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LIQUEFACTION ANALYSIS OF SILT INTERLAYERED SANDS BY MEANS OF A NUMERICAL METHOD

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ABSTRACT

Liquefaction phenomenon in sands during earthquakes is known to be the cause of many earthquake related damages including loss of bearing capacity, lateral flow and spreading, slope failures, etc. Therefore, a great deal of effort has been exerted in the past 40 years for understanding the mechanism of liquefaction and factors leading to liquefaction in the field. Field observations in recent earthquakes, including 1999 Kocaeli Earthquake in Turkey, have pointed out that silt inclusions and bands in the sandy deposits can have a significant effect on development of liquefaction. In this study, the behavior of sand deposits with and without a silt interlayer has been numerically analyzed using a computer code named LASS IV. It is also shown that taking into consideration the effects of two horizontal components of an earthquake rather than one can significantly increase the probability of occurrence of liquefaction and its extent of depth. Also, the behavior of an instrumented test site is numerically analyzed and the results are compared with the recorded behavior.

Keywords: Liquefaction,silt seams, two directional shaking,field monitoring

INTRODUCTION

Liquefaction phenomenon which might occur during a strong earthquake in saturated loose sandy soil deposits is known to be one of the major causes of damages experienced. In this study to shed light on some of the factors leading to liquefaction, a numerical study has been conducted by an effective stress soil analysis method capable of handling bi-directional cyclic motions. The results of analyses have shown that presence of less permeable silt seams in sand deposits can have a significant adverse effect on extend and depth of liquefaction potential. It is also shown that when the soil deposit is subjected to both horizontal components of the earthquake motion rather than a single component, the risk of liquefaction is increased.

THE ANALYTICAL METHOD AND THE MATERIAL MODEL

As mentioned above, an effective stress soil analysis method capable of handling bi-directional cyclic motion developed by Dikmen and Ghaboussi (1984) has been selected to conduct this numerical study. The method models the horizontally layered ground by a number of "layer elements". The response of the system is described in terms of the nodal plane displacement degrees of freedom. Each node has three degrees of freedom, two horizontal components of the solid displacement and one vertical for pore water relative to solid. Respectively, the stresses considered are the vertical normal stress, the horizontal shear stress and the pore water pressure.

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The method uses a plasticity based material model for analyzing the behavior of sands under cyclic loading. Hence, saturated sand is modeled as a two-phase medium subjected to two shear stresses τ_x, τ_y and the effective pressure, σ' .The failure is assumed to correspond to very large plastic shear strains, and a failure plane exists which acts as an asymptotic state for all shear stress paths.

A modified Masing type material model is used to define the stress-strain relationship. In this respect, the increments of shear strain are assumed to be the sum of the elastic and plastic shear strain increments,

$$d\gamma = d\gamma^e + d\gamma^p$$

And the relationship between the shear stress and strain increments are related through the following equation,

$$d\tau = \frac{GH}{G+H} d\gamma$$

in which G is the shear modulus and H is the plastic modulus.

As for the failure criteria, shear strength is assumed to be isotropic with respect to shear stresses and assumed to vary linearly with the effective pressure, implying a circular cone shaped failure surface in the stress space as expressed with the following equation,

$$F(\sigma) = \tau_x^2 + \tau_y^2 - (M\sigma')^2 = 0 \tag{1}$$

in which M=tanϕ, where ϕ is the internal angle of friction. Thus, M defines the failure condition $\tau / \sigma' = M$ which is assumed to have a conical shape.

The undrained effective stress path is assumed to be a quarter of an ellipse in $\tau - \sigma'$ plane, equation of which is given as follows

$$f_e = \tau_x^2 + \tau_y^2 + \lambda^2 \left[\sigma'^2 - \left(\frac{2\lambda}{\lambda + M} \right) \sigma' \sigma_O + \left(\frac{\lambda - M}{\lambda + M} \right) \sigma_O'^2 \right] = 0$$

(6)

Two parameters, namely σ_0' and λ completely define the effective stress path. The material parameter λ is the ratio of vertical to horizontal axis of the effective stress path which can be correlated to the density of sand, generally increasing with relative density. The effective stress path is followed only during the elasto-plastic shear stress increments when yielding takes place. The criterion for the onset of liquefaction is stated as the effective stresses reaching a residual value.

In the method, the damping is considered to comprise of two mechanisms in the nonlinear material model, hysteretic damping and pore pressure dissipation with time, and no additional viscous damping ratio is used. The soil deposit is separated into a number of horizontal sublayers with nodal planes and boundaries, which are assumed to remain planar during motion.

THE INFLUENCE OF A SILT LAYER WITHIN SAND DEPOSIT

In the natural sand deposits usually some silt seams and bands are present, and they can have a significant effect on development of pore pressures which might lead to liquefaction as well as to the after effects of liquefaction such as lateral spreading and slope instability. Experimental research have shown that sands mixed with silts of high plasticity increase the resistance against liquefaction, whereas silts of low plasticity can cause reduction in liquefaction resistance (Erken and Ansal,1994). On the other hand, presence of a silt layer of lower hydraulic conductivity within the sand deposit is believed to hinder the flow of pore water under the influence of cyclic loading, leading to increased excess pore pressure build-up. Additionally, due to the accumulation of water underneath the silt seam, a thin water film may develop as demonstrated by the experimental evidence (Özener et.al., 2005). In this study, a series of numerical analyses are carried out on a typical 20.0 m thick sand deposit with or without a thin silt seam (1.0 m or 2.0 m's thick) at various depths from the ground surface. Material properties used in the analysis are given in Table 1.

Table 1. Material Parameters used in numerical analysis

Material Parameter	Relative Density (Dr%)		
	30	50	80
Void ratio, e	0.85	0.75	0.60
Coefficient of permeability for the sand layer k (m/s)	10^{-4} and 10^{-6}	10^{-4} and 10^{-6}	10^{-4} and 10^{-6}
Coefficient of permeability for the silt interlayer, k (m/s)	10^{-7} and 10^{-8}	10^{-7} and 10^{-8}	10^{-7} and 10^{-8}
Internal Friction Angle, ϕ (°)	30	35	40
Constrained Modulus, (kPa)	61000	75000	99000
Initial Shear Modulus, G_0 [1] (kPa)	14000	17000	23000
Initial Plastic Modulus, H_0 [1] (kPa)	14000	17000	23000
Change in shear modulus, ΔG (kPa/m)	4000	6000	8000
Change in plastic modulus, ΔH (kPa/m)	4000	6000	8000

[1]The initial values are increased linearly with depth,z in meters $G_z = G_0 + (\Delta G)z$, $H_z = H_0 + (\Delta H)z$

In Figure 1 some typical results demonstrating the variation of excess pore pressure ratio (r_u) with depth are presented for a 20.0 m thick loose sand deposit (Dr=30%) subjected to ground motion recorded at a rock site (Arçelik-NS) during the August 17, 1999 Kocaeli earthquake with peak acceleration value scaled to 0.30g. The ground water level is considered to be at ground surface and the coefficient of permeability for sand is assumed to be 10^{-4} m/s, having a 1.0 m thick silt seam at 4.0 m depth with two different hydraulic conductivities (k=10^{-7} m/s and 10^{-8} m/s) and subjected to 1D shaking.

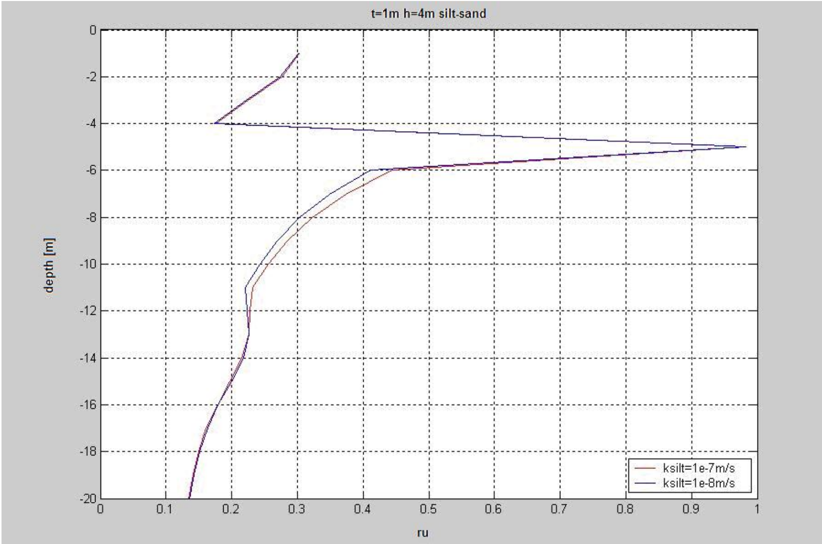

Figure 1. Variation of excess pore pressure ratio (r_u) with depth in a sand deposit with a silt seam.

The same sand deposit having either a 1.0 m silt seam with k=10^{-7} m/s or without any silt seam and groundwater level being at 2.0 m depth is also subjected to either 1D or 2D earthquake motion using both NS and EW components of the Arçelik site strong ground motion records, scaled to a 0.30g peak acceleration for NS component. The variation of excess pore pressure ratio (r_u) values with depth shown in Figure 2 demonstrate that within the homogenous sand deposit no liquefaction is expected under the influence of 1D shaking, but liquefaction will develop between 4 to 9 m depths under the influence of 2D shaking.

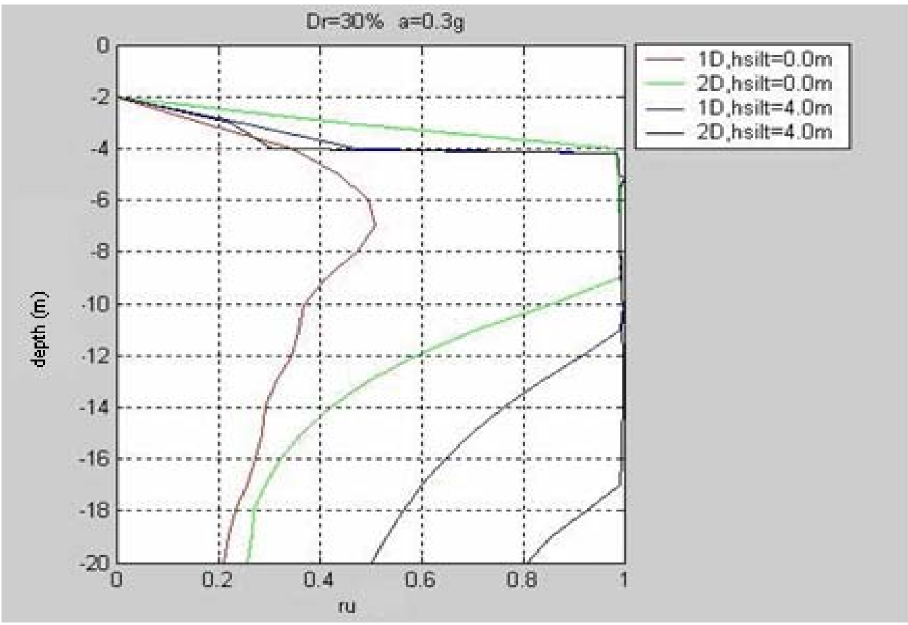

Figure 2. Variation of excess pore pressure ratio (r_u) with depth in a sand deposit with a silt seam under 1D and 2D shaking.

On the other hand, when a silt seam is present within the sand deposit, liquefaction will develop between 4 to 11 m depths with 1D shaking and between 4 to 17 m depths with 2D shaking. These results clearly show the significant influence of a silt seam being present in a sand deposit on development of excess pore pressures leading to liquefaction condition and its extend, as well as the importance of taking into consideration both horizontal components of earthquake motion in liquefaction analysis. Similar observations could be made from the results of analysis on sand deposits of other densities and hydraulic conductivities, influence of the silt seam being more pronounced when the ratio of permeability of sand/silt is higher.

ANALYSIS OF THE WILDLIFE LIQUEFACTION ARRAY TEST SITE BEHAVIOR

A test site has been instrumented by USA Geological Research Services (USGS) at a potentially liquefiable site near Alamo river about 160 km east of San Diego,CA. The site soil profile is comprised of a 2.5 m thick clayey silt at top, underlain by a 4.5 m thick sand deposit of high liquefaction potential followed by an overconsolidated silty clay layer down to at least 30.0 depth. At this site, 5 piezometers were installed at various depths within the sand layer and two accelerometers, one at ground surface and the other at 7.50 m depth were installed. The field instrumentation have recorded excess pore pressures developed and the acceleration time histories during the 1987 Superstition Hills Earthquake of 23 seconds duration.

The recorded peak ground acceleration values were 0.204 g and 0.170 g, at ground surface and at 7.50 m depth, respectively. The recorded excess pore pressure-time histories at four different depths are shown in Figure 3.

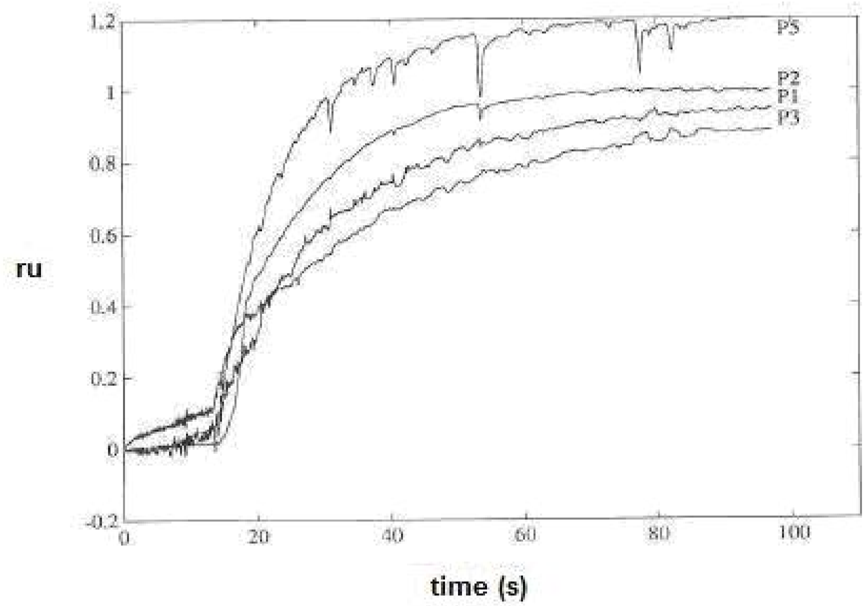

Figure 3. Variation of measured excess pore pressure ratios with time at Wildlife Liquefaction Array Test Site (Youd and Carter, 2003).

The behavior of the Wildlife Liquefaction Array Test Site during 1987 Superstition Hills Earthquake is analyzed using LASS IV depicting the required material parameters from the information available about the site soil conditions.The acceleration time history recorded at 7.50 m depth is exerted at the base of sand layer and the development of excess pore pressures at the piezometer levels within the sand deposit are computed, which are shown in Figure 4 and 5 for both the case of 1D shaking and 2D shaking.

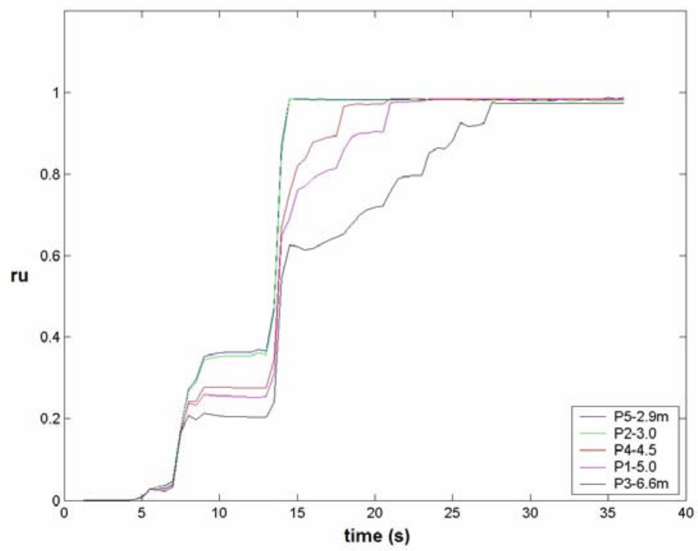

Figure 4. Variation computed excess pore pressure ratio with time for 1D at Wildlife Liquefaction Array Test Site.

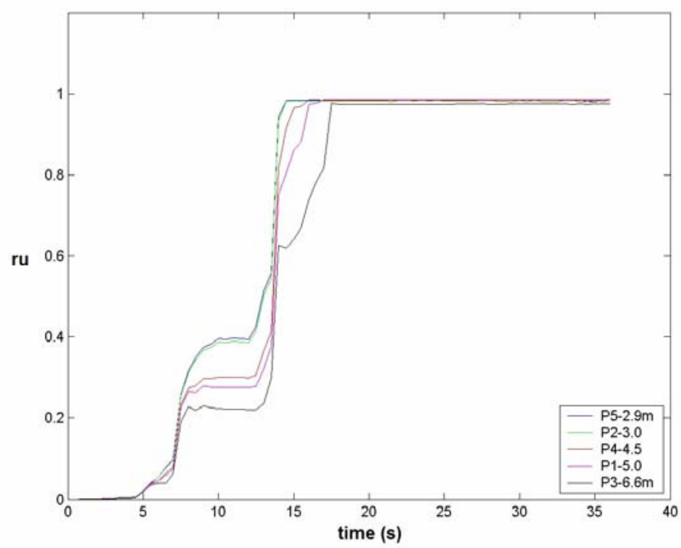

Figure 5. Variation computed excess pore pressure ratio with time for 2D at Wildlife Liquefaction Array Test Site.

When the computed values are compared with the recorded values, the following observations can be made:

The computed and measured maximum pore pressures are quite close to each other but unexpectedly their shape are more compatible when only one horizontal component of the earthquake is taken into account in the analysis.

The rate of pore pressure accumulation in the field seems to be slower than the predicted rate.The excess pore pressures measured in the field stayed limited until the peak acceleration value is reached (13.6 s) and continued to increase after the earthquake is over for another 70-80 seconds.

Even though the liquefaction phenomenon could be predicted, some contradictions are observed between the computed and measured excess pore pressure values which are believed to be due to following reasons:

i) In the analysis the excitation measured at 7.5 m depth is used as the base rock acceleration which is not the case in the field.
ii) The material parameters used in the analysis are estimated only from the limited data available about the field soil layers.

The excess pore pressure-time behavior observed in the field has been attempted to be explained differently by various researchers. Youd et.al (2004) have claimed that this might be a result of continued cyclic shear stresses due to oscilations experienced by the soil layers even after the earthquake is over. Others such as Husmand (1992) and Yoshida (1998) expressed some doubts about the reliability of field measurements.

CONCLUSIONS

It is shown that numerical analysis techniques can quite effectively be used to model the behavior of soil layers subjected to earthquake excitations in the field. The excess pore pressure generation and state of liquefaction in the sand deposits can be predicted quite realistically, and this enables to study the effects of various parameters known to be important in the development of liquefaction phenomenon. In this study, the effect of the existence of less permeable silt seams in a sand deposit is studied using the computer code LASS IV and it is shown that it can have a major effect on the field excess pore generation and occurrence of liquefaction, and can cause a significant increase in depth of liquefaction. It is also shown that taking into consideration the effects of two horizontal components of an earthquake rather than one can significantly increase the probability of occurrence of liquefaction and its extent of depth.

The behavior of an instrumented test site during an earthquake is also numerically analyzed and it is shown that the magnitude of excess pore pressures and development of liquefaction condition can be quite realistically predicted, even though significant discrepancies between measured and computed values are observed in the rate and duration of excess pore pressures.

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