NUMERICAL MODELLING OF DYNAMIC SOIL LIQUEFACTION IN SLOPING GROUND

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ABSTRACT

In sloping ground, before application of dynamic loading, the ground is subjected to a static shear stress due to the weight of the soil and the slope of the ground. Static shear stresses will act as driving forces and cause very large ground deformations even before the onset of soil liquefaction. Therefore reliable prediction of soil response is essential in the assessment of remediation methods to reduce liquefaction induced soil deformation. This paper investigates the application of a stress path model to simulate the soil liquefaction in sloping ground. Pore pressure generation and liquefaction strength of the soil predicted by the numerical model are compared with a series of simple shear tests performed on loose sand with and without an initial static shear stress simulating sloping and level ground conditions, respectively. Numerical predictions are shown to be in good agreement with test data.

INTRODUCTION

Soil liquefaction is the extreme manifestation of the excess pore pressure generation when saturated soil deposits are subjected to earthquake loading. With the increase in pore pressure, soil stiffness and strength decrease rapidly and the ability of the soil deposit to support foundations of buildings and bridges is reduced significantly. Many major earthquakes that have occurred in the past such as 1964 Niigata, 1964 Alaska, 1989 Loma-Prieta and 1995 Kobe events have demonstrated the devastating effects of soil liquefaction.

Mainly there are two approaches available to evaluate the amount of pore pressure generation in saturated ground during an earthquake. They are the effective stress and total stress based analysis methods. The deficiency of total stress analysis is that it calculates pore pressures based on the total stresses developed in ground during the earthquake loading and not based on the effective stresses. There is no way to evaluate the progressive degradation of soil stiffness and strength during a total stress analysis.

In general, the amount of resistance provided by the soil against deformation at any point in the soil deposit depends on the effective stress level at that point. Therefore, it is important to evaluate pore pressures based on the effective stress level and ground deformations using the degraded soil stiffness and strength as a result of increase in pore pressure during earthquake loading.

On level ground, effective stresses will reduce to near zero at the onset of liquefaction and any residual displacements will be small in the absence of driving stresses acting on ground. However, in sloping ground, the situation is different. A soil element near the surface of a slope is subjected to a driving static shear stress (Park and Byrne, 2004). Hence, with the increase in pore pressure, ground deformations can be very large due to these driving shear stresses but the effective stress may not reach near-zero at the onset of liquefaction.

In this paper, cyclic simple shear tests carried out at the University of British Columbia, Canada (Park and Byrne, 2004) have been simulated using a numerical model based on the stress path method proposed by Ishihara and Towhata (1982). The simulation results clearly show the patterns of failure that occur in level and sloping ground conditions.

NUMERICAL MODEL

The numerical model is one-dimensional and it is based on the finite element method. The analysis is carried out by dividing the soil deposit into a number of layers. The constitutive behaviour of the soil is modelled using a hyperbolic stress-strain relationship, which reflects the non-linear, strain-dependent and hysteretic behaviour of the soil. Pore pressure generation during cyclic loading is evaluated using the stress path model proposed by Ishihara and Towhata (1982). Figure 1 shows the stress path obtained from the numerical model for a typical case of sand subjected to a constant amplitude cyclic shear stress.

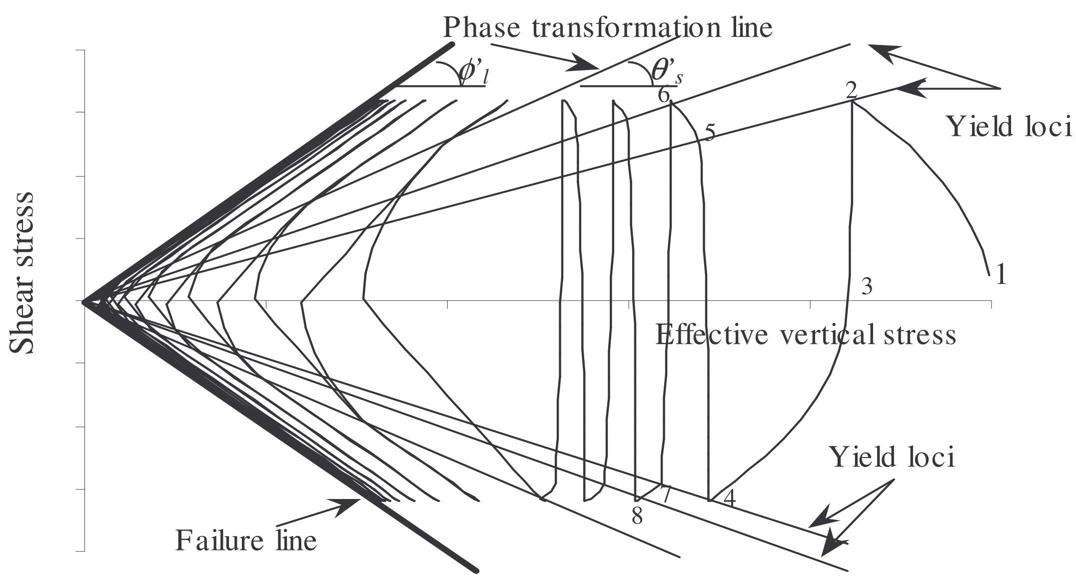

Figure 1. Stress path computed for a cyclic simple shear test.

The stress path from 1-2 is assumed to be a parabola given by,

$$\sigma_v' = m - \frac{B_p'}{m}\tau^2 \tag{1}$$

where m is a parameter to locate the parabolic stress path and $B'p$ is a soil constant representing the pore pressure characteristics of the soil. For stress path 1-2, m is the initial effective vertical stress. Stress path beyond the yield loci (e.g., 3-4, 5-6 and 7-8) is also defined by the same equation but m should be computed each time the stress path crosses the yield loci (e.g., points 5 and 7).
Within the current yield surface (e.g., 2-3, 4-5 and 6-7), the stress path is given by,

$$
\begin{aligned}
\Delta\sigma'_v &= -B'_u\left(\frac{\tau}{\sigma'_{v_o}} - \frac{\tau_m}{\sigma'_{v_o}}\right)\left(\frac{\sigma'_v}{\sigma'_{v_o}} - \kappa\right)\Delta\tau & \sigma'_v \geq \kappa\sigma'_{v_o} \\
\Delta\sigma'_v &= 0 & \sigma'_v < \kappa\sigma'_{v_o}
\end{aligned}
\tag{2}
$$

where B'_u is a soil parameter representing pore pressure characteristics of the soil and k is a parameter which takes into account the fact that pore pressure build up ceases when the σ'_v decreases to $\kappa\sigma'_{vo}$.

At point 'A' the stress path crosses the phase transformation line, which has a gradient of $0.625\tan\phi'_l$, and beyond point 'A' for both loading and reloading, the stress path is given by,

$$
\left(\frac{\sigma'_v}{m}\right)^2 - \left(\frac{\tau}{m\tan\varphi'_l}\right)^2 = 1 \tag{3}
$$

where ϕ'_l is the friction angle at very small effective confining stress. Equation (3) approaches the failure line asymptotically as shown in Figure 1. For unloading beyond point 'A', the stress path follows a straight line, which is tangential to the gradient of Equation (3) at the point of stress reversal. Theoretically, it is not possible to carry out an analysis with zero vertical effective stress. Therefore, when pore pressure has reached 97% of the initial effective overburden pressure, the stress path is allowed to follow the same hyperbolic curve without further increase in pore pressure. This assumption is reasonable because in normal ground the shear stress application is multi-directional. At the onset of liquefaction, always there will be a shear stress applied on the soil in the direction perpendicular to the direction of shear stress causing soil liquefaction (Ishihara and Towhata, 1982).

NUMERICAL SIMULATION OF CYCLIC SIMPLE SHEAR TEST DATA

The ability of the numerical model in simulating soil liquefaction in sloping ground has been evaluated using a series of constant volume, simple shear tests performed on Fraser River sand at UBC (University of British Columbia, Canada). These test data are available at http://www.civil.ubc.ca/liquefaction. Numerical simulations based on the plastic constitutive model UBCSAND (Byrne et al., 1995; Puebla et al., 1997) are described by Park and Byrne (2004).

When applying the stress path model presented here to simulate the constitutive behaviour of liquefying soil, four model parameters (θ'_s, κ, B'_p and B'_u) need to be determined prior to the analysis. In this study, the phase transformation angle, θ'_s, and angle of internal friction, φ_l', at very small effective confining stress are related to the friction angle of the soil, ϕ, as follows (Ishihara and Towhata, 1982):

$$tan\,\varphi_l' = 1.4\,tan\,\varphi \quad (4a)$$

$$tan\,\theta_s' = \frac{5}{8} tan\,\varphi_l' \quad (4b)$$

The soil constant, κ, is determined based on the cyclic shear strength curve and given by

$$\kappa = \frac{\tau_a}{\sigma_{vo}'} \frac{1}{tan\,\varphi'} \quad (5)$$

where τ_a/σ_{vo}' is the minimum cyclic stress ratio below which liquefaction does not occur. For all analyses carried out here for D_r = 44% Fraser River sand, κ = 0.01 has been used.

The third soil constant B_p' is determined using Equation (1). The effective stress level is determined from the pore pressure increase recorded in the first cycle during virgin loading in compression. The value of B_p' should be a constant for a particular soil type with a particular relative density. In this study B_p' of 8.2 has been used, based on the experimentally observed pore pressure increase in the first cycle during virgin loading in compression.

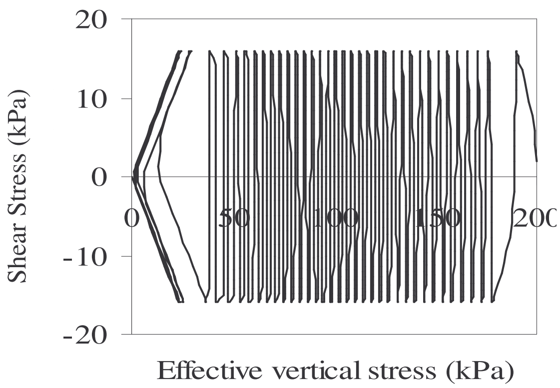

Figure 2. Stress path predicted by numerical model (CSR=0.08, α = 0.0).

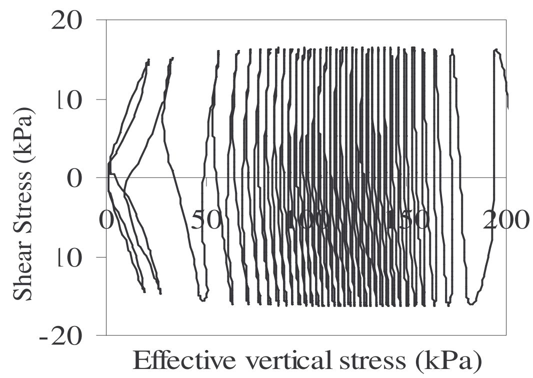

Figure 3. Stress path from the cyclic simple shear test (CSR=0.08, α = 0.0).

The fourth parameter B_u' governs the amount of pore pressure generation during unloading and reloading inside the current yield surface as given by Equation (2). A value for B_u' has been selected using the chart provided by Ishihara and Towhata (1982) where B_p' and B_u' are related to the Cyclic Stress Ratio (CSR) required to obtain liquefaction after application of 20 stress cycles to the soil.

Figures 2 and 3 show, respectively, the variation of shear stress with effective stress level predicted by the numerical model and that observed during the experiments for a soil sample subjected to a CSR of 0.08 without any static shear stress. Figure 4 shows the excess pore pressure generation predicted from the numerical model. When the effective stress of the soil is below 75 kPa, the experimental results show rapid softening of soil. According to the numerical model, soil softens rapidly when the effective stress of the soil is below 50 kPa. Pore pressure generation predicted using the stress path method is slightly high after the 10th cycle. However, the onset of soil liquefaction, which occurs after about 35 cycles, predicted by the numerical model agrees well with the simple shear test data.

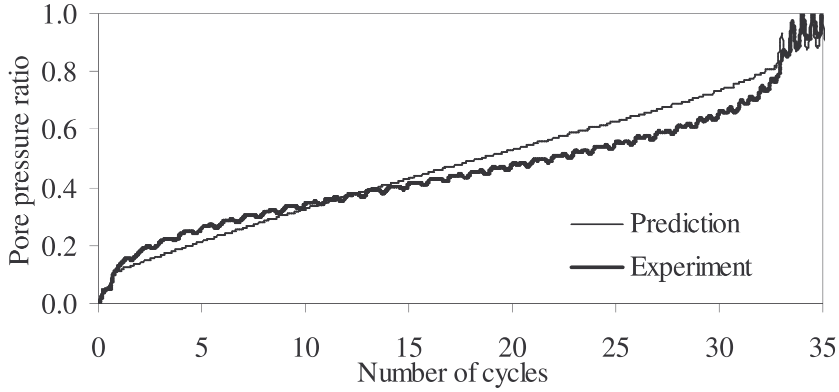

Figure 4. Pore pressure increase during cyclic loading (CSR=0.08, $\alpha = 0.0$).

Figure 5 shows the number of cycles required for liquefaction at different CSRs, when the static shear stress acting on the soil is zero. At cyclic stress ratios higher than 0.1, numerical prediction shows higher resistance to liquefaction compared to that observed during the cyclic simple shear test. The largest difference observed between the numerical model and the experiment is within $\pm 10\%$ and this is acceptable for design purposes.

In sloping ground, before application of cyclic loads leading to pore pressure generation and subsequent soil liquefaction, the ground is subjected to static shear stress due to the weight of the soil. Park and Byrne (2004) simulated this condition in cyclic simple shear tests, where tests were carried out with a static shear stress, τ_s. The static shear stress ratio, α, is defined as the driving static shear stress to initial effective vertical stress, σ'_{vo}.

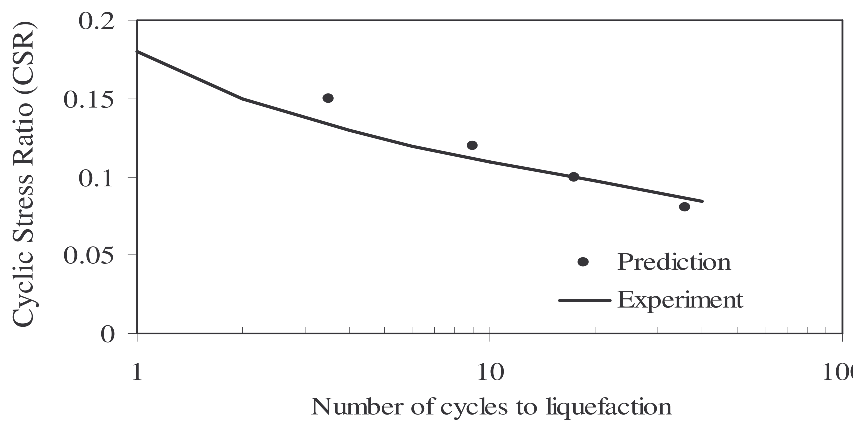

Figure 5. Predicted and measured liquefaction response of Fraser River sand ($\alpha = 0.0$).

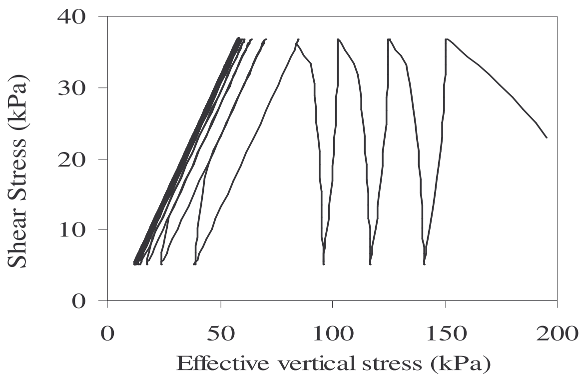

Figure 6. Stress path predicted by numerical model (CSR=0.08, α = 0.1).

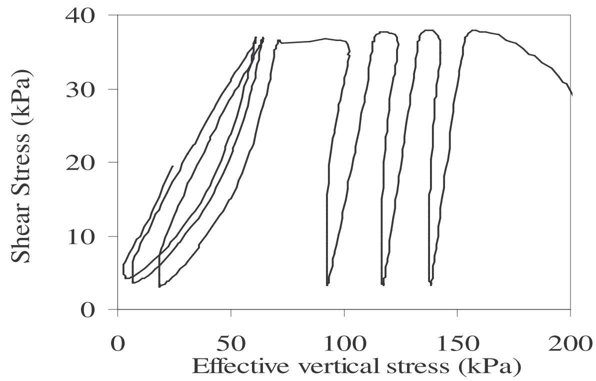

Figure 7. Stress path from the cyclic simple shear test (CSR=0.08, α = 0.1).

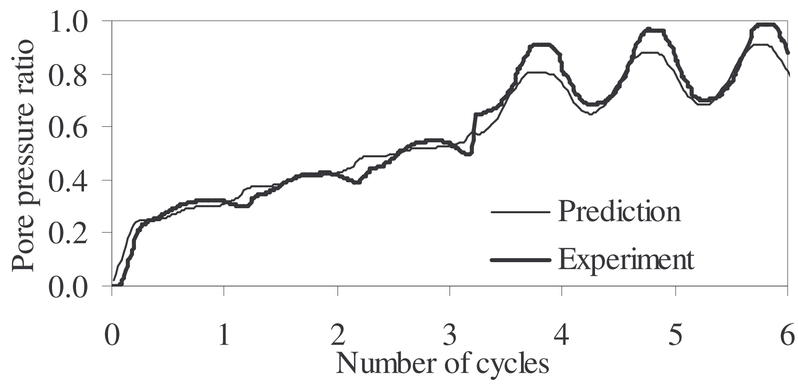

Figure 8. Pore pressure increase during cyclic loading (CSR=0.08, α = 0.1).

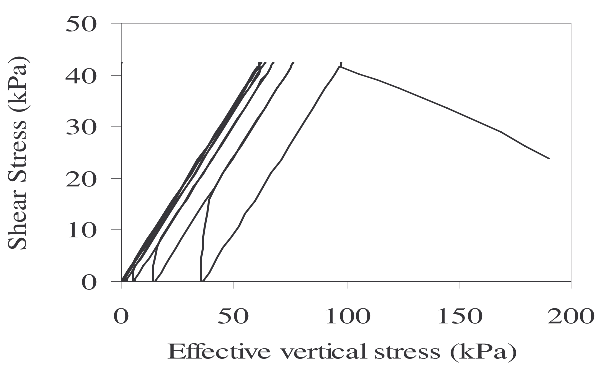

Figure 9. Stress path predicted by numerical model (CSR=0.1, α = 0.1).

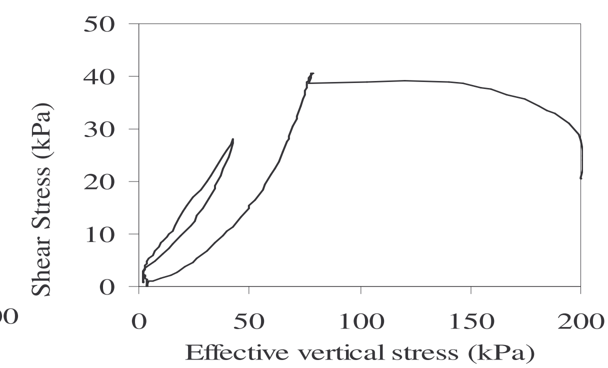

Figure 10. Stress path predicted by numerical model (CSR=0.1, α = 0.1).

Figures 6 and 7 show the stress paths obtained from the numerical model and the experiment for CSR=0.08 and $\alpha = 0.1$. In both cases, liquefaction is initiated after 4 stress cycles. When this result is compared with Figures 2 and 3, the influence of initial static shear stress on the soil liquefaction is clear. When there is no static shear stress, soil could sustain 35 stress cycles with CSR=0.08 before liquefaction but when there is a static stress of 0.1xσ'_{vo}, soil can sustain only 4 stress cycles before liquefaction. Figure 8 shows the predicted pore pressure generation and the predicted values closely agree with the experimental results. Figures 9 to 11 show the stress paths and excess pore pressure generation for CSR = 0.1 and α = 0.1. These results clearly show that the stress path model has the ability to simulate the soil behaviour in both level and sloping ground. In a companion paper (Liyanapathirana, 2007), application of this method to simulate a centrifuge test will be described.

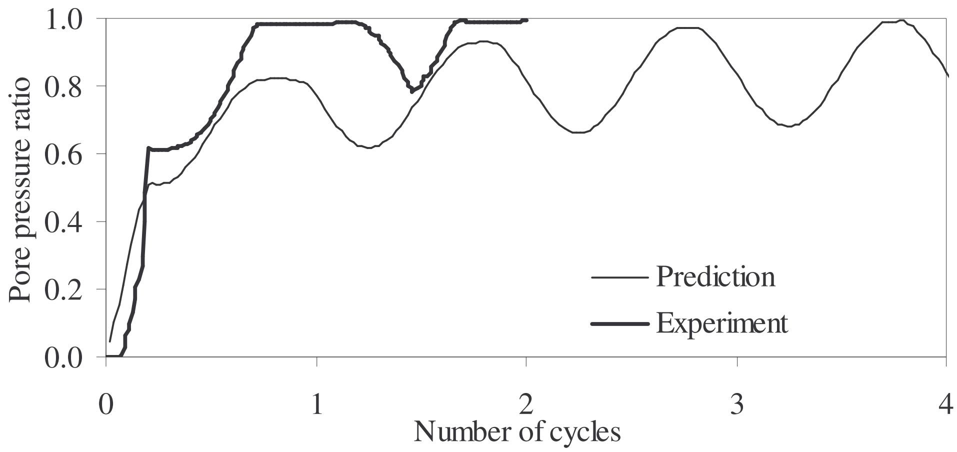

Figure 11. Pore pressure increase during cyclic loading (CSR=0.1, α = 0.1).

CONCLUSIONS

A numerical model based on the stress path model has been applied to predict soil liquefaction in conditions similar to level and sloping ground conditions, where the soil is subjected to a static shear stress prior to application of cyclic shear stresses. Simple shear tests simulates with this model clearly show that the model has the ability to simulate soil liquefaction in sloping ground. In a companion paper (Liyanapathirana, 2007), the method has been applied to simulate a centrifuge test performed in a sloping, layered soil deposit.

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