Report on "Design of Soil Nails for Upgrading Loose Fill Slopes"

Comments on Draft Final Report (circulated in Jun 2011)

Comments from DH(P&S)

No.	Comments	Response to Comments
1	The use of a hybrid system is supported. However, given the use of 2 types of soil nails with different inclinations, there could be site control problems to ensure that the nails are properly inclined as designed. Also, if the no. of sub-horizontal nails is less than 40% to 50% of the total no. of nails, is the vertical nail or embedded concrete footing needed? If so, will it matter if the division boundary of the sub-horizontal nails and steeply inclined nails are different in different sections of the slope?	As the proposed hybrid nail arrangement requires nails at the upper part of the slope to be shallowly inclined, and nails at the lower part of the slope to be steeply inclined (see Figure 4.2(c) of the report), it would be unlikely for the two nail sets to intercept each other. Hence, normal site control should be able to ensure nails of the hybrid arrangement are constructed in accordance with the design intent. In fact, the use of steeply inclined nails at one inclination would require more stringent site control as any deviations on the nail orientation would impose an unbalanced force on the grillage structure. The designer may adjust the nail arrangement to accommodate site constraints. However, if the nail arrangement deviates from the recommended ratio of sub-horizontal nails to steeply inclined nails, or different arrangements adopted for different sections of the slope, the designer should demonstrate the effectiveness and robustness of the proposed nail arrangement using numerical analyses. Vertical nails or embedded concrete footing generally have very little contribution to the robustness of hybrid nails arrangement, and may be omitted.
2.	The total size of the openings is about 50% of the overall plan area. Could the total opening size be increased without the need to carry out FLAC or other numerical analyses?	A simple upper bound estimate of the limiting earth pressure that the grillage facing could sustain is given in Section 4.4. If the designer opts to increase the total opening size to over 50%, the upper bound estimate could be used to justify whether squeezing out of the fill material is a concern. If external structural elements, such as geo-synthetics, are employed to increase the limiting earth pressure, numerical analyses would be required to demonstrate the adequacy of the proposed design.

Comments from CGE/ME

No.	Comments	Response to Comments
3.	Section 6, Item (a): We support the recommendation to encourage the use of site specific laboratory test results, it is desirable to provide relevant guidance in particular on sampling, testing and interpretation of results. Although TGN 7 has listed out the testing for loose fills, however, testing on fill samples retrieved from U tubes of which soil could have been densified during sampling, is still commonly found.	As soil samples retrieved from U tubes are liable to densification, design shear strength parameters of loose fill should be derived from triaxial tests on remoulded fill specimens obtained from representative bulk samples in accordance with TGN 7.
4.	Section 6, Item (b): We support the recommendation to adopt a more robust nail arrangement, it may be desirable to assess and illustrate other possible hybrid nail arrangements in addition to the one shown in Case C in Figure 4.2, for the practitioners to consider / select (e.g. staggered nail pattern, alternative rows / columns at sub-horizontal and perpendicular to the slope face?).	The hybrid nail arrangement shown in Figure 4.2(c) is considered to be the more practical nail arrangement that would facilitate load redistribution, enhance the system robustness, and cater for the uncertainties in the failure mechanisms and in the relative stiffness of the different components of the ground-nail-facing system. It is not the intention of this study to assess the feasibility and effectiveness of all the possible combinations of hybrid nail arrangements. The designer may adjust the nail arrangement to accommodate site constraints. However, if the nail arrangement deviates from the recommended hybrid nail arrangement or the ratio of sub-horizontal nails to steeply inclined nails is outside the 40 % to 50 % limit, the designer should demonstrate the effectiveness and robustness of the proposed nail arrangement using numerical analyses. A paragraph has been added to Section 5.4 to address the issue whereby alternative nail arrangement is proposed to accommodate site constraints.
5.	It is suggested to run some pilot projects for different conditions in order to review the performance based on the recommendations of the study.	Pilot projects for reviewing the performance of loose fill slopes upgraded on the basis of the proposed recommendations are beyond the scope of this study. These may be considered in future studies.

Comments from CGE/MW

No.	Comments	Response to Comments
6.	(1) Section 3.2 and Figure 2.1: The term "instability line" (which is called by Chu et al (2003)) does not reflect the physical behaviour of loose soil. It is suggested that a more commonly used term "collapse line", which better describes the triggering mechanism of liquefaction should be used. Besides, the term "collapse line" is consistent with many other terms that are used in various parts of the report, such as "collapse state", "undrained collapse", "collapse of a saturated soil microstructure" and "collapse behaviour".	The term "collapse line" or "collapse surface" has a different meaning according to Sladen et al (1985). The "collapse line" is defined as the line joining the peak strength and the steady state (or critical state) shear strength on the projection of the p′-q plane. This is different from the "instability line" which is used to represent the mobilised friction angle at the peak shear strength. There is a comment suggesting that use of the term "instability line" might generate confusion as it relates to micro-mechanical explanation of the observed stress-strain behaviour of the soil. In the revised report, the term "mobilised friction angle at the onset of liquefaction" is used to replace "instability line". The symbol "ϕ'_{col}" has been replaced by "ϕ'_{mob}" (to be consistent with the original HKIE-GD report, 2003).
7.	(2) Section 4.2.2, line 3. Please change "20^o to 30^o" to "10^o to 20^o"; this would tie in with the results of other studies.	The text has been amended in accordance with this suggestion.
8.	(3) Sections 4.1, 4.2 and Figure 4.2. Figure 4.2 shows that different nail lengths are used in the FLAC analysis for different nail arrangements. For the steeply inclined nail arrangement, the total length of the top three nail is 23.2m. For the hybrid nail arrangement, the total length of the top three nails (at sub-horizontal inclination) is 33.3m, about 43% more than the steeply inclined nail arrangement. It also appears from the figures presented in Appendix B that if compared with the steeply inclined nail arrangements, longer nails are used in the numerical analyses for the hybrid nail arrangements. With the significant difference in nail lengths adopted in the numerical models, it is difficult to compare the results of the analysis and assess the influence of nail inclination on stabilizing mechanism. In any case, it is apparent that when compared with steeply inclined nail	As shown in Figure 4.2, the total nail length for the steeply inclined nail arrangement is 79.5m, whilst that for the hybrid nail arrangement is 89.6m. This represents a percentage difference of 12.7%. As discussed in Section 5.4, the increased cost can be partially compensated by the omission of the vertical nails. The merit of the hybrid nail arrangement is the enhanced robustness of the system, especially when the failure mode could comprise a sliding type. If only steeply inclined nails are used, the slope deformation could be large if a sliding failure occurs. The large movement is caused by the limited structural stiffness along the potential sliding direction, which could not be enhanced by increasing the lengths of the steeply inclined nails. Therefore, even if the top three soil nails in the steeply inclined nail arrangement are

	arrangement, larger nail lengths are usually required for the hybrid nail arrangement in order for the nails to bond into the competent insitu materials.	increased to the same as those in the hybrid nail arrangement, the performance of the steeply nail arrangement would not be improved much in terms of slope movement. This is evident from Figure 4.6 (a) which shows that the mobilised nail forces in the top 3 steeply inclined nails are far below the design capacity.
9.	(4) Figure 4.9. The mobilized tensile nail force increases with depth for the top three nails (with small inclination) but it reduces sharply to almost zero (and even negative) at nail 4, then it increases again with depth in the bottom three nails. Furthermore, the structural displacement vectors seem to indicate that the whole grillage facing is rotating clockwise about nail 4 with the top half of the facing moving into the slope and the bottom half moving out. Similar observations in respect of nail forces and structural displacements can also be found in other hybrid cases presented in Appendix B, e.g. Figures B.20, B.21, B23 and B25. This suggests that the hybrid nail arrangement may not be a particularly efficient solution as some nails at about mid-slope height (e.g. nail no. 4) are ineffective in supporting the grillage facing. Furthermore, large bending moments may also be induced in the grillage facing near nail no. 4 and this bending load needs to be accounted for in the design.	Under interface liquefaction, the unbalanced earth pressure acting on the slope cover (i.e. grillage facing) is much reduced. Hence, this renders steeply inclined nails ineffective in nail force mobilisation. This is particularly the case for the upper rows of nails in the steeply inclined nail arrangement (see Figure 4.6(a)). For the hybrid nail arrangement, tensile forces are fully mobilized in the top three rows of sub-horizontal nails. However, tensile forces are not mobilized in the fourth row of steeply inclined nails immediately below the sub-horizontal nails (see Figures 4.9, B20, B21, B23 and B25). The results of the FLAC analyses indicate that the presence of the sub-horizontal nails significantly reduces the slope deformation (262mm in Figure 4.6(a) vs 164mm in Figure 4.9(a)). The hybrid nail arrangement is not only effective in resisting sliding failure (i.e. interface liquefaction), but also effective in resisting complete liquefaction failure. Under complete liquefaction mode, nail forces are effectively mobilised in all the soil nails as shown in Figure 4.8(a). For the hybrid nail arrangement under complete liquefaction, the induced bending moment in the grillage is not significant. The bending moment induced in the grillage beam near Nail No.4 is only 31 kNm/m according to the FLAC analysis. The maximum bending moment of 78 kNm/m occurs at the location between Nail No. 6 and Nail No. 7. For interface liquefaction, the bending moment in the grillage is even smaller.
10.	(5) Design of the nailing system is usually carried out using the limit equilibrium method. This method can hardly simulate the	The findings of the study indicate that two different failure modes (i.e. complete liquefaction and interface liquefaction) should be

	rapid changes in nail forces in the hybrid nail arrangement, as discussed in point (4) above.	considered in the design of soil nails for upgrading loose fill slopes. The main limitation of the limit equilibrium analysis is its inability to predict the mobilized nail forces in the slope under different failure modes. The FLAC analyses undertaken in this study confirms that the nail forces mobilized under complete liquefaction are similar to the stabilising nail forces predicted by the limit equilibrium analysis. However, under interface liquefaction, FLAC analyses indicate that only very low magnitude of forces could be mobilised in the upper rows of soil nails of the steeply inclined nail arrangement, and large soil and structural deformations were induced (see Figure 4.6). On the other hand, forces were effectively mobilised in the upper rows of soil nails of the hybrid nail arrangement, and the induced soil and structural deformations were much smaller than the steeply inclined nail arrangement (see Figure 4.9). This is precisely the underlying reason of introducing sub-horizontal nails at the upper part of the slope such that sufficient nail forces could be mobilised when sliding failure occurs.
11.	(6) For the steeply inclined nail arrangement, all the nail forces act more or less in the same direction of the uplift force and so they are efficient in resisting such force. However, in the case of hybrid nail arrangement, about 40% to 50% of the nails are placed at small inclinations and the direction of these nail forces deviates substantially from the direction of the uplift force, as such the nails are not efficient to resist the force. The hybrid nail arrangement would require more and longer nails than the steeply inclined nail arrangement for resisting the same amount of uplift load.	The challenge of designing soil nails in loose fill slopes is that two different failure modes – complete liquefaction and interface liquefaction – should be considered. Under complete liquefaction, large earth pressure is generated and exerted on the grillage. Undoubtedly steeply inclined nails would be very efficient in resisting such uplift force. However, if liquefaction is confined in a thin layer and causes a sliding failure mechanism, steeply inclined nail would become inefficient in resisting the sliding motion. The hybrid nail arrangement is a robust solution to tackle both failure mechanisms.
12.	(7) In examining the effect of toe fixity using the FLAC analysis, it appears that the vertical soil nail at the slope toe is modeled as “cable element”, like the other soil nails on the model slope. It	The vertical nails are very slender structural elements, and due to the relatively large horizontal spacing between the nails in comparison to the nail’s diameter, the effective bending resistance of the row of

	is probable that the tensile stresses mobilized in the vertical nails are relatively small and there is more structural capacity to resist bending/shear. If the vertical nails are modeled as a "pile element", the structural displacement of the facing would be substantially reduced. In any case, structural elements (e.g. steel channel) with larger bending stiffness than steel bars may be used in the "vertical nails" for reducing the amount of structural displacements. Furthermore, as the maximum uplift force is exerted at the toe of the slope, the vertical nails have an added advantage of helping to hold the grillage facing down.	vertical nails at the slope toe is rather insignificant. The use of a strong structural element at the slope toe is a possible option to enhance the structural stiffness of the entire system along the potential sliding direction. The results of the numerical analyses (Figures B.4 and B.18) indicated that the vertical nails were in compression instead of tension under complete liquefaction and interface liquefaction due to the self weight of the facing. The sign convention of the structural forces in the numerical analyses has been added in the figures in Appendix B.
13.	(8) Based on the discussions in points (4) to (6) above, it appears that the hybrid nail arrangement is not particularly superior to the steeply inclined nail arrangement (with toe fixity). In certain aspects, the hybrid nail arrangement is less efficient than the steeply inclined nail arrangement. Therefore, there are reservations with regard to the statement "designers should adopt a hybrid nail arrangement as far as possible" (Section 5.4). Perhaps, the report should be tuned in such a way that both hybrid nail arrangement and steeply inclined nail arrangement (with toe fixity) are workable options, and that a designer should adopt the option that best suits the specific site conditions. For example, a slope with thick tapered fill and dense/strong insitu materials at shallow depths below the slope toe, steeply inclined nail arrangement would be a more preferred option because: (i) it requires shorter soil nails at the upper portion of the slope; and (ii) only short vertical nails (probably at close spacing) would need to be installed into the insitu soil.	The results of the study indicate that the hybrid nail arrangement is capable of tackling both the complete liquefaction and interface liquefaction failure mechanisms. The use of steeply inclined nail arrangement is a consequence of designing the slope primarily against the earth pressure generated from the liquefied loose fill. The limited structural stiffness of the steeply inclined nails along the potential sliding direction renders the system heavily reliant on the toe fixity to resist sliding failure and to limit overall deformation. This significantly reduces the system redundancy to cope with deviations of actual ground conditions from design assumptions. The hybrid nail arrangement is a more robust system as the overall stiffness of the ground-nail-facing system along the potential sliding direction is enhanced. The need for toe fixity is also eliminated by the presence of sub-horizontal nails in the system.

Comments from CGE/LPM1

No.	Comments	Response to Comments
14.	The reasons given in Section 3.3 for excluding the data points below $C_{SS}/p'_{peak} = 0.2$ still seems not too convincing. I understand that the test samples were collected at shallow depths. The samples were remoulded and then subjected to a minimum confining pressure of 20kPa in the triaxial cell. In the process of consolidation, I would expect a loose test sample to densify. The void ratio of the test specimen would be governed by the confining pressure in the triaxial cell rather than its insitu void ratio. The argument of mismatch between the void ratio and the stress level of the test specimen seems vague.	The review of the laboratory test data reveals that loosely compacted CGD exhibits very stiff response during isotropic consolidation with very little change in void ratio (Figure A.5 in the report). Also, the isotropic consolidation line is not unique, but is dependent on the initial void ratio. If a soil sample taken from a very shallow depth is remoulded at the in-situ void ratio and tested at a higher stress level, it is likely that the test specimen is looser (i.e. of a higher void ratio) than the soil in the field corresponding to the same stress level (i.e. depth).

Comments from CGE/LPM2

No.	Comments	Response to Comments
15.	In Section 5.3, it is mentioned that fill at greater depth has a lower chance of undergoing liquefaction. It is suggested that a worked example be given to elaborate on how to delineate this zone.	Figure 5.1 has been added to Section 5.3 to illustrate how the zone of loose fill which has a lower likelihood of undergoing liquefaction can be delineated.

Comments from CGE/HD

No.	Comments	Response to Comments
16.	On p. 29 – 'Dsiaplcement' under item (b) of Fig. 4.9 should read 'Displacement'; and 'assuemd' in the Note should read 'assumed'.	The typos have been corrected accordingly.
17.	On p. 35 lines 11-13 and on p.37 para. 4 lines 4-6 – If the zone of non-liquefiable fill is delineated on this basis, the bond strength of the soil nail embedded in the non-liquefiable fill zone beyond the failure surface should be allowed in determining the required nail lengths.	Although the fill at great depth is unlikely to undergo liquefaction due to the low mobilised stress ratio, the fill in this "deep" zone may still exhibit volumetric contraction at the soil-nail interface when the nail force is being mobilised. Volumetric contraction of loose fill at depth may cause arching which may lead to very low effective confining stress along the bond length of the nail. Due to the uncertainties with regard to the development of bond strength in loose fill at depth, it is recommended not to rely on the bond strength of the soil nail

		embedded in any loose fill irrespective of the depth in the course of determining the required nail lengths.
18.	On p. 35 para. 2, the assumption that slopes with a thin fill layer (e.g. <1m) … would not undergo undrained collapse, or has a reduced likelihood of liquefaction, may need further justification. It would be useful if further researches are conducted and/or further parametric / numerical analyses are performed and presented in this report to substantiate this assumption.	The assumption that very thin (e.g. <1m) fill layer would not undergo undrained collapse is based on the following: (i) due to the low effective stress in a thin fill layer, the soil would exhibit dilative behaviour instead of contractive behaviour, which is one of the necessary conditions for the fill layer to liquefy; (ii) the short drainage path of thin fill layer facilities dissipation of excess pore water pressure generated during shearing of the loose fill.

Comments from Victor Li & Associates

No.	Comments	Response to Comments
19.	Figure 2.1 refers. The instability line usually refers to the demarcation line separating the zone of instability and zone of stability under undrained triaxial conditions for fully saturated soils. For a slope subjected to rain infiltration, the soils may not be fully saturated when static liquefaction occurs. The onset of liquefaction under the stress path in Figure 2.1 may not necessarily coincide with the instability line for fully saturated soils and in fact there may be an instability zone rather than an instability line for nearly saturated soils. Figure 2.1 may therefore be misleading.	In the revised report, the term "mobilised friction angle at the onset of liquefaction" is used to replace "instability line". The symbol "ϕ'_{col}" has been replaced by "ϕ'_{mol}" (to be consistent with the original HKIE-GD report, 2003). The onset of liquefaction is affected by many factors, such as the degree of saturation of the soil, the effective stress path through which liquefaction is triggered, etc. The mobilised friction angle derived from a series of isotropically consolidated undrained triaxial tests is normally a conservative measure of the actual stress conditions at which liquefaction is triggered by rainfall infiltration. Discussion on this point has been added to Section 3.2.
20.	Some guidelines on how to relate c_{ss} to some directly calculable quantifies will be useful to designers.	i. The concerned relationship implies that the mean effective stress in the slope can be estimated from the vertical effective stress by $p' = 0.67\sigma'_v$. This implies a K_0 value of 0.5 which can be considered as a

	i. In the footnote of p.19, it is suggested that $c_{ss} = 0.13 \sigma'_v$. Is this relationship applicable as a general rule? ii. Can we approximate σ'_v by γ z, where γ = unit weight and z is the design depth of liquefiable fill? iii. The follow-up questions to (ii) are: - What z should be used? The mid-depth or full depth of the body of liquefiable fill corresponding to a particular slip surface? - As σ'_v increases with depth, should we use increasing value of css with depth or a constant c_{ss} for the entire fill body? Which option is more correct and/or conservative? iv. Should we use the bulk density instead of submerged unit weight for γ when correlating c_{ss} with σ'_v? Although I believe that the designer should make his/her own judgment, some recommendations in the Report may help avoid potential arguments between designers and checkers.	general rule unless a more representative estimate of the K_0 value is available. ii. The vertical effective stress σ'_v can be estimated by γ' z, where γ' = effective unit weight and z is the depth below ground surface at which σ'_v is calculated. iii. z is defined as the depth below ground surface at which σ'_v is calculated. In other words, it is a variable within the loose fill. If a particular slip surface is considered, z would be the depth of the fill above the potential slip surface, and would vary along the slip surface. Since σ'_v increases with depth, c_{ss} increases with depth as well. Assuming a constant c_{ss} value within the entire fill body is a gross simplification as compared to adopting a depth increasing c_{ss} profile, and could either be too conservative or too aggressive depending on the adopted value. The approach of using an undrained shear strength that increases with depth is discussed in the HKIE-GD report (2003) (Section 7.3). iv. When correlating c_{ss} with σ'_v, the effective unit weight γ' should be used. Some discussion on the estimation of c_{ss} from the vertical effective stress has been added to Section 5.1.
21.	Steeply inclined soil nails are recommended for the hybrid arrangement. What formula should be used for calculating the pull out resistance of steeply inclined soil nails? Geoguide 7 recommends that the inclination of soil nails should not exceed 20°. A formula is also given in Geoguide 7 that relates the pullout resistance of soil nails to vertical effective overburden pressure. Will the same formula also be applicable to sub-vertical soil nails, if not, what other formula is recommended?	As part of a series of soil nail studies for the development of Geoguide 7, a review of the design methods for estimation of pullout resistance of soil nails was conducted by GEO and the results are presented in Cheung and Shum (2009). The review compared over one thousand pullout test data collected from LPM sites and concluded that the measured pullout resistance is generally higher than the theoretical value given by the effective stress method (i.e. the method described in Section 5.6.3 of Geoguide 7 which relates pullout resistance of soil nails to vertical effective overburden pressure). Although the review

		covered mainly sub-horizontal nails at an inclination of 10°-20°, the data set contained 38 pullout tests in which the soil nails were inclined at 30°-50°. Therefore, as described in Section 5.8.4 of Geoguide 7, the allowable capacity and pullout resistance of soil nails in loose fill slopes can be determined in accordance with the guidance given in Section 5.6.3 (2) of Geoguide 7. As stipulated in Section 5.6.3 (6) of Geoguide 7, when steeply inclined nails are used, considerations should be given to the effectiveness of the soil nails and the amount of slope deformation required to mobilise the design soil-nail forces. These aspects have been investigated in this study by FLAC analyses, which concluded that when steeply inclined nails are used in a hybrid nail arrangement, the earth pressure generated on the grillage near the lower portion of the slopes would facilitate mobilisation of soil nail forces. <u>Reference:</u> Cheung, W.M. & Shum, K.W. (2009). Review of the approach for estimation of pullout resistance of soil nails. Technical Note TN1/2009. Standard and Testing Division, Geotechnical Engineering Office.
22.	The report seems to suggest that the soil nails must be anchored in the weathered soils. As discussed in the report, the potential of soil liquefaction is low for fill at deep levels as the stress state is away from the instability line. Will it be acceptable to anchor the soil nails in fill at deep levels when dealing with slopes with a thick layer of fill?	Please refer to our response to the 2nd comment by CGE/HD.

Comments from Halcrow China Limited

No.	Comments	Response to Comments
23.	Possibly the two most important and welcome potential changes are the reduction in necessary thickness, (buried depth), of the	This comment is noted.

	grillage and the possibility of reducing the inclination of at least some of the soil nails.	
24.	From a landscaping point of view, somewhat disappointingly, the requirement for the grillage to cover 50% of the slope remains. In the draft of new GEO Technical Publication providing guidelines for landscaping slopes, we have suggested that the position of soil nail grillages around trees should be adjusted on sites to avoid surface and near surface tree roots. This suggests that either the buried depth of the grillage adjacent to trees is reduced and/or the grillage members are moved from the designed position to provide necessary space. The latter of these options means that over a certain area, which may be quite large where trees are clustered, the coverage of the grillage may be significantly less than 50%. On site this is sometimes addressed by closing up adjacent grillage member spacing so that over the total slope area, 50% coverage is provided. This closing up then restricts opportunities for planting in the reduced space between grillage members and in some cases where this has been done, it have given rise to adverse comment from some public commentators.	The need to limit the size of the grillage opening is mainly to avoid local squeezing out of the liquefied loose fill and to ensure that the grillage has sufficient structural capacity to withstand the earth pressure generated from the liquefied loose fill. In order to preserve existing trees, in some cases, the locations of the grillage beams might have to be adjusted and the size of the grillage opening increased locally. The presence of these existing trees may serve to reduce the likelihood of local instability through the grillage opening upon liquefaction of the underlying fill. Therefore the size of grillage openings where existing trees are present could be locally increased provided that the overall performance of the grillage as a monolithic structure to withstand the earth pressure from the liquefied loose fill is not compromised. On the other hand, the presence of these trees would not adversely affect the likelihood of squeezing out in the adjacent grillage panels; therefore the size of the opening of the adjacent grillage member could remain at 50%.
25.	It would be useful for designers if some guidance on this practice is provided i.e. the need to provide an overall 50% coverage for fill slopes where there is an extensive tree coverage or whether the 50% should only apply to the area outside of where the adjustment for trees have been made.	Some guidance on the need to provide 50% grillage coverage when there is extensive tree coverage has been added to Section 5.6.
26.	It is understood that the 50% coverage is based on fill bodies basically at Relative Density of around 75%. It may also be useful if there was some thoughts on necessary grillage coverage for fill bodies at say 80%, 85% and 90% of Relative Density i.e. could the percentage coverage be reduced as density increases. If a reduction could be made this would be a great help in development opportunities for planting on the slope.	As discussed in Section 4.4, the minimum coverage of the grillage is related to the undrained shear strength of the loose fill. One would expect the undrained shear strength to increase with the relative compaction, however, such correlation is not observed from the actual test data as discussed in the HKIE-GD 2003 report (see also Figure 5.11 in HKIE(2003)).

Comments from Professor Matthew Coop (CityU)

No.	Comments	Response to Comments
27.	There is an assumption in the report that Critical State Lines are straight in the e:lnp′ plane. They generally are at high pressures, but many coarse grained soils show a curvature to a horizontal asymptote at lower stress levels, as in the attached paper. Equation A3 is only valid if the Critical State Line is straight. Incidentally, in Equation A2, the intercept Γ is normally used with a specific volume axis not void ratio.	The purpose of the discussion in Section A.1 is to point out that if the location of the critical state line is unique, irrespective of its shape, the undrained shear strength of a soil would be uniquely related to its void ratio before undrained shearing. If the critical state line is a curve, Equation A.3 is still valid if λ is made a function of p′ instead of a constant. We agree that Γ is normally used with a specific volume axis and not void ratio. Its use with void ratio followed the convention adopted in the HKIE-GD 2003 report (HKIE, 2003). This has now been revised in this report.
28.	I agreed very much with your criticism of Fig.2.1 that the instability line is not a unique line of constant gradient. Many researchers have found that it depends on void ratio and stress level. As you have said about Fig. 3.1 it is commonly found that at higher stresses the value of your ϕ′col increases. As you will see from the attached paper, my own view is that it is not the stress level per se that is important, but the change of the void ratio that accompanies compression to a higher stress (see Fig.16 of Carrera et al.). Because of the lower void ratio the undrained stress path takes the soil to a lower point on the Critical State Line in the e:lnp′ plane. If the Critical State Line is curved, as the initial void ratio reduces, for the loosest possible sample there will be a much smaller decrease of lnp′ to reach that Critical State and so a much reduced degree of strain softening.	This comment is noted.
29.	Personally I am not an adherent to the steady state "heresy" and cannot see why we need that term and cannot just use critical state	The term "steady state" was used in the HKIE-GD 2003 report (HKIE, 2003). In this report, the critical state framework is introduced to

	consistently. I always thought that this was a red herring of soil mechanics. Isn't the implication of referring to steady states of undrained tests and critical states for drained that the soil in some way knows whether it is drained or undrained? I think that the demonstration that it is illogical as a terminology is a test that is carried out drained but under constant volume conditions. If the soil is saturated and we believe in effective stress the test must give the same behaviour as the undrained test, but would the failure state then be a critical state or a steady state? If we believed that the state we reached at ultimate failure depended on the stress path we took to get there, and that was the reason for the different terms, then we would of course need an infinity of different names for all the possible failure states reached by all the different possible drained paths, of which the constant volume path is just one.	explain the observed behaviour of loose fill materials. It is assumed in the report that "steady state" and "critical state" are identical.
30.	I did not really agree with the statement on P11 that when the soil state reaches ϕ'col there is a "collapse of the soil microstructure". I think that Mike Jefferies showed that was incorrect some time ago. As in the attached paper, for loose samples that reach the curved part of the Critical State Line under undrained loading, we would expect a very large reduction of p′ and so there must be a large degree of strain softening. There is therefore no need to invoke any micro-mechanical explanation for this, and as Jefferies showed, there is no special feature of the stress: strain behaviour for such tests that that might indicate some micro-mechanical collapse. Of course the engineering implications of reaching this state can be dramatic, but there are plenty of other soils that strain soften, so I am not sure why we suddenly adopt this vocabulary of "collapse" or "instability" for coarse grained soils. I therefore would have referred to ϕ'col as something else.	We agree that it is not necessary to invoke micro-mechanical explanation for the observed undrained strain softening behaviour of loose fill materials. This explanation has no implications whatsoever on the recommended usage and design of soil nails in loose fill slopes. The statement on P.12 has been modified.
31.	Similarly I am personally not that keen on referring to liquefaction for soils that simply have a peak in their stress: strain behaviour (P14). This is a common usage of the term, but I would	The term "liquefaction" has been used widely in the local geotechnical community to refer to the undrained strain softening behaviour. This definition is given in Section 2.1 in the report.

	reserve it for those cases where p′ and so the strength actually reduce to zero, i.e. the soil really becomes a liquid, which by definition will occur when the initial state is above the horizontal asymptote of the Critical State Line, as in the attached paper. It might be worth emphasising in the report that you do not see such cases here and you are able to define a minimum value of css/p′peak because you are testing soils at the typical in situ void ratios, which are not loose enough for this to occur.	It has been established in the HKIE-GD 2003 report (HKIE, 2003) that local loose fill materials would not undergo "complete liquefaction" whereby the effective mean stress and shear strength drop to zero. In fact, the use of soil nails in loose fill slopes hinges on the residual shear strength of the loose fill upon "liquefaction".
32.	On P12 there is an assumption that the effects of any natural structure of a CDG/CDV would be positive, i.e. increasing the strength and stiffness. I am not sure that this has been proven. I suspect it is probably correct, but it is not necessarily true. There are some soils in the literature that have negative effects of structure, so that they are weaker and/or softer in their intact state than when reconstituted or remoulded. These negative effects are generally related to fabric, and I am not sure what role this might play for residual soils like these. I guess that inter-particle bonding is important in residual soils, which I think can only have a positive effect.	Previous laboratory test results, as well as published results in the literature, suggest that natural structure has positive effects on CDG/CDV probably because of bonding. Therefore, we are of the opinion that this is a reasonable assumption.
33.	On P17 you might emphasise that the lack of a unique Critical State Line in the e:lnp′ plane probably has two origins, a) the natural differences that would occur in the location of the Critical State Line in the e:lnp′ plane because of variations of their nature (grading, mineralogy), which always have a much greater effect than in the q′:p′ plane, and b) the fact that some of them may also be transitional so even for a single soil the Critical State Line is non-unique. It is this non-uniqueness of the Critical State Line in the e:lnp′ plane that I suppose is the main source of the wide scatter of values of css/p′peak on Fig.3.2.	We agree that the lack of a unique Critical State Line in the e:lnp′ plane may be attributed to the two factors: (i) grading, mineralogy, and (ii) possible transitional behaviour exhibited by some completely decomposed soil. There are experimental results which suggest that CDG in Hong Kong does not exhibit transitional behaviour (i.e. Yan & Li, 2011). In fact, decomposed soils cover a very wide range of behaviour depending on the particle size distribution and mineralogy. We have amended the text to include the influence of soil grading and mineralogy on the location of the Critical State Line, instead of attributing the non-uniqueness of the Critical State Line solely to transitional soil behaviour. Yan, W.M. & Li, X.S (2011). Stress-strain response of a recompacted medium-fine-grained completely decomposed granite in Hong Kong. Engineering Geology [In press].

34.	On P44/45 I would refer to transitional soil behaviour as indicating that the soil fabric is robust, i.e. not easily changed, rather than stiff. Whether it is stiff or not is a separate question really.	We agree that the word "robust" better describes the soil fabric. The text has been amended accordingly.
35.	The effects of saturation of the strength of the soil on a slope may not simply be related to the loss of suction and decrease of effective stress as implied by Fig.2.1 and the analyses in the report. As shown by Lee & Coop (1995) and Coop & Lee (1995), an effect of saturating a decomposed granite can be that the location of the Critical State Line can change in the e:lnp′ plane because of the changing strength of the soil particles. This would cause the soil to become significantly more compressible in its behaviour i.e. more "collapsible".	Your comments are noted. We highlight the influence of soil suction on the location of the Critical State Line, as per the discussion in the original HKIE-GD report (HKIE, 2003). Soil suction does not play a part in the interpretation of the laboratory test data in this review as all the reported tests were conducted on saturated specimens. Nonetheless, it is obvious that further research is required to investigate the influence of soil suction on stress state at which liquefaction is triggered and the resulting shear resistance.