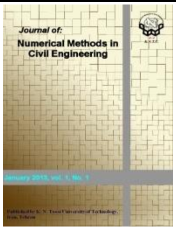

Numerical Methods in Civil Engineering

A simplified approach for evaluation of seismic displacements of pile group located in soil slope

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Abstract:

Seismically induced slope movements have imposed severe damage on the pile-supported structures located on soil slopes during past earthquakes. Consequently, evaluation of the lateral seismic response of pile-slope systems is an important measure towards safe design of pile groups. However, since the codes and techniques used in seismic design of pile groups for engineering purposes are neither practical nor easily accessible, it is difficult to employ them consistently in design procedures. Therefore, simplified approaches are required for pile design application. This paper presents a novel approach that practitioners can use to quickly evaluate the seismic displacements of pile-groups in soil slope. Such an approach is based on the reasonable and practical relationship found between the pseudo-static safety factor of pile-slope system and seismic displacements of pile groups. In order to explore this relationship, a parametric study was performed and a dimensional analysis was carried out to study the results of the relationship and achieve the dimensionless chart. Thus, the conclusions of this study are intended to provide practitioners with some practical guidelines such as low calculation efforts, and to incorporate the relationship between slope safety factor and pile group displacements into the design process.

1. Introduction

Many coastal and harbor structures, bridges, high-rise buildings and transmission towers are constructed on soil slopes and are supported by pile groups. These structures may be subjected to induced lateral loads due to the seismic movements of slope. In such conditions, existing analysis and design methods are mostly complex and time-taking and cannot be applied in analysis and design procedures consistently. In addition, project budgets often do not allow for in-depth numerical analysis. Therefore, practitioners are confronted with balancing computational efforts with anticipated benefits. Hence, simplified approaches become more pronounced for engineering purposes.

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Several analytical, numerical and experimental methods have been applied to study the seismic response of pile groups, but there appears to be little or no research that takes into account the seismic behavior of a pile group located within a soil slope, other than for the cases where the slope angle does not exceed 2° [1,2]. Consequently, a lot of complexities and unknown cases have remained in this area. The positioning of a pile within a soil slope has been considered in terms of the stabilization of unstable slopes in the absence of seismic activity [3].Since the approach in these studies has focused primarily on the increase in safety factor of the slope following the application of piles, the effects of piles has often not been considered. A number of studies can be found in the literature regarding the effects of inclined geometry of the ground on the lateral behavior of piles under static loading [4, 5].These observations have largely examined the effect of ground inclination on the lateral bearing capacity of piles. Moreover, numerous studies have been reported on the subject of seismic behavior of soil slopes [6, 7].Such analytical and/or experimental studies have focused on both the stability and the seismic deformations of slopes, and most involve the use of Newmark's sliding block theory [8].Pseudo static approaches for the seismic analysis of pile foundations are attractive for practicing engineers because they are simple when compared to difficult and more complex dynamic analyses. In the last years, several simplified approaches for analysis of single piles or pile groups have been developed that can be used with little computational effort [9, 10, 11]. These methods have produced results that are often in remarkable agreement with the mathematical models [12,13,14]. For example, Abghari and Chai [9] developed a pseudo-static procedure using a beam on nonlinear Winkler foundation model (BNWF) to evaluate the soil-pile-superstructure interaction. Following the pseudostatic approach, Tabesh and Poulos [14] presented a method based on simplified boundary element models for seismic analysis of a single pile with linear soil behavior. Elahi et al. [15] extended the Tabesh and Poulos [14] method to consider pile groups and soil yielding effects. They even produced a computer program named PSPG for pseudo-static analysis of pile group in horizontal ground. Afterwards, they employed relatively simple modifications and developed another program named PSPG-Slope, to evaluate the behavior of pile groups in the slope. While a number of simplified methods for seismic evaluation and design of pile foundations are available, there are comparatively few simple methods to consider the response of pile-slope systems. Thus, in this study, basic principles for achieving an engineering and simplified framework of seismic evaluation of pile group in soil slope leading to a better understanding of the seismic behavior of pile-slope systems are established, and a novel approach is developed. To this end, a pseudo-static method is used to take into account the seismic effects on slope stability and a comprehensive parametric study on the important parameters governing the response of pile-slope systems (including soil type, pile group configuration, slope angle, pile spacing (S) and input peak ground acceleration (PGA) is conducted to investigate the seismic behavior of pile groups which can be counted as a major difference from similar previous works. Subsequently, the results obtained from parametric analysis are correlated and practical charts and relationships enabling the estimation of displacements of pile groups are developed. The accuracy of the proposed approach is examined by the comparison of computed results with experimental and numerical results reported in the literature. Additionally, there is a strong emphasis in recent times that the seismic design of geotechnical structures should be based upon their performance during earthquakes. This situation is related to the public demand for cost effectiveness as well as desire for accurate information on extent of safety. To cope with this, the approach developed herein can be helpful. In this regard, the safety factor represents the performance of structure and based on its value and with respect to the amount of displacements, the structure is analyzed and designed.

2. Numerical Analysis

In the current study, two main stages have been considered for numerical analyses. The first stage is devoted to the safety factors and the second deals with dynamic analyses. In the following sections, each stage will be described thoroughly.

2.1. Slope Stability

The stability of a slope is represented by a factor of safety, which is the ratio of the shear strength along a critical failure surface and the shear stress induced on that failure surface by the slope. Owing to the main purpose of this study, which is investigation of seismic performance of the pile-slope system, seismic stability is considered. Pseudo static slope stability procedures are commonly used in engineering practice to evaluate the likely seismic performance of earth structures and natural slopes [15]. Hence, in order to account for the seismic effects on stability, the pseudo-static approach is employed, in which Seismic slope stability is evaluated using a pseudo static analysis where the whole model is assumed to be horizontally accelerated by the seismic coefficient k (in units of gravity), appropriately chosen for the expected seismicity of the site. It should be mentioned that only the horizontal component of earthquake shaking is considered and the vertical component is ignored because the effects of vertical forces tend to average out to near zero. In addition, since soil slopes are not rigid and the peak acceleration generated during an earthquake lasts for only a very short period of time, seismic coefficients used in practice generally correspond to acceleration values well below the predicted peak accelerations [17]. Terzaghi [18] originally recommended using k = 0.1, 0.2, and 0.5 for severe, violent destructive, and catastrophic earthquakes, respectively. In California, k is usually within the range of 0.15 to 0.3. Many researchers have recommended different factors of safety for various coefficients of k to attempt to remedy this situation. Thus, selection of the pseudo static coefficient is the most important aspect of pseudo static analysis, but it is also the most complicated. However, the choice of coefficients used in the slope stability analysis is very subjective and lacks a clear rationale and there are no hard and fast rules for the selection of this coefficient for designs [16]. Regarding selection of pseudo-static coefficient, it should be said that, as this coefficient is a function of maximum horizontal ground acceleration, only a fraction of peak acceleration should be selected for the coefficients. In addition, as mentioned above, there are no certain rules for selection of a pseudo static coefficient for design, but based on the recommendations cited in the literature, the most acceptable and commonly used value equals to one-half of the peak ground acceleration[17], which is used in this study. The shear strength reduction technique is utilized to calculate the factor of safety of slopes in this study. In this method, the factor of safety of a slope is estimated by the ratio of initial strength parameters to those that bring the slope to the point of failure. Accordingly, the shear strength parameters are reduced gradually with constant factors (factors of safety) in a series of analyses, until the occurrence of failure is detected [19]. Numerical examples appeared in the literature so far have shown that the shear strength reduction technique is an effective method for assessing the safety factor of the slope. The essence of the numerical methods such as FEM and/or FDM with shear strength reduction technique is the reduction of the soil strength parameters until the soil fails. The soil strength parameters c_f^* and Φ_f^* used in these procedures are defined as the actual shear strength parameters c^* and Φ^*divided by a shear strength reduction factor F_n(as shown in Fig. 1).

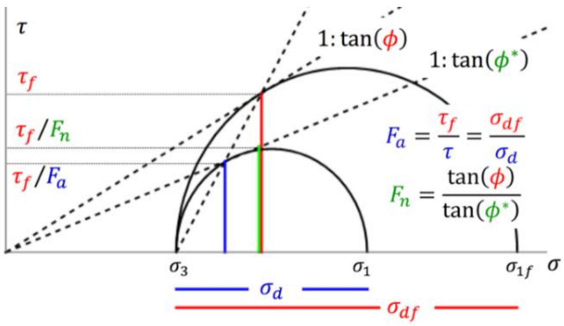


Fig. 1: Relationship between the actual strength and a strength reduced by a trial factor of safety

$$c_f^* = \frac{c^*}{F_n}, \Phi_f^* = \arctan(\tan\Phi^*/F_n) \quad (1)$$

The shear strength reduction factor F_n increases incrementally until collapse occurs. At this instant, the shear strength reduction factor comes to be the global minimal safety factor with the same meaning as the safety factor defined in the limit equilibrium method. Moreover, in the Fig below, F_a represents the ratio of potential failure stress to the shear stress acting on the same plane.

2.2 Numerical Simulation

The program used in this paper is finite difference program FLAC3D version 3.1 [19]. The calculation is based on the explicit finite difference scheme to solve the full equations of motion, using lumped grid point masses derived from the real density of surrounding zones rather than fictitious masses used for optimum convergence in the static solution scheme. Therefore, the response of slope-pile systems is analyzed by using a three-dimensional explicit-finite difference approach. The Mohr-Coulomb model was used to simulate the nonlinear soil behavior. This model was selected from among the soil models in the library of FLAC3D.The Mohr-Coulomb material model requires conventional soil parameters including unit weight (γ), friction angle (Φ), and cohesion intercept (c), shear modulus (G), and bulk modulus (B). Table 2 presents the soil parameters used in the FLAC model. The reason behind choosing this constitutive model is that currently, it is very common in practice to employ the Mohr-Coulomb model for simulating the soil behavior in the application of the soil-structure problems [20]. Factors of safety are computed using FLAC3D by means of strength reduction approach as mentioned earlier. FLAC has several options for modeling of structural elements, including beam and pile elements, both of which were utilized for this study. It should be noted that in this research, the pile heads are fixed (restrained). Restrained head condition is obtained by connecting the pile heads with beam elements. The structural elements are assumed to remain elastic at all times. Beam and pile elements are identical, except that pile elements include an interface element to model soil-structure interaction (SSI). The interface element utilizes springs to model both the shear and normal SSI behavior. The interaction of the piles and soil was modeled with SSI springs. The SSI springs had a strength (represented by cohesion and a friction angle) and stiffness. The properties of the SSI springs in the normal and shear directions were determined using a procedure outlined by Itasca (2002) [21]. For a dynamic analysis, FLAC3D program provides several mechanical methods of damping in which local damping is a simple and pragmatic method. The local damping coefficient α_L is defined as:

$$\alpha_L = \pi D \quad (2)$$

Where D is fraction of critical damping. Although the actual value given to the local damping has a profound influence on the dynamic wave transmission, if it is selected from a certain range, it has little influence on the predicted factor of safety in seismic slope stability analysis. Hence, local damping of 0.157 (i.e. fraction of critical damping is 5% which is a typical value for geologic materials) is used in the model. The input earthquake motions are recorded accelerograms, and applied at the base of each model as horizontal acceleration time histories. For the static solutions, the bottom boundary is fixed in the horizontal and vertical directions and the lateral boundaries are fixed in the horizontal direction. For the dynamic solutions, the lateral boundaries utilized the free-field option in FLAC that approximated a free-filed condition.

2.2.1. Model Mesh and Boundary Conditions

The soil was modeled with continuum zones, and the structure was modeled using structural elements, as mentioned before. The mesh size and the maximum unbalanced force at the grid points (i.e., error tolerance) were selected on the basis of a series of parametric analyses to concurrently optimize accuracy and computation speed. The element sizes varied from 1m in each dimension around the structure to about 2m far from the structure. Fig 2 shows the finite difference grid used in the FLAC model for the soil-structure system.

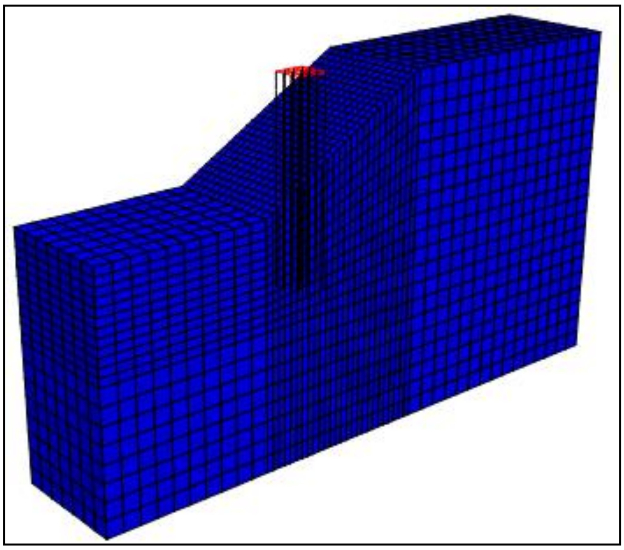


Fig. 2: Model Slope and Finite Difference Mesh

3. Parametric Study

In the current study, the procedure adopted for numerical analyses, consists of two main steps: the first step is to calculate safety factors of soil slope, and the second is to calculate dynamic displacements of the pile groups. Hence, a parametric study was performed on hypothetical and comparable examples to derive practical and representative results. As shown in Fig 3 and Table 1, this study examines the effects of key response factors and parameters on the response of pile-slope systems, such as configuration and number of piles in the group, soil profile (loose, medium and stiff categorized in terms of strength and stiffness), slope inclination angle, peak ground acceleration and pile spacing represented by the term s/d which is the ratio of spacing to diameter.

Table 1: Parameters and variables selected for parametric study

Group Configuration	*s/d*	*Soil Type*	*Slope Inclination Angle (β)*	*Peak Ground Acceleration (PGA) - (g)*
1x2 - 1x4 - 1x6	***3***	*A*	*25*	*0.15*
2x1 - 4x1 - 6x1	*6*	***B***	***30***	***0.25***
2x2 - 4x4 - 6x6	*10*	*C*	*37*	*0.35*

Note: Soils used in this study are classified into three types A, B and C and properties of each type have been presented in Table 2.

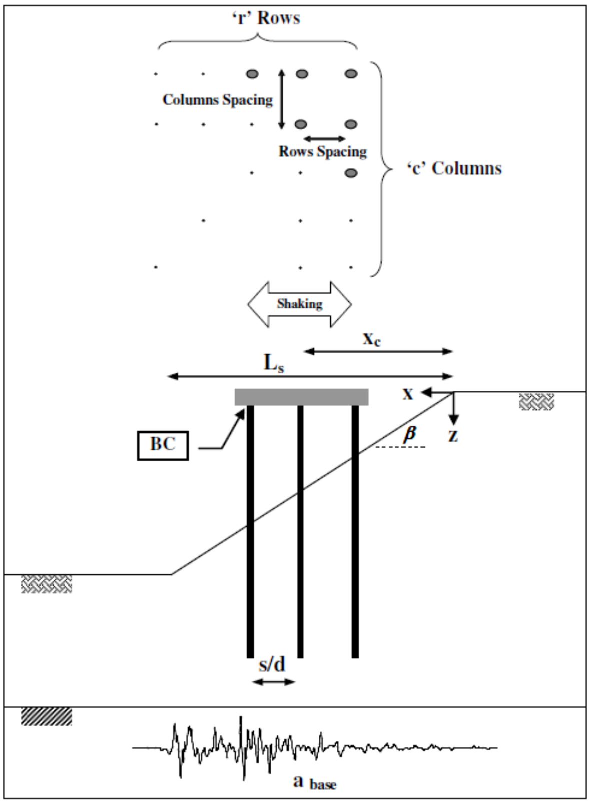


Fig. 3: Parameters used in the parametric study

Table 2: Major Modeling Properties for three different soil types used in the study

Model parameters	Loose (A)	Medium (B)	Stiff (C)
ρ = mass density (kg/m3)	1600	1800	2100
K=bulk modulus (kPa)	33333	55556	66667
G=shear modulus (kPa)	7143	18519	30769
υ=Poisson's ratio	0.4	0.35	0.3
E=elastic modulus (kPa)	20000	50000	80000
c=cohesion (kPa)	10	20	40
Φ=friction angle (deg)	20	30	40
Vs=shear wave velocity (m/s)	67	101	121

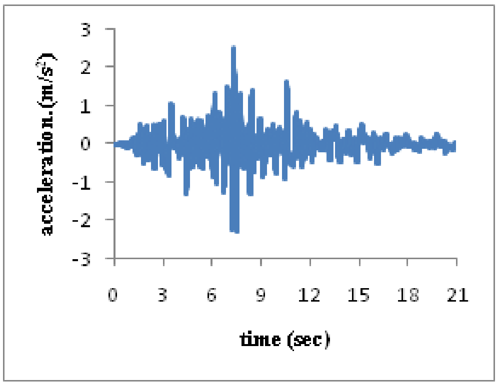
Table 3: Properties of piles and slope for parametric analysis

Case	Parameter	Value
Pile	E=elastic modulus (MPa)	2.00E+5
	Pile Length	24
	υ=Poisson's ratio	0.3
	ρ = mass density (kg/m3)	7850
	Thickness (cm)	1
	Diameter (cm)	40
	Moment of Inertia (m^4)	2.33E-04
Slope	H (height of Slope)-(m)	12

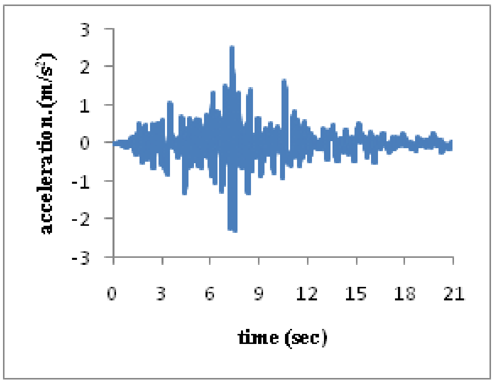
Regarding the stability and dynamic analyses, the following points should be noted:

1. Owing to the main purpose of this study, which is investigation of seismic performance of the pile-slope system, seismic stability should be considered. In order to account for the seismic effects on slope stability, pseudo-static approach is applied. It should be mentioned that only horizontal component of earthquake shaking is considered and vertical component is ignored because the effects of vertical forces tend to average out to near zero. Regarding selection of pseudo-static coefficient, it should be said that as this coefficient is a function of maximum horizontal ground acceleration, only a fraction of peak acceleration should be selected for the coefficients. As cited earlier, values equal to one-half of the peak ground acceleration have been employed in this study for pseudo-static coefficient.

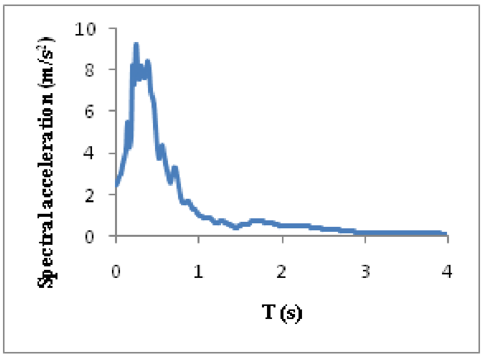
2. In order to study the effects of change in frequency content of the input motion on the response of pile-slope system, two earthquake accelerograms with different predominant periods are selected. One of these accelerograms is the horizontal component of the North Palm Spring earthquake, which has a long predominant period (1.78s), and the other one is related to the Northridge earthquake with short predominant period (0.24s). Fig 4 shows the time-history and response acceleration spectrum of the mentioned motions.



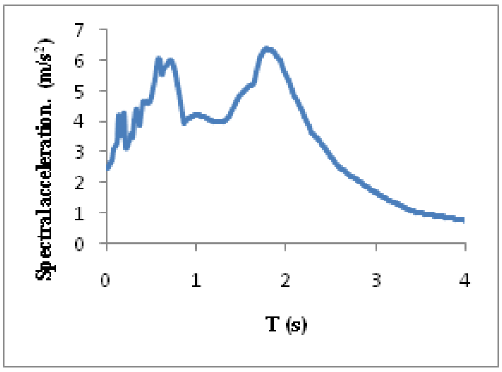
a: Northridge earthquake time-history



b: North Palm Springs time-history



c: Northridge Response Spectrum



d: North Palm Springs Response Spectrum

Fig. 4: Time-history and acceleration response spectrum of (a) Northridge (b) North Palm Springs

3. Due to the complexities of the effect of cap mass on group response, the effect of cap mass is ignored in the parametric study.

4. Concerning each one of the variable parameters, a value as the core is chosen, that is demonstrated in Bold, in the above Tables. The analyses that are performed with the core values provide core analyses that are considered as the basis for the comparison of each variable parameter.

4. Dimensional Analysis

In this section, an attempt has been made to perform a dimensionless analysis on the results obtained in the previous section (i.e. pseudo-static safety factors and seismic pile group displacements) so as to find the most influencing and representative dimensionless parameters and produce dimensionless charts in order to study the relationship between the mentioned safety factors and displacements. To perform dimensional analysis, the terms involved in the mechanical properties of soil, pile, slope and earthquake frequency are combined to form independent dimensionless variable parameters. The variable parameters used in this study and their description are presented in Tables 4 and 5, respectively.

Table 4: Dimensionless parameters used in dimensional analysis

Dimensionless Parameters	
Φ_1	$E_pI_p/E_sH_s^4$
Φ_2	$tan\,\Phi/tan\,\beta$
Φ_3	C/γ_sH_s
Φ_4	S/d
Φ_5	δ/d
Φ_6	$\omega d/V_s$

In the next step, different combinations of dimensionless parameters are formed and then, related charts are drawn and dimensional analysis is performed through a trial and error process. Combined dimensionless parameters are shown in Table 6.

Table 5: Description of dimensionless parameters used in dimensional analysis

Parameter	Dimension	Description
E_p	N/m^2	Pile Elastic Modulus
I_p	m^4	Moment of Inertia
E_s	N/m^2	Soil Elastic Modulus
H_s^4	m^4	Slope Height
$tan\,\Phi$	-	Friction Angle of Soil
$tan\,\beta$	-	Slope Inclination Angle
C	N/m^2	Soil Cohesion
γ_s	N/m^3	Soil Specific Weight
S	m	Pile Spacing
Δ	m	Maximum Displacement of Pile Group
d	m	Pile Diameter
ω	second	Earthquake Frequency
V_s	m/s	Shear Wave Velocity

Table 6: Combined Dimensionless Parameters

Combined Dimensionless Parameters including Displacement and Safety Factor used in the analysis	
Parameters including Safety Factor (FS)	*Parameters including Displacement (δ)*
$FS.\frac{C}{\gamma_sH_s}$	$\frac{\delta}{d}.\frac{E_pI_p}{E_sH_s^4}, \frac{\delta}{d}.\frac{E_sH_s^4}{E_pI_p}$
$FS.\frac{tan\,\Phi}{tan\,\beta}$	$\frac{\delta}{d}.\frac{tan\,\Phi}{tan\,\beta}$
$FS.\frac{C}{\gamma_sH_s}/\frac{S}{d}$	$\frac{\delta}{d}.\frac{C}{\gamma_sH_s}$
$FS/\frac{S}{d}$	$\frac{\delta}{d}.\frac{tan\,\Phi}{tan\,\beta}.\frac{E_pI_p}{E_sH_s^4}$
FS	$\frac{\delta}{d}.\frac{tan\,\Phi/tan\,\beta}{C/\gamma_sH_s}.\frac{E_sH_s^4}{E_pI_p}$
$FS.\frac{\omega d}{V_s}$	$\frac{\delta}{d}.\frac{tan\,\Phi}{tan\,\beta}.\frac{E_sH_s^4}{E_pI_p}$
-	$\frac{\delta}{d}.\frac{tan\,\Phi}{tan\,\beta}.\frac{E_sH_s^4}{E_pI_p}/\frac{S}{d}$

4.1 Dimensionless Charts

After determining different combinations of dimensionless parameters, the primary dimensionless parameter representing the relation between pseudo-static safety factor and maximum seismic displacements of pile group is established. For this reason, the dimensionless parameters were drawn versus each other in various charts and after comparing and investigating the charts, it was observed that in many cases either there is no adequate relationship between the selected parameters or these parameters do not demonstrate meaningful behavioral difference. Eventually, the parameter$\Psi = \frac{\delta}{d}.\frac{\tan\Phi}{\tan\beta}.\frac{E_sH_s^4}{E_pI_p}$was found to be the most appropriate and versatile involving all of the natural and geometrical properties of the pile-slope system and selected as the primary dimensionless parameter. Dimensionless diagrams illustrating the relationship under this parameter are shown in Fig 5.

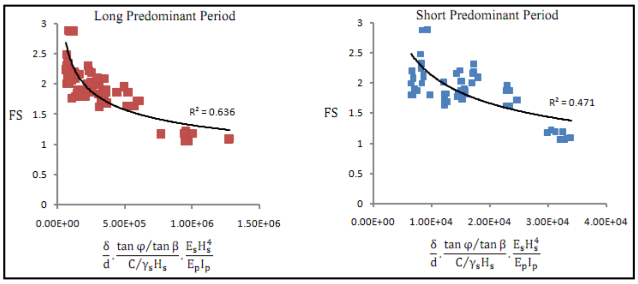


Fig. 5: Correlation between Pseudo Static Safety Factor and Seismic displacement of pile group under the primary dimensionless parameter

According to this Fig, a somewhat good and acceptable correlation between pseudo-static safety factor and displacement under the primary dimensionless parameter is found. As seen in this Fig, a reasonable and correct behavior of the system is illustrated under the primary dimensionless parameter that includes all of the factors and parameters involved in the problem. To further elucidate, it should be said that if this parameter is kept constant, a decrease in stiffness and strength of the system on the one hand decreases the safety factor, and on the other hand leads to increase in the displacements. In other words, safety factor and displacement that are two behavioral parameters are correlated to each other by the obtained dimensionless parameter, which is a natural-geometrical parameter.

4.2. Results Obtained From the Effects of the Primary Dimensionless Parameter and their Interpretation

According to Figs 6 and 7, as the safety factor of the slope decreases, the pile group displacements increase and this result can be interpreted through two geotechnical and seismic viewpoints. Regarding the geotechnical viewpoint, as seen in Fig 6, while the dimensionless parameter is kept constant, decrease in the safety factor results in the increase of induced pile group displacements.

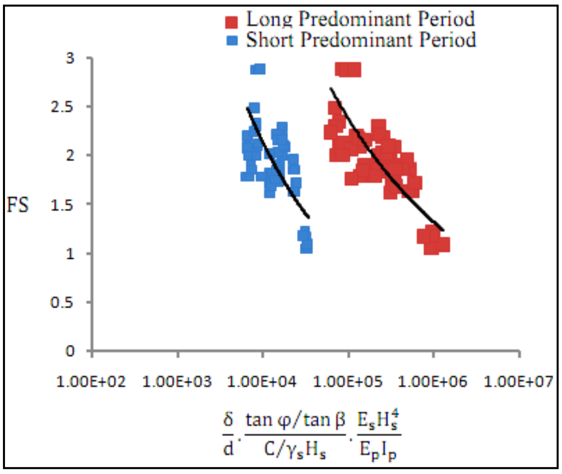


Fig. 6: Pile group displacements increase because of decrease in Safety Factor (Geotechnical viewpoint)

Fig7 also illustrates that the rate of increase in displacements induced by the short and long predominant periods are different. As shown in this Fig, following the decrease in the safety factor of the system, when the predominant period of input motion is long, pile groups experience much greater displacements compared to the short predominant period input motion. This finding is interpreted through a seismic viewpoint and it should said that as the safety factor decreases, natural frequency of the system also decreases, and due to the proximity of this frequency to the frequency of input motion, amplification phenomenon occurs whereas, in case of short predominant input motion, frequency of the motion is long. Due to this difference between the period of input motion and the natural period of the system, the rate of increase in displacements is insignificant. Thus, the difference in the increase in amount of displacements is attributed to the amplification phenomenon.

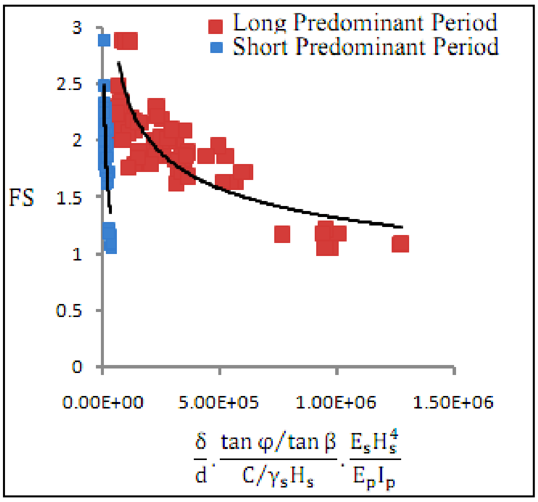


Fig. 7: Significant difference in the rate of displacement increase due to amplification (Seismic viewpoint)

In general, the abovementioned results are summarized in Fig8. According to this Figure, decrease in stiffness and strength of the system and/or generally, weakening of the system decreases the safety factors. On the other hand, as the predominant period of input motion increases, displacements become more sensitive to the increase.

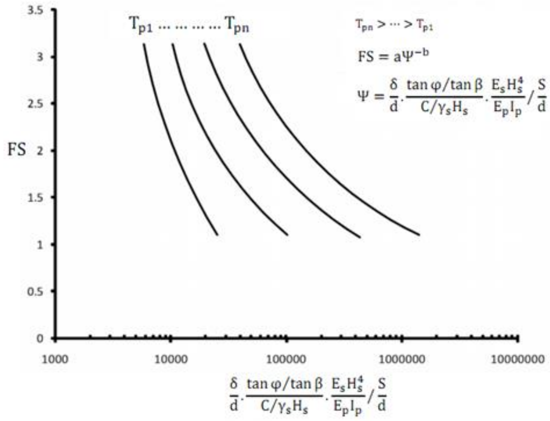


Fig. 8: Response of pile-slope system to the change in predominant period of input motion

5. Relationship between Safety factor and Pile Group Displacement

From the Figs above, it is concluded that there is a correlation between safety factor of soil slope and seismic displacements of pile group, and the diagrams developed can be utilized to only better understand the behavior of the structure (i.e. pile-slope system). On the other hand, the main objective of this research which is to develop an approach for estimation of the seismic displacements of pile groups has not been fulfilled yet. Thereby, to overcome this shortage and also achieve a practical relationship, the dimensionless parameter was modified and the terms incorporating the soil strength parameters were omitted. Furthermore, in order to account for the effects of the earthquake frequency content, a dimensionless frequency parameter $(\frac{\omega d}{V_s})$was added to the safety factor, as shown in Fig 9.

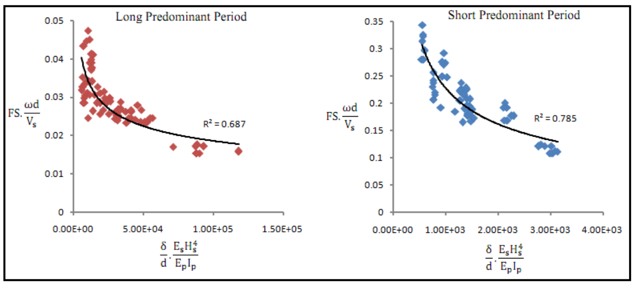


Fig. 9: Correlation between Pseudo static safety factor and seismic displacement by means of dimensionless stiffness and frequency parameters

In the next step, the two diagrams related to both input motions (earthquakes with short and long predominant periods) were combined and drawn as one diagram to evaluate seismic displacements of pile groups (Fig10).

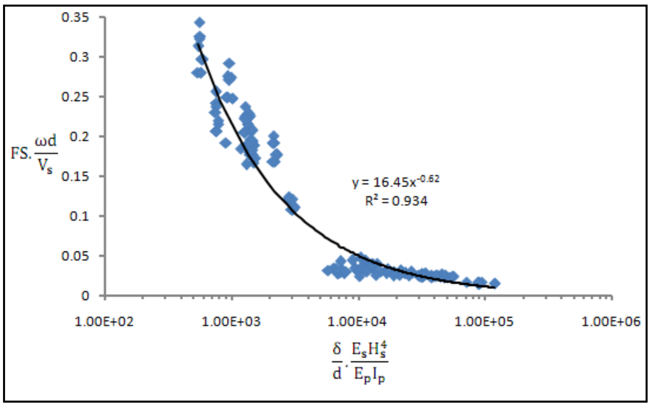


Fig. 10: Relationship between Pseudo Static safety factor and Seismic displacement.

According to this Fig, safety factors and displacements are very well correlated by means of the dimensionless parameters employed. Therefore, the following relationship can be used to estimate pile group displacements:

$$y = 16.45x^{-0.62} \tag{3}$$

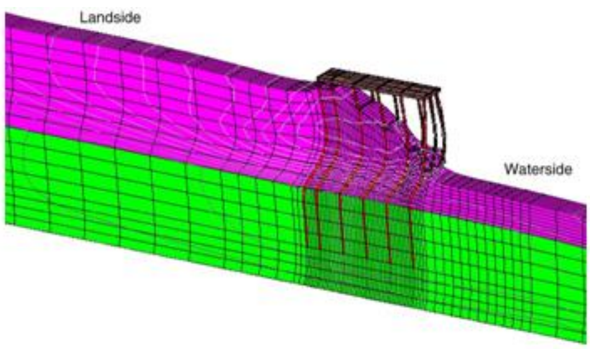
Where:

$$y = FS.\frac{\omega d}{V_s} \tag{4}$$

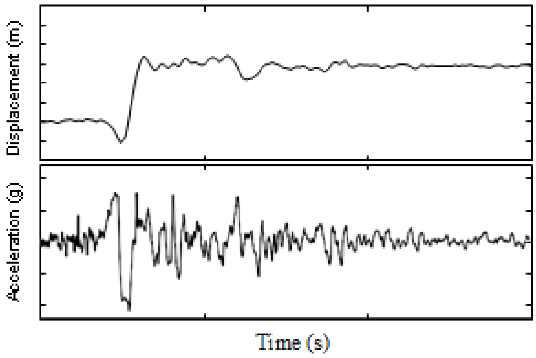
$$x = \frac{\delta}{d}.\frac{E_s H_s^4}{E_p I_p} \tag{5}$$

6. Verification of the Proposed Method

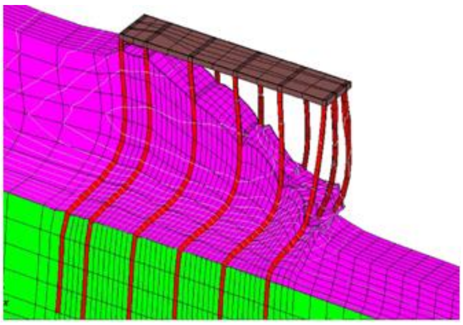
Pile supported wharves are key intermodal links between waterside and landside traffic at port facilities worldwide. These structures are commonly used to provide intermodal transition between waterside and landslide traffic and commerce. A pile-supported wharf refers to the combination of the wharf deck, piles, rock dike, backfill soils, and foundation soils. The seismic analysis of these structures is commonly based on methods that have not been developed or validated specifically for applications involving pile-supported structures on slopes, as there are few field case histories of pile-supported wharves subjected to design earthquakes. In fact, to the authors' knowledge, there are only a few filed case histories for which there are rough approximations of the ground deformations, ground motions, and structural response of the wharf and post-earthquake observation of pile performance. Moreover, the seismic performance of wharf structures is routinely analyzed with simplified procedures originally intended for structures founded on level, stable ground. In addition, Slope stability and its impact on the performance of the wharf structures constitute one of the key design issues. Therefore, this complex structure can be the best case to evaluate the suitability of the approach proposed in this paper. In this regard, the proposed simplified approach has been verified with experimental and numerical results reported in literature. To this end, Slope stability analyses were performed using the computer program SLIDE that is based on limit equilibrium concept. This program was chosen due to its simplicity to calculate safety factors. Afterwards, pseudo static safety factors were adopted as inputs to estimate the pile group displacements. The results obtained were compared with centrifuge tests carried out by McCullough et al. [22] and the numerical results of Lu et al. [23] that are described in the following. Due to the fact that the seismic response of wharves supported on piles involves significant pile-slope interaction, it is required to simulate this interaction by modeling the actual 3D configuration [23]. Lu presented a 3D dynamic model of pile and deck for the 100 container bearing wharf of Los Angeles harbor by means of the powerful software package, Parcyclic. The idealized model configuration is based on typical geometries of pile-supported wharf structures (Berth 100 Container Wharf at the Port of Los Angeles). In Fig. 7, a 3D slice in this wharf system (central section) is shown, that exploits symmetry of the supporting pile-system configuration[23]. In this idealized model (Fig. 7), there are 16 piles in 6 rows. Each pile is 0.6 m in diameter, and 43 m in length (reinforced concrete). The cracked flexural rigidity (EI) of the pile is 159 MN-m2, with a moment of inertia (I) of 7.09 x 10-3m4. The lower soil layer (25 m in height) was modeled as stiff clay (low strain shear modulus G = 486 MPa, Poisson's ratio ν = 0.46, and s_u of 255 kPa) with the upper layer being a weaker medium-strength clay (G = 80 MPa, ν= 0.46, and s_u of 44 kPa). The water table level was located at 16.6 m above the mud-line, and the slope inclination angle was 39 degrees. A scaled Rinaldi Receiving Station record from the 1994 Northridge Earthquake was employed as the base input motion (Fig. 11).



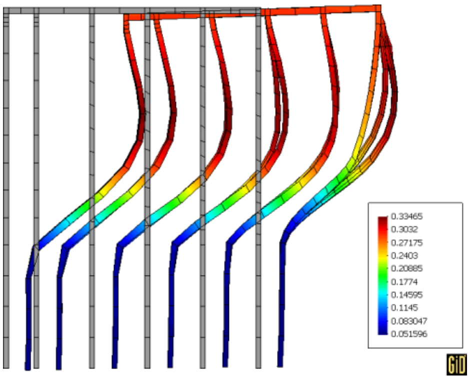
(a)



(b)



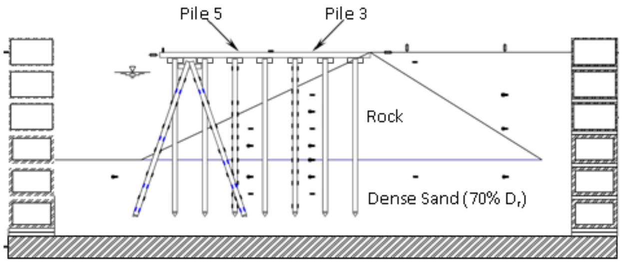
(c)



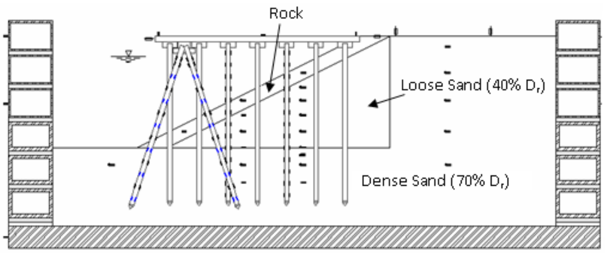
(d)

Fig. 11: Pile-supported wharf (Lu 2006): (a) Final deformed mesh, and (b) Close-up of final deformed mesh (c) Deck longitudinal displacement time history and base input record (Lu 2006) (d) Displacement Contours

In order to augment the limited number of instrumented earthquake case studies for modern wharves and evaluate the performance of the soil-foundation-structure system, McCullough et al. [21] constructed and tested a series of large-scale centrifuge models with typical pile-supported wharf configurations. The tests were carried out with and without the batter piles attached for each model at identical accelerations so as to evaluate the seismic performance of batter piles. Two of the centrifuge models are considered to evaluate the approach presented in this paper (SMS02 and JCB01). In model SMS02, the bottom layer of the model consisted relatively dense (Dr=70%) sand (G = 160 MPa,ν = 0.35, and φ of 36.55degree), used to provide a bearing and termination layer for the piles. A single monolithic rock dike with a 2.0:1.0 (H: V) slope was the waterfront face of the model. The reverse face (land-side) has 1.5:1.0 (H: V) slope. In addition, the rock dike geometry for JCB01 was modified to a 6-foot-thick (prototype scale) sloping rock facing (2:1 slope) placed over loose (Dr=40%) sand (Fig12). In addition to the loose sand, the backland area incorporated an improved (Dr=70%) region. In addition, in both models, the height of slope and water depth are 15.2 and 12.4, respectively. The structural geometry, piles and wharf model were identical for both tests. In plain view, a total number of 21 pre-stressed concrete vertical piles with diameter and length of 0.637 and25m, respectively (prototype scale)are used in three rows of seven piles. When attached, two sets of batter piles are spaced between the three rows of piles near the outboard of the wharf. Two earthquakes were used as input motions. One motion was from the 1989 Loma Prieta Earthquake, recorded at the port of Oakland Outer harbor. The other motion was from the 1994 Northridge earthquake, recorded at the Rinaldi station. These input acceleration time histories are shown in Fig 13:

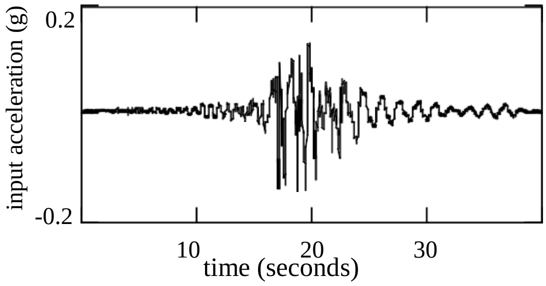


(a)

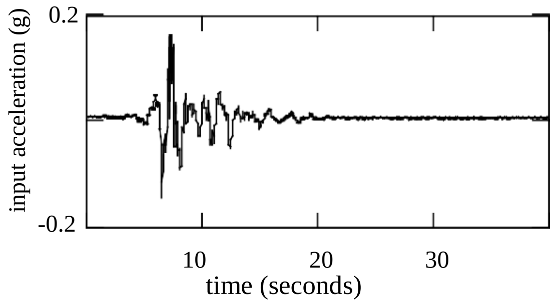


(b)

Fig. 12: Structural geometry, piles and model wharf of centrifuge models (a) SMS02 (b) JCB01



(a)



(b)

Fig. 13: Scaled input acceleration time histories: a) 1989 Loma Prieta-Oakland Outer Harbor, Port of Oakland, and b) 1994 Northridge Earthquake-Rinaldi Station

The comparison of the proposed method estimates of seismic displacement with the maximum observed seismic displacements of Lu's 3D model and the centrifuge tests are presented in Table 7.

Table 7: Comparison of the maximum seismic displacements using proposed method and numerical (calculated) and centrifuge (measured) results

Model	Input Earthquake	PGA (g)	T_p(s)	Batter Pile Configuration	Pseudo-Static Safety Factor	Displacement (cm)		
						Calculated and Measured	Proposed Method	Error (%)
Lu	Northridge	0.45	0.72	-	0.65	35	40	12.5
JCB01				attached	1.528	12	10.6	11.67
				detached	1.423	15	13.3	11.33
SMS02	Northridge			attached	1.07	28	24.2	13.57
				detached	0.951	34	30.35	10.73
SMS02	Loma Prieta	0.15	0.66	attached	1.07	25	23.4	6.4
				detached	0.951	30	27.8	7.33

Note: As two sets of centrifuge tests are carried out, with and without batter pile, safety factors are calculated in two forms of attached and detached batter piles for each model.

As can be observed in Table 7, the difference between displacements obtained from numerical analysis and centrifuge tests with those derived from the proposed approach, severely fall below 15% indicating an acceptable agreement. Herein, it can be concluded that the proposed approach is capable of yielding promising results at least for the sake of preliminary designs where sophisticated methods are not welcomed. Hence, one can simply use the obtained relationship to estimate the displacements of pile groups. Most notably, the proposed relationship requires to be scrutinized by applying some more parameters in the process of parametric study, which can be a good subject for further research.

7. Concluding Remarks

A novel simplified approach for estimating seismic displacements of pile groups located in soil slopes has been developed. The approach is based on a relationship explored between pseudo-static safety factors of slope and maximum seismic displacement of pile groups. In this respect, first the safety factors are calculated and used as the input and then, by means of the relationship, displacements are obtained. The numerical results obtained from the proposed simplified approach were compared with experimental and numerical results reported in literature. It has been shown that this procedure can be used successfully for determining the response of a pile group to ground movements developed during an earthquake. Furthermore, as in performance-based design procedures, the design relies on the earthquake-induced displacement, which is a seismic performance of a structure; this procedure can be an adequate method for assessing the performance of a structure. Therefore, performance objectives that are used to define the state of a structure following a design earthquake, can be defined by means of the safety factor of the system and then, by means of the relationship derived, displacements can be reasonably estimated.

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