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ABSTRACT

In this paper two numerical analysis methods (i.e. cyclic time history and pseudo-static) are applied to simulate the seismic behaviour and failure mechanism of soil-nailed structures. The numerical simulations are performed by using a finite difference software (Flac). Nevada sand soil parameters are used and construction sequences of nailed-structures are simulated prior to the cyclic and pseudo-static analyses. The results revealed that the failure pattern of two kinds of analyses are approximately similar and comprised of bilinear sliding surfaces. Furthermore, good agreement is found between failure pattern of two types of numerical analyses and previous experimental tests. based on comparison between facing displacements in two considered analysis methods, a simple process is presented to achieve the seismic coefficient consistent with the peak ground acceleration. Presentation of considered method is based on supposition that failure occurs at the constant pullout displacement of bottom-row nails for both analysis methods.

INTRODUCTION

There are several limit equilibrium analysis methods currently available for the design of soil-nailed slopes such as the German method (Stocker et al. 1979), the Davis method (Mitchell and Villet 1980) and the French method (schlosser 1982). However the limit equilibrium analyses yield only a global safety factor with respect to a rotational or translational failure of the soil-nailed block along the proposed potential sliding surface and don't allow for estimate of forces mobilized in structural elements such as nails and facing, therefore can not be used to evaluate the local pull-out stability of soil-nailed systems under both static and dynamic loading conditions. Furthermore assumption of input seismic force acting on sliding block is typically determined by reducing by some factor the peak seismic acceleration to be resisted during the lifetime of the slope. Some researches conducted to estimate the pseudo-static forces. Hong et al. (2005) based on a regression of the critical seismic amplitudes obtained in the shaking table model tests with the critical seismic coefficient gained by the proposed two-wedge limit analysis presented 0.63 for the ratio of critical seismic coefficient to the critical seismic amplitude. It is noteworthy that the critical seismic coefficient and critical seismic amplitude respectively were defined as the horizontal seismic coefficient that correspond to a stability safety factor of 1 and the value which any further increase in the amplitude of acceleration greatly displace the slope. In orther to estimate the pseudo-static inertia forces, Chokeir (1996) applied the simplified spring mass model in which internal nails were represented by external springs. Then the seismic coefficient was determined as a function of earthquake frequency and maximum base acceleration:

$$k_h = \left(\frac{0.5}{1-\left(\frac{\omega}{\omega_n}\right)^2}\right)\times\left(\frac{a}{g}\right) \tag{1}$$

Where ω is the applied earthquake frequency, ω_n is the soil-nailed natural frequency, a is the maximum base acceleration and g is the gravity acceleration.

The numerical pseudo-static methods can evaluate the forces mobilized in the structural elements and the slope deformations, in contrast with the limit equilibrium methods, and there is no need to propose a failure surface for analyses. However, selection of appropriate inertia seismic forces based on design earthquake parameters is essential.

The objectives of current paper are to compare the failure mechanism and stability limits in cyclic and pseudo-static analyses and estimate the seismic coefficient based on peak cyclic acceleration.

MODELS DESCRIPTIONS

Two models representative of deep excavations were selected for analyses. The finite difference numerical models were set up to represent 9.6m and 15m high excavations with a face angle of 90° to the horizontal.

Hereafter in the paper, the model with 9.6 m height is considered as model 1 and other is khnown as model 2.

Models 1 and 2 respectively comprise 6m and 9m of underlying soil, 30m and 43.5m laterally soil behind the facing and approximately 10m of soil for both models in front of the facing. For the 9.6m nailed slope, 6 rows of steel bar of 30mm diameter and 7m length and for the 15m wall, 9 rows of nails of 32mm diameter and 10m length are used. The vertical and horizontal spacing of the nails is 1.5m for both models except for the lowest-row nails horizontal spacing of model 2 (15m high model) which is considered 1m. Models dimensions are shown in Fig.1. Two types of facing modeled, namely, a 10cm shotcrete facing with wire mesh as a temporary facing and a 30cm reinforced cast in place concrete as a permanent facing.

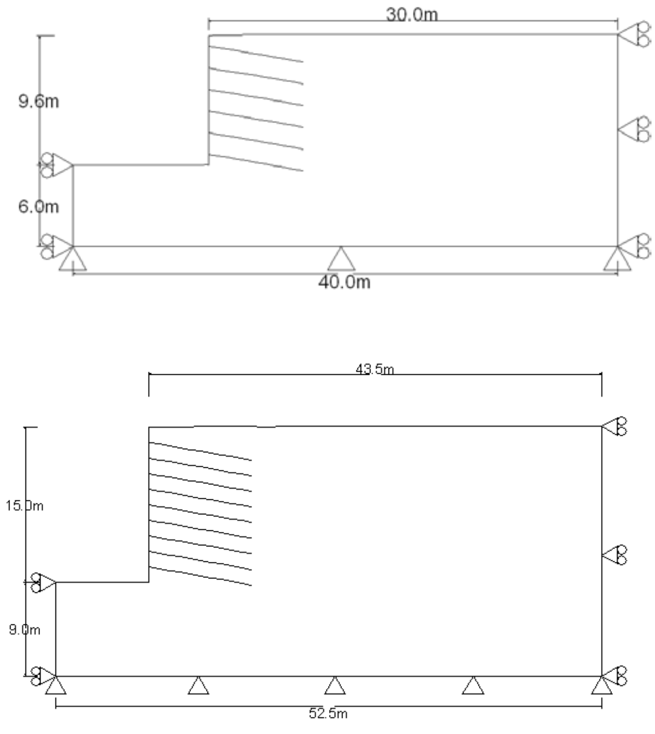


Fig.1. Models dimensions

Mohr-Coloumb elasto-plastic constitutive model is considered to model the soil, beam elements with plastic moment resistance were considered to model the facing and elastic pile elements to model the nails. To simulate the mobilization of forces due to relative displacement between soil and structural elements (i.e. nails and facing), coupling elastic perfectly-plastic springs with elastic stiffness and yield strength are used. For the soil-facing interface, shear stiffness and shear strength are modeled so that they increase as a result of an increase in the normal stress acting on the interface. Other interface parameters (i.e. stiffness and strength) are considered as a fix value.

As shown in Fig.1, The boundary conditions of the models are taken as full fixity at the base with vertical rollers on the whole right and left boundaries of foundation soils for construction stage and pseudo-static analyses. However for dynamic analyses free-field boundary conditions specified along the lateral edges of models. It should be noted that in initial phase conditions, The left boundary was totally taken as vertical rollers.

Using proper size of finite difference zones increase the numerical accuracy of the propagating wave in the model. As a result, Kuhlemeyer and Lysmer (1973) present that zone dimensions must be smaller than one-tenth to one-eight of the highest frequency component of the input wave length. In the present study for 9.6m and 15m height models, grid spacings are considered, 0.2m and 0.3m respectively. It is noting that zones dimensions for model 2 were considered larger to decrease time of analysis.

SOIL MODEL

For all analyses, the soil material model is selected to behave under "Mohr-Coloumb" criteria. In this model the elastic perfectly-plastic behaviour is applied to the soil zones. The basic characteristics of the model are: shear moduls (G); bulk modulus (K); internal friction angle (φ); Soil cohesion (C); dilatancy angle (ψ) and density (ρ). In the present study, Mohr-Coloumb parameters of Nevada sand at relative densities of 40% and 60% calibrated on basis of data from tests performed by Alrumoli et al. (1992). With respect to tests results, shear modulus and bulk modulus at different confining pressures could be found as a function of mean effective stress as follow:

$$G = G_0 \left(\frac{P}{P_{at}}\right)^n \tag{2}$$

$$K = K_0 \left(\frac{P}{P_{at}}\right)^n \tag{3}$$

Where G is the bulk modulus, G_0 is shear modulus at atmospheric pressure, K is the atmospheric pressure, K_0 is bulk modulus at atmospheric pressure, P_{at} is the atmospheric pressure, n is the dimensionless material constant and p is the mean effective stress estimated using the following expression:

$$p = \left(\frac{1 + 2k_0}{3}\right) \times \rho \times g \times h \quad (4)$$

Where k_0 is the at rest soil pressure coefficient and h is the height of soil overburden. k_0 may be estimated using the relation (i.e., k_0 =1-sin(φ)) proposed by Jaky (1994).

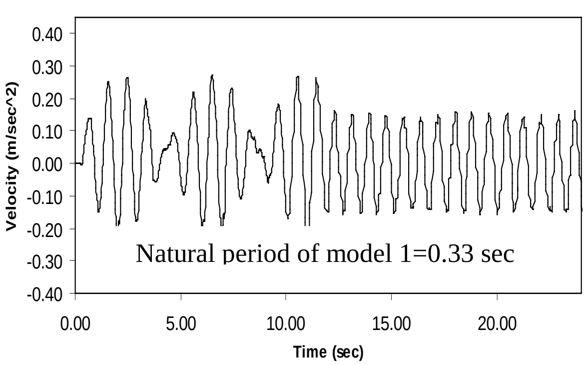
The input values of characteristics relevant to the Nevada sand are included in Table 1. Soil cohesion and dilatancy angle are negligible and they are not included in the following table.

Table 1. Soil input values for Nevada sand

Model no	h	Dr	φ	G_0	K_0	γ(wet)
1	9.6m	40%	34°	73kpa	165kpa	17.3kPa/m
2	15m	60%	36°	85kpa	175kpa	18kPa/m

DAMPING

Rayleigh damping is specified for dynamic analyses. The damping ratio and the corresponding centeral frequency need to be specified. 3% rayleigh damping is used for the soil profile and the central frequency of the damping is set up to the fundamental frequency of structure. In the present study fundamental frequency estimate by the following procedure. Soil and soil structure constutive models assumed to be elastic, additional damping is neglected and cyclic amplitude with the specified duration apply to the models, Then time history responses of some points at the structures were recorded for longer time than duration of applied cyclic amplitudes. The time history responses of models 1 and 2 are presented in Fig. 2. As shown in the implicit plot, period of responses in the free vibration phase are the natural period of structures. However it is necessary that the period of input acceleration be large enough in comparison with second period of structure to resist the resonance in the second frequency.



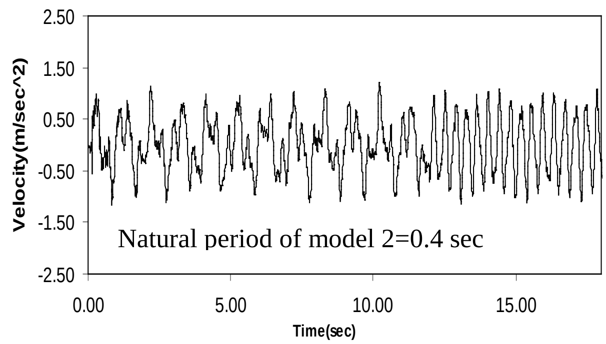


Fig. 2. computed responses on selected points of elastic models

ANALYSES TYPES

Analyses have been done in the current research are: construction stage, time history with cyclic amplitude, pseudo-static and c-φ reduction analyses that all are described as following.

<u>Construction stage analysis</u>

For simulation of excavation in soils, it is essential that excavation carried out in steps that reflect the actual excavation sequences because plastic deformation and stress redistribution in intermediate sequences affect the total excavation results. Therefore, prior to dynamic and pseudo-static analyses, the construction stages should be simulated. In this simulation procedure, stages include successive excavation followed by installation of nails and placement of shotcrete.

<u>Cyclic analysis</u>

Both models were subjected to at least 5 cycles of in-plane sinusoidal base excitation at a frequency of 3Hz with peak amplitudes of 0.035g, 0.1g, 0.17g, 0.35g and 0.5g. Plastic points indicators, facing displacements, displacements contours and forces mobilized in the structural elements recorded during analyses for evaluation of failure mechanism and investigation of dynamic behaviour of models.

<u>Pseudo-static analysis</u>

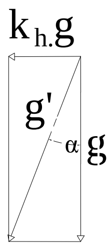
Siesmic effects is simulated by horizontal forces (F_h) equal to the product of weight of soil mass zones (w) and horizontal coefficient of earthquake (k_h) as follows:

$$F_h = k_h.w \quad (5)$$

It is obvious that w is given as product of soil mass (m) and gravity acceleration (g). So inertia forces could be expressed as:

$$F_h = k_h.g.m \tag{6}$$

Using this concept in the current research, pseudo-static analysis is done by changing the magnitude and angle of gravity acceleration as the resultant of gravity acceleration and virtual acceleration equal to $K_h.g$. The subject stated above is indicated in Fig. 3.



$$\alpha = \tan^{-1}\left(\frac{k_h.g}{g}\right)$$

$$g' = (g^2 + (k_h.g)^2)^{0.5}$$

Fig. 3. acceleration resultant of gravity and virtual seismic accelerations

The seismic coefficients applied to models are 0.05, 0.11, 0.146, 0.192 and 0.238.

C-φ reduction analysis

The model can estimate stability factor of safety by strength reduction method. In this approach both friction angle and cohesion are reduced by a constant factor until failure occurs in model. The least factor which makes the system to be in non-equilibrium is named factor of safety. C-φ reduction analyses have been done on models subjected to inertia seismic forces.

ANALYSES RESULTS AND DISCUSSION

Figure. 3 shows the plastic surfaces comprising sliding blocks in cyclic and pseudo-static analyses. It is obvious that the failure pattern includes two sliding blocks, one reinforced block which act as a semi-rigid block and other a block located behind the first block and produce active pressure behind the first block. As shown in Fig. 4, three specified plastic surfaces (one curved shap and two linear surfaces) enclose the sliding blocks.

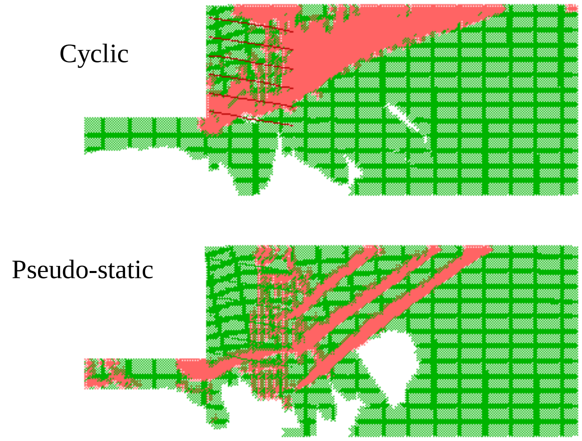


Fig. 4. failure surfaces at the cyclic and pseudo-static analyses

As obsereved above, Predicted failure mechanisms in both numerical analysis methods are approximately similar and are similar to that observed in soil-nailed centrifuge test by Tufenkjian and Vucetice (2000). Shematic failure pattern is shown in Fig. 5.

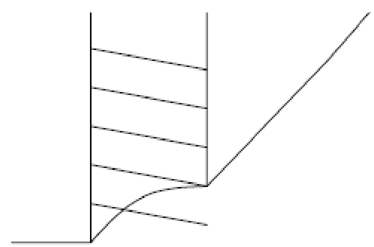


Fig. 5. schematic predicted soil-nailed structures failure pattern

It is clear in Fig. 4 that the nails along the bottom row connect the semi-rigid sliding block to the intact soil mass. Furthermore numerical simulations revealed that with increasing the seismic coefficient and peak seismic amplitude, plastic surfaces propagate behind the whole nails, It means that the bottom-row nails lost their's anchoring effect and the structure has failed.

The maximum nails forces mobilized in pseudo-static analysis and the second cycle of dynamic analysis in comparison with forces obtained from construction stages simulation resluts are presented in Fig. 6. It is obsereved in Fig. 6 that dynamic loading has little effect on mobilization of axial forces in upper-row nails, however the two or three lowest-row nails have sensible increase in axial forces. In other word the maximum axial forces mobilize in the bottom-row nails with increasing the dynamic forces. Aforesaid issue indicates that bottom-row nails avoid from slope failure by having anchoring effect when the failure surface is formed as shown in Fig. 5.

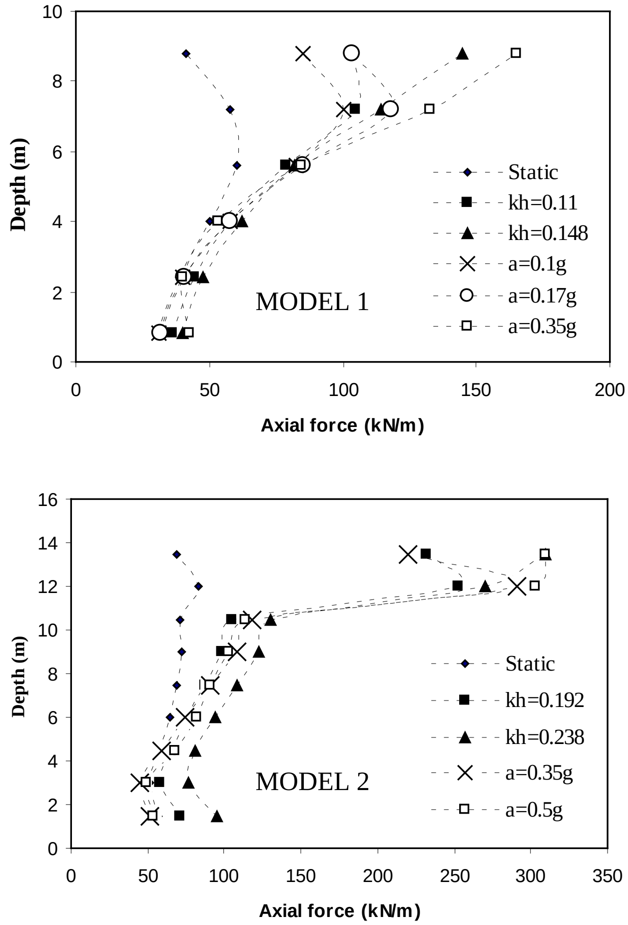


Fig. 6. maximum mobilized forces in different analyses

As stated above it seems that failure may occur at a specified pullout displacement of bottom-row nails or a constant sliding of semi-rigid block. Hereafter the mentioned pulling out displacement will known as critical pulling out displacement. Using this concept, relation between the critical seismic coefficient and the critical peak acceleration can be achieved. In the following, the hunt is on for the evaluation of peak cyclic acceleration consistent with the critical seismic coefficient.

Factors of safety for models subjected to inertia forces regarded for pseudo-static analyses are listed in Table 2.

Table 2. Factors of safety for models subjectes to horizontal forces

k_h	0.11	0.146	0.192	0.238
Model 1	1.09	1.03	0.97	-
Model 2	1.26	1.17	1.08	0.98

As observed in Fig. 7, using linear regression of seismic coefficient with the factor of safety values, the critical seismic coefficient can be obtained. 0.169 for model 1 and 0.228 for model 2 are estimated.

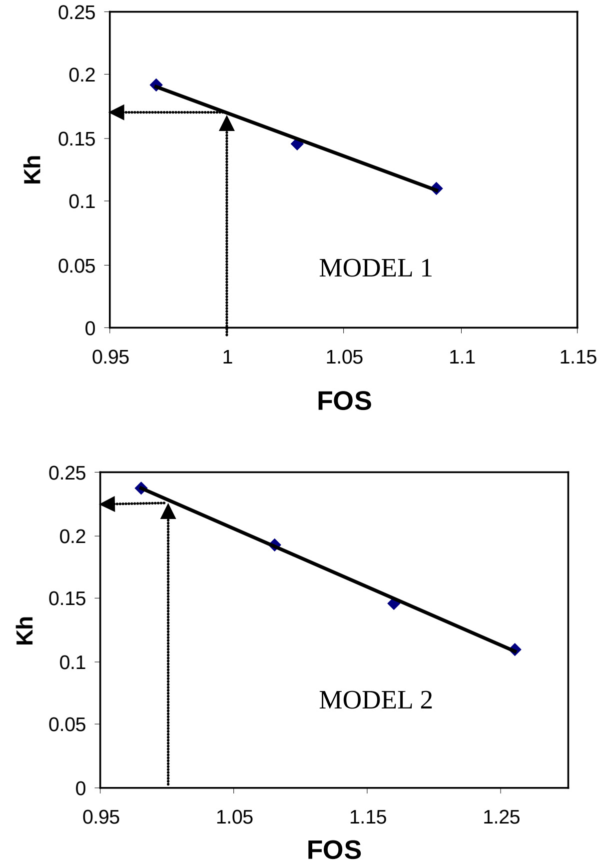


Fig. 7. Input seismic coefficient versus factor of safety

Plot of pullout displacement for bottom-row nails versus seismic coefficient is presented in Fig. 8. Note that seismic coefficient in mentioned plot is for seismic coefficients less than critical seismic coefficient because that there is no limited displacements for models subjected to larger seismic coefficients. Thus the critical pullout displacement would be gained by extrapolation as shown in Fig. 8. Considering the seismic coefficients of 0.169 and 0.228, For models 1 and 2, the critical pullout displacement are estimated 2.25cm and 2.69cm, respectively.

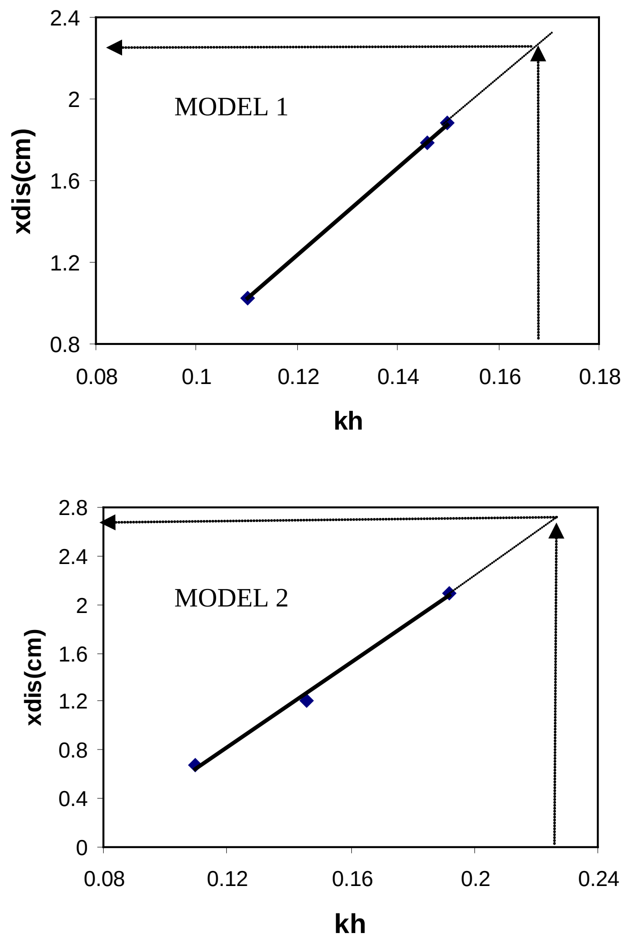


Fig. 8. pullout displacement versus seismic coefficient

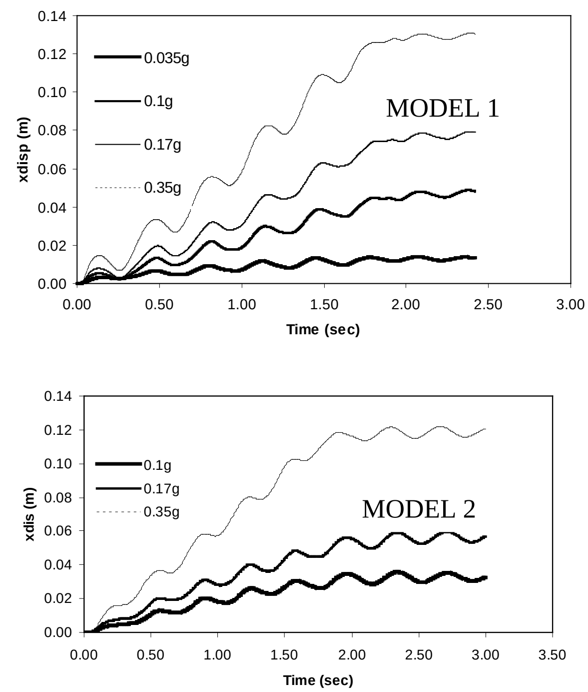


Fig. 9. Facing displacements from the cyclic analyses

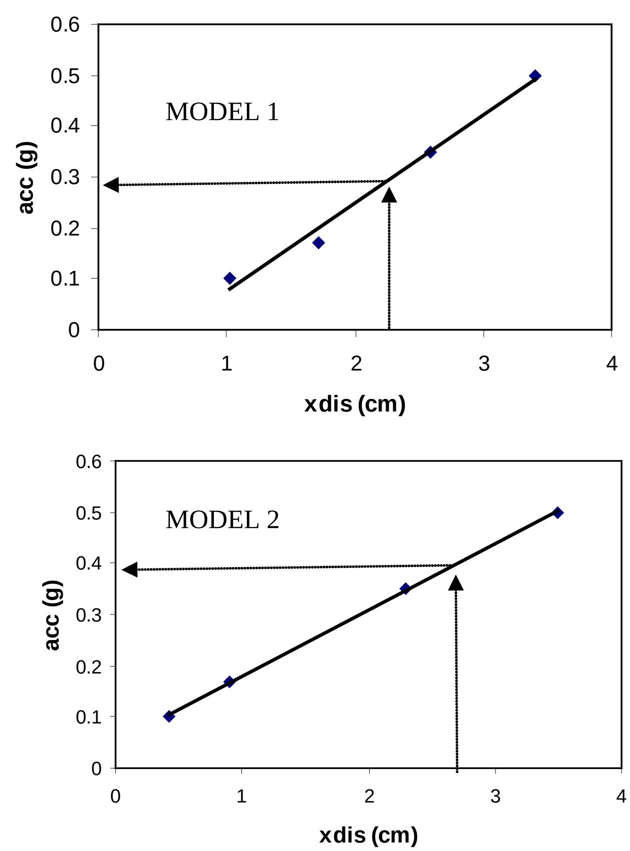


Fig. 10. Peak cyclic acceleration versus bottom-row pullout displacement

From the above plots, the critical seismic coefficient and the critical pullout displacement achieved. Now, the critical peak cyclic amplitude should be estimated. Figure. 9 shows the facing displacements for both models obtained from cyclic analyses. The peak cyclic acceleration that lead to critical pullout displacement is the critical seismic amplitude. Using plot of peak seismic amplitude versus mean bottom-row nails pullout displacement in one cycle (Fig. 10) and estimate of seismic amplitude at the critical pullout displacement, the critical seismic amplitude can be obtained. 0.29 for model 1 and 0.39 for model 2 are extracted from the plots in Fig. 10. As a result, it would be noted that the ratio of seismic coefficient to peak cyclic acceleration is estimated 0.58 for both models.

CONCLUSION

Numerical pseudo-static method is presented for analysis of nailed soil-slopes. Earthquake effects are considered in terms of seismic coefficient-depended forces. Results show that good agreement is found between pseudo-static and cyclic analyses methods. The ratio of the input seismic coefficient in pseudo-static analysis to the design peak amplitude in cyclic analysis can be evaluated regarding the index of critical pullout displacement. Results of considered models indicate that the mentioned ratio ($k_h.g/a$) may be estimated about 0.58.

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