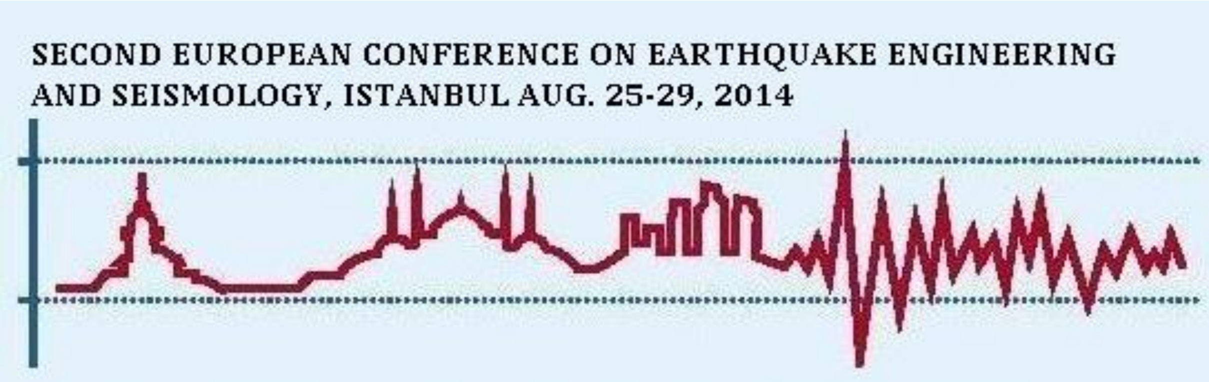

PERFORMANCE BASED STABILIZATION OF SEISMICALLY UNSTABLE SLOPES BY LARGE DIAMETER PILES

Ali Güney ÖZCEBE[1,2], Carlo Giovanni LAI[3,4], Mario MARTINELLI[5]

ABSTRACT

This paper aims at summarizing the methodology, proposed by Özcebe (2014) to estimate the amount of reduction in residual displacements demand of pile-reinforced slopes through *normalized mean stabilization curve*s. Özcebe (2014) carried out numerous advanced numerical analyses using FLAC^{2D} and FLAC^{3D} commercial computer programs (Itasca, 2011 and Itasca, 2009) which were functional for setting up an innovative performance-based design method. Relatively simple elastic-plastic constitutive law with hysteretic unloading-reloading associated with Mohr-Coulomb yield criterion has been chosen to idealize soil response which had been tested and calibrated on the basis of two seismic centrifuge tests carried out in the ISMGEO laboratory at Seriate in Northern Italy, which stands out of scope of the current paper.

INTRODUCTION

Modern day seismic performance evaluation of slopes is being carried out in terms of residual or irreversible displacement demand. Increased computational performance permits advanced numerical modelling of the seismic response of slopes which, to a great extent, is earthquake-dependent. Still, numerical modelling is expensive in terms of time and effort which may be impractical for preliminary and rapid assessment purposes. Moreover, accurate numerical modelling requires skilled and experienced analysts to avoid erroneous estimations. As an alternative to numerical analysis, use of simplified analytical methods (e.g. Newmark, 1965) or predictive frameworks (e.g. Jibson, 1993; Bray and Travasarou, 2007; Saygili and Rathje, 2008) could also be used in rapid and preliminary assessment of residual displacements, particularly in unreinforced and natural slope settings. Once the estimated displacement demand is found to exceed the predetermined threshold displacement (which can be considered as seismic capacity), slope is considered as "seismically unstable"; so that a mitigation action needs to be implemented.

Reinforcement with large diameter piles represents just one possible mitigation measure. In this method, reinforcement design (i.e. location, configuration, capacity, etc.) could be finalized through advanced numerical load-deformation modelling or tentatively obtained by making use of modified limit-equilibrium techniques (e.g. Yamin, 2007). The use of the former is expected to be more accurate and precise compared to the latter; however, it again suffers from the same concerns mentioned previously. On the other hand, the latter option is very practical to determine the internal actions on the reinforcement; yet, it does not consider the amount of reduction in the displacement

[1] Dr., Politecnico di Milano, Milano aliguney.ozcebe@polimi.it
[2] PhD graduate of ROSE Programme, UME School, IUSS Pavia, Pavia, agozcebe@roseschool.it
[3] Prof. Dr., University of Pavia, Pavia carlo.lai@unipv.it
[4] Prof. Dr., Head of Geotechnical Earthquake Engineering Group, EUCENTRE, Pavia, carlo.lai@eucentre.it
[5] Dr., Deltares, Delft, mario.martinelli@deltares.nl

demand which is the engineering demand parameter used in the assessment of seismic stability. Being consistent with the initial engineering demand parameter, this paper aims at summarizing a methodology, which is based on numerical modelling, in terms of the reduction in the reversible displacement demand.

NUMERICAL MODELING

Complex slope-pile-rock interacting system is influenced by (i) the seismic motion, (ii) slope yield acceleration (k_y), (iii) reinforcing structure properties (geometry, strength, stiffness, and ductility), (iv) slope setting and geometry (soil profile, slope depth, length, possibly also spacing) and (v) properties of the slope material (stress-strain behaviour, initial and secant shear moduli, degree of saturation, etc.) and (vi) stable rock/firm soil properties (i.e. shear strength, shear wave velocity, etc.). This section gives very brief information on each component of the numerical FLAC^{2D} (Itasca, 2011) and FLAC^{3D} (Itasca, 2009) models.

Earthquake set contains 10 actual rock outcrop records which are selected and scaled to stand for the representative ground motions in Europe with moderate to high intensity ($PGA_r \cong 0.30g$) (Zuccolo, 2012). In order not to modify the real record significantly, scaling factors had been kept between 0.35-3.35 with an average of 1.76. Figure 1 shows the spectrum compatible records used in the non-linear dynamic analyses.

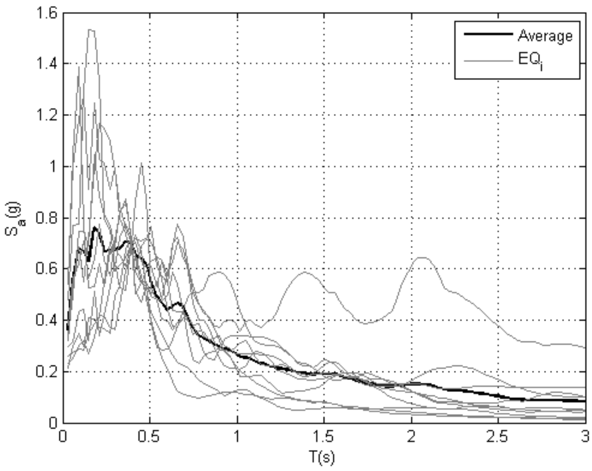

Figure 1. 5% damped elastic pseudo acceleration response spectra of the earthquake set (records are scaled so that linear average of spectral accelerations is spectrum compatible)

Slope yield acceleration is another important parameter, which is effectively combined with the PGA throughout the seismic performance evaluation of the natural slopes. Two different yield acceleration value has been chosen as 0.05g and 0.10g so that k_y/PGA_r ratios are be roughly equal to 0.16 and 0.32; respectively.

Slope geometries chosen in the numerical simulations have been selected to show heterogeneous setting with a large impedance contrast (i.e. bottom layer V_s= 700 m/s and top layer $V_s \cong$ 130-140 m/s), so that soft layer overlies on a much stiffer stable layer. Such configuration has been used to generate slopes having yield acceleration (k_y) of 0.05g and 0.10g for shallow (H= 4m) and relatively deep (H= 6m) seismically unstable layer thickness values under fully saturated (S=100%) and nearly fully dry conditions (S$\cong$ 0%). All of the slopes have the deterministic length of 50 m (which has been studied to be long enough, since D=1m positioned with maximum spacing of 4m). Figure 2 shows an example slope geometry in the absence of the reinforcement (i.e. pile layer).

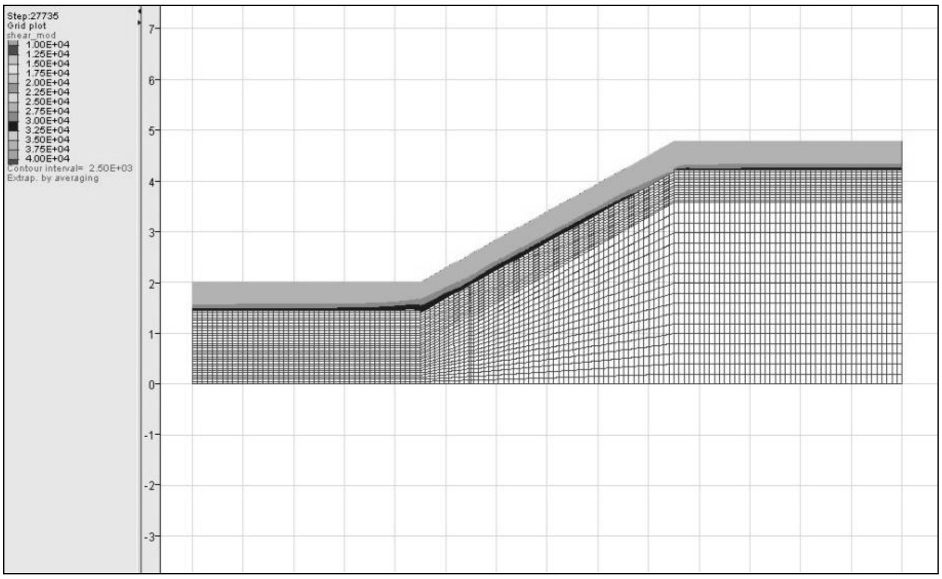

Figure 2. Geometry of one example case study in determining the seismic response of unreinforced slopes ($H=6m$, $S\cong0\%$, $ky=0.05g$, $L=50m$, $FLAC^{2D}$ model)

Seismically unstable soil layer is assumed to be a generic material with PI= 30, in which the hysteretic response may be represented by Darendeli (2001) family of curves. Unloading and reloading branches are represented by Masing's rule (Masing, 1926). Figure 3 shows the stress-strain curve of the PI=30 material implemented in the FLAC analyses. Small strain shear wave velocity of the mentioned layer is 165 m/s at an isotropic effective confining pressure of 1 atm conditions. Corresponding V_s values are then implemented according to the real stress state through in terms of FISH functions.

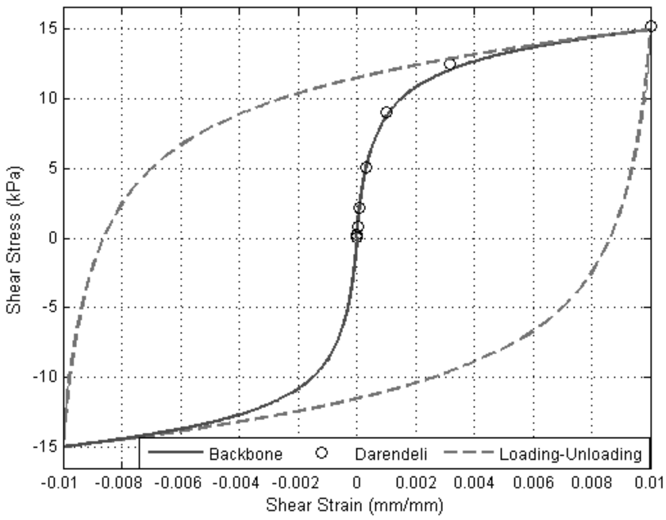

Figure 3. Comparison of calibrated hysteretic relation (of PI=30 material) in terms of stress-strain

Assignment of the strength parameters has a strong dependence on the water content. It has been previously mentioned that the degree of saturation values are either selected to be nearly completely dry ($S\cong0\%$) or completely saturated ($S=100\%$). Several researchers have shown that partial saturation causes the capillary cohesion due to the micro-scale interaction between the pore fluid and the soil particles. In fact, this effect becomes more and more significant as the particle size decreases and the fine (silt and clay) content increases (Bishop, 1959; Bishop and Blight, 1963; Fredlund et al., 1979, and many others). Interaction forces between the pore water and the small size particles can be so high that the perfectly dry conditions are unlikely to occur in the nature. Due to this reason, modest cohesion of 5kPa is added along with the 27^o of internal friction angle under nearly dry conditions. For fully saturated condition, the strength is modelled through undrained shear strength utilizing the total stress concept. Softening due to excessive pore pressures are not considered due to the presence of the fines. As it is widely accepted, undrained shear strength is a function of the effective confining pressure. This dependency has been modelled through the use of Skempton's A and B parameters (Skempton, 1957). Equation 1 formulizes the Skempton's equation with A=1/3, valid for linear elastic material/ lightly OC clays and B=1 for fully saturated conditions. In Equation 1; Δu represents the change in the pore water pressure, $\Delta\sigma_1$ is the change in the major principal stress and $\Delta\sigma_3$ is the minor principal stress.

$$\Delta u = \left[\Delta\sigma_3 + \left(\Delta\sigma_1 - \Delta\sigma_3\right)/3\right] \qquad (1)$$

After the relevant mathematical manipulations, Equation 2 could be obtained (Özcebe, 2014). In Equation 2; S_u is the undrained shear strength (in kPa) and σ'_0= isotropic effective confining pressure (atm)

$$S_u = 53.756\sigma_0^{'} \qquad (2)$$

Poulos (1995) and Kourkolis et al. (2011) suggested that earth-induced bending moment should stay below the yield limit, since they perform better in stabilizing purposes (larger forces could develop). Due to this reason, in this study, reinforcing piles are modelled through linear elastic continuum elements (with D= 1m, ρ_l=3%, f_y=500 MPa, f'_c=35 MPa) and the actual elastic moment has been checked not to exceed its yield value. Cubrinovski et al. (2006; 2009) showed that as the amount of relative displacement between soil and pile increases, the amount of pressure acting on the reinforcing piles also increase with a decreasing pace. So it would not be surprising that the majority of the maximum soil pressure could be obtained with relatively closer spacing values. Actually, optimum stabilization has also been reported with the spacing of s= 4D (Yamin, 2007; Kourkolis et al., 2011) under static and creeping failure conditions, such that the force acting on the piles could be maximized without significantly loosing the group effect. Understanding the fundamental similarity between steady state part of the dynamic response with the mono-directional static/creeping conditions, this study takes s= 4D as the maximum spacing that could be allowed to satisfy sufficient group effect between the individual piles, so that the response of the reinforced systems are investigated with spacing/diameter (s/D) configurations of 1, 2, and 4. S/D=1 represents continuous wall (so that it is plane strain and modelled in FLAC^{2D}) while s/D=2 and s/D=4 configurations impose 3D effects and they are modelled in 3D solver (i.e. FLAC^{3D}). Figure 4 shows a portion of the 3D grid for one of the combinations.

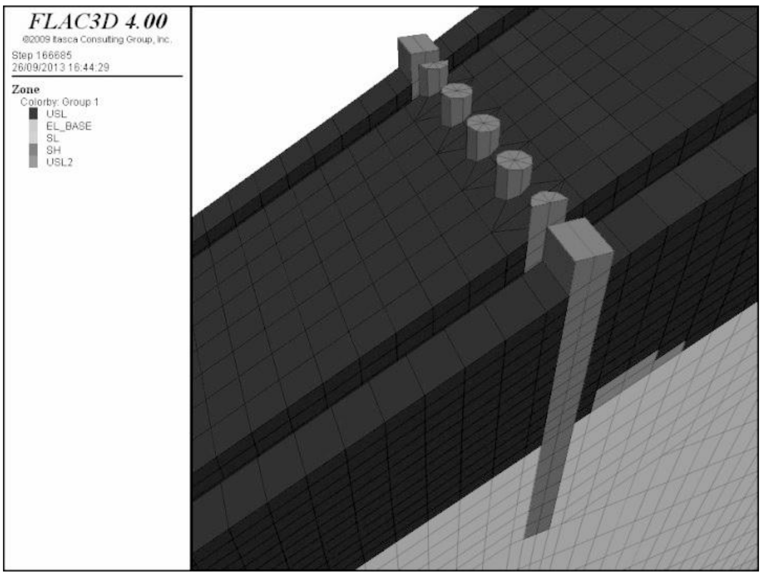

Figure 4. Screen capture of the grid used in 3D finite difference simulations

Locations of the piles are also studied by changing the position of the stabilizing pile row between 10m (ST1), 25m (ST2), and 40m (ST3) distant away from the toe side.

Once the static geo-stresses are found through explicit time-stepping, earthquake excitation is given in terms of shear stress in the explicit codes. As for the dynamic boundaries of the finite difference models, free-field boundary conditions (Cundall, 1980; Itasca, 2009; Itasca, 2011) are imposed for the sides of the numerical grid while Lysmer and Kuhlemeyer (1969) is used for the bottom faces.

METHODOLOGY, SAMPLE RESULTS AND FORMULIZATIONS

In this study, the following steps have been followed in the given order:

1. Slope inclinations are decided for different slope thicknesses under different yield accelerations and saturation conditions through factor of safety type analyses by making use of uniform yield acceleration.
2. Un-reinforced slope responses are obtained in FLAC^{2D} for all of the configurations in terms of the irreversible seismic-induced displacement amount. Figure 5 shows one illustrative example.

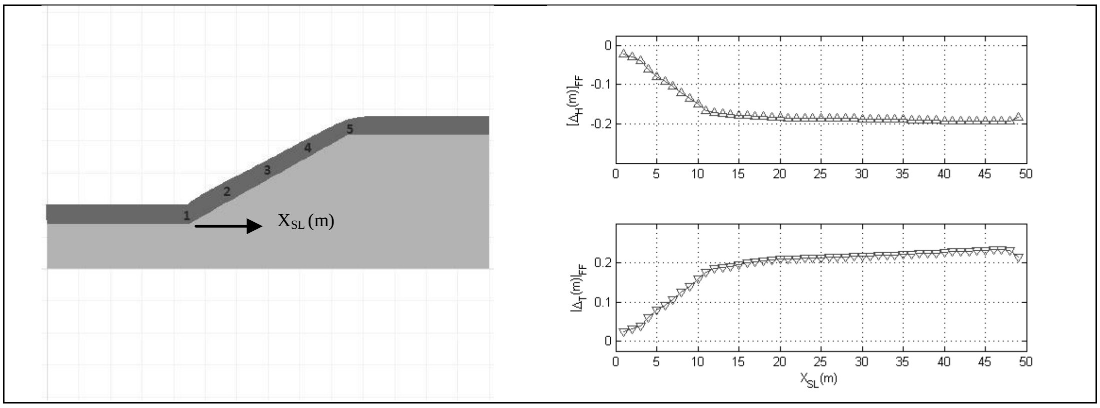

Figure 5. Performance of an unreinforced slope obtained at the end of an earthquake

3. Reinforced slope response is obtained under plane strain conditions in FLAC^{2D}. Figure 6 shows one illustrative example. This is repeated for all of the yield acceleration/soil thickness/saturation/earthquake/pile position combinations.

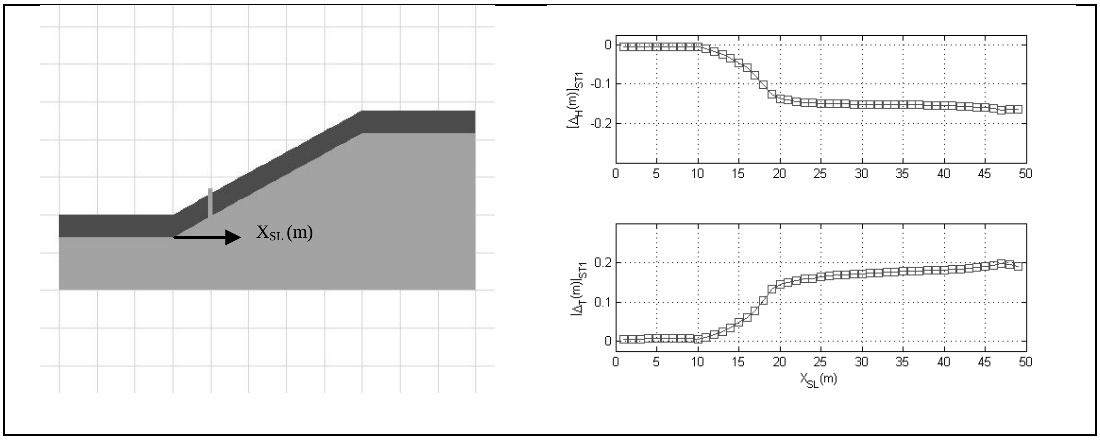

Figure 6. Performance of a reinforced slope obtained at the end of an earthquake

4. For specific cases, FLAC^{3D} analyses are run and their results are compared with the plane strain conditions. Good accordance has been noted in most of the cases, which means that

s=4D could be used as the upper-bound with care. Figure 7 shows one example correspondence.

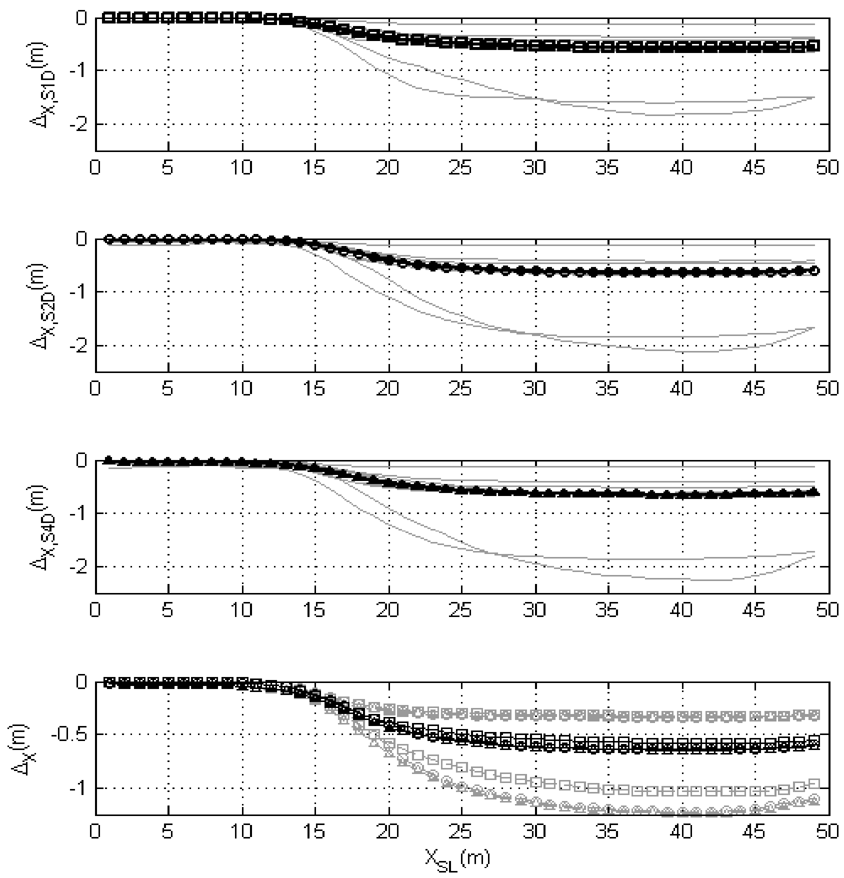

Figure 7. Comparison of FLAC^{3D} analyses results with FLAC^{2D} results for one pile location, yield acc., slope thickness and saturation condition (Top: s=1D (i.e. FLAC^{2D}), Second, s=2D (i.e. FLAC^{3D}), Third, s=4D (i.e. FLAC^{3D}), Last: All)

5. *Stabilization curves* are obtained by dividing Fig 6 results to the mid displacement of the Fig 5 results for all of the cases (Equation 3).

$$\delta_{ST}(x_{SL}) = \frac{\Delta_{ST}(x_{SL})}{\Delta_{FF}(X_{SL} = 25m)} \tag{3}$$

In Equation 3; $\Delta_{ST}(x_{SL})$ shows the residual displacement profile of the stabilized slope, $\Delta_{FF}(X_{SL}=25m)$ is the residual displacement of the mid-slope for the unreinforced case, $\delta_{ST}(x_L)$ is the stabilization profile for the case considered. For each set of parameter, excluding the earthquake set, *mean ± 1 standard deviation* values are obtained to represent the aleatory variability in the input motion.

6. Normalized charts are constructed from *mean stabilization curves* which were obtained by taking the ratio of the residual displacements obtained in the previous step. Figure 8 illustrates one example case (H=6m, S≅ 0%, k_y=0.05g, k_y/PGA_r=0.16) of such calculation.

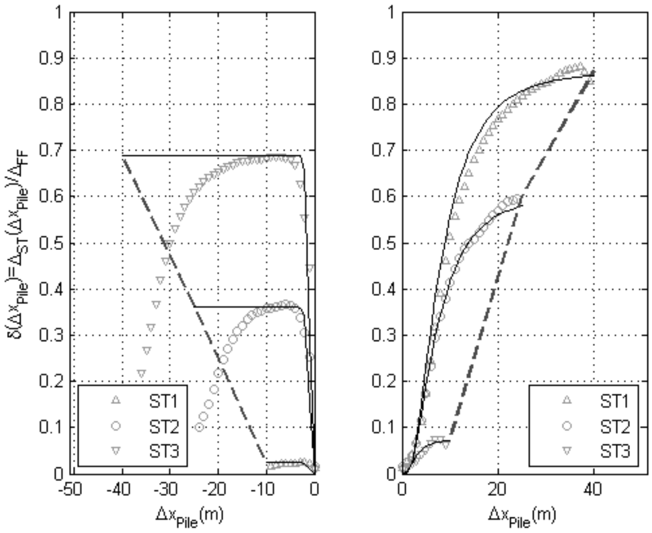

Figure 8. *Sample normalized mean stabilization curves* for three different reinforcement locations: ST1 (closer to the toe), ST2 (at the middle), ST3 (closer to the crest) for one example case (H=6m, S≅0%, k_y=0.05g, k_y/PGA_r=0.16)

It must be noted that spurious stabilization effect at the toe side is purely due to slope-dependent reasons and occurring due to planar failure wedge obtained in nearly dry soil with some fines. A scaled logarithmic cumulative density function containing three parameters was chosen to fit the results of numerical simulations as shown below:

$$\delta_{ST}(x_{SL}) = A \cdot CDF(\ln(x_{SL}), \mu, \sigma) \tag{4}$$

In Equation 4, A stands for scaling parameter, Δx_{pile} is the distance of a point in the mid-slope from the center of the reinforcing pile (always positive), μ is the median and σ is the standard deviation parameters that mathematically define the equation of the fitting model. CDF is the cumulative density function.

For Figure 8; behind of the pile row: A= [0.024 0.36 0.69], μ= [0.075 0.075 0.075], σ= [0.375 0.375 0.375]. In front of the pile row: A= [0.87 0.60 0.075], μ= [2.05, 1.95, 1.05], σ= [0.70 0.70 0.70] for [ST1 (10 m distant to toe) ST2 (25 m distant to toe) ST3 (40 m distant to toe).

CLOSURE AND COMMENTS

This manuscript serves as an introductory paper for the work included in the PhD thesis of Özcebe (2014) by explaining the methodology and illustrating the sample results obtained. Methodology is much more in-detail explained with complete set of parameters inside the aforementioned reference to which interested readers are kindly referred.

To sum up, in this proceeding, effect of stabilization provided by discretely spaced single row pile system has been studied in terms of improved residual seismic-induced soil displacements by numerical modelling in finite difference solvers $FLAC^{2D}$ and $FLAC^{3D}$. At the end, a framework has been offered to be used for preliminary design purposes of such systems. For the final or any other stages of detailed designs, carrying out site and project specific dynamic time history analyses are strongly recommended. Finally, the proposed methodology:

- Is believed to be a handy relation in the first estimation of the stabilized displacement amount, however, it has never been claimed to be accurate for all possible range of parameters (it is subject to change with different slope settings/slope materials/input motion combinations),
- Is only applicable for preliminary design purposes, and should never be replaced advanced stress-deformation analyses that are necessary to be carried out at the final design stage,

- Is developed for slopes not having a discrete failure surface (i.e. there is no pre-determined weak plane),
- Is developed for heterogeneous slope formations which contain significant stiffness contrast,
- Is developed for non-softening type of geo-materials. For softening soils, residual properties should be used while determining the yield acceleration,
- Does not consider the embedment depth and short pile type of behaviour should be omitted with sufficient embedment depth,
- Does not consider the effect of liquefaction and it must be avoided for the slopes where significant pore water pressure is likely to develop.

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