[TH-08] BEHAVIOUR OF SHALLOW FOUNDATION ON LIQUEFIABLE SOIL

Amrendra Kumar 1
1Researcher (Ph.D), Department of Civil Engineering, National Institute of Technology Patna, 800005.
E-mail: amrendraroy2k8@gmail.com

Sunita Kumari 2
2Assistant professor, Department of Civil Engineering, National Institute of Technology Patna 800005.
E-mail: sunitafce@nitp.ac.in

ABSTRACT: Soil liquefaction has led to excessive settlement, tilt and lateral displacement of many buildings on shallow foundations during previous earthquakes, causing damage to the structures and their nearby lifelines. The present paper describes the results of numerical modeling of a surface footing resting on loose liquefiable deposit using Biot’s basic theory for dynamics of saturated porous media. Pastor–Zienkiewicz Mark III constitutive model is used to describe the inelastic behaviour of soils in the dynamic simulations. Acceleration base input motion of sinusoidal nature is applied to the model for monitoring the displacements, liquefaction potential and excess pore pressures (EPP). Generalized Newmark-beta method is employed for integration in time. It is noticed that liquefaction occurs just below the footing.

KEYWORDS: Liquefaction, Newmark-Beta method, Pastor–Zienkiewicz Mark III, Excess pore pressures.

1. INTRODUCTION:
Earthquake-induced liquefaction is most commonly observed in loose, saturated, clean sand deposits. This is due to the tendency of the loose soil to compact under loading resulting dilatation during shearing. If the soil is fully saturated, water pressure tends to increase and attempts to flow out from the soil to regions with lower pore water pressure. However, if the loading is rapidly applied and is large enough, or if it is repeated many times at relatively high frequencies, as in an earthquake, the nearly undrained condition may result. Due to this, partial or total effective stress loss occurs resulting cyclic mobility or liquefaction respectively. When this phenomenon occurs, the strength and stiffness of the soil decreases and the ability of a soil deposit to support structural load is dramatically reduced.

Initially researchers had focused on experimental work to understand the liquefaction phenomenon and cyclic mobility (Seed and Lee, 1966), Seed and Idriss (1971), Castro (1975), Castro and Poulos (1977), Seed (1979), Seed et al. (1985), and Kramer (1996). While the physical phenomenon is well understood, analytical modelling of soil liquefaction and numerical simulation remains a challenge. The development of appropriate constitutive models, capable of predicting soil liquefaction has been a challenging problem for the researchers. Oka et al. (1994) discussed the FEM–FDM coupled liquefaction analysis of a porous soil using elasto plastic model. Elgamal et al. (2003) developed a computational model for analysis of cyclic mobility scenario based on general fully coupled (solid–fluid) finite element formulations. Mesgouez et al. (2005) presented the applications of Biot’s theory in transient wave propagation in saturated porous media. Taiebat et al. (2007) worked on numerical analyses of liquefiable sand using critical state two-surface plasticity model and densification model for bounded soil domain. Some of researchers study the shallow foundation situated on liquefiable soil analysis both experimental and numerical model analysis. Dashti et al. (2010) conducted Centrifuge test and results shows that building settlement was not proportional to the thickness of the liqueifiable layer and that most of this settlement occurs during earthquake strong shaking i.e. depend on intensity of earthquake. Karamitros et al. (2013) studied mechanisms that control the seismic liquefaction performance and results shows that a naturally or artificially created non-liquefiable soil crust may effectively mitigate the detrimental effects of liquefaction and allow for a performance-based design of surface foundations, without additional improvement measures. Dashti and Bray (2013) simulate fully-coupled numerical approach with the UBCSAND model implemented in FLAC-2D captured building settlements measured in these experiments reasonably well for one scaled input motion, mostly within factors of 0.7 and 1.8.

In the present study, a finite element based coupled algorithm is proposed for simulation of behaviour of shallow foundation on liquefiable soil domain. The generalized Biot’s theory is used to develop the governing equations, which couple the equilibrium and continuity equations for a deforming saturated porous medium. These are solved with Newmark-beta time marching scheme. The Pastor–Zienkiewicz Mark III model Kumari and Sawant (2015) has been used to describe the inelastic behavior of soils under isotropic cyclic loadings.
2. FINITE ELEMENT FORMULATION

The equilibrium and continuity equation have been used for the simulation of liquefaction. Generalized Newmark-beta method is employed for integration in time. In terms of incremental displacements \( \Delta q \) and pore pressure \( \Delta p \) as primary unknowns, the final set of equations from Kumari and Sawant (2015) are as follows:

\[
(L_1 [M] + [K]) \{ \Delta q^1 \} - [Q] \{ \Delta p^1 \} = \\
\Delta F_u + [M] (L_2 \dot q_{i-1} + L_3 \ddot q_{i-1})
\]

\[
(L_1 [G] + L_4 [\dot Q]^T) \{ \Delta q^1 \} - (L_2 [S] + [H]) \{ \Delta p^1 \} = \\
\Delta F_p + [G] (L_4 \dot q_{i-1} + L_5 \ddot q_{i-1}) + \\
[\dot Q]^T (L_6 \dot q_{i-1} + L_6 \ddot q_{i-1})
\]

\[
L_1 = \frac{1}{\beta \Delta^2}; L_2 = \frac{1}{\beta \Delta}; L_3 = \frac{0.5}{\beta} \\
L_4 = \frac{\alpha}{\beta \Delta}; L_5 = \frac{\alpha}{\beta}; L_6 = \frac{0.5}{\beta - 1}
\]

In which, \( \alpha \) and \( \beta \) are the parameters of the generalized Newmark constant and \( \Delta \) is the time step. The vectors \( \dot q \), \( \ddot q \), and \( \dot p \), can be evaluated explicitly from the information available at time \( t_n \).

The surface footing assumed at top surface. The load vector is distributed as:

\[
N_5 = \frac{1}{2} \{ 1 + \xi \} \xi; N_6 = \frac{1}{2} \{ 1 - \xi^2 \} \xi; N_7 = \frac{1}{2} \{ 1 - \xi \} \xi
\]

\[
\{ Q \}_e = \frac{1}{2} \xi \left\{ \begin{array}{c} \\
\frac{1}{2} (\xi + \xi^2) \\
\frac{1}{2} (1 - \xi^2) \\
\frac{1}{2} (1 - \xi + \xi^2) 
\end{array} \right. 
\]

(3)

In the present study, the saturated loose soil domain of size 12 m × 15 m had been considered for numerical simulation Figure 1. The top 10 m depth is loose soil whereas bottom 5 m depth soil is gravel. The soil domain in XZ plane is discretized into 180 elements of uniform mixed element as shown in Figure 1. The 8–4 node mixed element having eight displacement nodes and four pore pressure has been used. As a result, displacements are continuous biquadratic and pore pressures are continuous bilinear in the element. The top surface excluding surface footing node have atmospheric pressure. On the top of soil domain a surface foundation of vertical loading of 0.1 t/m\(^2\) is placed for analysis of liquefaction.

3. VALIDATION

Centrifuge modeling has been considered among the best experimental methods for modeling and observing soil liquefaction phenomena. It creates stress conditions in the model which closely simulate those in the full-scale prototype.

Fig. 1 Soil domain under consideration

Fig. 2 Comparison of vertical settlement at the top below the centre of footing

The correctness and accuracy of the proposed FEM based solution algorithm are validated by comparing the numerical results with the centrifuge model test results conducted at the RPI geotechnical centrifuge.
facility by Liu and Dobry (1997). The settlement curve of experiment and numerical modeling shows good agreement.

4. NUMERICAL STUDY

Material properties of the purposed model has been given in Table 1. The variation of displacement and pore pressure with time at different nodes had been calculated using finite element code written in FORTRAN-90. The variation of both parameters with time has been considered for comparing the response.

Table 1 Material properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma )</td>
<td>Unit weight of soil</td>
<td>18 kN m(^{-3} )</td>
</tr>
<tr>
<td>( \mu )</td>
<td>Poisson’s ratio</td>
<td>0.31</td>
</tr>
<tr>
<td>( K_1 )</td>
<td>Co-efficient of permeability (Top layer)</td>
<td>( 6.5 \times 10^{-5} ) m s(^{-1} )</td>
</tr>
<tr>
<td>( K_2 )</td>
<td>Co-efficient of permeability (bottom layer)</td>
<td>( 6.5 \times 10^{-5} ) m s(^{-1} )</td>
</tr>
<tr>
<td>( G_1 )</td>
<td>Shear modulus</td>
<td>10000 kPa</td>
</tr>
<tr>
<td>( G_2 )</td>
<td>Shear modulus</td>
<td>80000 kPa</td>
</tr>
<tr>
<td>( D_R )</td>
<td>Relative density</td>
<td>0.4</td>
</tr>
</tbody>
</table>

5. RESULT AND DISCUSSION

The method predicts the phenomenological features of dynamic response of behaviour of shallow foundation situated on saturated sand layers that commonly occur as pore water pressure rises in the sand during cyclic loading. It allows the distribution of pore water pressure and the effects that drainage and internal flow have on the time of liquefaction to be determined quantitatively. Figure 3 shows the vertical settlement of 16.2 cm, 19.1 cm and 20.4 cm are observed at left side, centre and right end of the footing respectively.

Figure 4 horizontal displacement of 1.02 cm, 2.73 cm and 4.33 cm are observed at left side, centre and right end of the footing respectively. Vertical displacement is high with respect to horizontal displacement.

Figure 5 and Figure 6 shows the typical variation of vertical and horizontal acceleration. The variation of acceleration just below the footing at top central point. Figure 7 shows the variation of Excess pore pressure (EPP) at different depth. At top and 10 m depth of footing EPP are 53 kPa and 84 kPa respectively whereas at 2 m depth, EPP is less than 23 kPa. It is noticed that liquefaction occurs just below the footing due to very high rise in EPP whereas after 2 m depth generation pore pressure is low compare to effective overburden pressure.

Due to placement of surface footing nearly undrained condition is prevailed below footing and hence high rise in EPP is seen. It is also observed that generation of EPP depends upon the drainage path and permeability. At 10 m depth liquefaction does not occur, due to high overburden pressure and presence of the gravel layer. It is noticed that liquefaction occurs at shallow depth.
CONCLUSION
The proposed formulation is capable of capturing the features of pore water pressure buildup and strength loss in granular soil deposit under cyclic loading. The maximum vertical settlement of 20.4 cm and horizontal displacement of 4.33 cm is observed. Maximum EPP developed below the footing is 53.6 kPA and at the 10 m depth EPP is 84kPa. Liquefaction occurs only at the immediate below the footing not at the 10 m depth because of effective overburden and drainage. The developed numerical formulation can be easily adapted to provide confidence for practicing engineers to use fully coupled procedures to predict liquefaction potential for unbounded domain.

References


