Permanent Displacement of Nailed Soil Slopes Subjected to Earthquake Loading

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SUMMARY:
In-situ soil slopes and embankments are often reinforced with nails to improve their static and seismic performance. Michalowski and You (2000) developed an approximate method based on kinematic approach of limit analysis to estimate the permanent displacement of geosynthetic-reinforced soil slopes subjected to earthquake loading. In this paper, this approximate method is verified through finite element (FE) analysis of nailed soil slopes considering the soil and the nail as nonlinear and linear elastic materials, respectively. Radiation damping has been considered by using Lysmer-Khulemeyer (L-K) dampers at the soil boundaries of the FE model. Soil is assumed to be dry and cohesionless, and analyzed under plane strain conditions. The permanent displacements from approximate method and FE analysis have been compared. It is found that the displacements from FE analysis are considerably (more than 10%) less than those from approximate method.

Keywords: Nailed slope, soil nonlinearity, FE model

1. INTRODUCTION

The stability of soil slopes under static as well as seismic loading is one of the major concerns in geotechnical engineering. The stability of soil slopes is usually enhanced by reinforcing them with different reinforcing materials such as geosynthetics, nails, etc. Among the different reinforcing materials, nails are popular one. Nails are slender steel bars, usually galvanized or coated with epoxy resins to protect them against corrosion. In soil nailing, nails are either inserted into drilled bore holes, and then covered by grout along their length or driven directly into the soil. Nails are provided with flexible or rigid facing called as shotcrete. Sometimes isolated nail heads are also provided. Nails are installed either horizontally or inclined. The spacing between nails is usually in the range of 1m to 2m.

There are several well established methods to assess performance of slopes under static loading. However, seismic stability of reinforced slopes needs further understanding to gain sufficient confidence in design. The conventional design of nailed soil slopes subjected to earthquake loading is done by pseudostatic method. However, this method results in large length of nails for a significant magnitude of design acceleration and, thus becomes uneconomical (Ausilio et al., 2000). If soil slopes are designed for some allowable permanent displacement instead of force, the amount of reinforcement gets reduced significantly (Michalowski and You, 2000). However, there is no well-established method for estimation of the permanent displacement of reinforced soil slopes subjected to earthquake shaking. Newmark (1965) devised a method to estimate permanent displacement of unreinforced soil slopes subjected to earthquake loading. However, this method cannot be applied directly to estimate the permanent displacement of reinforced soil slopes. To overcome this difficulty, Michalowski and You (2000) developed a method based on kinematic approach of limit analysis to estimate the permanent displacement of soil slopes reinforced with geosynthetic. However, this method needs further verification to get accepted as reliable method for seismic design of reinforced soil slopes.

In the present work, the soil slopes reinforced with nail have been designed by the approximate method first and then the finite element (FE) analysis has been carried out using the open-source code OpenSees (Mazzoni et al., 2006).

2. DESIGN OF NAILED SOIL SLOPE BY APPROXIMATE METHOD

Michalowski and You (2000) performed displacement analysis of geosynthetic-reinforced slopes subjected to seismic loading and produced design charts. They used the kinematic theorem of limit analysis for this purpose. They assumed two types of failure mechanisms of slope. These include rotational collapse and direct sliding of slope over the bottommost layer of reinforcement. In this analysis they assumed that all reinforcement layers are of equal length and reinforcement strength distributions are of two types, uniform and linear. They calculated critical acceleration by incorporating pseudostatic method along with limit analysis. This critical acceleration was used in Newmark's sliding block method (Newmark, 1965) to estimate the permanent displacement of slope. The steps to be followed in design of nailed soil slope by the approximate method are described in brief in the following section. The charts produced by Michalowski and co-authors (Michalowski, 1998; Michalowski and You, 2000) have been used in this design.

2.1. Step 1: Assume Critical Acceleration (k_c)

First, a value of critical acceleration (k_c) is assumed for the slope under consideration. There is no hard and fast rule to select the critical acceleration. Higher value of critical acceleration signifies that the slope will take higher seismic load before it starts sliding. To take care of this large seismic load, the amount of required reinforcement will be large. It means that higher the critical acceleration, larger the amount of reinforcement required and, lesser the permanent displacement of the slope. To begin with, small value of critical acceleration (say, 0.1g) is assumed and, amount of reinforcement and permanent displacement is calculated. If this calculated permanent displacement is higher than permissible permanent displacement, then a higher value of critical acceleration is chosen and design is repeated till the calculated permanent displacement is less than or equal to permissible permanent displacement.

2.2. Step 2: Determine Dimensionless Strength Parameter

In this step, a dimensionless strength parameter $k_t/\gamma H$ is determined corresponding to k_c. In this expression, k_t is the average strength of reinforcement to be used in slope and is given by the following equation:

$$k_t = \frac{nT}{H} \tag{2.1}$$

where n is the number of reinforcement layers to be used, T denotes the tensile strength of single reinforcement layer per unit width, γ is the weight density of soil and, H is the height of slope. k_t gives the amount of reinforcement to be used in the slope. The value of k_t depends on the design friction angle of soil and the slope angle. The dimensionless strength parameter is read from the design charts produced by Michalowski (1998).

2.3. Step 3: Calculate Length of Reinforcement

The length of reinforcement required to prevent rotational collapse of slope is different from that required to prevent direct sliding of slope. This length is read in the form of length ratio in the design charts. Length ratio is the ratio of length of reinforcement required to prevent particular slope failure (L) to the slope height (H). Two length ratios are read from the design charts, one for rotational collapse and another for direct sliding. The failure mechanism corresponding to this greater length ratio governs the design and is used to calculate the required length of reinforcement.

2.4. Step 4: Check Toe Displacement

The toe displacement (u_x) is determined from the following equation:

$$u_x = CI \tag{2.2}$$

where C denotes the displacement coefficient which takes care of slope geometry, failure mechanism and, soil and reinforcement properties. The value of this coefficient differs according to the failure mechanism. The value of C corresponding to the governing failure mechanism is read from the design charts and used in the Eqn.2.2 to calculate the toe displacement. I is the displacement integral which is double integration of that part of design earthquake record for which design acceleration exceeds critical acceleration. The parameter I can be obtained from the charts given by Michalowski and You (2000) corresponding to the design earthquake record and (k_m- k_c), where k_m is the peak acceleration of design earthquake record.

If the toe displacement is greater than the maximum permissible displacement then design is repeated from step 1 to step 4. The maximum permissible displacement is 0.005H, where H is the height of slope in meter.

2.5. Step 5: Calculate Number and Depth of Reinforcement Layers

Number of reinforcement layers (n) is calculated by using Eqn.2.3 as given below,

$$n = \left(\frac{k_t}{\gamma H}\right)\left(\frac{\gamma H^2}{T}\right) \tag{2.3}$$

If the reinforcement strength distribution is assumed to be uniform then the depth is calculated by Eqn.2.4 as given below,

$$z_i = (i - 0.5)\left(\frac{H}{n}\right) \tag{2.4}$$

If the reinforcement strength distribution is assumed to be linearly increasing then the depth is given by the Eqn.2.5 as given below,

$$z_i = \frac{2}{3} nH\left[\sqrt{\left(\frac{i}{n}\right)^3} - \sqrt{\left(\frac{i-1}{n}\right)^3}\right] \tag{2.5}$$

where z_i is the depth of i^{th} layer, i is the sequence of layer, H is the slope height in meter and n is the number of reinforcement layers.

3. FINITE ELEMENT MODELING

The finite element modeling of slope in present study has different issues such as constitutive behavior of soil and nail, meshing, boundary conditions, application of the earthquake loading, etc. These issues have been discussed in brief in the following paragraph.

The constitutive behavior of the soil has been modeled using a pressure dependent multi-yield (or nested yield) surface plasticity model available in OpenSees. For meshing of the soil, the four node

quadrilateral element has been used. This element has four gauss points. Each node has two translational degrees of freedom. The nail has been modeled as linear elastic beam-column element with three degrees of freedom at each node. Out of which two are translational and one is rotational. To have better accuracy of results, the meshing has been made finer near the nails as shown in Fig. 3.1. Two cases of soil-nail interface have been considered. In first case, it is defined as perfect bonding contact and in another case it is defined with interface element. The interface element works on the principle of penalty method. To take care of the reflection of waves from the side and base boundaries of the model, the Lysmer-Kuhlemeyer's (Lysmer and Kuhlemeyer, 1969) radiation dampers have been provided. For the very same purpose side boundaries are kept 100 m apart from the toe of slope. The earthquake motion has been applied at the base of the model in the form of equivalent nodal shear forces proposed by Joyner and Chen (1975), and Ayala and Aranda (1977).

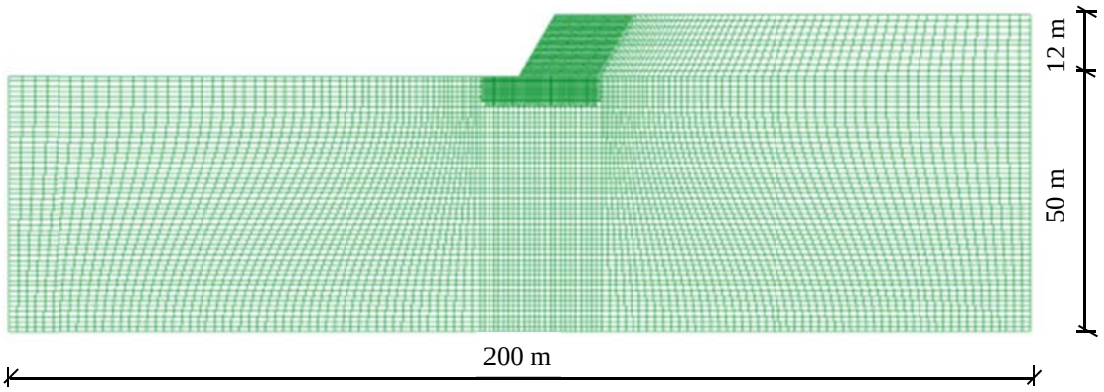


Figure 3.1. Finite element meshing

3.1. Verification of FE Model with Shake2000

Verification of the FE model in OpenSees has been done with Shake2000. Shake2000 is used for equivalent 1-D Linear analysis of soil subjected to earthquake (Ordóñez, 2004). For verification purpose, the material used is elastic isotropic. In Shake2000, this task is achieved by defining G/G_{max} ratio to be 1 for all strain values and zero damping is assumed. Discretization of OpenSees model and Soil column in Shake2000 is same. It is specified that maximum thickness of layer to be defined in Shake2000 should be less than 2.0 m. Total 46 layers have been defined in Shake2000. The topmost 20 layers have thickness of 0.5 m each, the middle 20 layers have thickness of 1.0 m each and the bottommost 6 layers have thickness of 1.5 m each.

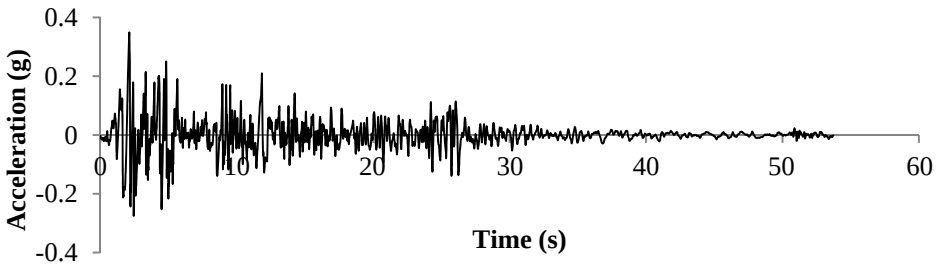


Figure 3.2. Input acceleration time history for verification

The north-south component of the 1940 Imperial Valley Earthquake recorded at El-Centro has been used as the input motion (Fig. 3.2). This motion has peak acceleration of 0.314g. Verification is done by comparing peak horizontal acceleration profile across the depth from both models as shown in Fig.

3.3. There is about 20% of difference in the two profiles, for a small depth range at the base of model. However, for the rest of the depth, there is good match between the two profiles.

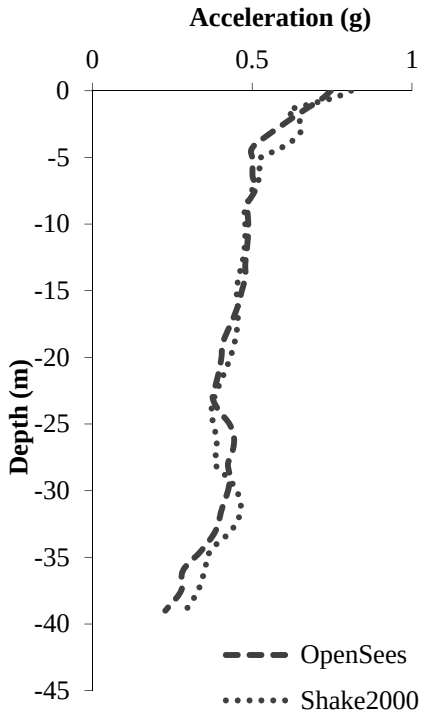


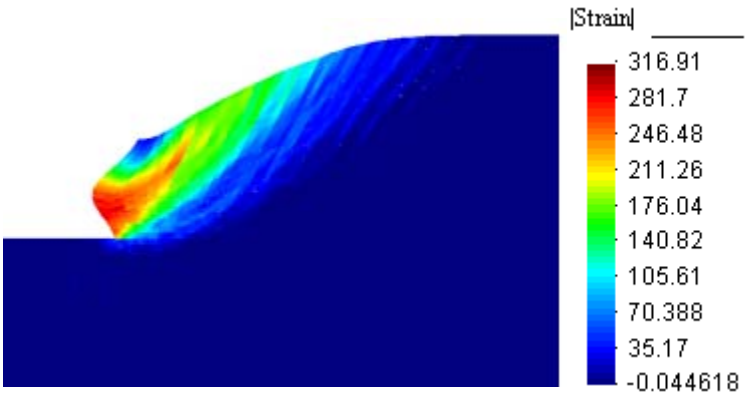
Figure 3.3. Peak horizontal acceleration profile across the depth

4. RESULTS AND DISCUSSIONS

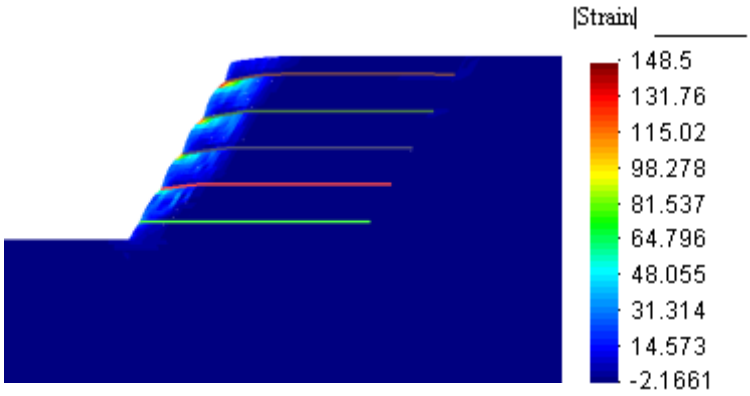
In the present study, three models with slope angles 60°, 45° and 30° have been considered. Each of these slopes has been designed by approximate method to obtain the number of nails, depth of nails following the procedure described in Section 2. The slope with 60° inclination angle has been designed for the critical acceleration k_c equal to 0.2g and the other two slopes are designed for k_c equal to 0.1g. The finite element analysis of the slope with the designed nails is performed in OpenSees. The south-east component of Imperial Valley earthquake (1940) recorded at El Centro has been used as the design earthquake motion.

Seismic analysis has been performed for three cases, namely, slope without nail, slope with nail and perfect bonding contact, and slope with nail and interface element. Strain contours at the end of the seismic analysis of soil slope (60°) for the three cases are shown in Fig. 4.1. It is observed from the Fig. 4.1a that the failure surface is more or less log-spiral and it passes through the toe of the slope. From the Figs. 4.1b and 4.1c, it is clear that the failure surface becomes planer and narrow in presence of nails. Further, it is observed that the failure surface passes through toe of the slope. This slope had in fact experienced severe deformations (failure) under gravity loading itself. Displacement at crest of the 60° slope has been given in the Table 4.1. It is clear that the deformation of slope is maximum for the slope without nail and minimum for the slope with nail and perfect bonding. It is observed that there is significant reduction in the deformation of slope in presence of nails.

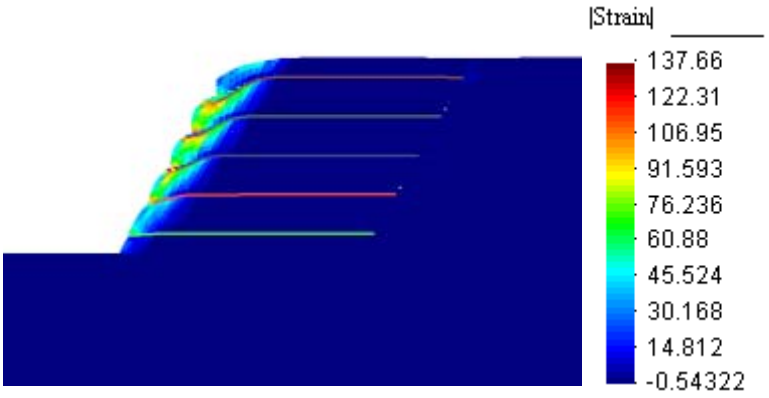
The permanent displacement of nailed slope obtained from the approximate analysis and FE analysis have been compared in the Table 4.2. In case of 60° slope, the displacements from approximate method and FE analysis were found to be very close. But for rest of the two slopes, there is large discrepancy between the two displacements. It is observed that for all the considered cases of slope the displacement from the FE analysis is less than that from the approximate method.



(a) Slope without nail



(b) Perfect bonding between soil and nail



(c) Interface elements between soil and nail

Figure 4.1. Strain contours at the end of seismic analysis (slope 60°)

Table 4.1. Displacement at the crest of the slope (60°)

Cases	Crest displacement (mm)	
	X	Y
Without nail	5690	6259
Nail with perfect bonding	312	538
Nail with interface elements	937	1409

Table 4.2. Permanent displacement at toe

Slope angle (degree)	k_c	Cases	Toe displacement (mm)
60	0.2	Approximate method	16.9
		FEM-with nail (perfect bonding)	21.9
		FEM-with nail (interface elements)	15.0
45	0.1	Approximate method	63.5
		FEM-with nail (perfect bonding)	17.3
		FEM-with nail (interface elements)	14.7
30	0.1	Approximate method	64.3
		FEM-with nail (perfect bonding)	14.3
		FEM-with nail (interface elements)	12.4

This is the reason why the displacement from approximate method for slope 60° is considerably less than that for rest of the two slopes. One more observation contradictory to the intuition is that the displacement for perfect bonding contact case is more than that for contact defined with interface element. The perfect bonding contact adds rigidity to the soil-nail system because of which the soil-nail acts as a single unit. This results into the increase in inertia force of the system. This increase in inertia force causes increase in the displacement at toe. In case of interface elements the soil-nail system is flexible and hence it does not act as single unit. This results into smaller values of toe displacement in comparison with those from the case of perfect bonding contact.

5. CONCLUSIONS

Pseudo-static method and Newmark's sliding block method are the most widely used methods for the seismic design of slopes. The pseudo-static design of such slopes results into nails of very large length and thus the design becomes uneconomical. On the other hand, Newmark's sliding block method cannot be directly applied to estimate this permanent displacement of nailed slope. In this study, approximate method proposed by Michalowski and You (2000) is used to design the nailed slope. Permanent displacement obtained from the approximate analysis is compared with that obtained from finite element analysis. It is observed that the permanent displacements obtained from finite element analysis are smaller (10% to 80%) than those from approximate method irrespective of the slope angle. This difference reduces with increase in the critical acceleration chosen in the approximate method of design. However, for small values of critical acceleration, say 0.1g, this difference is significant. The permanent displacement obtained for perfect bonding contact is higher than that when sliding and separation are allowed at the interface between soil and nails. In the present study the slope was designed and analyzed for only one earthquake motion. To get more confidence on the approximate method the slope should be designed for different earthquake motions and analyzed.

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