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SOIL–STRUCTURE INTERACTION EFFECTS DURING LANDSLIDES IN STRAIN–SOFTENING SOIL

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ABSTRACT

The impact of landslides on neighboring structures has been observed during recent earthquakes to range from no or minor damage, to total collapse, suggesting the possibility of interaction between the sliding soil and the overlying structure. This study attempts a parametric investigation of the effect of foundation type, load, and size, on the position of the failure surface and on the generated soil displacements for various levels of ground shaking. Dynamic analyses are performed utilizing a plane-strain non–linear finite element model of both the soil and the structure which captures the possible separation of the foundation from the supporting soil. The model is calibrated against published data to simulate the observed strain-softening behavior of soil during sliding. The shear zone within the soil mass is formed as a result of strength degradation due to strain softening during the earthquake and under the action of gravitational forces. The response of both piled and shallow foundations is examined.Soil–structure interaction is demonstrated either as modification of the position of the sliding surface due to the existence of the structure or as a modification of the dynamic response of the structure itself.

Keywords: seismic, slope, structure, interaction

INTRODUCTION

Many researchers (e.g. Sarma & Iossifelis, 1990; Sawada et al., 1994; Durmieux & Pecker, 1995; Soubra, 1997) have investigated the seismic bearing capacity of shallow footings with limit-state procedures, and assuming that the seismic load being applied pseudo-statically. However, only few of them have focused on cases of foundations on the vicinity of sloping ground. Among them, Sarma and Chen, (1995), Sarma (1999), Askari and Farzaneh, (2003), Kumar and Rao (2003)), by means of a pseudostatic limit equilibrium approach that takes into account the inertia of soil mass, concluded that the bearing capacity is minimal when the footing is located at the edge of the slope, and increases as the footing is carried away from it until it ultimately converges to its level-ground value.

Despite its broad acceptance among engineering practitioners, limit equilibrium technique is inappropriate to simulate the soil-structure-interaction effects during earthquakes; while it is inherently incapable to capture the actual strain-development phenomena. The method usually requires a pre-assumed sliding surface that is not the “natural” product of strength reduction due to strain accumulation. Similarly, despite its ease of use, the pseudostatic approach is generally not applicable to soils that soften with increasing number of cycles (Loukidis et. al (2003)), such as those examined in the present study. As is well known, the resistance of soils whose strength is characterized by different peak and residual values is progressively reducing with increasing strain (Terzaghi and Peck, (1948), Skempton (1964), Bjerrum (1967). The complex mechanism of progressive failure of slopes

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apparently cannot be modelled with simplified limit equilibrium techniques and rather necessitates advanced numerical techniques capable to simulate strain-localization phenomena.

Among others Hoeg 1972, *Lo and Lee, 1973, Chen et al 1992, Modaressi et al (1995) ,Potts et al (1997, 1999), Loukidis et al (2003), Troncone, (2005) Pradel et al (2005)* have used the finite elements method to model the shear zones development and the progressive failure of soil. *Dounias et al (1988)* and *Potts et al (1990)* applied the finite element method in the analysis of the stability of embankments and rockfill dams, while *Potts et al (1997) and Troncone (2005)* back analyzed the slow collapse of cuts in strain-softening stiff clays. All the above analyses were performed under static conditions. In all of them, the elements in the vicinity of the expected failure surface were modelled so as to obey a certain strain-softening law while the rest of the soil model was assumed to behave elastically.

For the dynamic analysis of slopes with foundations placed on their top, the present study, utilizes a fully non-linear finite element model with a strain-softening material law (Figure 1a). The potential separation of the foundation from the underlying soil, due either to uplift or to the downward slope movement during landslide, is also taken into account. Since this model can exhibit non-uniform straining, it realistically captures not only the mechanism of progressive slope failure but also the effects of the foundation on the position of the generated failure surface. Because the sliding surface is not pre-defined but rather a product of strain softening, the model encapsulates the effect of the structure on the produced displacements field and on the position of the failure surface itself. The soil is at this stage assumed to be dry, neglecting the effect of pore pressure build up due to cyclic loading.

FINITE ELEMENT MODEL

The analyses are performed in the finite element code ABAQUS (HKS 2001) Plane strain dynamic analyses are executed, utilizing a fully non-linear elasto-plastic model. A 30 m high, 23^{o} steep slope is analysed. Two different soil strata are considered. The top 40 m layer exhibits strain softening behaviour according to the material law described below. The bottom stratum is considered to be elastic, while its thickness till the bedrock is 20 m

Quadrilateral 4-noded plane strain elements are used for the representation of the soil while the foundation slab is modeled with beam elements. The interface between the foundation slab and the underlying soil is modeled with frictional elements which allow both slippage and separation between the soil and the foundation nodes. Both shallow and piled foundations are examined. The piles are modeled as beam elements; their diameter is 0.30 m, while their length extends to 30 meters. This way the piles are founded within the stiff soil layer below the level of the observed slope failure surface. Free field conditions are applied at the model boundaries. The optimum element dimension that would ensure computational efficiency without endangering the accuracy of the simulation was parametrically investigated. A size of L=0.5 m has been finally selected. It was shown that the use of smaller elements had no effect on the position of the failure surface, while at the same time the computational time was exponentially increased.

SOIL MODELING AND CALIBRATION

A non-associated flow rule is adopted for the soil. The pre-failure behavior and the loading-unloading of the soil obey the theory of elasticity, while strain softening and the post-failure behavior are modeled with s Mohr-Coulmb failure criterion. The model parameters are the cohesion, c, the friction angle φ and the dilatancy angle ψ. Strain softening is incorporated into the finite element code through a user defined subroutine that reduces the strength parameters c and φ with increasing plastic strain (Figure 1a). The strain softening law has been effectively utilized by Anastasopoulos et al (2005a, 2006) and Anastasopoulos (2005b) for the analysis of fault propagation through soil. In the present sudy, the material behavior is calibrated against a viscous-plastic model for the calculation of the strain and strength of strain-softening cohesive soils developed by Gerolymos et al., (2006), Based on several experimental results (e.g. Lupini et al.,1981, Bishop et al., 1971, Bromhead and Curtis, 1983, Skempton, 1985, Tika and Hutchinson, 1999) and utilizing an artificial neural network, they proposed analytical relationships for the calculation of the strength parameters of cohesive soils depending on their characteristics. The residual friction angle (Fig. 1b) has been found to be a function of the clay content (C_F) and the clay activity (A):

$$A = I_P / C \tag{1}$$

where Ip the plasticity index.
Skempton (1985), suggests typical values of displacements during various stages of the ring shear test. The critical state friction angle is calculated according to Mitchell (1976) as:

$$\varphi_{cv} \approx \arcsin [\ 0.6 - 0.14\log(I_P - 5)] \tag{2}$$

Finally, the peak value of the friction angle was found to be best calculated as follows:

$$\frac{\tan \varphi_P}{\tan \varphi_{cv}} = 1 - 0.85 \ln \frac{2}{OCR} \quad , \text{OCR} > 2 \tag{3}$$

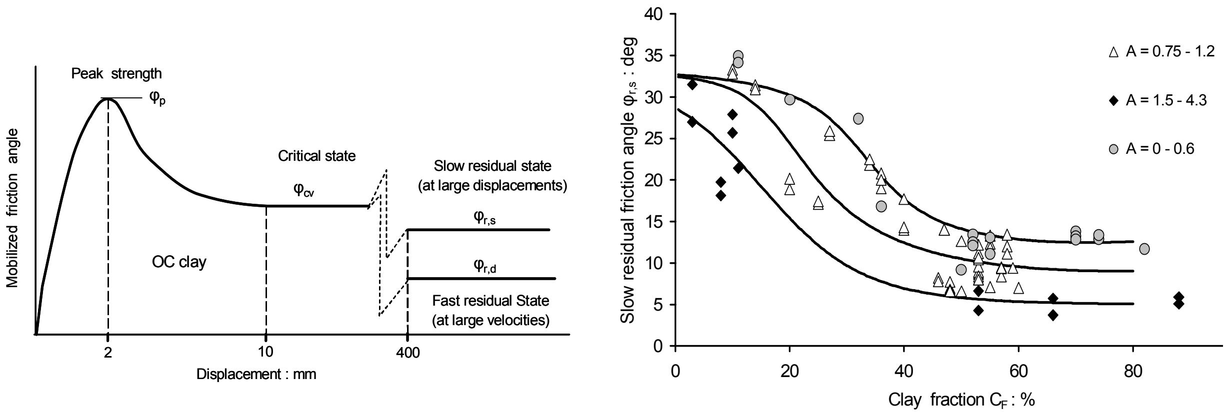


Figure 1. (a) Typical behavior of strain-softening soils (after: Gerolymos et al 2006) (b) Curve for the calculation of the slow residual angle as a function of clay fraction and parameter A (after: Gerolymos et al 2006)

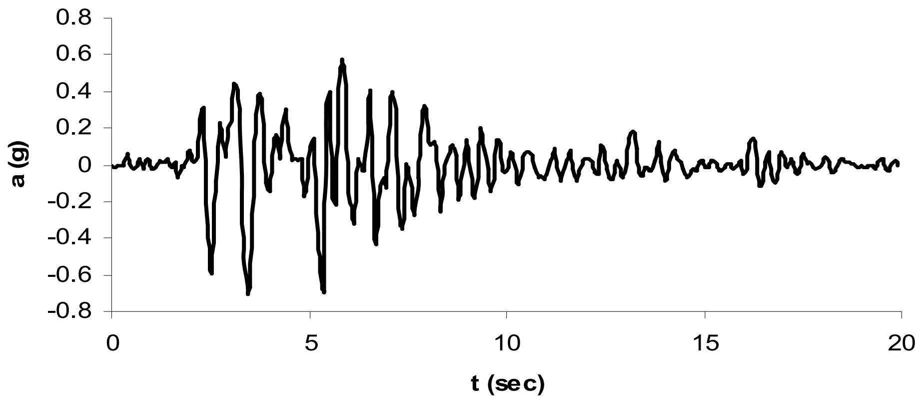
PARAMETRIC ANALYSES

Parametric dynamic analyses are performed in order to investigate the influence of various factors on the development of the shear zone within the slope. In the first set of analyses the strong motion is applied on the base of our model producing the initial “free-field” (i.e. without any structure on the slope crest) strains and displacements. The analyses are repeated after adding the shallow foundation model on the top of the slope in order to evaluate its influence on the position of the failure surface and the generated soil displacements. Finally, a third set of analyses are executed, this time the slab being supported through piles to a depth below the landsliding soil stratum in order to assess piled foundation as a method of enhancing the stability of structures in the vicinity of sliding slopes.

Analyses are performed in subsequent stages. First, the purely geostatic loads are applied, while on the second stage the dynamic loading is applied on the base nodes of the model. A third step follows, during which all external loads but gravity are zero. In case a footing exists on the slope crest, an extra step is interpolated before the application of the earthquake loading. If the deformations caused by the earthquake are significant, a failure surface is being developed and hence soil displacements keep increasing due to gravity.

Two acceleration time-histories are applied at the base nodes of the model (bedrock). the Ricker1 wavelet (Fig.2a) and the JMA accelerogram recorded during the devastating Kobe 1995 earthquake (Fig 2b). Both strong motions are scaled at PGAs of 0.5 g and 0.8 g. This way, the effect of various time history characteristics (PGA, strong motion duration, number of cycles etc) on the slope response can be examined. The ricker1 wavelet is a narrow –band excitation lasting 3.5 seconds, while the JMA time history is comprised of many significant cycles of loading, with a total duration of 20 seconds. Depending on its characteristics, the seismic loading might either trigger a landslide or not.

(a)



(b)

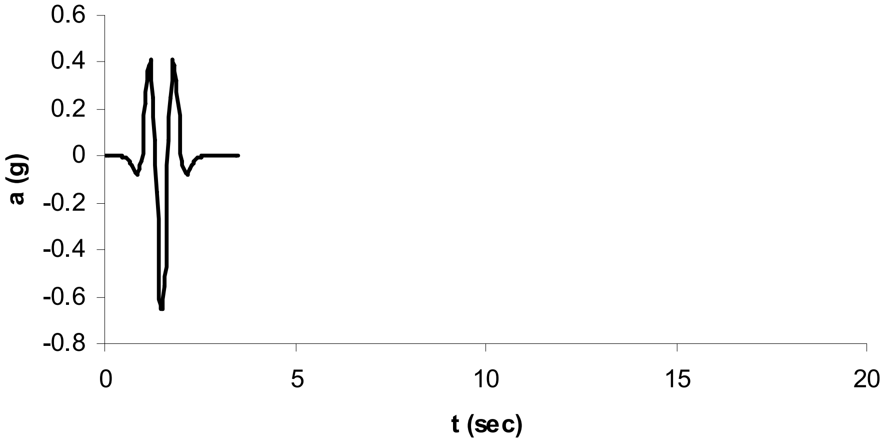


Figure 2. (a) JMA accelerogram (b) Ricker1 Wavelet. Both scaled at PGA of 0.8g

RESULTS AND CONCLUSIONS

Position of the failure surface

Figure 3 plots the generated displacement field when no foundation is present and the slope is subjected to the JMA (Fig. 3a) accelerogram and the Ricker 1 wavelet (Fig 3b) respectively. Interestingly, even though the two applied motions exhibit the same PGA, the produced displacements are significantly higher when the model is excited by the JMA motion. This is evidence of the inadequacy of PGA as a predictive tool of slopes displacements, as already reported by many authors (Fardis et al 2003, Sarma et al 2004). The strain distribution (again for the field field case) when the input motion is the JMA accelerogram is depicted in Fig4.

(a)

(b)

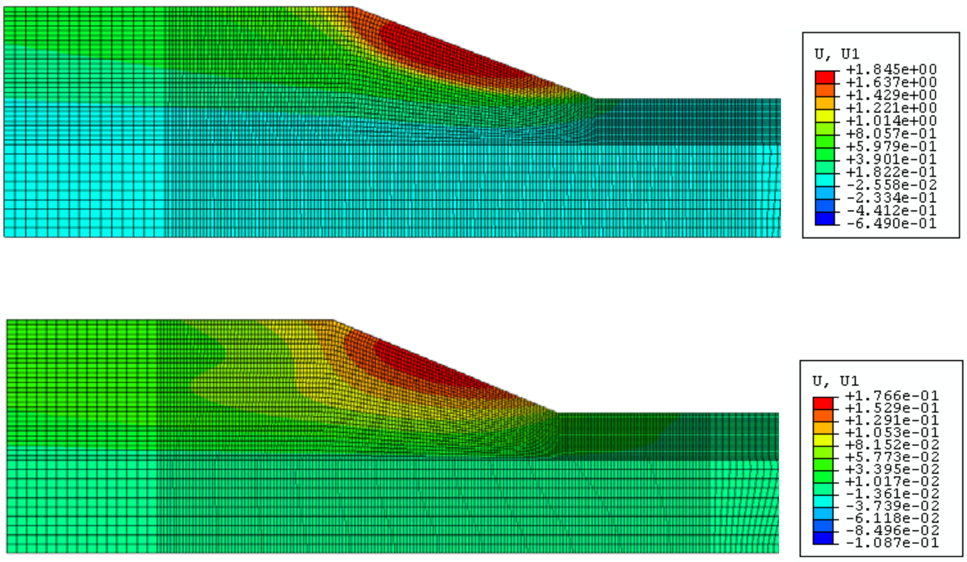


Figure 3. Contours of horizontal displacement for the "free field" case when the model is subjected (a) to the JMA accelerogram scaled at 0.8 g and (b) to a Ricker 1 pulse scaled at 1 g

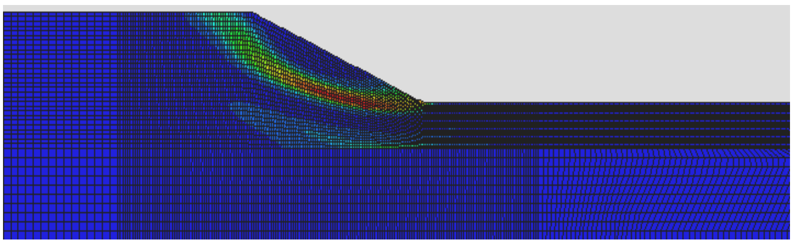


Figure 4. Contours of plastic deformations generated for the "free field" case

When a 20 m rigid footing of q = 40 kPa lies at 5 m distance from the slope crest the picture of the generated plastic deformations is notably changed. Fig 5a illustrates the new failure surface when the slope is excited by JMA accelerogram (at 0.8 g PGA). It is clear that the footing causes the failure surface to deviate from its free field position, as it imposes an extra load on the slope crest which in turn provokes failure of its underlying soil. As the footing is taken further away from the crest at a distance of 8 m (Fig. 5b), the failure surface seems to be less affected until it finally returns to its initial free field position once the footing has been moved far away (Fig. 5c). At that distance, the influence of the footing on the slope failure is minimal.

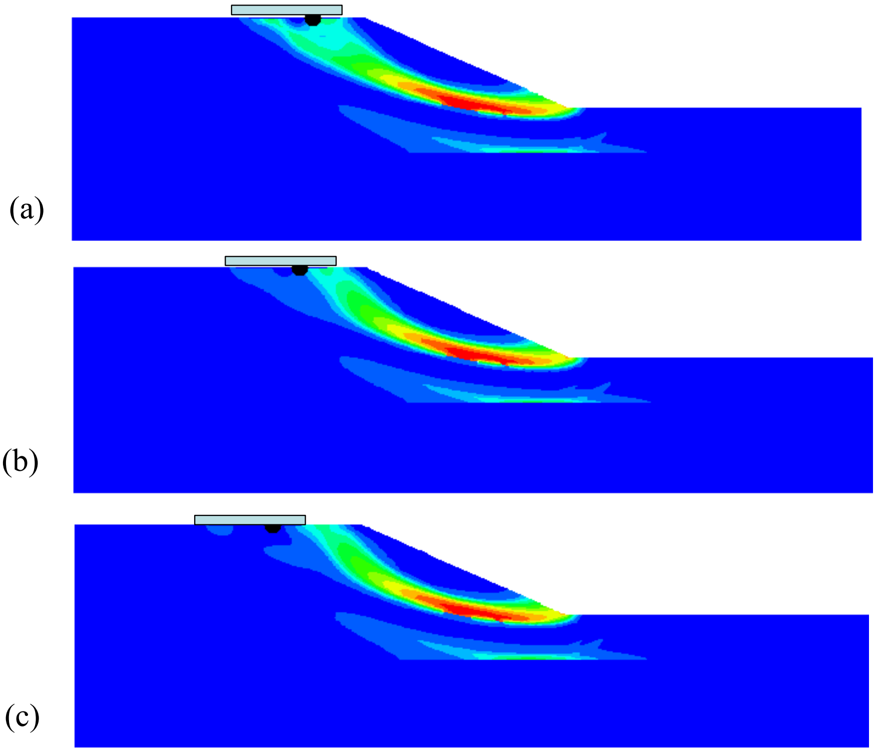


Figure 5. Contours of plastic deformations generated for the case of a 20 m wide footing of q=40kPa lying at a distance from the crest of (a) 5m, (b) 8 m, (c) 11 m, subjected to 0.8g JMA excitation

When the footing load is reduced to the half (q = 20 kPa) the failure surface (Figure 6a) is practically unaffected by the footing's presence. Conversely, in case the foundation load is large (80 kPa), the shear zone is indeed influenced by the footing: now the shape of the generated failure surface is reminiscent of a "bearing capacity"–type failure surface (Fig. 6c). As can be deduced by the intensity of plastic deformations generated underneath the footing (Figs 5 and 6), the bearing capacity of the ground increases as the footing is moved away from the crest.

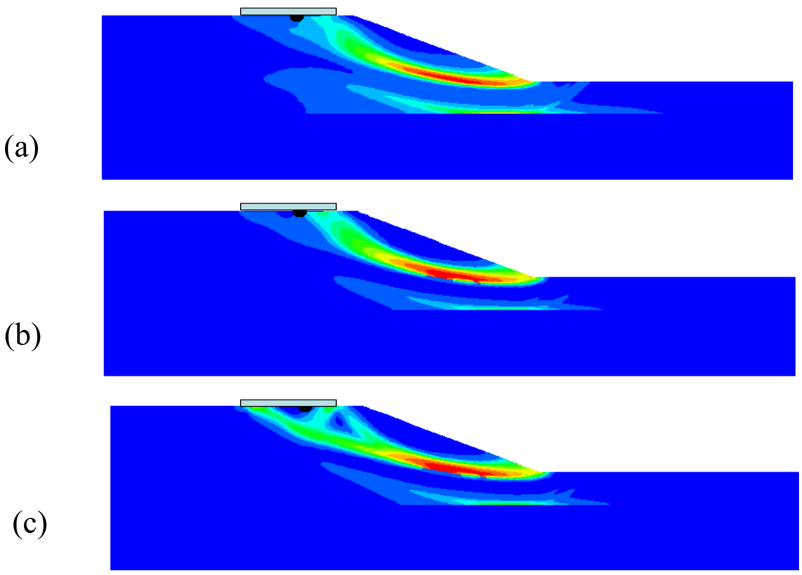


Figure 6. Contours of plastic deformations generated for the case of a 20 m wide footing lying at 8m from the crest carrying load of (a) q=20kPa (b) q= 40 kPa, (c) q = 80 kPa, subjected to the JMA excitation at 0.8g.

Soil Displacements Profil*e*

The presence of the foundation does not seem to affect the peak values of the soil displacements regardless of its position. However, there is a considerable effect of the foundation load on both the values and the distribution of vertical displacements.

Footing Response

As expected, the tilt angle of the shallow footing reduces as it is carried away from the crest and increases with increasing load.

Figure 7 plots the horizontal sliding of the soil nodes underneath the two footing edges (nodes 1 and 2) along with the sliding of the footing itself (Node 3). As expected, soil node 1 which lies on the side of the sloping ground systematically displaces the most, while soil node 2 displaces less as it is less affected by the slope movement. In case of the immense JMA input motion scaled at 0.8 g., the displacement of node 1 keeps increasing with time –an evidence of devastating landslide. On the other hand, as the footing detaches from the ground once the friction force is exceeded, the footing node 3 displaces less than compared to the underlying soil node 1. When the footing is founded through piles (Fig.8) the tilt angle diminishes.

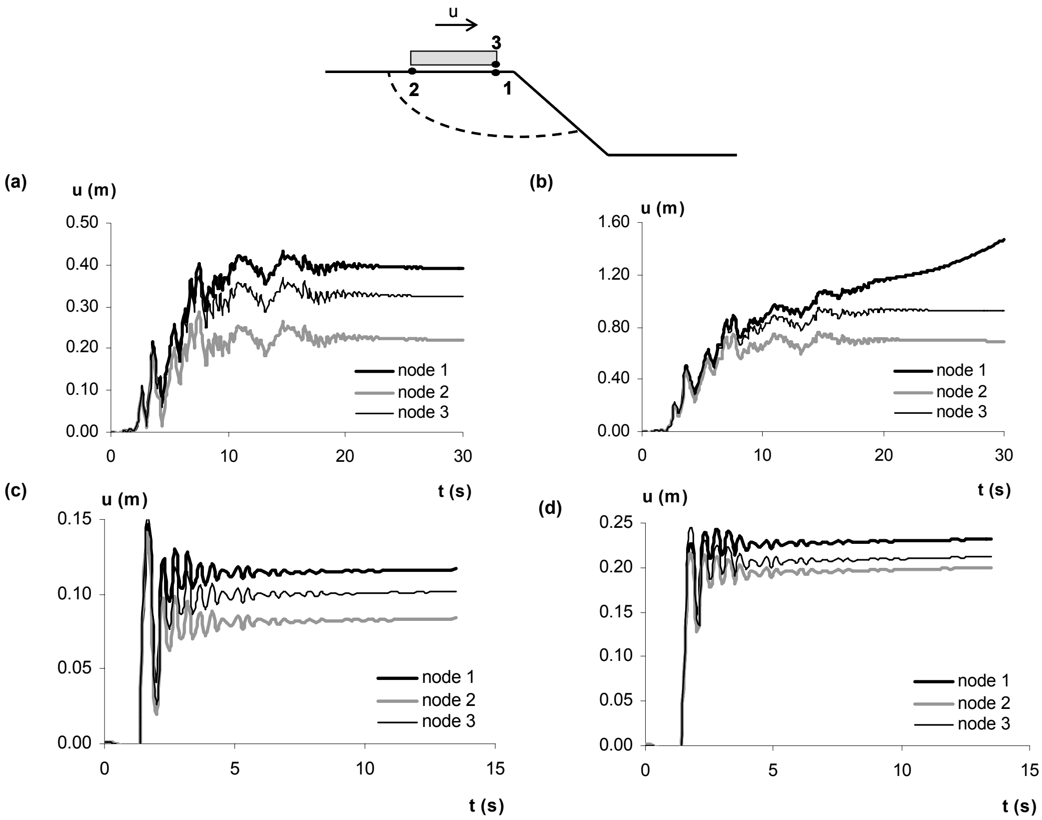


Figure 8. Horizontal displacements at nodes 1, 2 and 3 respectively when the input motion is (a) a JMA scaled at 0.5 g,(b) JMA scaled at 0.8 g, (c) a 0.5 g ricker 1 pulse (d) a ricker1 pulse at 0.8 g

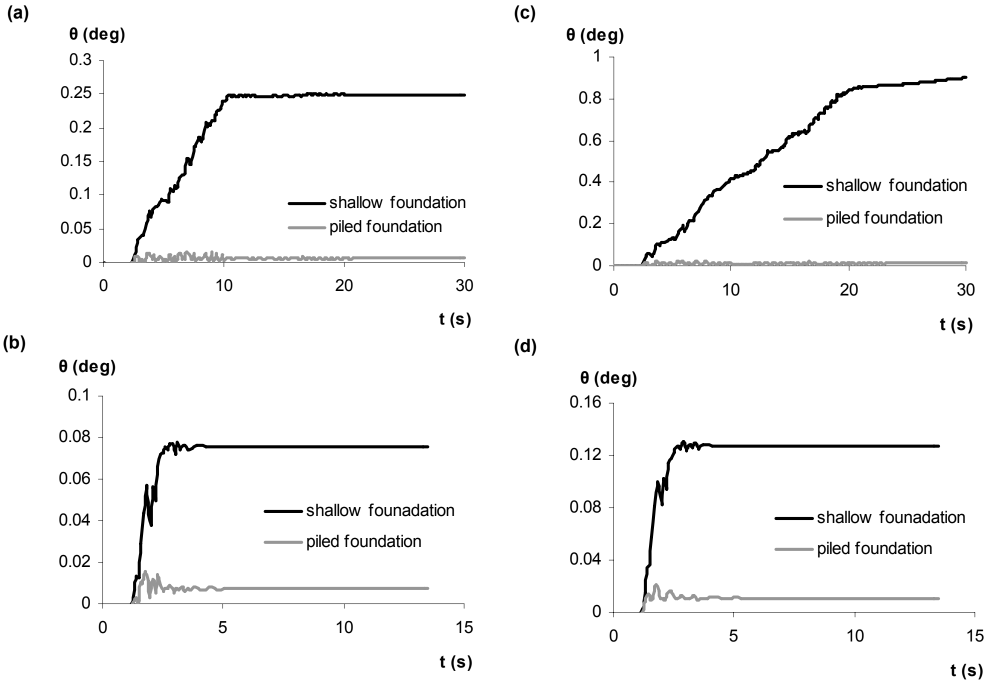


Figure 9. Tilt angle for a shallow and a piled foundation respectively when the input motion is (a) a JMA scaled at 0.5 g, (b) a JMA at 0.8 g, (c) a 0.5 g ricker1 pulse and (d) a 0.8g ricker1 pulse

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