under the condition of “full liquefaction” (i.e., the entire loose fill undergoes undrained strain-softening), the most efficient nail arrangement is to have the nails nearly perpendicular to the slope facing, rendering the soil nails steeply inclined. This is particularly the case for fill slopes that normally have a gentle slope angle in the range of 30°–45° to the horizontal (Sun 1999). However, the steep orientation may reduce the effectiveness of the nails if stabilizing forces are to be mobilized from relative movement between the nail and the surrounding soil. Previous studies have revealed that an increase in soil nail inclination would decrease the tensile forces mobilized in the nails, in turn reducing the stabilizing effect, and compressive forces may even be mobilized in steeply inclined nails (Jewell and Wroth 1987; Shiu and Chang 2006). The steep nail orientation leads to the concern as to whether sufficient stabilizing forces could be mobilized if the mode of the landslide involves sliding without static liquefaction, or when static liquefaction is confined to a thin layer leading to a deformation mechanism resembling a sliding failure — a scenario denoted as “interface liquefaction.” The potential for interface liquefaction is demonstrated by the 1972 Sau Mau Ping landslide in Hong Kong, which led to 71 fatalities (Yang et al. 2008).

Objectives of the study

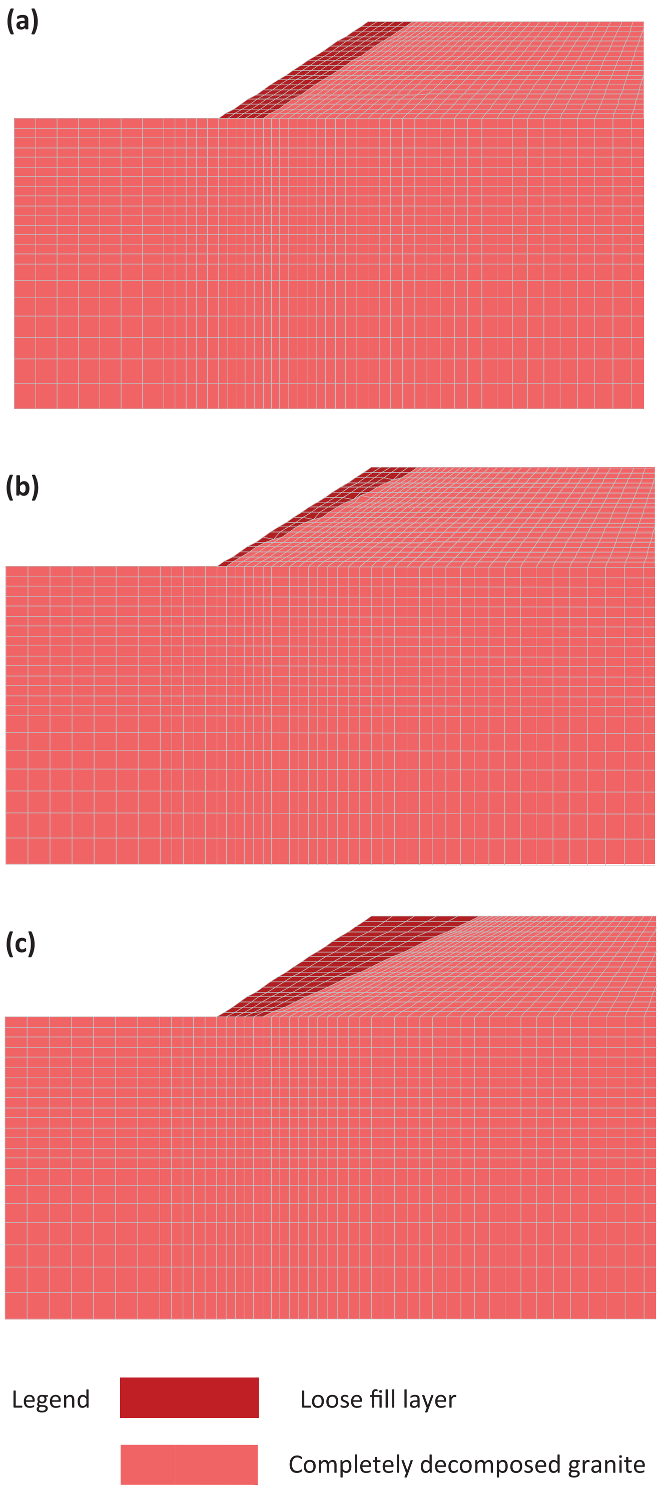
The paper presents an investigation into the stabilizing mechanisms of soil nails in loose fill slopes. A series of numerical analyses have been conducted using two-dimensional finite difference computer program FLAC (version 4.0). The objectives of these analyses are to examine the nail force mobilization mechanisms for steeply inclined soil nails and to optimize the inclinations of the soil nails. Details of the numerical analyses are presented in this paper. Based on the analysis results, new design recommendations for enhancing the robustness of upgrading loose fill slopes by soil nails are given.

Numerical analysis

Model geometry

The numerical model representing the benchmark case considers a 10 m high, 34° (i.e., 1:1.5) loose fill slope with 3 m uniform depth of loose fill overlying completely decomposed granite (CDG) (see Fig. 2). The assumed ground profile simplifies the highly variable nature of a loose fill profile originated from end-tipping. Two different types of tapered fill geometry have also been considered as a parametric study (Fig. 2). The benchmark model consists of seven rows of soil nails that are connected together by a structural

Fig. 2. Finite difference grids adopted in the numerical analyses: (*a*) uniform 3 m fill; (*b*) thin tapered fill; (*c*) thick tapered fill.



facing on the slope surface. The bottom boundary is restrained vertically and horizontally, and the vertical boundaries on both sides are allowed to displace vertically only.

Three different nail arrangements as shown in Fig. 3 have been examined. The first nail arrangement consists of steeply inclined soil nails that are perpendicular to the slope surface. This represents the typical nail arrangement in current practice. In the second case, the nails are subhorizontal (i.e., inclined at 20° to the horizontal), which is a typical nail inclination in cut slopes. The third nail arrangement, denoted as a hybrid nail arrangement, adopts a combination of subhorizontal and steeply inclined nails. The nail lengths are determined using the procedures described in Appendix A. The adopted nail lengths for the benchmark cases (i.e., 10 m high slopes) are shown in Fig. 3. In some cases, the presence of a 0.5 m deep embedded toe wall is also considered (Fig. 3).

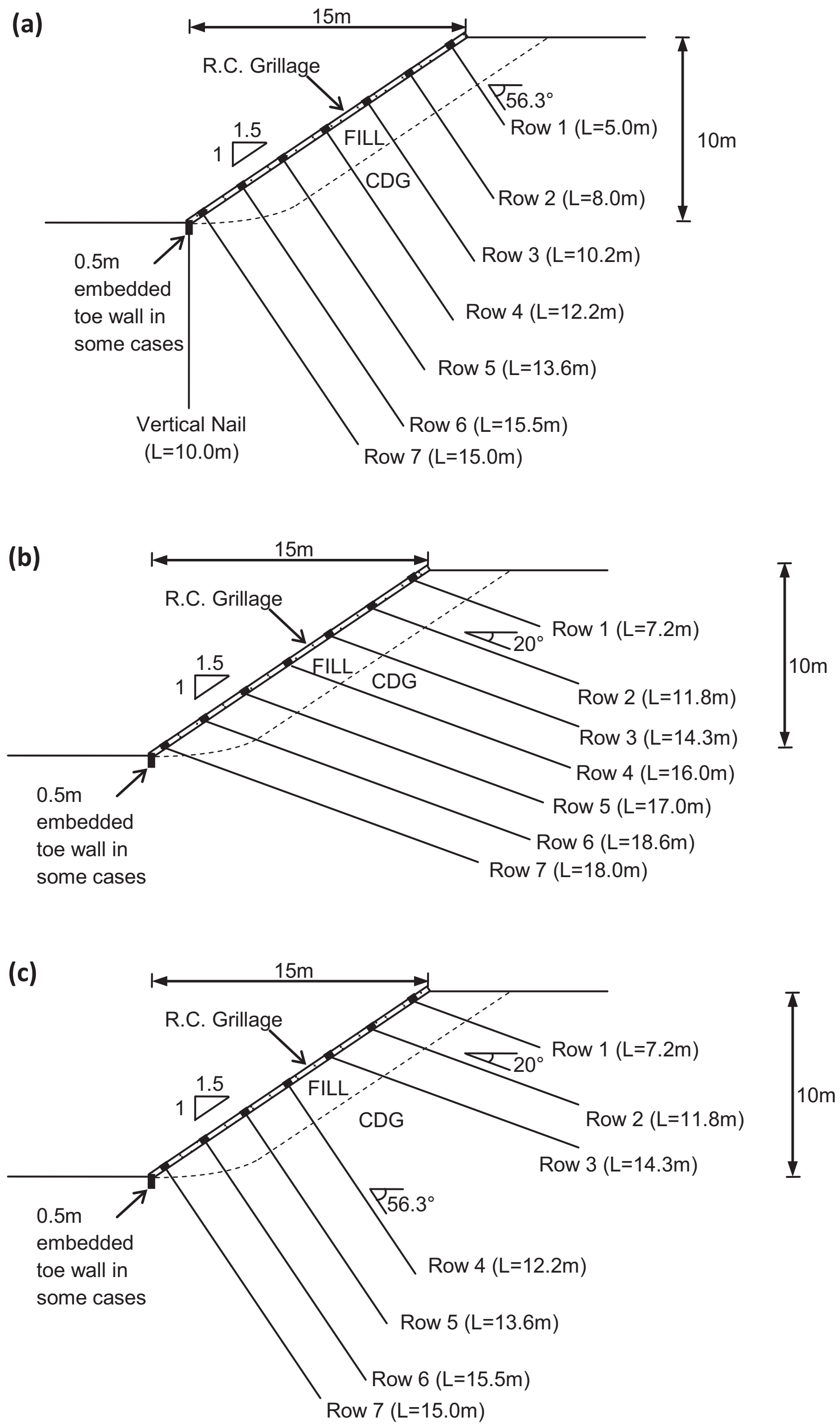
Constitutive models and model parameters

Both the loose fill and CDG underneath were modelled as an elastic – perfectly plastic soil continuum with a Mohr–Coulomb failure criterion. The adopted soil parameters are summarized in Table 1. Before liquefaction, the shear (G) and bulk (K) moduli of the loose fill are calculated based on an assumed Young's modulus (E) of 5 MPa and a Poisson ratio (ν) of 0.3. The drained shear strength is characterized by typical effective strength parameters for loose fill materials (i.e., cohesion parameter $c' = 5$ kPa and friction angle $\phi' = 35°$). Static liquefaction of the loose fill was modelled by a gradual reduction of the shear strength. A total stress approach was adopted to mimic the low shear strength as a result of static liquefaction. This simplified approach ignores the initiation of the undrained strain-softening, and was considered conservative as initial mobilization of nail forces at small deformation was not taken into account. The friction angle (ϕ') and dilation angle (ψ) are taken as zero, while the c' parameter corresponding to the critical-state undrained shear strength is calculated assuming $c_u = 0.13\,\sigma'_v$ (where σ'_v is the in situ vertical effective stress; see Appendix A). A large undrained bulk modulus (K) of 10 GPa is assumed to mimic the constant volume condition upon liquefaction. The corresponding shear modulus (G) is determined based on the same Young's modulus (E) of 5 MPa.

Soil nails at 1.5 m centre-to-centre spacing were modelled as cable elements in the analyses, which are elastic elements with axial (tension or compression) capacity only. The adopted model parameters are tabulated in Table 2. They are determined based on a 25 mm diameter steel bar installed in a 100 mm diameter drilled hole. The cross-sectional area (A) of the cable element is determined from the geometry of the grouted nail (i.e., outer diameter of 100 mm). The Young's modulus (E) is calculated from that of a high yield steel reinforcement, and divided by 1.5 m to take account of the horizontal spacing of the soil nails in the plane-strain model. The contribution from the grout material surrounding the steel reinforcement has been conservatively ignored. The nail perimeter (P) is used to determine the mobilized shear resistance along the soil–nail interface. It is therefore calculated from the outer diameter of the grouted nail (i.e., 100 mm), and divided by the horizontal spacing of 1.5 m.

Due to possible "flow" behaviour of the liquefied loose fill around the soil nails, structural nodes have been omitted along the portion of the nails located within the loose fill body (Fig. 4). This "decoupling" approach is conservative as it ignores the possible interaction between the soil nails and liquefied loose fill. It is therefore only necessary to specify the interface properties for the portion of the soil nails embedded in the in situ ground (e.g., CDG). The behaviour along the soil–nail interface is governed by the properties of the shear coupling springs at the structural nodes of the cable elements. The stiffness of the shear coupling spring (K_s) is calculated based on the shear modulus of the surrounding soil and an assumed thickness of the shear zone, which can be difficult to estimate. In this study, a comparison has been made between the results of laboratory pull-out tests and the numerical simulation of a pull-out test. A scaling factor of 10 is found to be appropriate to match the pull-out test results, which implies a shear zone of approximately 0.1 m in thickness. The shear spring stiffness (K_s) is therefore calculated by

Fig. 3. Soil nail arrangements considered in the numerical analyses: (*a*) steeply inclined nail arrangement; (*b*) subhorizontal nail arrangement; (*c*) hybrid nail arrangement.



(1) $$K_s = \frac{10G\pi D}{S}$$

where K_s is the stiffness of the shear coupling spring, G is the shear modulus of the surrounding soil, D is the diameter of the grouted soil nail, and S is the horizontal spacing of the soil nails.

To calculate K_s from eq. 1, the G value has been taken as the shear modulus of CDG (i.e., 9615 kPa). The maximum frictional resistance that can be developed along the soil–nail interface is dictated by the cohesive strength (C_s) and the friction coefficient (ϕ_s) of the shear coupling spring. The cohesive strength (C_s) is calculated from a cohesion parameter (c') of 5 kPa, while the friction coefficient (ϕ_s) is taken as 35°, which is the same as the surrounding CDG.

The slope facing is modelled by pile elements in the analyses. The model parameters are tabulated in Table 3. The structure being modelled is a grillage consisting of 600 mm wide × 300 mm deep reinforced concrete beams at 1.5 m horizontal–vertical

Table 1. Model parameters for soils.

	Input value			
		Loose fill		
Parameter	In situ soil (CDG)	Before liquefaction	Saturated before being liquefied	After liquefaction
Shear modulus, G (kPa)	9615	1923	1923	1667
Bulk modulus, K (kPa)	20 833	4167	4167	1×10^7
Density, ρ (Mg/m^3)	1.8	1.8	1.8	1.8
Cohesion parameter, c' (kPa)	5	5	0	0.13 σ'_v
Friction angle, ϕ' (°)	35	35	26	0
Dilation angle, ψ (°)	0	0	0	0

Table 2. Model parameters for soil nails.

Type of parameter	Parameter	Input value
Structural parameter	Area, A (m^2)	7.85×10^{-3}
	Perimeter, P (m^2/m/m)	0.209
	Young's modulus, E (kPa/m)	8.33×10^6
	Tensile yield strength, Y_t (kN/m)	1×10^7
	Compressive yield strength, Y_c (kN/m)	1×10^7
Shear coupling spring	Stiffness, K_s (kPa/m)	20 138
	Cohesive strength, C_s (kN/m/m)	1.047
	Friction coefficient, ϕ_s (°)	35

Fig. 4. Decoupling of soil–structure interaction for cable elements in liquefied loose fill.

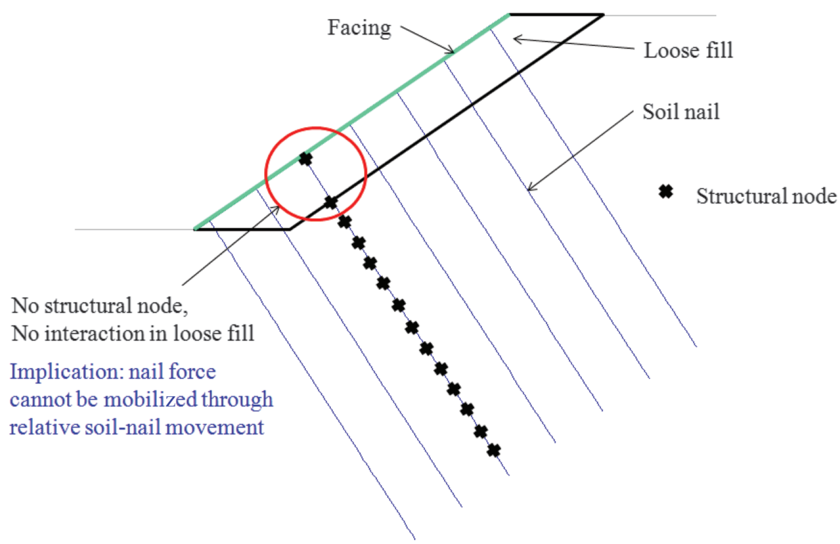


Table 3. Model parameters for slope facing.

	Input value	
Parameter	Before liquefaction	After liquefaction
Structural parameters		
Area, A (m^2)	0.18	0.18
Perimeter, P (m^2/m/m)	0.533	0.533
Density, ρ (Mg/m^3)	2.45	2.45
Young's modulus, E (kPa/m)	1.48×10^7	1.48×10^7
Second moment of area, I (m^4)	1.35×10^{-3}	1.35×10^{-3}
Interface between slope facing and soil		
Shear coupling spring		
Stiffness, K_s (kPa/m)	10 250	8890
Cohesive strength, C_s (kN/m/m)	2.7	1.6
Frictional coefficient, ϕ_s (°)	35	0
Normal coupling spring		
Stiffness, K_n (kPa/m)	2665	2665
Cohesive strength, C_n (kN/m/m)	30 000	30 000
Friction coefficient, ϕ_n (°)	0	0

Table 4. Model parameters for toe embedment.

Parameter	Input value
Structural parameters	
Area, A (m^2)	0.5
Perimeter, P (m^2/m/m)	2.0
Density, ρ (Mg/m^3)	2.45
Young's modulus, E (kPa/m)	2.22×10^7
Moment of inertia, I (m^4)	1.04×10^{-2}
Interface between toe embedment and soil	
Shear coupling spring	
Stiffness, K_s (kPa/m)	192 300
Cohesive strength, C_s (kN/m/m)	10
Frictional coefficient, ϕ_s (°)	35
Normal coupling spring	
Stiffness, K_n (kPa/m)	25 000
Cohesive strength, C_n (kN/m/m)	33
Friction coefficient, ϕ_n (°)	0

Table 5. Summary of numerical analyses for full liquefaction condition.

Analysis No.	Fill geometry	Slope height (m)	Slope angle (°)	Nail arrangement	Toe condition	Maximum soil displacement (mm)	Maximum structural displacement (mm)
1	Uniform	10	34	Steeply inclined	No toe fixity	381	147
2	Uniform	10	34	Steeply inclined	Connected to 10 m long vertical nail	422	85
3	Uniform	10	34	Steeply inclined	Connected to 0.5 m embedded toe wall	404	100
4	Uniform	10	34	Subhorizontal	Connected to 0.5 m embedded toe wall	2688	815
5	Uniform	10	34	Hybrid	No toe fixity	350	35
6	Uniform	10	34	Hybrid	Connected to 0.5 m embedded toe wall	344	43
7	Uniform	20	34	Steeply inclined	No toe fixity	1981	127
8	Uniform	20	34	Hybrid	No toe fixity	1392	81
9	Uniform	10	40	Steeply inclined	No toe fixity	307	141
10	Uniform	10	40	Hybrid	No toe fixity	284	48
11	Thin tapered	10	34	Steeply inclined	No toe fixity	300	132
12	Thin tapered	10	34	Hybrid	No toe fixity	283	16
13	Thick tapered	10	34	Steeply inclined	No toe fixity	267	136
14	Thick tapered	10	34	Hybrid	No toe fixity	238	42

Table 6. Summary of numerical analyses for interface liquefaction condition.

Analysis No.	Fill geometry	Slope height (m)	Slope angle (°)	Nail arrangement	Toe condition	Maximum soil displacement (mm)	Maximum structural displacement (mm)
15	Uniform	10	34	Steeply inclined	No Toe Fixity	262	149
16	Uniform	10	34	Steeply inclined	Connected to 10 m long vertical nail	267	110
17	Uniform	10	34	Steeply inclined	Connected to 0.5 m embedded toe wall	286	110
18	Uniform	10	34	Hybrid	No toe fixity	164	52
19	Uniform	10	34	Hybrid	Connected to 0.5 m embedded toe wall	172	43
20	Uniform	20	34	Steeply inclined	No toe fixity	569	302
21	Uniform	20	34	Hybrid	No toe fixity	516	62
22	Thin tapered	10	34	Steeply inclined	No toe fixity	114	117
23	Thin tapered	10	34	Hybrid	No toe fixity	32	21
24	Thick tapered	10	34	Steeply inclined	No toe fixity	194	151
25	Thick tapered	10	34	Hybrid	No toe fixity	109	51

spacing. The interaction between the slope facing and the loose fill is controlled by the shear and normal coupling springs at the nodal points. Before liquefaction, the stiffness of the shear coupling spring (K_s) is determined from eq. 1 with G being taken as that of the loose fill before liquefaction (i.e., G = 1923 kPa). The cohesive strength (C_s) is calculated from a cohesion parameter (c') of 3 kPa and the friction coefficient (ϕ_s) is taken as 35°. Upon liquefaction, the stiffness of the shear coupling spring K_s is reduced to match the reduction in the shear modulus of the liquefied loose fill.

In the analyses where an embedded toe wall is present to support the slope facing, the embedded wall is modelled as pile elements and is assumed to be connected to the base of the slope facing. The model parameters for the embedded toe wall are tabulated in Table 4. The embedded toe wall being modelled is a 0.5 m wide × 0.5 m deep continuous reinforced concrete toe wall. Assuming that the toe wall is embedded in competent ground, the stiffness parameters of the shear and normal coupling springs can be determined from the properties of CDG.

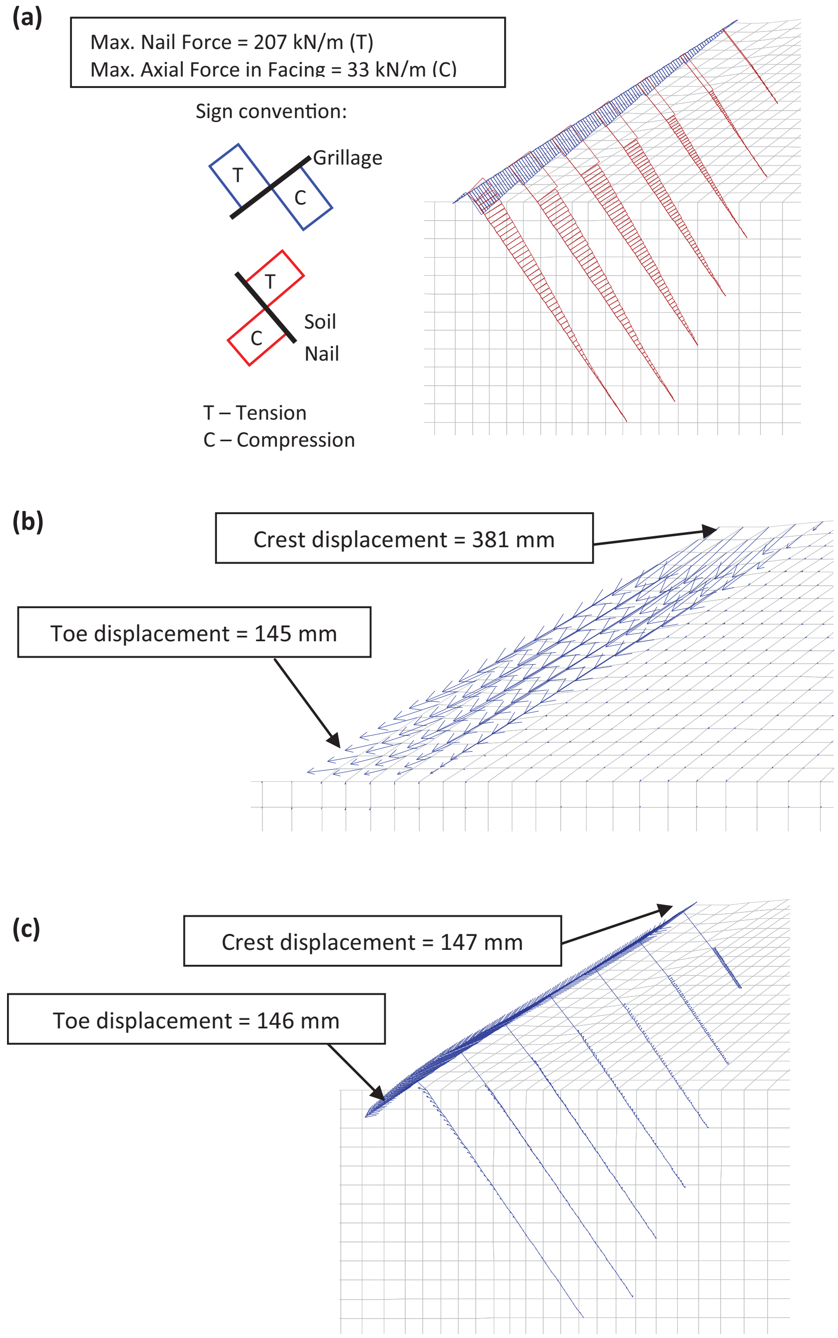
Modelling procedure

The modelling procedure in each analysis consisted of three main stages. In the first stage, initial stresses were generated by adopting the model parameters corresponding to the state before liquefaction (refer to Table 1). The initial stresses in the in situ ground (i.e., CDG) were first calculated assuming that the loose fill was not present. The geometry of the loose fill was then built up layer by layer to mimic the deposition of loose fill. The second stage mimicked the construction of soil nails and slope facing, as well as the toe embedment if applicable. The locations and material properties of the soil nails and grillage facing were specified, and the model was solved for equilibrium. All the displacements incurred during the first and second stages were reset to zero before the third stage began.

The third stage modelled static liquefaction of the loose fill. The ϕ' value of the loose fill that was assumed to be saturated and liquefied was reduced gradually from 35° to 0° in steps. In the last step when the ϕ' value was reduced from 10° to zero, the c parameter was changed to the critical-state undrained shear strength ($c_u = 0.13\ \sigma'_v$) simultaneously. The resulting c_u ranged from 3–6 kPa. In addition, the shear modulus was reduced slightly to reflect the undrained conditions (refer to Table 1). The static equilibrium solution was obtained in each intermediate step. The mobilized nail forces and deformation at the final step were examined. The matric suction initially present in the loose fill has not been considered in this study. This assumption conservatively underestimates the mobilized nail force, especially for subhorizontal nails, as small deformation is expected to be triggered during the saturation process due to infiltration.

Two major loading scenarios were considered in the numerical analyses. The first scenario assumed full liquefaction in which the entire fill body liquefied and reached the critical-state undrained

Fig. 5. Predicted nail force distribution and deformations of steeply inclined nail arrangement under full liquefaction: (*a*) nail force distribution, (*b*) soil displacement vectors, and (*c*) structural displacement vectors.



shear strength. The second loading scenario assumed that only a 0.5 m thick fill layer liquefied (i.e., interface liquefaction). The saturated fill above the liquefied layer was modelled by drained parameters ($c' = 0$ kPa and $\phi' = 26°$). This is to simulate a sliding failure corresponding to liquefaction occurring within a relatively thin layer of loose fill.

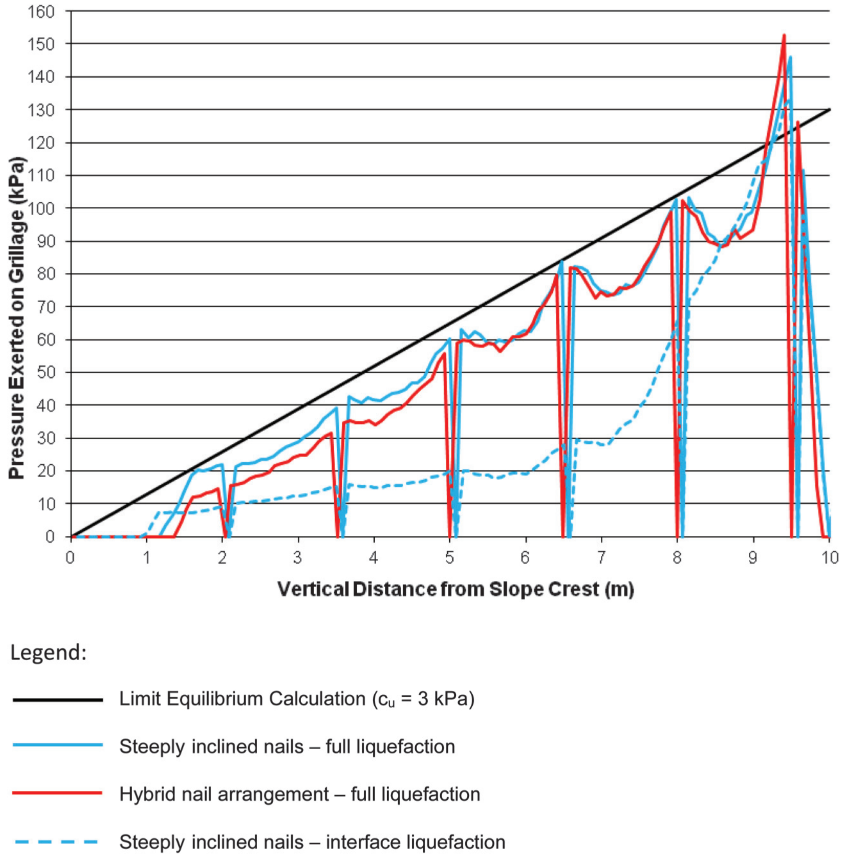
Model conditions

A total of 25 analyses were conducted. The model conditions are summarized in Tables 5 and 6. The three benchmark cases that examine the effect of nail orientations under the full liquefaction loading condition are analyses 1, 4, and 5. For the interface liquefaction scenario, the performance of steeply inclined nails and the hybrid nail arrangement is compared in analyses 15 and 18. The performance of subhorizontal nails under interface liquefaction was not considered due to the nonconvergence of the analysis that mimicked a nailed slope subjected to full liquefaction. A comprehensive parametric study was carried out to investigate the influence of slope height, slope angle, fill geometry, and toe fixity conditions on the key observations obtained from the benchmark cases. The maximum predicted slope deformation is also tabulated in Tables 5 and 6 for direct comparison. Detailed discussion is presented below.

Steeply inclined nails

The results of analysis 1, which represent the typical behaviour of a loose fill slope upgraded by steeply inclined nails under full liquefaction, are shown in Fig. 5. The numerical analysis results

Fig. 6. Earth pressure exerted on slope facing.



suggest that, when the entire fill body liquefies, sufficient nail forces can be mobilized to maintain overall stability (Fig. 5*a*). Fig. 6, which plots the normal stresses exerted on the slope facing, suggests that the tensile forces in the steeply inclined nails are mobilized by the unbalanced earth pressure acting on the slope cover. The nail arrangement therefore satisfies the design objective of sustaining the earth pressure exerted on the structural facing upon liquefaction of the loose fill. The distribution of earth pressure determined from the numerical analyses is triangular in shape, which is a direct result of not including any nail–ground interaction within the fill layer. The triangular distributed earth pressure in Fig. 6 is found to be comparable with that determined in the limit equilibrium calculation, which assumes the undrained shear strength of loose fill to be 3 kPa. Despite the fact that overall stability can be maintained by the mobilized nail forces, a large slope and structural deformations (Figs. 5*b* and 5*c*) are triggered. The deformation pattern suggests that the ground nail–facing system has very limited structural rigidity to counteract the sliding movement of the liquefied fill mass. Sensitivity analyses demonstrate that the deformation could be reduced, to some extent, by incorporating a structural element (e.g., vertical nails or embedded toe wall) at the slope toe.

The major concern regarding the use of steeply inclined nails is that nail forces may not be mobilized effectively in the event of a sliding failure (e.g., interface liquefaction) and that the orientation of the nails is not favourable for counteracting sliding failure. As illustrated in Fig. 7, if the soil nails are perpendicular to the sliding motion, the driving force is only resisted by the soil shear strength along the slip surface; any mobilized tensile forces in the nails would not contribute to counteract sliding failure under undrained conditions. The soil nails need to bend to such an extent that the component of the nail forces along the sliding direction as shown in Fig. 7*b* can be mobilized.

Fig. 7. Steeply inclined nails under sliding failure: (*a*) force diagram for original nail configuration and (*b*) force diagram for deformed nail configuration.

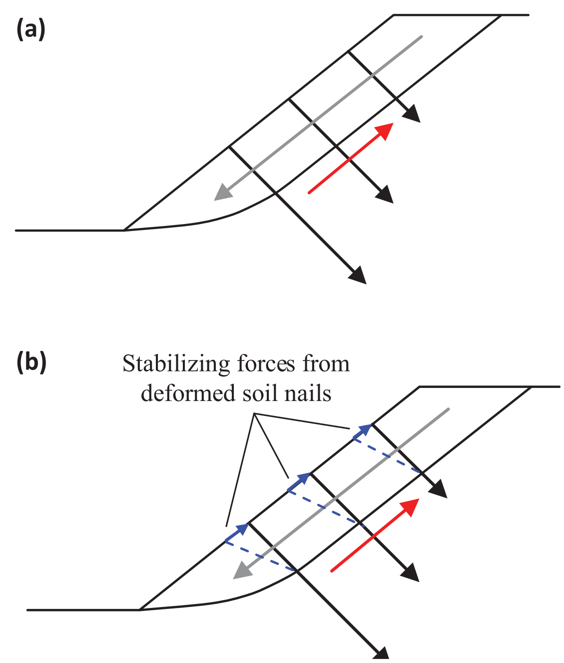
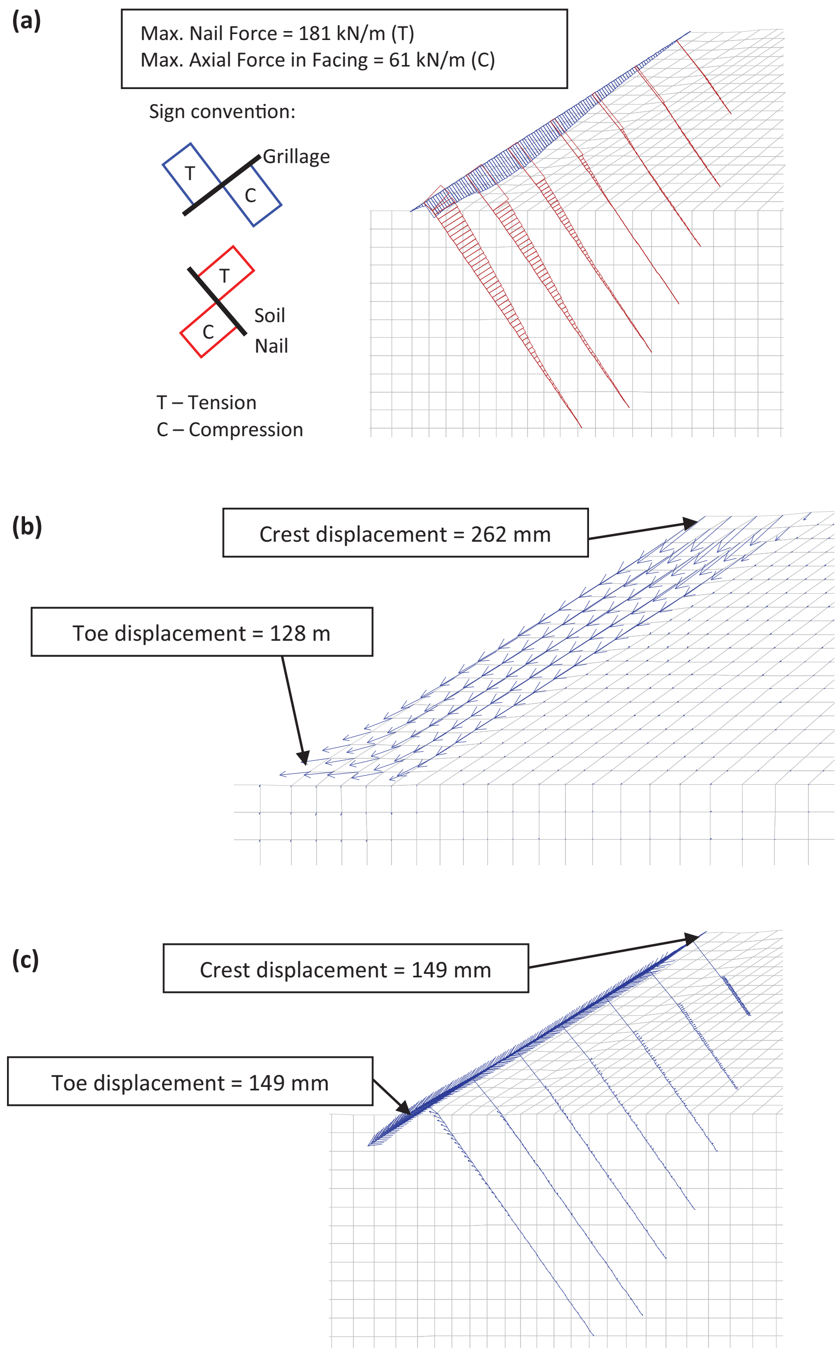


Figure 8 presents the numerical analysis results for steeply inclined nails under interface liquefaction (i.e., analysis 15). Under interface liquefaction, the unbalanced earth pressure acting on the grillage facing is reduced (Fig. 6). The mobilized nail forces

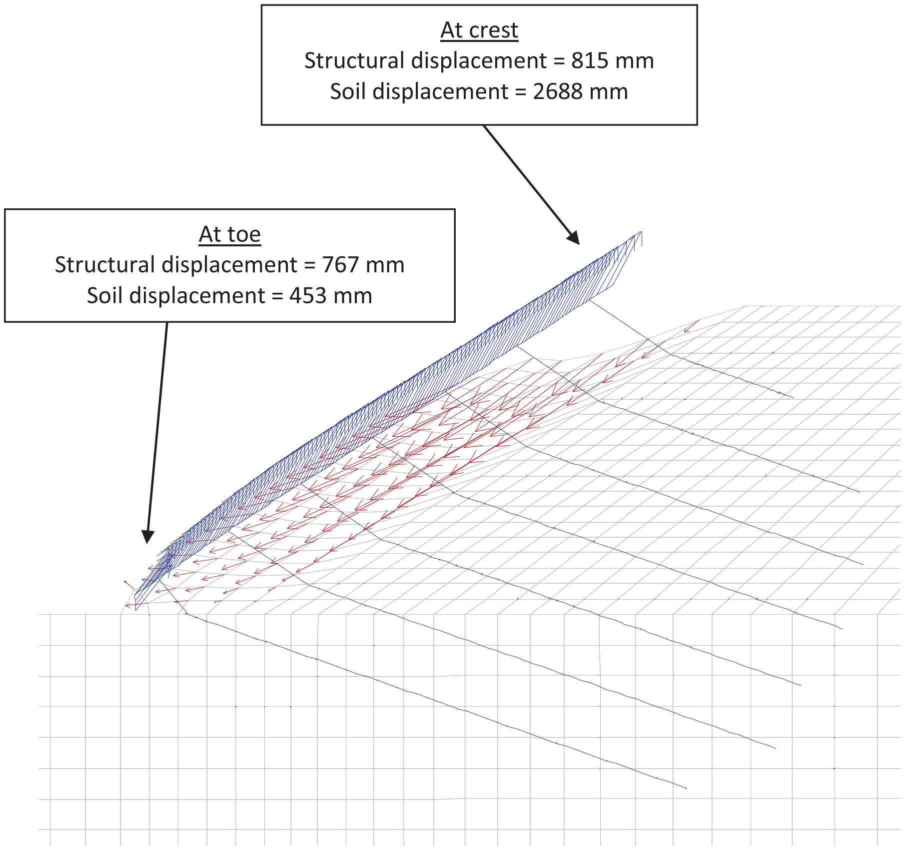
Fig. 8. Predicted nail force distribution and deformations of steeply inclined nail arrangement under interface liquefaction: (*a*) nail force distribution, (*b*) soil displacement vectors, and (*c*) structural displacement vectors.



predicted by FLAC are much lower, compared to the case of full liquefaction, especially in the soil nails near the slope crest (Fig. 8*a*). Although numerical convergence (i.e., overall system stability) could be achieved in the numerical model, the bending of the soil nails is prominent. As in the case of full liquefaction, large soil and structural deformations are triggered along the potential sliding direction due to limited structural rigidity of the ground nail–facing system. Whilst the unbalanced earth pressure acting on the grillage facing is reduced, the bending of the soil nails towards the sliding direction to gain sufficient stabilizing force against sliding failure has given rise to large structural facing and soil deformations (Figs. 8*b* and 8*c*).

Despite the large deformation, steeply inclined nails still serve to improve the stability of the system for the selected scenarios considered in the analyses. As the instability condition in the event of interface liquefaction is less severe than that of full liquefaction, and given the reduced brittleness of the system, the risk of uncontrolled failure could be reduced even if steeply inclined soil nails are used, albeit the overall stability of the system has to rely on the large deformation behaviour of the system in the cases analysed. Given the low bending stiffness of the soil nails, the bending action may not affect the structural integrity of the system, but may incur considerable structural facing movement, especially when the free lengths of the soil nails are large (i.e., in a thick fill deposit).

Fig. 9. Predicted failure mechanism for subhorizontal nail arrangement under full liquefaction.



Subhorizontal nails

Subhorizontal nails are effective in countering sliding failures in cut slopes. The most effective orientation would be for the nail reinforcement to align in the tensile-strain direction of the soil, implying a nail inclination of 10° to 20° for typical slope angles. However, the numerical analyses conducted in the present study show that, if only subhorizontal (20°) nails are used in the loose-fill slope, the system is ineffective in resisting uplift of the grillage facing and therefore can not maintain overall stability in the case of full liquefaction.

As shown in Fig. 9, which presents the predicted movement in analysis 4, the movement of the grillage facing is primarily upwards if subhorizontal nails are used throughout the slope. This uplift of the grillage facing is caused by the upward components of the nail forces as tensions are mobilized in the nails upon liquefaction of the loose fill. The upward movement of the grillage facing creates local instability at the slope toe, which allows the liquefied loose fill to "flow" through the gap between the grillage facing and the slope surface. This leads to very large soil deformation, and is also accompanied by the bending of the soil nails in the upward direction as shown in Fig. 9.

Hybrid nail arrangement

The discussion presented above clarifies the shortcomings of using soil nails at a single orientation throughout a loose fill slope that may be vulnerable to two different failure mechanisms: liquefaction and sliding. In this study, the potential merit of using a hybrid nail arrangement comprising soil nails at two different inclinations has been examined.

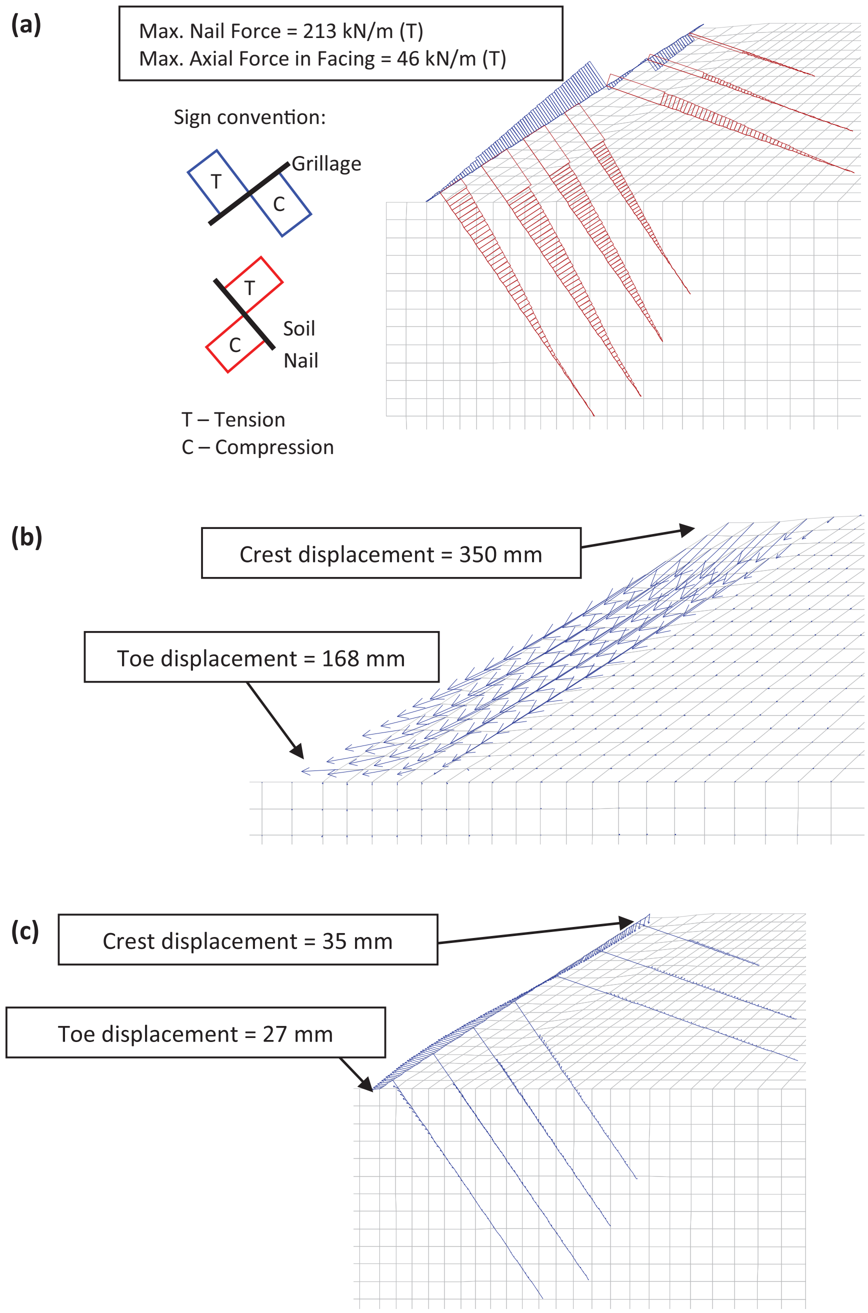
Uniform fill geometry

The results of analyses 5 and 18, which represent typical behaviour of a loose fill slope upgraded by soil nails at two orientations, are shown in Figs. 10 and 11 for the case of full liquefaction and interface liquefaction, respectively. The numerical analyses show that a hybrid nail arrangement incurs smaller deformation under both the full and interface liquefaction failure modes, as compared to the steeply inclined nail arrangement. Under full liquefaction, the nail forces (Fig. 10*a*) are mobilized effectively at much smaller slope and structural deformation (Figs. 10*b* and 10*c*) even when toe fixity is absent. This is due to the increase in structural rigidity of the system along the sliding direction. In the case of interface liquefaction, the unbalanced earth pressure acting on the grillage facing is much reduced, leading to smaller mobilized nail forces. The smaller soil and structural deformations (Figs. 11*b* and 11*c*) for the hybrid nail arrangement can also be attributed to the effective mobilization of nail forces in the subhorizontal nails near the upper part of the slope (Fig. 11*a*). The numerical analysis results for other slope heights and slope angles in the parametric study also show similar observations: that the deformation of the system is much reduced when the hybrid nail arrangement is adopted (refer to Tables 5 and 6).

Influence of fill geometry

The ground nail–facing interaction mechanisms in tapered fill geometry (i.e., fill thickness decreases from slope crest towards slope toe) have been examined as part of a parametric study. Figure 12 presents the predicted deformation pattern for the case of a thin tapered fill (i.e., analyses 12 and 23). The predicted failure mechanism in the event of full liquefaction involves only the top part of the fill body and does not extend to the slope toe (Fig. 12*a*). The earth pressure exerted on the grillage facing is therefore

Fig. 10. Predicted nail force distribution and deformations of hybrid nail arrangement under full liquefaction: (*a*) nail force distribution, (*b*) soil displacement vectors, and (*c*) structural displacement vectors.



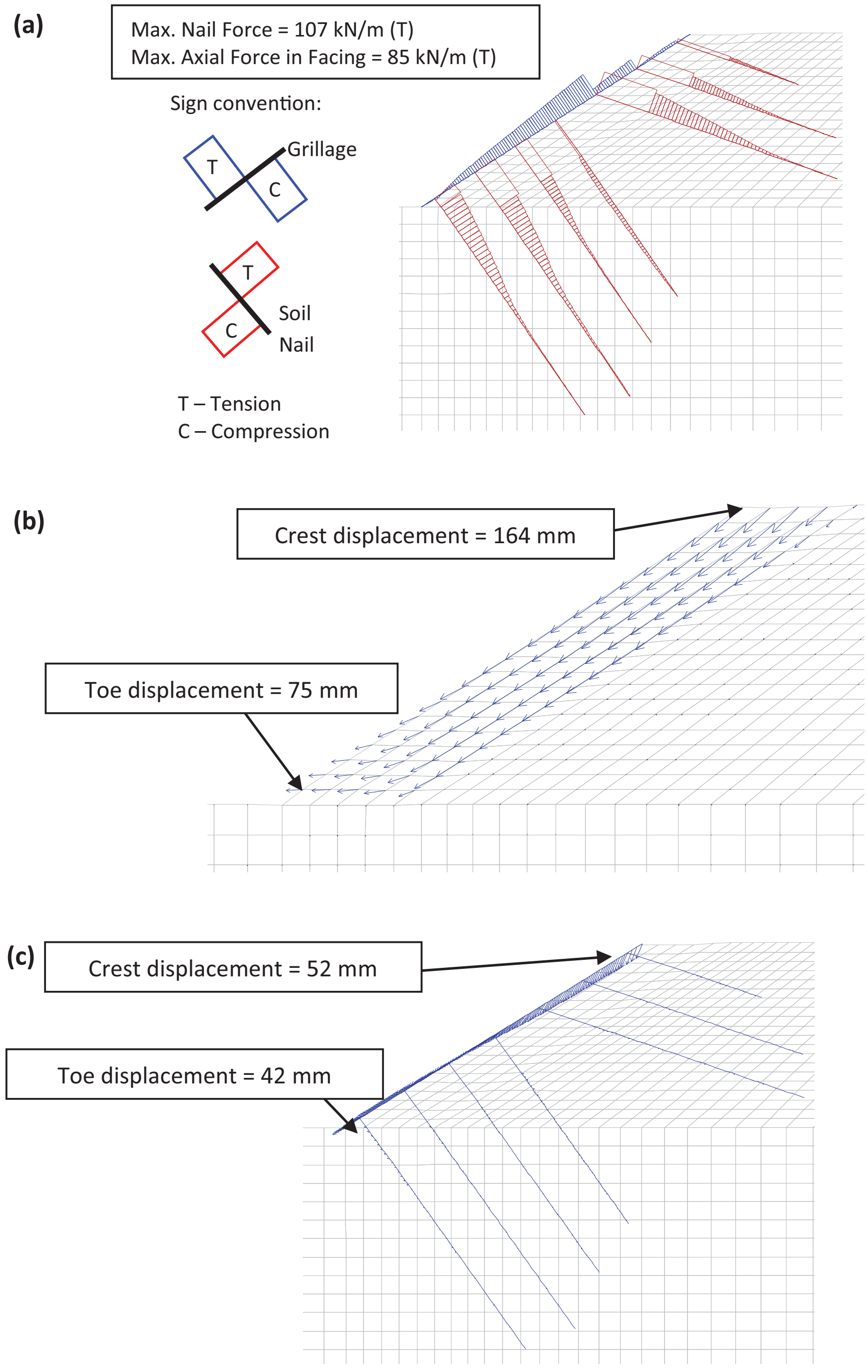
smaller when compared with that in a uniform fill body with the same slope height. The distribution of earth pressure remains triangular in shape, increasing from the slope crest to the lowest point of the failure mass, suggesting that the current design approach of assuming a triangular stabilizing surface pressure is appropriate. For interface liquefaction (Fig. 12*b*), the slip surface in a tapered fill body is gentler when compared with that in a uniform fill. This implies that even where steeply inclined nails are used, the nail orientation is not exactly perpendicular to the sliding direction, and there would be a small component of nail force that directly resists the sliding motion. This is a less critical scenario as far as stability condition is concerned. Nonetheless, the hybrid nail arrangement significantly reduced the mobilized deformation (refer to Table 6).

For the case of a thick tapered fill, the observations are generally similar to those of a uniform fill except that the failing soil mass has a larger extent and a slightly gentler sliding surface. Much smaller deformations are mobilized when the hybrid nail arrangement is adopted for both full liquefaction and interface liquefaction conditions (refer to Tables 5 and 6).

Discussion

Under normal circumstances, the tensile force developed in a soil nail originates from the bond resistance in the passive zone and is balanced by the shear resistance along the soil–nail interface in the active zone together with the bearing pressure at the nail head. In soft soil, like the liquefied loose fill considered in this

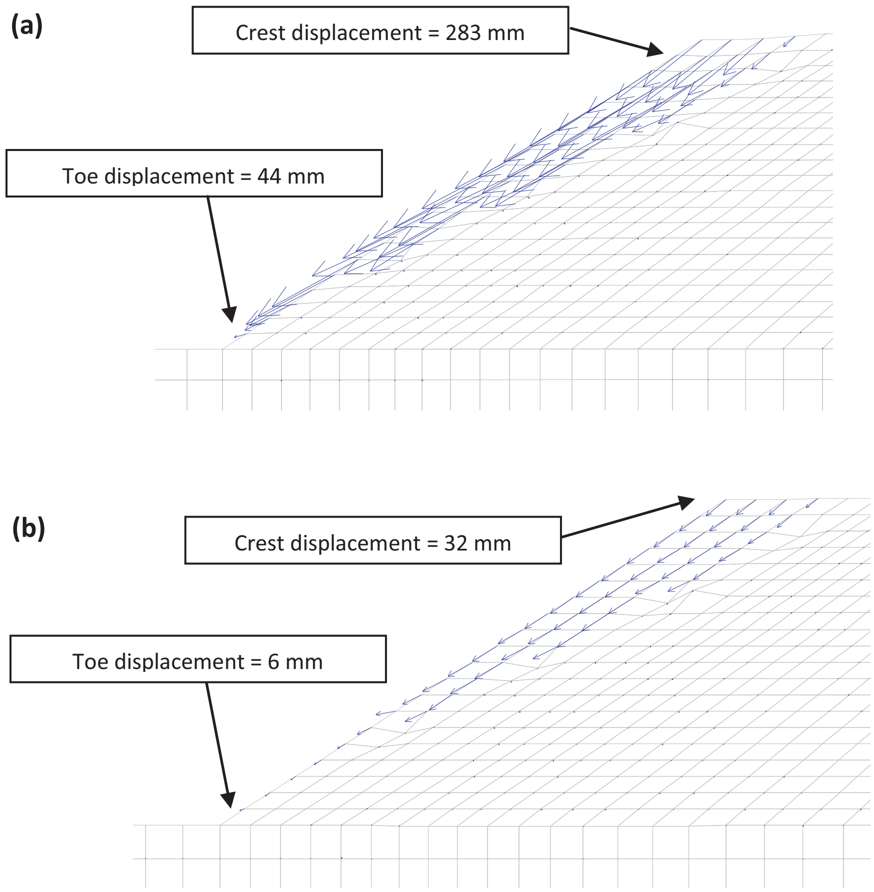
Fig. 11. Predicted nail force distribution and deformations of hybrid nail arrangement under interface liquefaction: (*a*) nail force distribution, (*b*) soil displacement vectors, and (*c*) structural displacement vectors.



study, the bond resistance developed in the active zone is limited. This gives rise to the need to provide a continuous structural facing to resist the earth pressure generated from the failing soil mass such that the bond resistance in the passive zone could be mobilized. The working principle therefore becomes more like a passive anchor. With limited bond resistance in the active zone, nails that are nearly perpendicular to the slope face are effective in resisting the earth pressure acting on the structural facing, but would cause large slope deformation due to the limited structural rigidity of the ground nail–facing system along the potential sliding direction. Although a sliding mechanism initiated from interface liquefaction may represent a less critical loading scenario, the slope deformation required to mobilize sufficient stabilizing force is also excessive due to the mechanism of generating the tension forces in the soil nails.

The numerical analyses conducted in this study suggest that providing a hybrid nail arrangement with some soil nails at a gentler orientation and some steeply inclined could reduce the overall deformations. The presence of the subhorizontal nails in the upper part of the slope facilitates early development of stabilizing nail forces at small deformation and enhances the rigidity of the system along the potential sliding direction; in this case, no additional fixity would be required at the slope toe. The steeply inclined nails near the bottom part of the slope facilitates effective force mobilization when an unbalanced earth pressure is

Fig. 12. Predicted deformation patterns in nonuniform (thin tapered) fill adopting hybrid nail arrangement: (*a*) full liquefaction and (*b*) interface liquefaction.



exerted on the slope facing upon liquefaction of the loose fill material.

The numerical analyses conservatively ignored any bond resistance that could be developed in the active zone. The numerical solutions reflect the ultimate condition whereby the loose fill has reached the large-strain critical-state undrained shear strength — the situation assumed in the design procedure. In reality, some bond resistance could be developed in the active zone at small slope deformation when loading due to rainfall infiltration has not yet reached a critical level and therefore undrained strain-softening has not taken place in the loose fill. This requires the soil nails to be aligned in the direction of the minor principal strain, such that tensile resistance can be mobilized (Jewell and Wroth 1987). This corresponds to an inclination of about 10°–20° to the horizontal. The provisions of some subhorizontal soil nails would promote early development of stabilizing nail force at working conditions. This is particularly crucial in preventing liquefaction failure, which may initiate from a local zone and develop into a global failure progressively.

From a practical point of view, the number of subhorizontal nails should be approximately 40% to 50% of the total number of soil nails required to ensure that sufficient subhorizontal nails are present to counter sliding failure. It is also necessary to ensure that the upward component of nail force in the potential sliding direction is sufficient to support the weight of the facing structure upon liquefaction of the underlying fill. This can be checked by considering force equilibrium of the slope facing.

The use of some subhorizontal nails in the hybrid system may incur a slight increase in cost and possible encroachment with the adjoining lots. The increase in construction cost arises from the increase in nail lengths to compensate for the reduction in the overburden pressure acting on the subhorizontal nails. The increased cost is partially compensated by the omission of the vertical nails. The overall cost can also be further reduced by optimization of the nail arrangement in the hybrid system through numerical analysis. As the required stabilizing pressure increases linearly with slope height, soil nailing may not be the most cost-efficient design solution for loose fill slopes with a significant height.

Conclusions

The design of soil nails in loose fill slopes formed by loosely compacted fill material derived from decomposed granitic or volcanic rocks needs to consider two key mechanisms; namely, static liquefaction and sliding. The orientation of the soil nails has a direct influence on the stabilizing mechanisms. The numerical analyses conducted in the present study suggest that installing the nails to an inclination of nearly perpendicular to the slope face could lead to significant slope movement especially when sliding failure prevails, for instance, due to interface liquefaction. The slope movement could be reduced by the provision of an embedded toe wall that increases the structural rigidity of the overall soil nail–facing system along the potential sliding direction.

The numerical analyses also demonstrate that a hybrid nail arrangement comprising nails at two different orientations (i.e., subhorizontal nails at the upper part and steeply inclined at the lower part) would limit slope movement and enhance the robustness of the system. Apart from incurring smaller soil and structural deformations under both the full and interface liquefaction failure modes, the hybrid nail arrangement would also facilitate load redistribution, enhance the system robustness, and cater to the uncertainties in the failure mechanisms and in the relative stiffnesses of the different components of the ground nail–facing system.

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Appendix A

To determine the nail lengths in the three scenarios considered (Fig. 3), limit equilibrium analyses were first carried out to determine the required stabilizing surface pressure to prevent overall instability of the slope, with a global safety factor of 1.1 in accordance with HKIE (2003). The analyses assume that the entire loose fill has reached the critical-state undrained shear strength of $c_u = 0.13\sigma'_v$, where σ'_v is the in situ vertical effective stress. The c_u/σ'_v ratio of 0.13 is recommended in HKIE (2003) as a lower bound estimate based on a review of the laboratory test data on loose fill materials derived from decomposed granitic or volcanic rocks in Hong Kong. No perched water table was assumed in the analysis. A small basal shear of 3 kPa was assumed at the interface between the base of the slope facing and the surface of the fill slope. The required nail lengths were then calculated by transforming the triangular surface pressure to discrete line forces.