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Response of Liquefaction Phenomenon on Variation of Permeability

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Abstract: Liquefaction phenomenon is usually accompanied by large amounts of settlement owing to disruption of soil structure. Loose granular materials such as sands are susceptible to liquefaction, resulting failure of civil structures resting on ground. The ability to predict the dissipation of the excess pore water pressure is important in assessing liquefaction phenomenon. This paper investigates the effect of variation of permeability on the assessment of liquefaction potential of a soil deposit during earthquake. A formulation based on Finite Element (FE) method is developed for solving coupled liquefaction problem in FORTRAN90, and its validity is examined with the variation of permeability. The basic matrix equations for coupling analysis using finite elements are derived, based on the basic equations of Biot's formulation of the soil-pore fluid mixture, coupled with the material constitutive model and the equilibrium equations. Spatial variables are discretized using the shape function and Newmark's Beta scheme is used to discretize the time domain. The numerical results reveal that there is a direct relationship between permeability coefficient and rate of excess pore pressure generation and dissipation in liquefying the soil profile.

Keywords: Liquefaction, Pore water pressure, Permeability, Coupled Analysis

1. Introduction

The failure of the civil engineering structures occur during earthquake due to various reasons amongst which liquefaction of sandy soils is one of the most important, interesting, complex, and controversial phenomena in geotechnical engineering. When soil liquefaction occurs, the strength and stiffness of the soil decreases and the ability of the soil deposit to support foundations of buildings and bridges is reduced. The interaction of soil and pore fluid under loading may lead to the build-up of pore pressure, which results in material softening and loss of shear strength. In extreme case the soil loses all the shearing resistance and fails like a viscous liquid, a phenomenon known as 'liquefaction' (Seed and Idriss, 1971). Major earthquakes such as the 1964 Niigata, 1989 Loma-Prieta, 1964 Alaska and 1995 Kobe events have demonstrated the damaging effects of soil liquefaction.

Loose granular materials such as sands are susceptible to compaction under vibration or cyclic loading. However, the reduction in volume is often prevented by lack of drainage, due to relatively low permeability, long drainage path or high frequency of load, during the period of vibration. Hence, a nearly untrained conditions prevail which result in build-up of pore pressures to counter such contractive behaviour.

As described above, soil settlement during seismic shaking is due to the dissipation of pore water pressure, and settlement increases with the increase in the amount of generated pore pressure and drainage. The main phenomenon responsible for generation of pore water pressure is the contractive soil response which subsequently leads to the obvious conclusion that more pore water pressure is developed in loose deposits under earthquake loading. On the other hand, the most important parameter affecting drainage is the permeability coefficient. A high value of the permeability coefficient causes a rapid dissipation of excess pore water pressure and consequently more settlement occurs during seismic loadings. Field observations of settlement due to seismically induced liquefaction range from fractions of an inch to over a foot (Tokimatsu

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and Seed, 1987). This large amount of settlement is mostly due to disruption of soil structure by liquefaction. Field observations and investigations show that the rate of water flow increases during liquefaction period. It has been reported in a number of investigations that one of the main mechanisms causing substantial settlement is significant increase in the soil permeability during seismic excitation.

The work presented in this paper utilizes a fully coupled dynamic analysis based on Biot's formulation to investigate the effects of variable soil permeability on the seismic behaviour of saturated sandy soils.

2. General Formulation

There are several different approaches to model the behavior of a two-phase medium. Generally, they can be classified as uncoupled and coupled analyses. In the uncoupled analysis, the response of saturated soil is modelled without considering the effect of soil–water interaction, and then the pore water pressure is included separately by means of a pore pressure generation model.

In the coupled analysis a formulation is used where all unknowns are computed simultaneously at each time step. This is a more realistic representation of the physical phenomena than that provided by uncoupled formulation. For a fully coupled analysis, equilibrium or momentum balance for the soil–fluid mixture, momentum balance for the fluid phase, and finally mass balance for the whole system of soil and fluid must be satisfied. The unknowns in this complete set of equations are displacement of solid phase (u_s), displacement of fluid phase relative to the solid phase (u_{rf}), and pressure of fluid phase (P). It is convenient to reduce the number of variables by neglecting the terms that have little influence on the results. For dynamic problems in which high-frequency oscillations are not important, such as problems under earthquake loading, the relative velocity of fluid phase has little influence on the system and can be eliminated (Zienkiewicz et al. 1999). Therefore, the equations for fluid momentum balance and mass balance can be mixed together and as a result the governing equations are reduced to two. The primary variables in this form of equations are solid displacement and fluid pressure. Thus, this form is called u_s–P or for simplicity u–P formulation. The contribution of the solid acceleration in the equation of momentum balance of the fluid phase will render the final system of equations non-symmetric and can give unstable solutions in some cases (Park et al. 1983). This contribution has little effect on the system (Zienkiewicz et al. 1987) and can be neglected.

Using the finite element method for spatial discretization, the u–P formulation with the above mentioned simplification is as follows:

$$[M]\ddot{q}_e + [K]\ddot{q}_e - [Q]\{P_e\} = \{f_u\} \quad \ldots (1)$$

$$[G]\ddot{q}_e + \left[Q^T\right]\dot{q}_e + [S]\dot{p}_e + [H]p_e = \{f_P\} \quad \ldots (2)$$

Where $[M]$ is the mass matrix , $[K]$ is the stiffness matrix, $[Q]$ is coupling matrix, , $[G]$ is dynamic coupling matrix , $[S]$ is the compressibility matrix , $[M]$ is the permeability matrix , $\{f_u\}$ is force vector for solid phase, $\{f_p\}$ is the force vector for fluid phase. q_e is the solid displacement vector, p_e is the pore pressure vector. For numerical solution of above coupled equations, it is necessary to integrate the equations in the time domain which can extrapolated to

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the next time instance (t_{n+1}) using known previous initial conditions by employing generalized Newmark method (Katona et al.1985). At time t_{n+1}, the acceleration, velocity, and pore pressure gradient are expressed as:

$$\Delta \dot{q}_i = \ddot{q}_i \left(1 - 0.5\alpha/\beta\right)\Delta t + \alpha/\left(\beta \Delta t\right)\Delta q_i - \left(\alpha/\beta\right)\dot{q}_i \qquad \ldots (3)$$

$$\Delta \ddot{q}_i = \left(1/\beta \Delta t^2\right)\Delta q_i - \left(1/\beta \Delta t\right)\dot{q}_i - \left(1/2\beta\right)\ddot{q}_i \qquad \ldots (4)$$

$$\Delta \dot{p}_n = \left(1/\alpha \Delta t\right)\Delta p_n - \left(1/\alpha\right)\dot{p}_n \qquad \ldots (5)$$

By employing the above equation and choosing Δq_e and Δp_e as primary unknowns, the final set of equations is obtained as follows:

$$\left(C_1[M]+[K]\right)\{\Delta q_i\} - [Q]\{\Delta p_i\} = \Delta F_u + \left(C_3\dot{q}_i + C_4\ddot{q}_i\right)[M] \qquad \ldots (6)$$

$$\left(C_1[G]+C_2[Q]^T\right)\{\Delta q_i\} - \left(C_3[S]+[H]\right)\{\Delta p_i\} = \Delta F_p + \left(C_3\dot{q}_i + C_4\ddot{q}_i\right)[G] + \left(C_5\dot{q}_i + C_6\ddot{q}_i\right)[Q]^T$$

$$where,\ C_1 = \frac{1}{\beta \Delta t^2};\quad C_2 = \frac{\alpha}{\beta \Delta t};\quad C_3 = \frac{1}{\beta \Delta t};\quad C_4 = \frac{1}{2\beta};\quad C_5 = \frac{\alpha}{\beta};\quad C_6 = \frac{1}{2\beta} - 1 \qquad \ldots (7)$$

In which, α and β the parameters of the generalized Newmark method and Δt is the time step. The vectors $\dot{q}_t$, $\ddot{q}_t$ and $\dot{p}_t$ can be evaluated explicitly from the information available at time t_n. The soil profile will be discretised using the 8 – 4 – Node mixed element which satisfies the Babuska-Brezzi stability condition and the constraint count is used. The mixed element (as shown in Fig. 1) has 8 displacement nodes and 4 pore pressure nodes. Therefore, displacements are continuous biquadratic and pore pressures are continuous bilinear in the element.

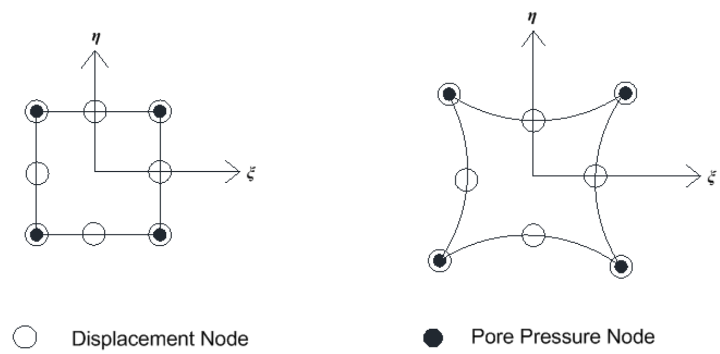

Fig. 1 Mixed Displacement-Pore Pressure Element

3.Analysis Of 1940 El-Centro Earthquake

To demonstrate the ability of the new method to predict the earthquake-induced ground response, the dynamic response of an instrumented site at El-Centro during the 1940 earthquake has been simulated. Loose Sand Strata of 20m width and 10m depth underlain by gravel soil strata of 5m depth is considered under the present study having properties as given in table 1. The input ground acceleration as given in Fig.2 is simulated at 10m depth of soil profile.

Table.1 Material parameters for Soil

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Shear Modulus *G*	50 MPa
Porosity	30 %
Void Ratio	0.43
Poisson's Ratio *υ*	0.499
Bulk modulus of soil grain	100 MPa
Bulk modulus of pore fluid	2000 MPa
Time stepping coefficients	α = 0.5, β = 0.25

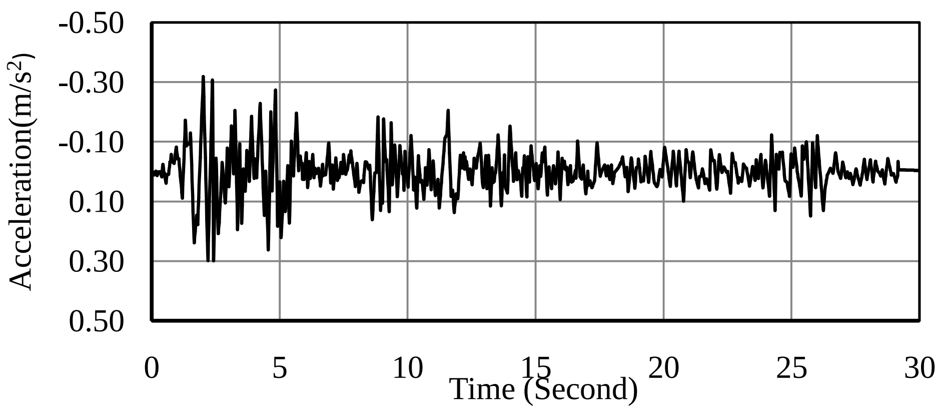

Fig. 2 Acceleration time history of El-Centro 2001 Earthquake

The distributions of displacement and pore pressure that developed at various times when drainage occurs during the earthquake are presented in Figs. 3 and 4 respectively at different value of permeability. When the permeability of soil domain is high, no liquefaction occurs where as at low permeability liquefaction occurs.

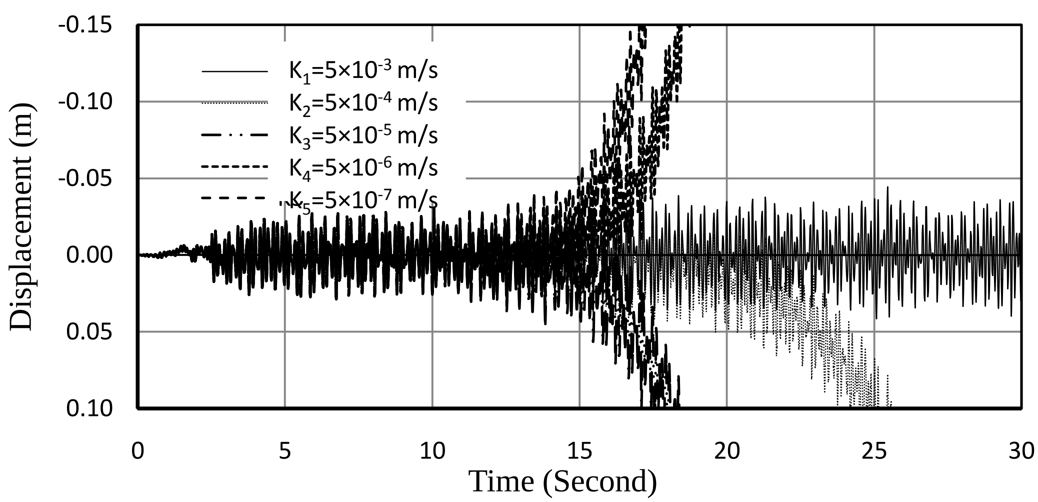

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Fig. 3: Displacement *vs* time with variation of Permeability

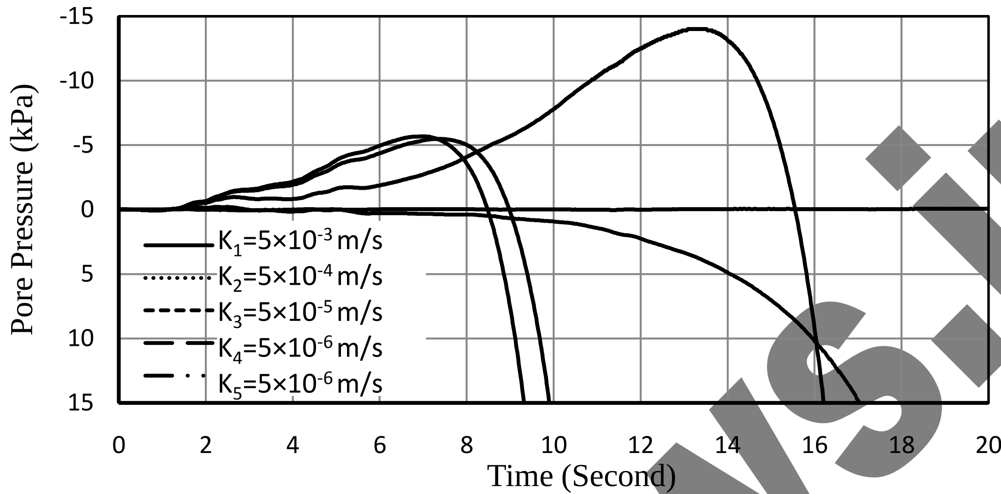

Fig. 4: Pore Pressure distribution for different value of Permeability

It can be observed that drainage increased the time of liquefaction and raised the level at which liquefaction first occurred for $K_2 = 5\times10^{-4}$ m/s whereas the high permeable sand did not liquefy at all under the given earthquake. Figs. 3 and 4 show that at about t = 14 sec or so, the earthquake motions are such that pore-water pressure dissipation is occurring slower than pore-water pressure generation.

4. Conclusion

The mathematical advantage of the coupled finite element analysis considered in the fact that excess pore pressure and displacement can be evaluated at any time numerically. At higher value of co-efficient of permeability, liquefaction does not occur within the sand strata. As the co-efficient of permeability value is decreased, liquefaction time is also decreased because of generation of higher pore pressure resulting in reducing the effective stress.

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