Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 356

BEHAVIOUR OF A PILE SUPPORTED STRUCTURE UNDER STRONG GROUND MOTION CONSIDERING LIQUEFACTION OF THE SOIL MEDIUM

B.K. Maheshwari[1] and Rajib Sarkar[2]

ABSTRACT

The behaviour of a pile-supported structure under strong ground motion is reasonably a complex problem. Further, the complexity gets aggravated when piles are embedded in liquefiable soil medium.

In this study, a finite element code in MATLAB has been developed to model soil-pile-structure system. Radiation boundary condition is properly simulated by considering frequency dependent Kelvin elements (springs and dashpots). The developed algorithm is validated with the established literature. The soil medium is modeled with a work-hardening plastic cap model. The pore pressure generation for liquefaction is incorporated by two-parameter volume change model established in the literature. For simplicity, a four-storied portal frame is considered for the structure. First, the kinematic interaction factors for soil-pile model are calculated for discrete frequencies considering both non-liquefying and liquefying soil medium. Then the inertial interaction due to structure is analyzed for a system consisting of portal frame on a 2×2 pile group-soil subsystem. The response of the structure is investigated at discrete frequencies of excitation and also for real earthquake time histories. The importance of considering nonlinearity and liquefaction of the soil medium for analysis and design of a pile-supported structure is highlighted in the paper.

Introduction

Recent earthquakes such as North Ridge (1994), Kobe (1995), Chi-chi (1999), Koeceli (1999) and Bhuj (2001) have caused foundation failure and subsequently the failures of many important massive structures (e.g. power plants, bridges, dams and offshore structures etc). Since many of such structures are founded on pile foundations, an adequate dynamic analysis of the Soil-Pile-Structure (SPS) systems is essentially required. The effect of nonlinearity on the response of structure resting on the pile cap of the soil-pile subsystem was investigated by Maheshwari et al. (2004). Shakib and Fuladgar (2004) performed seismic analysis of asymmetric buildings considering soil-structure interaction effects, but soil nonlinearity was not considered. Wilson (1998) studied the dynamic response of pile foundations in liquefying sand and soft clay during strong shaking. Uzuoka *et al.* (2007) performed three dimensional dynamic analyses of pile groups considering liquefaction of the soil medium and presented seismic analysis for a five storied

[1] Associate Professor, Dept. of Earthquake Engg., IIT Roorkee, India-247667, e-mail: bkmahfeq@iitr.ernet.in
[2] Sr. Design Engineer, Lahmeyer International (I) Pvt. Ltd., Kolkata, India, e-mail: rajibdeq@gmail.com

building which tilted during Kobe earthquake (1995). In the present paper, the effect of nonlinearity and pore pressure on the kinematic interaction factors of a soil-pile system is investigated for harmonic excitations with wide range of frequencies. The effect of inertial interaction for a four-storied portal frame with a wide range of frequency was demonstrated and the investigation is carried out for nonlinear work hardening plastic cap model for soil. Then the study for soil-pile-structure interaction is performed for soil medium capable of generating pore water pressure leading to liquefaction. Investigations are performed for harmonic as well as real time earthquake excitations.

Modeling of the System

A finite element code is developed in MATLAB for analyzing SPS system. Study was performed for 2×2 pile group with s/d = 5 (where s = centre to centre distance between the piles, d = width of the piles). Square piles of width 0.5 m are considered. The length of the piles is taken as 7.5 m. Piles are considered to be floating. A four storied portal frame is modeled above the pile cap. The plan dimension of the portal frame is 4.0 m $\times$ 4.0 m and the storey height is 3.5 m. A full three dimensional view of the soil-pile-structure system is shown in Fig. 1. Radiation boundary condition is considered at the lateral boundary of the soil-pile system.

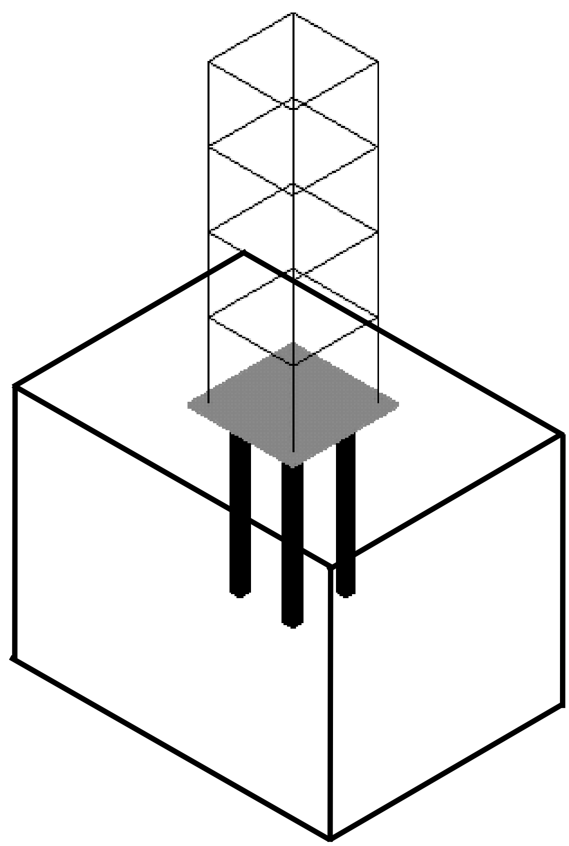


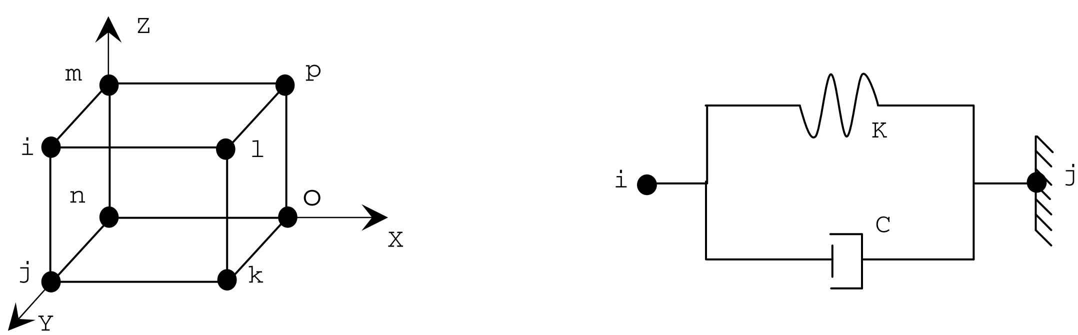
Figure 1. Full three dimensional view of the Soil-Pile-Structure system.

Elements Used

The soil and piles are modeled using eight-noded solid elements (Fig. 2a). Each node has three translational degrees of freedom in all three i.e. X, Y, and Z coordinate directions. Since with eight-noded solid elements, piles under bending deformations may show the shear locking phenomenon, therefore, for the pile elements, the Extra Shape Functions (ESF) was used.

To simulate infinite soil medium, two-noded Kelvin elements (Fig. 2b) are attached in all three directions (i.e. X, Y and Z) along the mesh boundaries. The Kelvin element constants in the two horizontal directions are calculated using solution by Novak and Mitwally (1988). The constants for vertical directions are given by Novak et al. (1978). To determine the stiffness and damping of the Kelvin elements, the constants are multiplied by the area of the element face (normal to the direction of loading) because they assume constant unit area of contact. For static loading, i.e. zero frequency, the damping term vanishes for horizontal case reducing to a spring only whereas for vertical case both the constants reduce to zero.

For modeling portal frame, two-noded beam element with six degree of freedoms (dofs) at each node is considered. The mass of the infill, mass of the slab and imposed load are considered for the portal frame. The cross section of all beams and columns for the portal frame are assumed to be the identical and is taken as 300 mm $\times$ 300 mm.



(a) Eight-noded solid element (b) Two-noded Kelvin element

Figure 2. Finite elements used for the study.

Nonlinear soil model

Drucker-Prager soil model with work hardening (Sandler and Rubin, 1979 and Chen and Baladi, 1985) is considered in the study. The basic assumptions of the model are: the yield surface must be convex in stress space, the yield surface and plastic potential surface must coincide and work softening must not occur. Of primary importance is its ability to model volumetric hysteresis through the use of a strain-hardening yield surface or cap. This model effectively simulates the stress-strain-pore pressure response of fluid saturated granular materials. The details are presented in Chen and Baladi (1985).

Liquefaction model

Effective stress approach is being used for simulating saturated condition. The total stress can be written as

$$\sigma_{ij} = \sigma'_{ij} + u\delta_{ij} \quad (1)$$

where σ_{ij} = total stress tensor; σ'_{ij} = effective stress tensor; u = pore water pressure; δ_{ij} = Kronecker delta. The pore pressure is allowed to develop in the material by considering undrained condition. The pore pressure model, implemented here, is a simplified version of Martin *et al.* (1975) and given by the following incremental equation (Byrne 1991).

$$\frac{\Delta\varepsilon_v}{\gamma} = C_1 \exp\left(-C_2\left(\frac{\varepsilon_v}{\gamma}\right)\right) \quad (2)$$

where γ and $\Delta\varepsilon_v$ are current shear strain amplitude and the increment in volumetric strain, respectively. The C_1 and C_2 are constants for given sand, at a particular density. The non dimensional constants C_1 and C_2, can be derived as follows

$$C_1 = 7600(D_r)^{-2.5} \quad (3)$$

$$C_2 = 0.4 / C_1 \quad (4)$$

where D_r is the relative density and is expressed in percentage. For undrained behaviour, the increase of pore water pressure is computed as follows (Martin *et al*., 1975)

$$\Delta u = E_r \, \Delta\varepsilon_v \quad (5)$$

in which Δu = increment in pore water pressure, E_r = rebound modulus of sand skeleton. The rebound modulus E_r at an effective stress level σ'_v is given by

$$E_r = \frac{(\sigma'_v)^{1-m}}{mk_2(\sigma'_{v0})^{n-m}} \quad (6)$$

where σ'_{v0} is the initial value of effective overburden pressure at that level; and k_2, m and n are experimental constants for a given sand. The increment in pore water pressure is distributed over the unloading portion of the load cycle and the current shear modulus (G) and bulk modulus (K) is modified progressively for changing effective stresses in each time interval as follows (Arduino *et al*., 2002)

$$G = G_{max}\left(\frac{\sigma'_v}{\sigma'_{vo}}\right)^{0.5} ; \qquad K = K_{max}\left(\frac{\sigma'_v}{\sigma'_{vo}}\right)^{0.5} \quad (7)$$

where G_{max} and K_{max} are initial shear and bulk modulus respectively. For the cases where due to rise in pore water pressure, effective stress becomes zero (complete liquefaction), to maintain stability in the calculations, shear modulus equal to 3% of initial shear modulus (G_{max}) is retained.

Material Properties Assumed

Effective parameters are assumed for all the cases. It is assumed that the soil stratum is fully saturated at all depths. The following material properties are used.

- Properties of pile: Young's modulus, E_p = 20 GPa; Poisson's ratio, ν_p = 0.30 and Density ρ_p = 2300 kg/m^3.
- Properties of soil: Young's modulus, E_s = 23.9 MPa; Poisson's ratio, ν_s = 0.30; Density ρ_s = 1922 kg/m^3; Cohesion, c' = 38.8 kPa and Internal friction angle, ϕ' = 30^0;
  Cap parameters: R = 4.0; W = 6 %; D = 1.262×10^{-6} Pa^{-1}.

The relative density of the soil medium is assumed as 20%. The coefficients required for computing elastic rebound modulus (Eq. 6) are assumed as m = 0.43; n = 0.62; $k_2 = 0.227 \times 10^{-3}$ Pa^{-1}(Martin *et al.*, 1975). Pile cap is assumed to be rigid massless and the material properties of the piles are assumed to be linear elastic. It is assumed that the portal frame is made of concrete and the properties are the same as that used for the pile material.

Solution Procedure

The global dynamic equilibrium equation is solved. Consistent mass matrix is used. The material damping is considered to be hysteretic in nature. Newmark's constant average acceleration method of integration (Bathe 1982) is being used for solution. For nonlinear analysis, stiffness and damping matrix changes with each time step and for solution, modified Newton-Raphson iteration technique is used. For convergence out of balance load, displacement and energy criteria each with tolerance value equal to 10^{-6} are simultaneously used.

Results and Discussion

Results are presented for following 3 conditions related to behaviour of soil medium:

(i) Elastic : Linear **Elastic**
(ii) DPWC : **Drucker-Prager** soil model **With Cap** (work hardening) and without pore pressure generation capability
(iii)DPWCP : **Drucker-Prager** soil model **With Cap** (work hardening) and with **Pore-pressure** generation capability

The effects of soil nonlinearity and the pore pressure generation of the soil medium on the Kinematic Interaction Factor (KIF) of the soil-pile system are discussed below.

Effects on Kinematic Interaction Factor

Translational and rotational KIFs are computed for the soil-pile system considered in the study without the superstructure for seismic base acceleration of 2.0 m/s^2. The investigation was conducted for wide range of frequencies. The KIFs for horizontal displacement and rotation plotted against the dimensionless frequency ($a_0 = \omega d/V_s$, where ω = frequency of excitation and V_s = shear wave velocity of soil) are shown in Fig. 3. From the results, it may be observed that the effect of work-hardening nonlinearity without pore pressure generation is not very significant on both the translational and rotational KIFs.

When the pore pressure generation capability causing liquefaction is incorporated along with the work-hardening soil nonlinearity, the responses increase significantly for almost all the frequencies. Both translational and rotational KIF increase significantly. So it is worth to state that though the rotation is not an important design criterion for piles in nonliquefiable soil medium, however, it may be an important factor for liquefying soil medium. It is also observed that the effect of liquefaction is more in lower frequency range. It can be observed from Fig. 3, at low frequencies (a0 < 0.2), there is hardly any change in displacements due to nonlinearity, however, the same is increased by about 50% due to liquefaction.

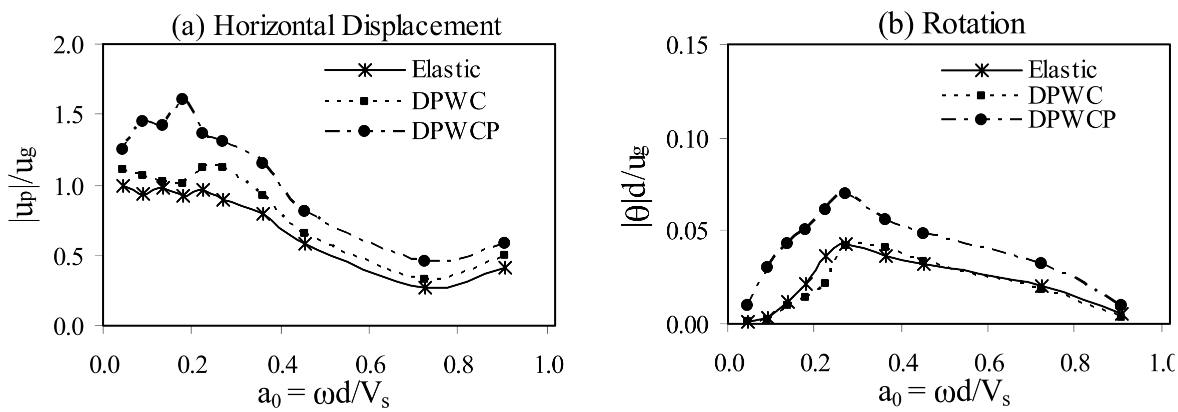


Figure 3. Nonlinearity (DPWC) and pore pressure (DPWCP) effects on seismic response: 2×2 pile group (s/d = 5)

Effects of Inertial Interaction

The effects of inertial interaction are investigated for linear properties. In the first case, horizontal harmonic base acceleration of different discrete frequencies is applied at the base of the soil layers without considering the superstructure. The motion at the points on the pile cap where the portal frame will be connected is noted. This motion is then applied as the input motion at the base of the portal frame and the response of the structure is computed. This procedure neglects the feedback of the structure, known as inertial interaction. In the second case, full system consisting of portal frame and the soil-pile system is considered. For this case also, the base acceleration at the bed rock is applied for different discrete frequencies and the response of the structure is computed. Thus both the inertial and kinematic interaction is considered.

The effect of inertial interaction on response of the structure is shown Fig. 4 in terms of amplification of response with respect to the input bedrock motion. It can be observed that due to inertial interaction, the fundamental period of the structure increases and the structure peak response decreases significantly. So it may be concluded that the soil-pile-superstructure interaction increases the period of the structure and tends to decrease the peak response. Veletsos (1991) also had similar observations. Next, the effects of soil nonlinearity and pore pressure generation in the soil on the response of the structure are investigated and discussed in the following section.

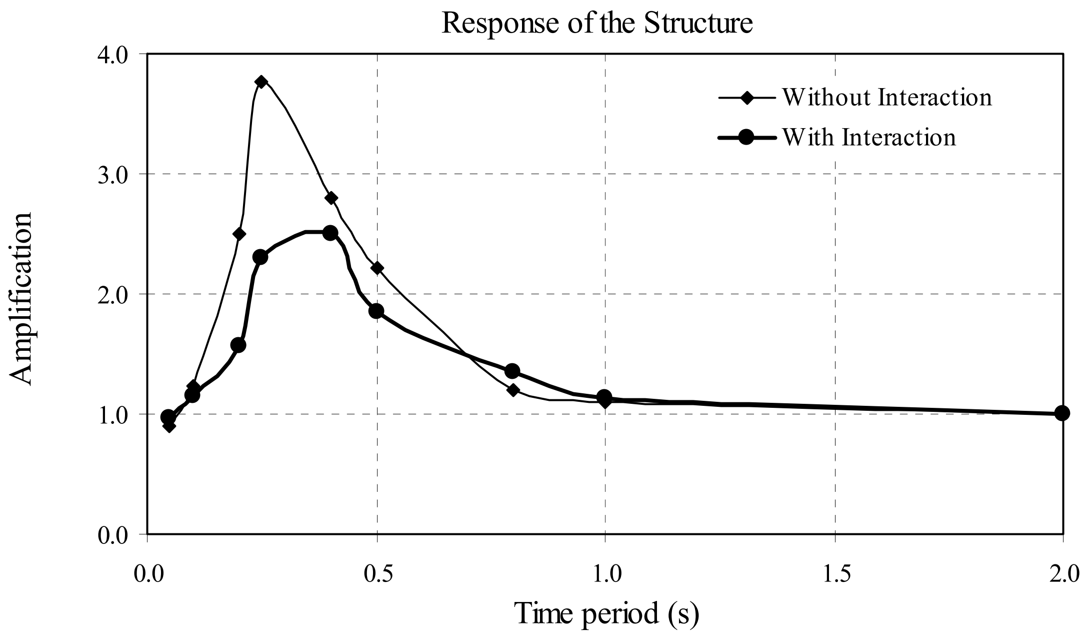


Figure 4. Effects of inertial interaction on the response of the structure for elastic soil

Effects on Response of Superstructure

The analyses are performed for harmonic excitations of discrete frequencies and real earthquake excitations for the three soil conditions mentioned above.

Analysis for Harmonic Excitations

Horizontal harmonic base accelerations of amplitude 2.0 m/s^2 (0.2g) at different discrete frequencies are applied at the bed rock and the corresponding steady state response of the structure is computed. The amplification with respect to the bedrock motion is presented in Fig. 5. It can be observed that that the work-hardening plastic soil medium without considering the pore pressure generation (DPWC) is increasing the response of the structure especially in the lower frequencies (near the fundamental frequency of the structure). It also demonstrates the importance of considering soil nonlinearity during the design and analysis of any structure subjected to dynamic condition. The analyses have also been performed for soil medium capable of simulating pore pressure (DPWCP) and presented in Fig. 5. It shows that when the pore pressure generation is considered, the response of the structure is increased by about 40%. This much increased in response is quite significant for design of structures. The maximum response also falls in range of the fundamental frequency of the structure.

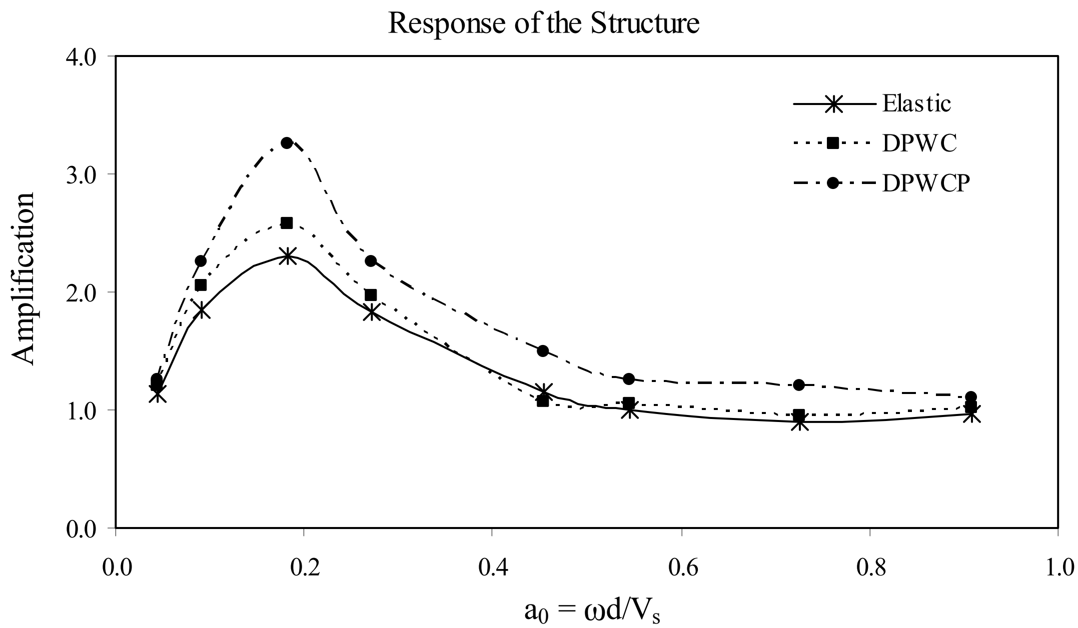


Figure 5. Effects of nonlinearity and pore pressure generation on the response of structure

Analysis for Real Earthquake Excitation

Real time earthquake motion is applied for soil-pile-superstructure system. Kobe earthquake (1995) time history with PGA value 0.141g and predominant frequency 1.5 Hz (i.e. $a_0 = 0.07$) is used for the investigation.

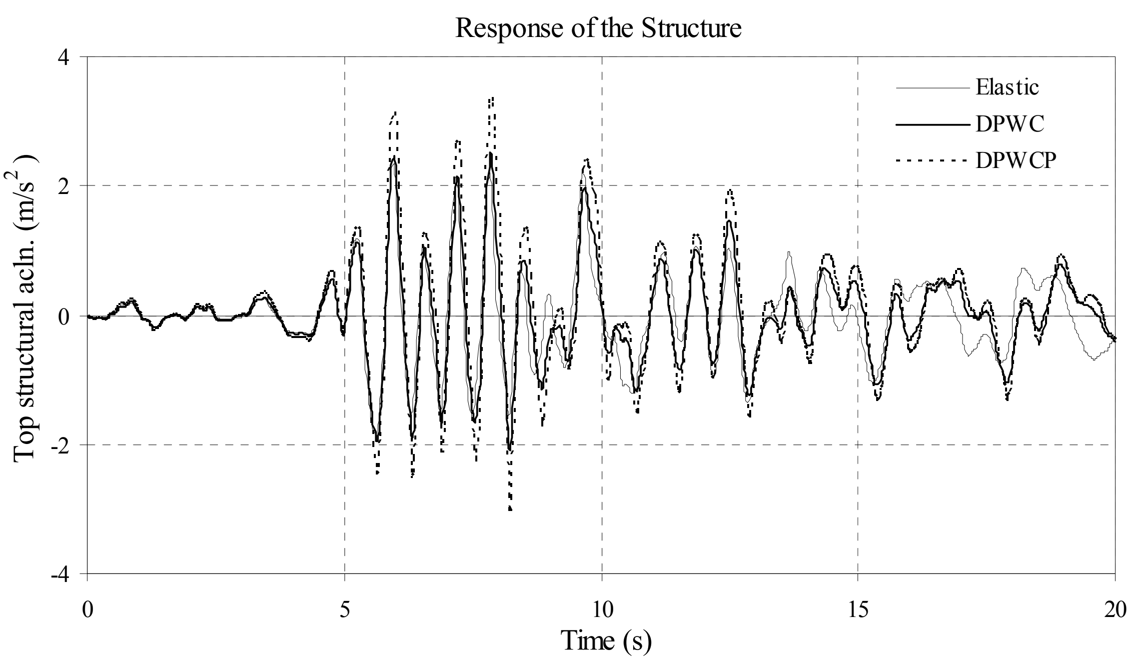


Figure 6. Effect of nonlinearity (DPWC) and pore water pressure on response of the structure for Kobe Earthquake input motion

The structural response was computed and accelerations at the top of the portal frame have been compared for all the three cases. The comparison is shown in Fig. 6. It may also be noted that the peak value of structural acceleration considering linear elastic soil medium is 2.34 m/s^2. But when Drucker-Prager soil nonlinearity with cap is considered, this value is increased to 2.52 m/s^2. Further with inclusion of pore pressure generation capability of the soil medium, the maximum response increases to 3.36 m/s^2. Thus, it may be noticed that the transient results follow the same trend as those shown in Fig. 5 in both the qualitative and quantitative ways.

Thus it can be concluded that the effect of material nonlinearity is significant on the response of structure, where the response is increased by a margin of 10-15%. However the effect of pore water pressure is very significant and the response is increased by a margin of about 40% for the given conditions.

Conclusions

The effects of soil nonlinearity and the pore pressure generation in the soil medium on the response of soil-pile system and soil-pile-structure system is examined in this paper for particular pile spacing. Radiation boundary condition is approximately simulated by using Kelvin elements at the lateral boundary. From the study, following major conclusions may be drawn.

- The nonlinearity of the soil medium affects the seismic response of pile groups slightly at all frequencies for nonlinear soil model.
- The effect of pore water pressure generation, increased seismic response (both translational and rotational) significantly. So for design and analysis of piles in liquefying soil under seismic loading, both the translation and rotation have to be taken into account.
- The soil-pile-superstructure interaction increases the period of the structure and tends to decrease the peak response.
- The nonlinearity of the soil medium increases the response of the structure especially in the low frequency range. The pore pressure generation leading to liquefaction of the soil medium induces significant increase in structural response.

Acknowledgement

The research presented here is supported by Department of Science and Technology, Govt. of India under Project No. SR/S3/MERC/31/2005. The support is gratefully acknowledged. Authors would like to thank Prof. & Head, Dept. of Earthquake Engineering for extending every possible help for smooth conduct of the research.

References

Arduino, P., Kramer, S.L., Li, P. and D. A. Baska, 2002. Dynamic stiffness of piles in liquefiable soils, *Research Report*, Dept. of Civil and Environmental Engineering, University of Washington, Washington.

Bathe, K.J., 1982. *Finite element procedures in engineering analysis,* Prentice-Hall, Inc., Englewood Cliffs, New Jersey.

Byrne, P.M., 1991. A cyclic shear-volume coupling and pore pressure model for sand. *In Proc. of the 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, St Louis, Report 1.24, 47-56.

Chen, W.F. and G.Y. Baladi, 1985. *Soil plasticity: theory and implementation,* Elsevier, Amsterdam.

Maheshwari, B. K., Truman, K. Z., El Naggar, M. H. and Gould, P. L. (2004). Three-dimensional nonlinear analysis for seismic soil-pile-structure interaction, *Soil Dynamics and Earthquake Engineering*, 24, 343-356.

Martin, G.R., Finn, W.D.L. and H.B. Seed, 1975. Fundamentals of liquefaction under cyclic loading, *Journal of Geotechnical Engineering Division, ASCE* 101(GT5), 423-438.

MATLAB, *Reference manual*, Version 7.1.0.246 (R14), Service Pack 3, The MathWorks Inc., USA.

Novak, M., Nogami, T. and F. Aboul-Ella, 1978. Dynamic soil reaction for plane strain case, *Journal of Engineering Mechanics, ASCE* 104(4), 953-956.

Novak, M. and H. Mitwally, 1988. Transmitting boundary for axisymmetrical dilation problems, *Journal of Engineering Mechanics, ASCE* 114 (1), 181-187.

Sandler, I. and D. Rubin, 1979. An algorithm and a modular subroutine for the cap model, *International Journal for Numerical and Analytical Methods in Geomechanics* 3, 173-186.

Shakib, H. and A. Fuladgar, 2004. Dynamic Soil-Structure Interaction Effects on the Seismic Response of Asymmetric Buildings, *Soil Dynamics and Earthquake Engineering* 24, 379-388.

Uzuoka, R., Sento, N., Kazama, M., Zhang, F., Yashima, A. and F. Oka, 2007. Three-dimensional numerical simulation of earthquake damage to group-piles in a liquefied ground, *Soil Dynamics and Earthquake Engineering* 27, 395–413.

Veletsos, A. S., 1991. Effects of Soil-Structure Interaction for Structures Subjected to Earthquakes, *In Proc. of 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, St. Louis, Missouri, 2419-2421.

Wilson, D.W. 1998. *Soil pile superstructure interaction in liquefying sand and soft clay,* Ph.D. Thesis, University of California, Davis, USA.