COMPDYN 2015
5th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, V. Papadopoulos, V. Plevris (eds.)
Crete Island, Greece, 25–27 May 2015

# A SEISMIC PERFORMANCE-BASED DESIGN APPROACH FOR SLOPE STABILIZING PILES

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**Keywords:** slope stability, slope stabilizing piles, seismic factor of safety, performance based design.

**Abstract.** *Piles have been used to increase the stability of landslides in static regime for many decades, as they provide resistance against a laterally moving soil mass by transferring the loads into more stable underlying ground.*
*Nevertheless, the presence of such a reinforcement has beneficial effects even under seismic shaking. The paper aims at proposing a simple procedure for the estimation of permanent displacements of slopes stabilized with piles. The method is made up of 3 different stages: (a) evaluation of stability conditions of the natural slope and the ultimate load offered by the piles; (b) combination of the two ingredients above, through limit equilibrium considerations, to assess the critical acceleration of the reinforced slope; (c) use of the classical Newmark method to calculate the permanent displacement under specified input motions. A parametric study is also performed to quantify the beneficial effect of piles under earthquake shaking.*

## 1 INTRODUCTION

In the last decades, permanent displacement analysis of slopes in earthquake-prone areas has gained popularity in current engineering practice. This is mainly due to the fact that the traditional capacity-based seismic design of a geotechnical structure (i.e., a pseudo-static analysis with a factor of safety larger than unity) appears to be quite over-conservative, as does not take into account that the seismic action lasts for a short period of time. In reality, a factor of safety temporarily lower than one may lead to acceptable permanent displacement. On the other hand, the use of piles to stabilize active landslides or as a preventive measure in safe slopes has become one of the most common slope reinforcement techniques and, accordingly, numerous methods have been developed for the analysis of piled slopes in static regime by means of analytical [1-6] and numerical [7-9] approaches.

Few contributions [10] investigated the behaviour of reinforced slopes under seismic shaking and, thus, the present paper tries to outline a simple procedure for estimating permanent displacements of the combined pile-slope system. The method involves three steps:

(a) evaluation of stability conditions of the natural slope and the ultimate load offered by the piles in static regime;

(b) combination of the two ingredients above, through limit equilibrium considerations, to assess the critical acceleration of the reinforced slope;

(c) employment of the Newmark method [11] to calculate the permanent displacement under a specified time-history. In addition, the paper presents a parametric study to assess the beneficial effect, in terms of reduction of permanent seismic displacements, offered by statically-designed piles.

## 2 STABILITY ANALYSIS OF UNREINFORCED SLOPES

Slope stability analysis is generally formulated in terms of the safety factor with respect to soil shearing strength parameters: it is defined as the factor by which the soil shearing strength parameters should be divided to give the condition of incipient failure.
The critical slip surface is the surface that produces the minimum factor of safety in static analysis or the minimum critical acceleration in pseudo-static analysis.
Several methods, based on the principle of limiting equilibrium and the method of slices, are available to find the factor of safety in a rigorous way given a possible failure surface [12-16] and all of these methods require a predefined possible slip surface.
In the method of Sarma [15], used and modified in the following paragraphs, the process of finding the critical slip surface is linked to the technique for finding the minimum factor of safety (or critical acceleration). It was developed as a means of computing the critical horizontal acceleration that is required to bring the mass of soil upon slip surface to a state of limiting equilibrium.

## 3 ULTIMATE LATERAL LOADS ON SLOPE STABILIZING PILES IN STATIC REGIME

The increase of the safety margin in slope stability depends on the amount of resisting force which can be developed by the pile at the level of the sliding plan and the mechanism of interaction between the sliding soil mass and the pile.

Viggiani [4] proposed analytical expressions for a single rigid pile of length L, crossing an unstable horizontal layer of thickness $l_1$ and penetrating the underlying stable soil for a length

$l_2 = \lambda\ l_1$. Both layers are modeled as purely cohesive materials and thereby such a formulation is suitable for treating undrained conditions in fined grained soils, where upper and lower layers have undrained shear strength equal to $s_{u1}$ and $s_{u2}$, respectively. The Author, following an approach derived from that proposed by Broms [17] for piles loaded by transverse loads applied at their upper end, identified 6 possible failure mechanisms, corresponding to a stiff pile having infinite (mechanisms A, B, C) and finite (mechanisms $B_1$, $B_Y$, $B_2$) cross-section yielding moment $M_y$ (Figure 1). Defining the dimensionless quantities:

$$\chi = \frac{k_{u1}\ s_{u1}}{k_{u2}\ s_{u2}} \qquad \lambda = \frac{l_2}{l_1} \qquad m = \frac{M_y}{k_{u1}\ s_{u1}\ d\, l_1^2} \tag{1a,b,c}$$

where $k_{u1}$ and $k_{u2}$ are bearing capacity factors whose values can be taken as 4 and 8 respectively, the shear force offered by the pile for the six mechanisms are found to be[1]:

<u>Mode A</u>

$$\frac{T_A}{k_{u1}\ s_{u1}\ d\, l_1} = \frac{k_{u2}\ s_{u2}\ d}{k_{u1}\ s_{u1}\ d}\frac{l_2}{l_1} = \frac{\lambda}{\chi} \tag{2}$$

<u>Mode B</u>

$$\frac{T_B}{k_{u1}\, s_{u1}\, d\, l_1} = \sqrt{\frac{(1+\lambda)^2}{(1+\chi)^2} + \frac{(\chi+\lambda^2)}{\chi(1+\chi)}} - \frac{(1+\lambda)}{(1+\chi)} \tag{3}$$

<u>Mode C</u>

$$\frac{T_C}{k_{u1}\, s_{u1}\, d\, l_1} = 1 \tag{4}$$

<u>Mode $B_1$</u>

$$\frac{T_{B1}}{k_{u1}\, s_{u1}\, d\, l_1} = \frac{\lambda}{\chi+2}\left[\sqrt{\frac{2\chi+2}{\chi} + 4\frac{\chi+2}{\lambda^2}m} - 1\right] \tag{5}$$

<u>Mode $B_Y$</u>

$$\frac{T_{BY}}{k_{u1}\ s_{u1}\ d\, l_1} = 2\sqrt{\frac{m}{(1+\chi)}} \tag{6}$$

<u>Mode $B_2$</u>

$$\frac{T_{B2}}{k_{u1}\ s_{u1}\ d\, l_1} = \frac{1}{2\,\chi+1}\left[\sqrt{1+(2\,\chi+1)(1+4m)} - 1\right] \tag{7}$$

[1] Expression (5) contains a typographical error in the original work.

For a homogeneous soil ($\chi$ = 0.5), the stabilizing shear force corresponding to the minimum value among the above mechanisms are depicted in Figure 2 as function of $\lambda$ and m. For further details, the reader can refer to the original work.

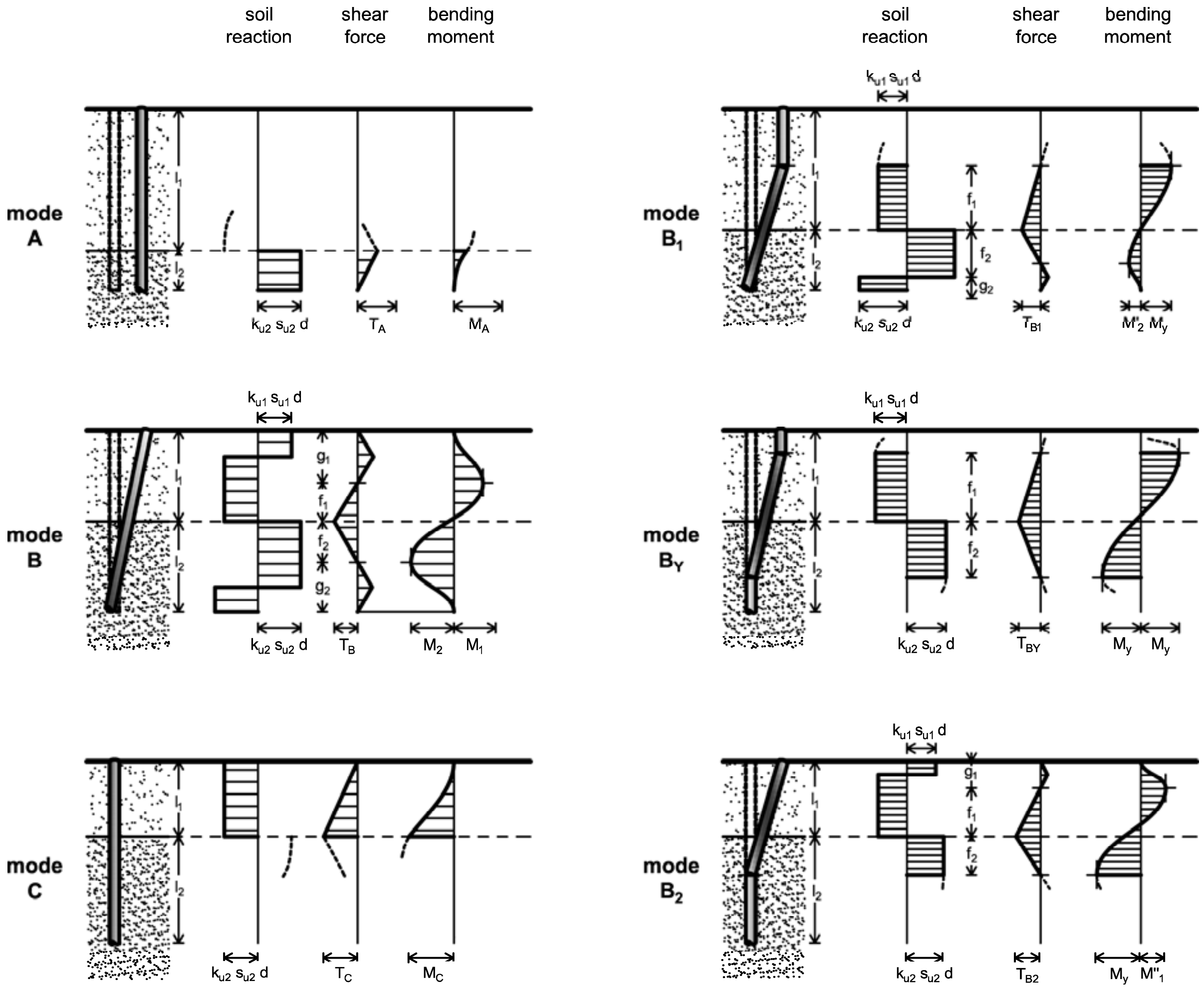


Figure 1: Failure modes for a pile with infinite cross section capacity (left column) and with plastic hinges (right column), modified from Viggiani, 1981.

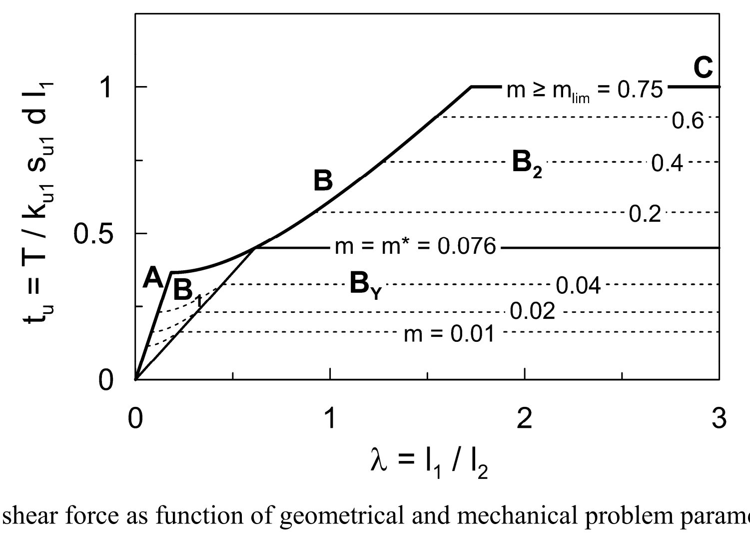


Figure 2: Stabilizing shear force as function of geometrical and mechanical problem parameters (modified from Viggiani 1981 for $\chi$ =0.5).

## 4 ANALYSIS OF REINFORCED SLOPES

Stabilizing piles are often utilized in slopes as a remedial measure to increase safety margin against collapse under static actions (i.e., in absence of any seismic loading). It is indeed evident that an additional resisting force provided by the presence of a structural element crossing the unstable soil mass and penetrating the stable soil will increase safety against a collapse mechanism. The amount of such resisting contribution depends on geometrical and mechanical features of the specific slope-pile system. In reinforced slopes, landslide body is subjected to a constant driving force due to body forces (self-weight and seismic body forces), while the resisting forces are due to the shear strength along the failure surface and to the lateral shear strength produced by pile contributions to stability (Figure 3). The sliding mass is divided in n slices and the forces acting on ith slice are shown in the following figure.
As in the original work, geometric quantities:

- $E_i$ and $E_{i+1}$: lateral thrust on vertical side of section i and i+1 in terms of total stresses;
- $X_i$ and $X_{i+1}$: shear forces on vertical side of section i and i+1;
- $W_i$: weight of ith slice;
- $k_hW_i$: horizontal force on ith slice acting at center of gravity slice;
- $N_i$: normal force at base of slice in terms of total stresses;
- $T_i$: shear force at base of slice;
- $z_i$: height of point of application of $E_i$ above slip line;
- $c_i'$ and $\phi_i'$: effective shear strength parameters of material at base of slice;
- $H_i$: height of sliding mass at section i;
- $b_i$: breadth of slice;
- $\alpha_i$: angle made by slip line mid point and the horizontal;
- $x_i$ and $y_i$: coordinate of mid point of base of slice with respect to the axes;
- $x_g$ and $y_g$: coordinate of centre of gravity of sliding mass with respect to the axes

were calculated for each slice.
It is assumed that slices are of thickness sufficiently small to consider that normal force $N_i$ acts at the mid point of the slice and, since there are no other external forces on the free surface, $\Sigma\Delta E_i = 0$ and $\Sigma\Delta X_i = 0$.

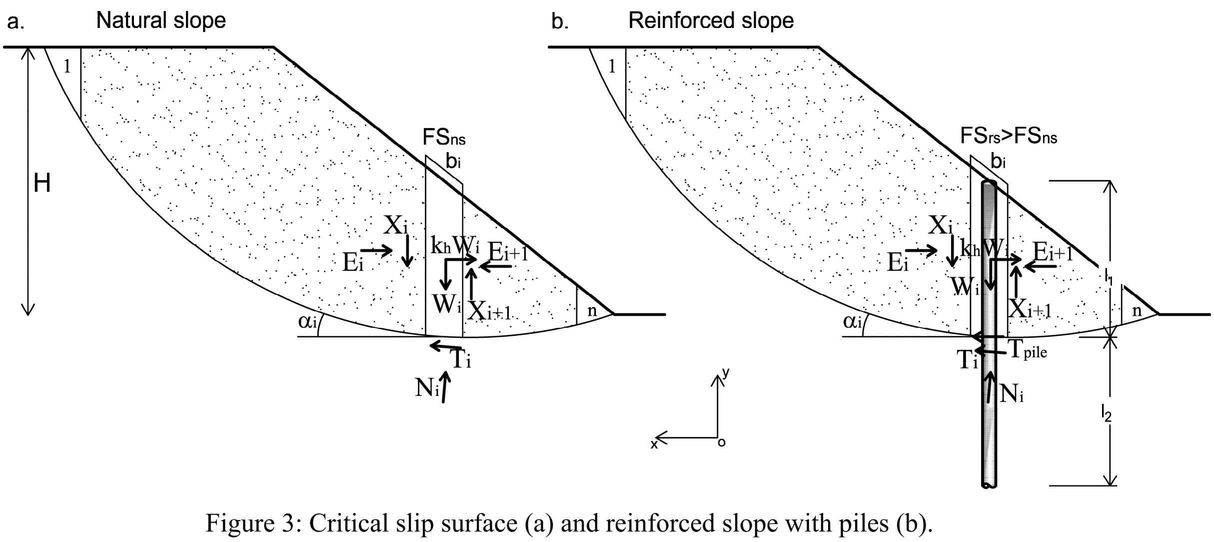


Figure 3: Critical slip surface (a) and reinforced slope with piles (b).

From the vertical and horizontal equilibrium of the slice, considering an additional resisting force due to piles and assuming that, under the action of the seismic force $k_h W$, the complete shear strength of the surface is mobilized, the following system may be considered:

$$N_i \cdot \cos\alpha_i + T_i \cdot \text{sen}\alpha_i + T_{pile} \cdot \text{sen}\alpha_i = W_i - \Delta X_i \tag{8}$$

$$T_i \cdot \cos\alpha_i - N_i \cdot \text{sen}\alpha_i + T_{pile} \cdot \cos\alpha_i = k_h \cdot W_i - \Delta E_i \tag{9}$$

$$T_i = N_i \cdot \tan\varphi_i^{'} + c_i^{'} \cdot b_i \cdot \sec\alpha_i \tag{10}$$

where, using equations (8), (9) and (10), are obtained:

$$Ti = \left[c_i^{'} \cdot b_i + Wi \cdot \tan\varphi_i^{'}\right] \cdot \frac{\cos\varphi_i^{'}}{\cos(\varphi_i^{'} - \alpha_i)} - \left[\Delta X_i \cdot \frac{\text{sen}\varphi_i^{'}}{\text{sen}(\varphi_i^{'} - \alpha_i)}\right] - \left[T_{pile} \cdot \frac{\text{sen}\alpha_i \cdot \text{sen}\varphi_i^{'}}{\cos(\varphi_i^{'} - \alpha_i)}\right] \tag{11}$$

$$Ni = \left[W_i - c_i^{'} \cdot b_i \cdot \tan\alpha_i\right] \cdot \frac{\cos\varphi_i^{'}}{\cos(\varphi_i^{'} - \alpha_i)} - \left[\Delta X_i \cdot \frac{\cos\varphi_i^{'}}{\cos(\varphi_i^{'} - \alpha_i)}\right] - \left[T_{pile} \cdot \frac{\text{sen}\alpha_i \cdot \cos\varphi_i^{'}}{\cos(\varphi_i^{'} - \alpha_i)}\right] \tag{12}$$

$$Di = \left[W_i \cdot \tan(\varphi_i^{'} - \alpha_i)\right] + \left[c_i^{'} \cdot b_i \cdot \frac{\cos\varphi_i^{'} \cdot \sec\alpha_i}{\cos(\varphi_i^{'} - \alpha_i)}\right] + \left[T_{pile} \cdot \frac{\cos\varphi_i^{'}}{\cos(\varphi_i^{'} - \alpha_i)}\right] \tag{13}$$

Following the appendix of the original work [15] for derivation of the formula for $X_i$ for homogeneous slope and neglecting the contribution of pore pressure, the quantities are given for each slices:

$$\beta_i = 2 \cdot \alpha_i - \varphi_i^{'} \tag{14}$$

$$E_i = \frac{\gamma \cdot H^2}{2} \cdot \left\{\frac{1 - \text{sen}\beta \cdot \left[\text{sen}\varphi' + \left(4 \cdot c'/\gamma \cdot H\right) \cdot \cos\varphi'\right]}{1 + \text{sen}\beta \cdot \text{sen}\varphi'}\right\} \tag{15}$$

$$X_i = E_i \cdot \tan\varphi' + c' \cdot H_i \tag{16}$$

$$F_i = X_{i+1} - X_i \tag{17}$$

The $X_i$ values that were obtained from this solution were used to determine the $F_i$ (the factor of safety on the vertical sections of the slices i.e. local safety factor) values which were subsequently used in equations (20) and (21). These quantities are implemented in the well-known parameters of the original method:

$$S_1 = \sum D_i \tag{18}$$

$$S_2 = \sum W_i (x_i - x_g) + \sum D_i (y_i - y_g) \tag{19}$$

$$S_3 = \sum F_i \cdot [(y_i - y_g) \cdot \tan(\varphi_i^{'} - \alpha_i) + (x_i - x_g)] \tag{20}$$

$$S_4 = \sum F_i \cdot \tan(\varphi_i^{'} - \alpha_i) \tag{21}$$

Since $F_i$ is assumed to be known, equations (18), (19), (20) and (21) can be solved simultaneously to obtain λ and *k*:

$$\lambda = S_2 / S_3 \tag{22}$$

$$k = (S_1 - \lambda \cdot S_4) / \sum W_i \tag{23}$$

In order to obtain the static factor of safety, the strength parameters of the material along the slip surface must be reduced by a known factor of safety, and the critical acceleration computed. The value of the factor which gives zero critical acceleration is the factor of safety which obtains without the earthquake forces.
For further details the reader can refer to the original work [15].

## 5 PERMANENT DISPLACEMENTS OF REINFORCED SLOPES UNDER EARTHQUAKE SHAKING

In order to reduce damages, due to activation of landslides during seismic events, a performance based design approach is proposed, by applying Newmark's sliding block method [11]. The model involves critical accelerations related to the unreinforced and stabilized slopes computed through limit equilibrium analysis proposed in the previous paragraphs. This procedure is able to estimate how the increase in global resisting force due to slope stabilizing piles can increase the seismic performance of the slope in terms of cumulative displacements.

### 5.1 Parametric study

Slope stability analyses in plane strain conditions through conventional methods, as the global limit equilibrium, have been performed for three homogeneous slopes in fined grained soils and undrained conditions (cohesive soils). This hypothesis is certainly acceptable when dealing with the seismic performance of slopes in fined grained soils, as done in the present paper. It follows that safety is increased by considering the presence of different pile layouts for each critical sliding surface. In addition, the increase in global resisting force is beneficial under seismic shaking, as a higher magnitude of driving force (that is, a larger value of acceleration) is required to generate displacements.
The Sarma's limit equilibrium method [15], as upon modified for piled slopes, has been implemented in Matlab code and used for direct calculation of critical seismic coefficient ($k_c$) and to determine the factor of safety (FS): the soil is assumed to follow the Tresca failure criterion and slice interfaces are vertical. Assuming undrained shear strength $s_u$=36, 42, 50 kPa, slope height H=10, 12.5, 15 m and slope angle α= 63°, 55°, 50° (respectively slope n°1, slope n°2 and slope n°3), critical slip surfaces for each slope are reported in Figure 4. These critical slip surfaces are characterized by the minimum safety of factor (FS=1.02, 1.04, 1.05), associated with the minimum critical acceleration ($k_c$=0.020, 0.017, 0.035).

In order to improve the stability of such precarious slopes, a row of stabilizing piles is embedded within landslides body. Following the general procedure suggested by Kourkoulis [8], the total shear force needed to increase stability is evaluated by imposing desired value of safety factors. Figure 5 shows the optimum pile configurations providing the required resisting force for each slopes: different layouts are obtained varying pile-to-pile distance (s).

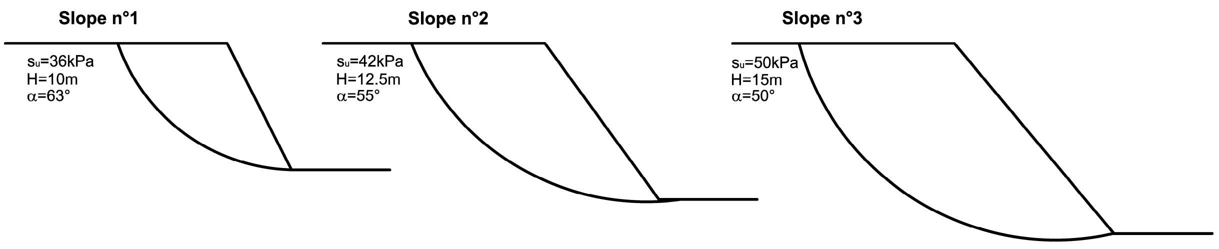


Figure 4: Case studies: unreinforced slopes

Table 1: Slopes features

| | | s/d | d (m) | s (m) | FS | $k_c$ | $T_{pile}$(kN/m) | $M_y$(kNm) |
|---|---|---|---|---|---|---|---|---|
| **SLOPE 1** | *CONF. 1* | 3 | 0.8 | 2.4 | 1.304 | 0.1835 | 134.8 | 577.2 |
| | *CONF. 2* | 4 | 0.8 | 3.2 | 1.248 | 0.1563 | 112.3 | 769.0 |
| | *CONF. 3* | 5 | 0.8 | 4 | 1.197 | 0.1291 | 89.9 | 769.0 |
| | *UNREINFORCED* | 0 | 0 | 0 | 1.027 | 0.0204 | 0.0 | 0.0 |
| **SLOPE 2** | *CONF. 1* | 2.5 | 0.8 | 2 | 1.302 | 0.1641 | 212.2 | 749.0 |
| | *CONF. 2* | 3 | 0.8 | 2.4 | 1.252 | 0.1423 | 180.7 | 805. 8 |
| | *CONF. 3* | 4 | 0.8 | 3.2 | 1.203 | 0.1195 | 147.6 | 1049.0 |
| | *UNREINFORCED* | 0 | 0 | 0 | 1.040 | 0.0179 | 0.0 | 0.0 |
| **SLOPE 3** | *CONF. 1* | 2.5 | 0.8 | 2 | 1.301 | 0.1598 | 288.0 | 518.8 |
| | *CONF. 2* | 3 | 0.8 | 2.4 | 1.250 | 0.1382 | 238.0 | 769.0 |
| | *CONF. 3* | 4 | 0.8 | 3.2 | 1.200 | 0.1151 | 184.4 | 913.8 |
| | *UNREINFORCED* | 0 | 0 | 0 | 1.054 | 0.0356 | 0.0 | 0.0 |

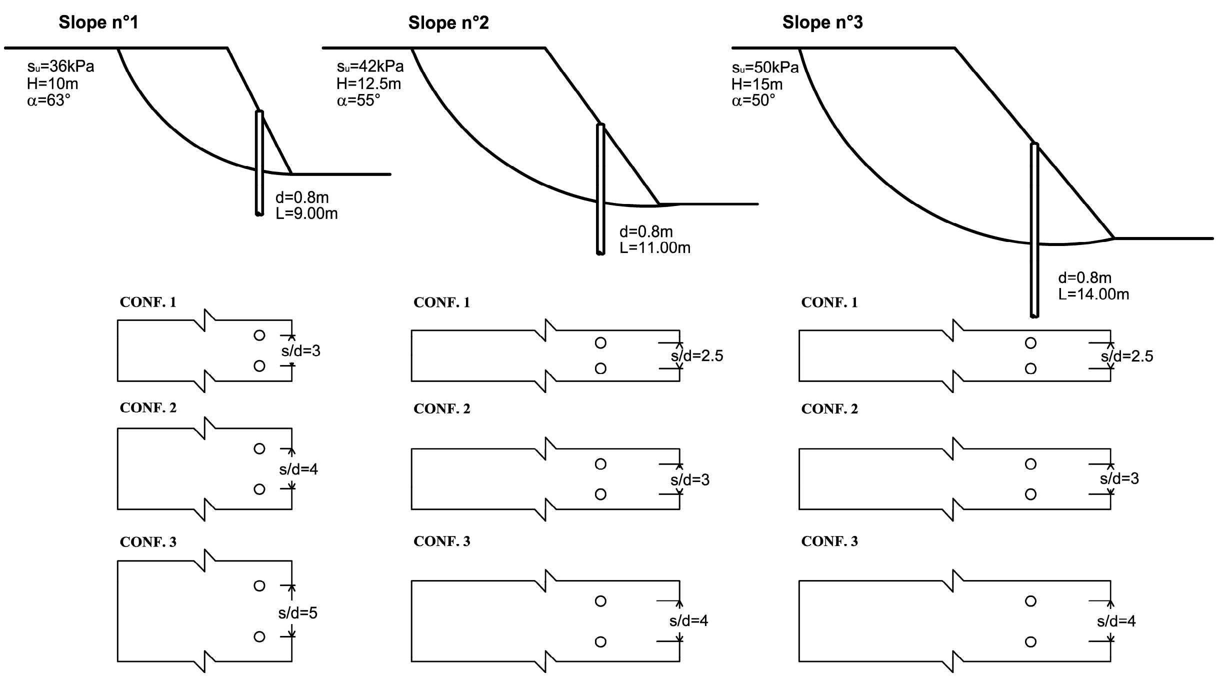


Figure 5: Case studies: piled slopes

In Table 1 the yield moment $M_y$ is defined by conventional formulae for cylindrical concrete piles whereas shear forces of each pile ($T_{pile}$) referring to the above pile modes of failure in order to verify desired value of safety factors (FS= 1.20, 1.25, 1.30). The Sarma method, modified adding the resisting force due to piles, leads to evaluation of $k_c$ and FS on piled slopes related to the previous critical slip surfaces of Figure 4.

## 5.2 Results

A total of 30 earthquake strong-motion records selected from different databases [18-20] are plotted in Figure 6: the parts of the record exceeding the critical acceleration are integrated to obtain the relative velocity-time history of the block and then cumulative relative displacements of the landslide block. Peak accelerations were scaled to values of $a_{max}$=0.25g.

Table 2: Seismic database.

| N° | Event | Station | PGA (g) | N° | Event | Station | PGA (g) |
|---|---|---|---|---|---|---|---|
| 1 | Chi Chi, 1999 | TCU045 | 0.361 | 16 | Bingol, 2003 | 007142xa | 0.515 |
| 2 | Friuli, 1976 | ATMZ270 | 0.315 | 17 | Avej, 2002 | 007718xa | 0.446 |
| 3 | Irpinia, 1980 | ASTU270 | 0.320 | 18 | South Iceland, 2000 | 006349xa | 0.744 |
| 4 | Irpinia, 1980 | ABAG 270 | 0.189 | 19 | South Iceland, 2000 | 004674xa | 0.318 |
| 5 | Izmit, 1999 | 001231xa | 0.161 | 20 | Duzce, 1999 | 006500xa | 0.496 |
| 6 | Izmit, 1999 | GBZ000 | 0.244 | 21 | Oelfus, 1998 | 004992xa | 0.145 |
| 7 | Loma Prieta, 1989 | CYC285 | 0.484 | 22 | Mt. Hengill area, 1998 | 005079XA | 0.173 |
| 8 | Tabas, 1978 | 000182xa | 0.338 | 23 | Kozani, 1995 | 006115xa | 0.208 |
| 9 | Ardal, 1977 | 000158xa | 0.908 | 24 | Firuzabad, 1994 | 007156xa | 0.310 |
| 10 | Montenegro, 1979 | 000198xa | 0.181 | 25 | Spitak, 1988 | 000465xa | 0.202 |
| 11 | Hollister,1961 | USGS 1028 | 0.484 | 26 | Etolia, 1988 | 000428xa | 0.166 |
| 12 | Montenegro, 1979 | 000200xa | 0.224 | 27 | SE_Tirana, 1988 | 003802xa | 0.113 |
| 13 | Northridge, 1994 | 24278 090 | 0.568 | 28 | Umbria Marche, 1997 | ENCB090 | 0.383 |
| 14 | Olfus, 2008 | 013006xa | 0.665 | 29 | Umbria Marche, 1997 | IBCT090 | 0.162 |
| 15 | Olfus, 2008 | 013010xa | 0.536 | 30 | Trinidad, 1983 | CDMG 1498 | 0.194 |

Critical accelerations ($k_c$), reported in Table1, have been estimated by iteratively performing a pseudo-static analysis to determine the yield factor of safety in slope equilibrium Sarma method. The significant reduction of displacements due to piles used to stabilize landslides can be judged in Figure 7, where the permanent displacements evaluated with the Newmark analyses using the Tolmezzo earthquake (ATMZ270) are reported for three unreinforced slopes (red lines) and for the same piled slopes.

It is worth to note that displacements in the piled slopes are significantly lower, than the ones in the unreinforced slopes. The reductions is ranging between 99 to 93% in the first slope (subjected to maximum displacement of 40cm in unreinforced slope), a reduction of 98 to 91% in the second slope (subjected to maximum displacement of 43cm in unreinforced slope) and a reduction of 97 to 83% in the third slope (subjected to maximum displacement of 27cm in unreinforced slope).

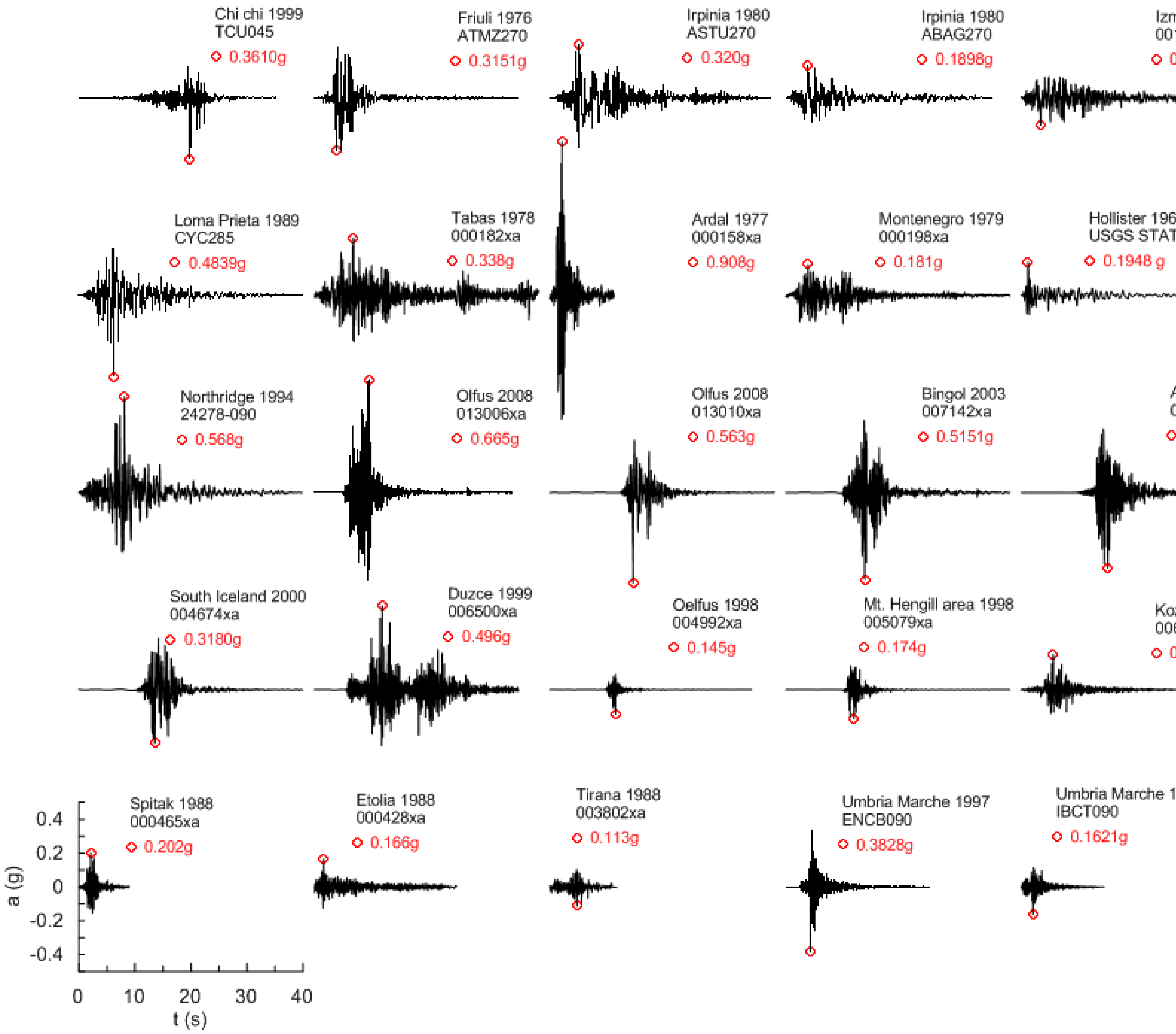


Figure 6: Time histories used in the analyses

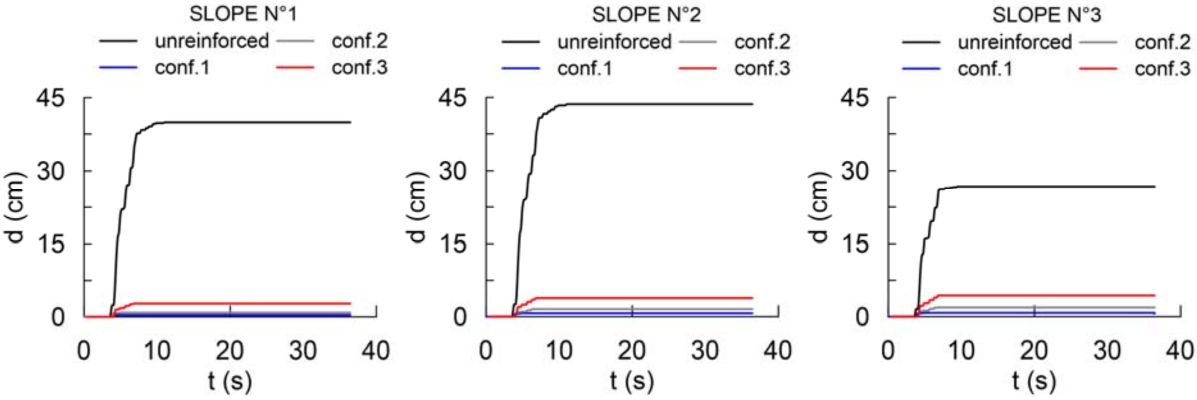


Figure 7: Newmark analyses (ATMZ270 earthquake)

Newmark analyses were carried out for the whole database of Figure 6 in order to evaluate the reduction of dynamic displacements on unreinforced and stabilized slopes.
The results are shown in Figure 8 where seismic permanent displacements are plotted in function of the static safety factors (FS): on the left hand displacements of unreinforced precarious slopes whereas, on the right, displacements of piled slopes.
It is evident that the effect of piles stabilization can be evaluated in terms of reduction of seismic induced displacements more than in terms of increment of a conventional seismic safety factor.

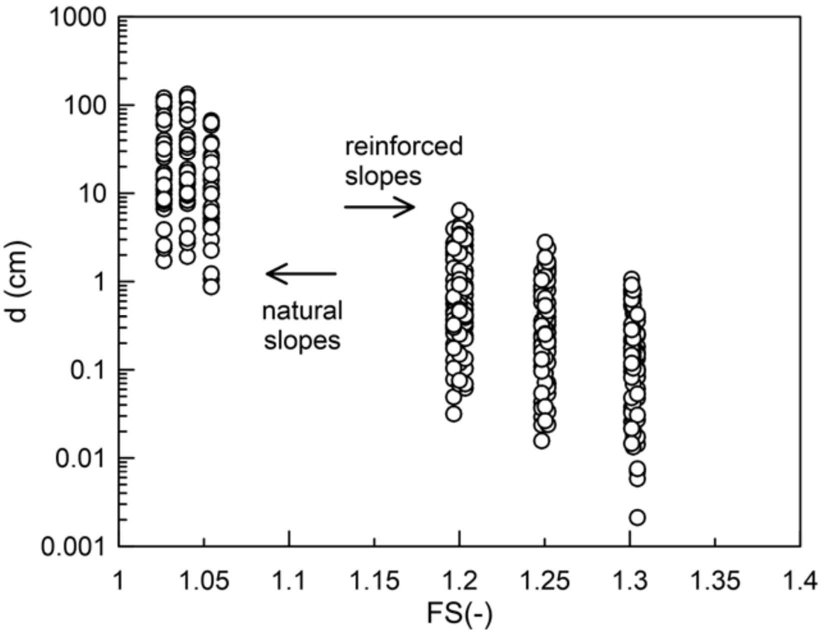


Figure 8: Effects of stabilization by piles in terms of seismic displacements and safety factors

## 6 CONCLUSIONS

In the present work a simple methodology has been proposed for evaluating the permanent displacements of slopes reinforced by piles under earthquake shaking. After evaluating the ultimate load of pile stabilizing slope under static regime (in the application this was done by means of Viggiani [4] method in undrained conditions), the procedure extends the well-known Sarma [15] method, by adding in the equilibrium equations the contribution of the pile, to obtain the critical acceleration. A classical Newmark method was then employed for the estimation of permanent displacements under different earthquake time-histories. By making use of this procedure, a parametric study has been conducted to evaluate the ability of statically-designed piles in reducing earthquake-induced permanent displacement of slopes. The study shows that when piles are designed to increase the safety factor in static regime up to typical values (i.e., designed according to classical capacity-based concepts) they can reduce drastically the seismic displacements. It is fair to mention that this study presents different limitations due to the simplifying hypotheses adopted. Therefore, while the work highlights interesting trends, the generalization of these results requires caution and will be investigated in future research.

## ACKNOWLEDGEMENTS

This research was supported by National relevant Interest Research Program (PRIN 2010-2011). The Authors would like to express their gratitude to the responsible of the research project, Prof. Leonardo Cascini.

## REFERENCES

[1] Ito T, Matsui T, Hong PW. Design method for stabilizing piles against landslide – one row of piles. Soils Found 1981;21(1):21–37.

[2] Chen LT, Poulos HG. Piles subjected to lateral soil movements. J Geotech Geoenviron Eng ASCE 1997;123(9):802–811.

[3] Zeng S, Liang R. Stability analysis of drilled shafts reinforced slope. Soils Found 2002; 42(2):93–102.

[4] Viggiani C. Ultimate Lateral Load on Piles Used to Stabilize Landslides. Proc. 10th Int. Conf. Soil Mech. Found. Engng. Stockholm, 1981; 3: 555-560.

[5] Poulos HG. Design of reinforcing piles to increase slope stability. Can Geotech J 1995; 32:808–818.

[6] Ausilio E, Conte E, Dente G. Stability analysis of slopes reinforced with piles. Comp and Geotec 2001; 28(8): 591-611.

[7] Won J, You K, Jeong S, Kim S. Coupled effects in stability analysis of pile-slope systems. Comp and Geotech 2005; 32:304-315.

[8] Kourkoulis R, Gelagoti F, Anastasopoulos I, Gazetas G. Slope stabilizing piles and pile-groups: parametric study and design insights. J Geotech Geoenviron Eng ASCE 2011;137(7):663–678.

[9] Kourkoulis R, Gelagoti F, Anastasopoulos I, Gazetas G. Hybrid method for analysis and design of slope stabilizing piles. J Geotech Geoenviron Eng ASCE 2012;138(1):1–14.

[10] Li X, He S, Wu Y. Seismic Displacement of Slopes Reinforced with Piles. J. Geotech. Geoenviron. Eng. ASCE 2010.136:880-884.

[11] Newmark N.W. Effects of earthquakes on dam and embankments. The Fifth Rankine lecture of the British Geotecnical Society, Geotecnique, 1965, vol. 15, n° 2.

[12] Janbu N. Stability analysis of slopes with dimensionless parameters. Harvard Soil Mechanics Series no. 46, 1954.

[13] Bishop, A. W. & Morgenstern, N. R. Stability coefficients for earth slopes. Geotechnique 1960: 10, No. 4, 129-150.

[14] Morgenstern, N. R. & Price, V. E. The analysis of the stability of general slip surfaces. Geotechnique 1965: 15, No. 1, 79-93.

[15] Sarma S K. Stability analysis of embankments and slopes. Geotechnique 1973; 23(3):423-433.

[16] Sarma, S K, Stability analysis of embankments and slopes. Journal of the Geotechnical Engineering Division, ASCE 1979: 105, 1511-1524.

[17] Broms B. Lateral resistance of piles in choesionless soils. J soil Mech Found Div 1964; 90(SM3): 123-156.

[18] Scasserra G, Lanzo G, Stewart J P, D'Elia B. SISMA (Site of Italian Strong Motion Accelerograms): a web-database of ground motion recordings for engineering applications. Seismic Engineering Conference commemorating the 1908 Messina and Reggio Calabria Earthquake, Reggio Calabria, Italy 2008: 2:1649-56.

[19] Chiou B, Darragh R, Gregor N, Silva W. NGA Project Strong-Motion Database. Earthquake Spectra 2008; 24(1): 23-44. Pacific Earthquake Engineering Research Center PEER Ground Motion Database Beta Version 2010; Available online from URL: http://peer.berkeley.edu/

[20] Seismosoft, Earthquake Engineering Software Solutions. Seismosignal and Seismospectra - computer programs for signal processing of strong-motion data 2013; available online from URL: http://www.seismosoft.com