Determination of Dynamic p-y Curves for Pile Foundations Under Seismic Loading

15 WCEE
LISBOA 2012

A. Rahmani , M. Taiebat, W.D. L. Finn, and C. E. Ventura
Department of Civil Engineering, University of British Columbia, Vancouver, BC, Canada.

SUMMARY:
Beam on a Nonlinear Winkler Foundation (BNWF) approach is widely used in practice since the simulation process is rather simple and requires less computational efforts compared to three-dimensional continuum modeling approaches. In the recent decades, this approach has been extended by proposing advanced one-dimensional models in order to better simulate the soil-pile interaction. In this paper, the performance of two different soil-pile interaction elements under earthquake loading is investigated. A three-dimensional continuum model is employed to determine dynamic p-y curves. Firstly, the continuum model is validated by comparing the computed data with the data of two centrifuge tests. Then, the p-y curves obtained from BNWF approach are compared and assessed with the p-y curves obtained from continuum model. It is concluded that the initial slope, shape, and ultimate value of the dynamic p-y curves are not represented properly suggesting the necessity for further assessment of the state of the practice for simulating seismic soil-pile interaction.

Keywords: soil-pile interaction, dynamic p-y curve, continuum model, BNWF approach.

1. INTRODUCTION

It is well-established that the simulation of soil-pile-structure interaction plays a key role in accurate design of both the structure and the piles. There are two common approaches for the seismic analysis of soil-pile-structure systems:

1. *Beam on a nonlinear Winkler foundation (BNWF):* The BNWF model has three major disadvantages: (a) the subgrade reaction is governed only by the pile displacement at the depth where the soil reaction is considered, and the shear transfer between layers of soil is ignored; (b) the three dimensional displacement pattern of the soil as the pile pushes into the soil is ignored and (c) the spring stiffness values are usually foundation size dependent and difficult to determine. Despite of the mentioned drawbacks, the approach is being widely used in practice. The main reason for its popularity is that the approach is very simple and requires less computational efforts. Dynamic BNWF approach has been extended, and its capability in prediction of the soil-pile-superstructure response has been discussed by several researchers such as Badoni and Makris (1996), El Naggar and Novak (1996), Wang et al. (1998), Boulanger et al. (1999), Gerolymos and Gazetas (2005), Taciroglu et al. (2006), and Allotey and El Naggar (2008).

2. *Continuum modeling:* This approach potentially provides the most powerful means for conducting soil-pile-structure analysis, but it has not yet been fully realized as a practical tool. The advantages of a finite element approach include its capability for performing soil-pile-superstructure analysis of pile groups in a fully-coupled manner, without resorting to independent calculations of site or superstructure response, or application of pile group interaction factors. Capability of this approach in accurate simulation of soil-pile-superstructure systems has been demonstrated by many researchers such as Finn and Fujita (2002), Cheng and Jeremic (2009), Rahmani and Pak (2012).

In this paper, the simple BNWF approach commonly used in practice, the BNWF model extended by Boulanger et al. (1999) and the continuum modeling approach are assessed and validated by comparing the computed data with the measured data of two different dynamic centrifuge tests. The focus of attention is on dynamic p-y responses at soil-pile interface. For this purpose, p-y relationships are compared with the ones developed by continuum model in order to characterize the properties of dynamic p-y curves.

Data from two different centrifuge tests are used to assess the capabilities of the BNWF approach and the continuum modeling approach to predict soil-pile-superstructure seismic performance. The centrifuge tests conducted by Wilson et al. (1998) at the University of California at Davis and by Gohl (1991) at the California Institute of Technology have been selected to be simulated in this research. In Table 1, a brief description of the centrifuge tests is presented.

Table 1. Case description for the centrifuge tests

Case	Test	Soil profile	Level of shaking
I	Wilson et al. (1998)	Two-layered Nevada sand (saturated)	Low-level (PGA=0.04g)
II	Gohl (1991)	Loose Nevada sand (dry)	Moderate-level (PGA=0.15g)

Centrifuge tests by Wilson et al. (1998): The soil profile consists of two horizontal layers of saturated, fine and uniformly graded Nevada sand. The lower dense layer (initial void ratio of 0.595) is 11.4 m thick, and the upper medium dense layer (initial void ratio of 0.684) is 9.1 m thick at the prototype scale. Shear wave velocity is calculated to be 117.5 and 141 m/s for loose and dense layer, respectively. The single pile is equivalent to a steel pipe pile with a diameter of D=0.67 m and wall thickness of 19 mm at the prototype scale. The pile is extended 3.8 m above the ground level and carries a superstructure load of 480 kN. The embedded length of pile is about 16.8 m. The soil-pile-superstructure system was spun at a centrifugal acceleration of 30g.

Centrifuge tests by Gohl (1991): The soil profile consists of one layer of dry fine-grained Nevada Sand with the thickness of 12 m. The test was conducted for two different soil densities; initial void ratio of 0.77 (loose Case) and 0.57 (dense Case) with shear wave velocity of 160 m/s and 220 m/s, respectively. The single aluminium pile in the model scale is equivalent to a steel pipe pile with diameter of D=0.57 m and wall thickness of 13 mm at the prototype scale. The pile is extended about 2 m above the ground level and carries a superstructure load of 522 kN. The embedded length of the pile is about 12 m. The soil-pile-superstructure system was spun at a centrifugal acceleration of 60g.

2. NUMERICAL MODELING OF SOIL-PILE-SUPERSTRUCTURE SYSTEM

Figure 1 depicts the employed dynamic BNWF approach in this paper. The analysis is performed in two separate stages: (1) site response analysis is carried out to determine the time history of the displacement at each elevation, (2) a seismic analysis of the pile is carried out by applying computed ground displacements dynamically to the pile. Fig. 1 presents the schematic view of the procedure.

Site response analysis, the first step in BNWF approach, are done using the OpenSees finite element program (Mckenna et al. 2000). The behavior of the sand in the free field is modeled using a critical state two-surface plasticity model from the SANISAND class of models (Dafalias and Manzari, 2004) that is implemented, verified and validated for application in seismic wave problems in OpenSees by Jeremic et al. (2008) and Taiebat et al. (2010). Such a nonlinear model is selected because Boulanger et al. (1999) and Allotey and El Naggar (2008) conducted a wide range of sensitivity analysis on the BNWF approach and concluded that the site response calculations were a major source of uncertainty in predicting system response.

Soil-pile interaction is modeled through two different constitutive models. Model No. 1 is the modified version of the one proposed by Wang et al. (1998). This model is composed of a non-linear spring in series with Kelvin-Voigt element to provide radiation damping for simulating wave propagation to the far field. The components of the model have been explained in details in the paper by Rahmani et al. (2012). Model No. 2 was proposed by Boulanger et al. (1999). This model is composed of model No. 1 in series with a gap element which consists of a non-linear closure spring in parallel with a non-linear drag spring (details about this model can be found in Boulanger et al, 1999). For both models, API (2007) p-y curves are used as backbone curves for the nonlinear components.

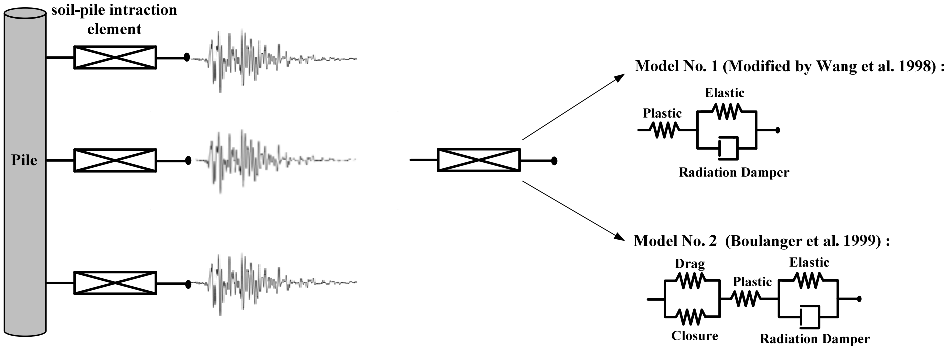


Figure 1. Dynamic BNWF approach using two one-dimensional models.

The performance of these two models under a sinusoidal loading is shown in Fig. 2; it is observed that for a low level shaking the performance of the models are very similar since the gap component of second model is not activated; however, for a higher level of shaking the inclusion of the gap components result in an inverted S-shaped p-y curve.

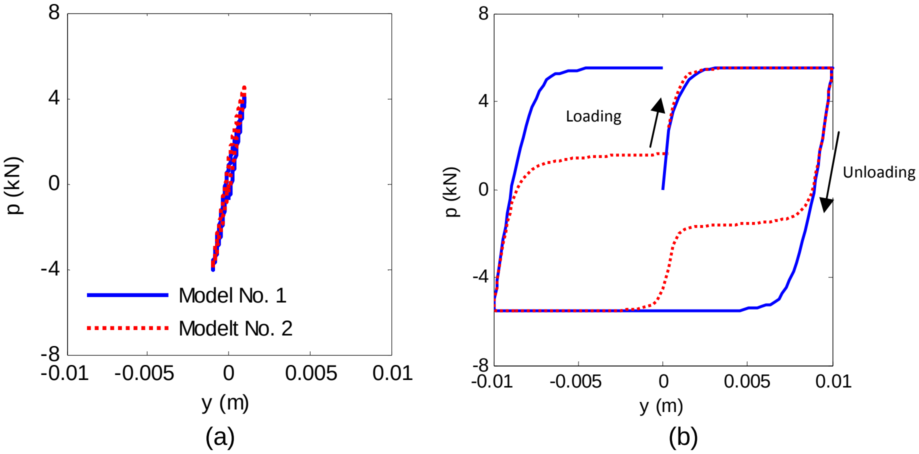


Figure 2. The p-y response of model No. 1 and No. 2 under (a) low amplitude, and (b) high amplitude sinusoidal loading.

The finite element mesh used in the continuum modeling approach is shown in Fig. 3. Soil elements and constitutive model are the same as the ones used in free-field analysis. Soil elements far from the pile are coarser, while the elements in the radial distance of eight times pile diameter (8D) from the pile are finer. Boundary conditions are set in the following way: (i) base of the mesh is fully fixed in all directions; (ii) the nodes at equal depths on the lateral boundaries are constrained to have equal displacements in the direction of excitation to simulate laminar box boundary conditions; (iii) the nodes on the plane which halves the model constrained in y direction and (iv) all other internal nodes are free to move in any direction.

Pile is modeled by beam-column elements which have six degrees of freedom for each node: three for displacements and three for rotations. Each node is connected to the surrounding soil at the same elevation through rigid elements as shown in Fig. 3 in order to simulate the pile region in the finite element mesh. In addition to this, computed forces in these elements are used to obtain p-y curves in which p is the summation of forces in the direction of loading, and y is the pile deflection relative to the soil displacement at the same elevation in free field. Due to the symmetry of the model, the model is halved at the line of symmetry along the centre-line of the pile, and stiffness, cross-sectional area of the pile and the applied static loads are halved. The lumped mass on the pile head represents the superstructure.

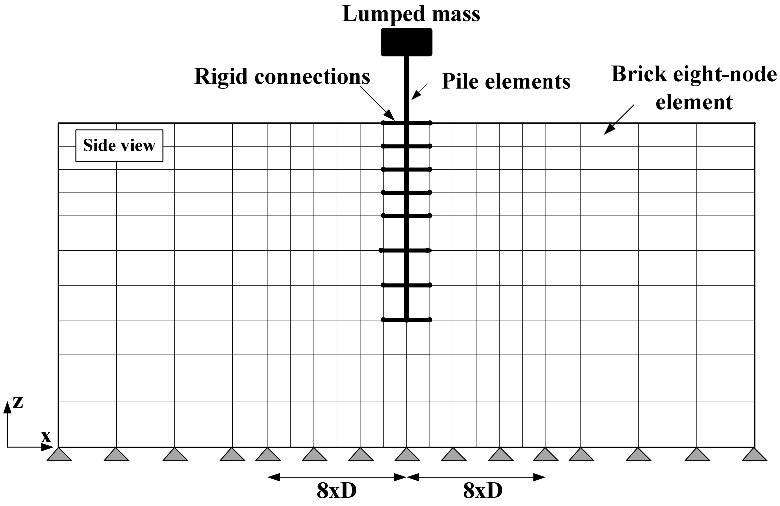


Figure 3. Schematic view of the finite element mesh (D is pile diameter).

3. ASSESSMENT OF THREE APPROACHES FOR SIMULATING SOIL-PILE INTERACTION

The computed acceleration response spectrum at the ground surface is shown in Fig. 4. The frequency content and the amplitudes of the motion at the ground surface agree reasonably well with the measured ones. The concluded accuracy can be attributable to the employed non-linear time domain analysis in which an advanced constitutive model has been used to simulate the soil deposit.

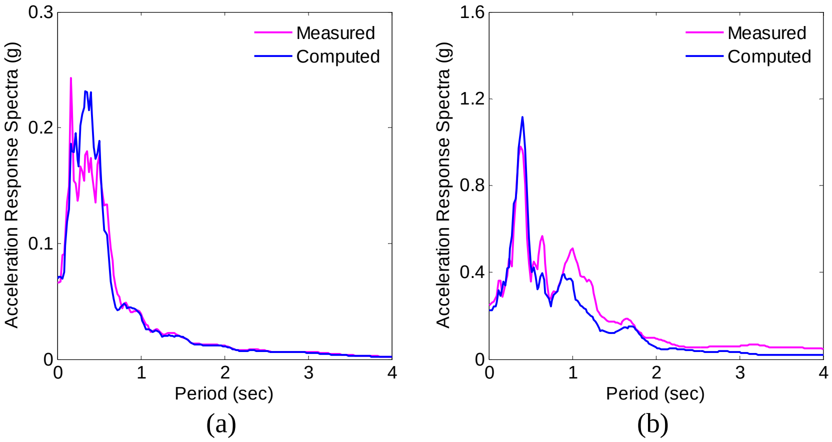
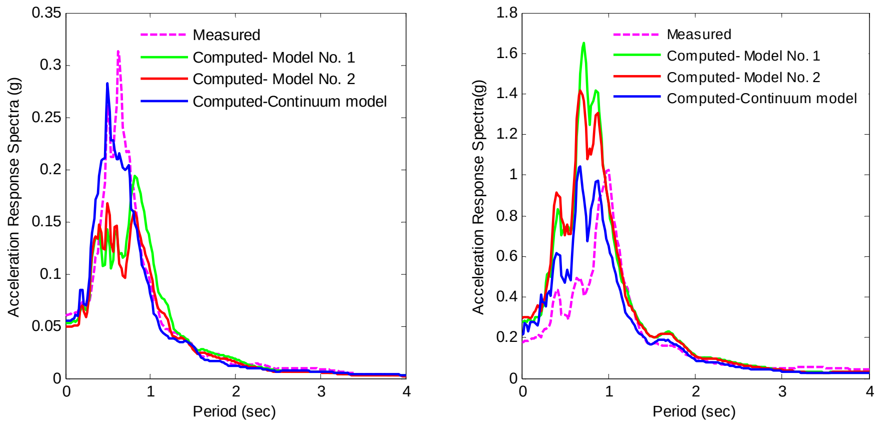


Figure 4. Measured and computed acceleration response spectra at the ground surface for (a) Case I, and (b) Case II.

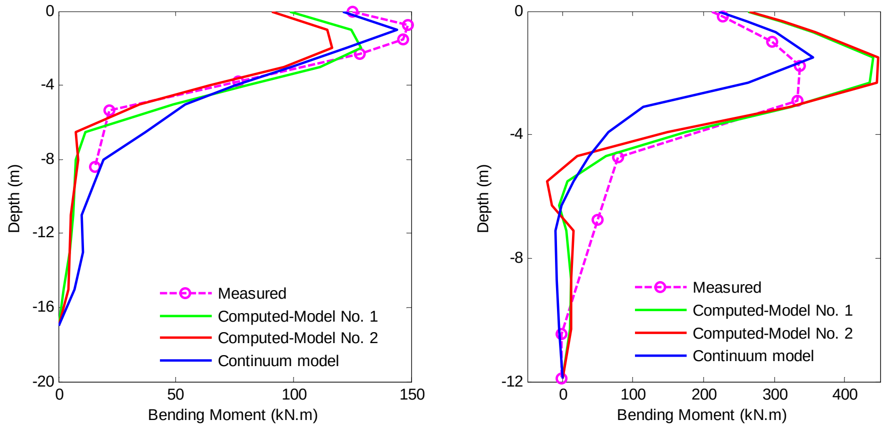
Figure 5 presents the acceleration response spectra (ARS-5% damping) of the superstructure using models No. 1 and No. 2. The results of the continuum model are presented also in Fig. 5. It is observed that for both Cases I and II the superstructure performance is poorly predicted by employing either model No. 1 or model No. 2. The prediction of frequency content and amplitudes of computed acceleration are not satisfactory even if a gap component is added to the soil-pile interaction model. It is to be noted that inclusion of gap component in the second model did not even change the performance of the system because of the low level of plastic behavior in the system. However, as shown in Fig. 5 inclusion of gap element in Case II led to a slight decrease in the amplitudes of acceleration due to reduction in hysteretic damping predicted by model No. 2. Furthermore, as expected, it is concluded that the continuum model predicts the superstructure response more accurately compared to the other approaches.

The bending moment envelope for Case I and bending moment distribution at peak pile head displacement for Case II are shown in Fig. 6. For Case I, pile performance is roughly similar using models No.1 and No. 2 because the gap element is not mobilised at low levels of excitation. Even in Case II, the inclusion of the gap element does not affect the bending moment values along the pile shaft. It should be noted that the level of error in estimation of maximum bending moment is relatively less when the continuum model is used (with error less than 10%).



(a) (b)

Figure 5. Measured and computed (using three different approaches) acceleration response spectra of superstructure for (a) Case I, and (b) Case II.



(a) (b)

Figure 6. Measured and computed (using three different approaches) (a) maximum bending moments distribution in Case I, and (b) bending moments at peak pile head displacement in Case II (Depth zero corresponds to the ground surface).

4. DETERMINATION OF DYNAMIC P-Y CURVES

In this section, dynamic p-y curves are determined. Using all 3 computational models, the p-y curves for model No. 1 and No. 2 are obtained through calculating the forces and relative deflections of the interaction elements while for the continuum model p is the summation of forces, exerted on the rigid connection elements, in the direction of loading, and y is the pile deflection relative to the soil displacement at the same elevation in the free field.

Dynamic p-y curves under low level of shaking (PGA=0.04g) from all 3 computational models are shown in Fig. 8. The shape of p-y curves using model No.1 and No. 2 is similar to an ellipse suggesting that the Kelvin-Voigt element is the dominant component of the model due to the low level of shaking and consequently the elastic response of the system.

The API (2007) recommendations are employed to define the backbone curves for model No. 1 and model No. 2. The recommended initial subgrade reaction at the depth of 2D in the case under study is 10900 kN/m. However, the initial slope of the dynamic p-y curve determined by continuum model is approximately 26000 kN/m (Fig. 8). Similar conclusion has been made by Finn (2005), who could not simulate the pile performance in a centrifuge test by the initial subgrade reaction modulus recommended by API. He concluded that the modulus of subgrade reaction at any depth which is proportional to the soil modulus at the same depth improves prediction of response considerably.

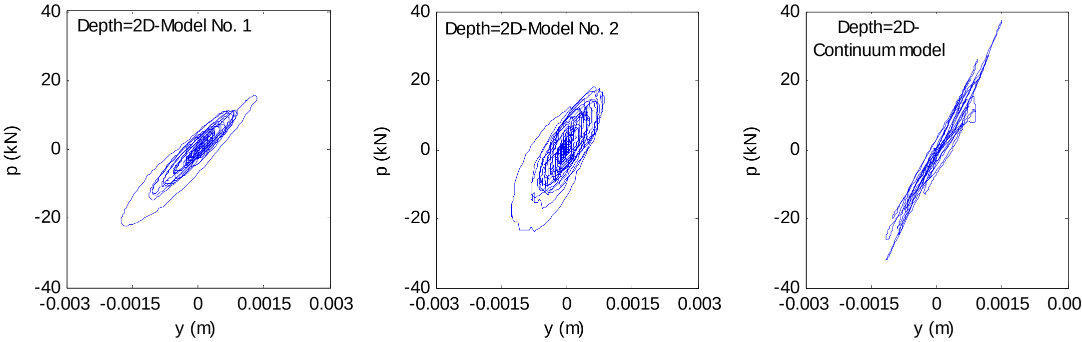


Figure 8. Dynamic p-y curves for Case I under low level shaking (PGA=0.04g).

Figure 9 shows the dynamic p-y curves developed in Case II under moderate level of shaking (PGA=0.15g). Due to higher level of the applied forces, the plastic component of the models used in BNWF approach dominates the system response. It is observed that even by including the gap component in model No. 2 the p-y curve predicted by continuum model has not been satisfactorily estimated. It would appear that the API p-y curve employed as the backbone curve for soil-pile interaction models is not accurate enough to capture the correct dynamic p-y loop. The p-y curve obtained from continuum model demonstrates that the ultimate soil-resistance is larger than the value recommended by API (for the depth of 1D: p_{ult} = 25 kN). The ultimate soil resistance is not even mobilized for the lateral displacement of 0.013 m; while, API suggests that soil reaches its ultimate strength at the pile deflection (y) of 0.006 m. Similar scenarios has been observed in the experimental study of Yan and Byrne (1992) who suggested an ever-increasing power-function, with no defined ultimate pressure, to relate pile deflection (y) to soil resistance (p).

As shown in Fig. 9, the slope of p-y curves obtained from the continuum model decreases because of the stiffness degradation of the soil during earthquake loading. This observation demonstrates the necessity of employing a soil-pile interaction model which is capable of taking soil stiffness degradation into account. It is worth referring to the study of Allotey and El Naggar (2008). They developed an advanced one-dimensional model which takes into account both stiffness degradation and strength degradation of the soil.

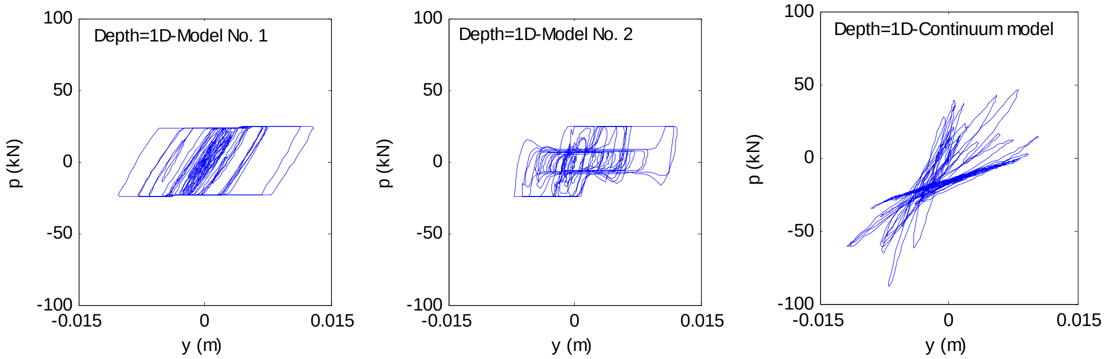


Figure 9. Dynamic p-y curves for Case II under moderate level shaking (PGA=0.15g).

Lastly, as shown in the Figs. 8 and 9, the p-y loops obtained from the BNWF approach is significantly different from the ones obtained from the continuum model. Nevertheless, it was concluded that maximum bending moment along the pile shaft is acceptably predicted by the BNWF approach (see Fig. 6). A research is under progress to investigate in details the reliability of the BNWF approach.

5. CONCLUSIONS

The objective of this paper is to provide better insight into the dynamic lateral response of pile foundations under earthquake loading. Measured data of two different centrifuge tests was used to investigate the efficiency of the BNWF approach and the continuum modeling approach. In the BNWF approach two different one-dimensional models were used to simulate soil-pile interaction. The only difference between the two models was that the second model was capable of modeling gap at the soil-pile interface. The capabilities of these two models together with the capabilities of the continuum model in prediction of the soil-pile-superstructure response were discussed in details. Furthermore, dynamic p-y curves were determined in both of the BNWF models and compared with the dynamic p-y curves determined by the continuum model.

Results of the BNWF model demonstrated that the model poorly predicts the acceleration response spectra of the superstructure, while acceptably predicts the bending moment distribution along the pile shafts. Inclusion of the gap component slightly improved the prediction of the acceleration response spectra of superstructure but the level of error in capturing the measured values remained significant. The continuum model was much more successful in predicting the acceleration response spectra of the superstructure and the bending moment distribution along the pile shafts.

Finally, dynamic p-y curves obtained from the BNWF approach and the continuum modeling approach were compared. The calculated p-y curves from models No. 1 and No. 2 appeared to be considerably different from the p-y curves obtained from the continuum model. The initial slope and the ultimate resistance were poorly predicted by BNWF approach, but interestingly, in terms of the overall response of the pile foundation the maximum bending moment along the pile shaft was acceptably predicted. This suggests the need to further investigate the reliability of both the recommended backbone curves and the soil-pile interaction models.

AKCNOWLEDGEMENT

Support to conduct this study is provided by grant from Natural Sciences and Engineering Research Council of Canada (NSERC).

REFERENCES

Allotey, N. and El Naggar, M.H. (2008). Generalized dynamic Winkler model for nonlinear soil-structure interaction analysis. *Canadian Geotechnical Journal*, **45**:560–73.

API (2007). Recommended practice for planning, designing, and constructing fixed offshore platforms. American Petroleum Institute.

Badoni, D. andMakris, N. (1996). Nonlinear response of single piles under lateral inertial and seismic loads. Soil Dynamics and Earthquake Engineering, **15**:29–43.

Boulanger, R. W., Curras, C. J., Kutter, B. L., Wilson, D. W. and Abghari, A. (1999). Seismic soil-pile-structure interaction experiments and analyses. *Journal of Geotechnical and Geoenvironmental Engineering*, **125(9)**, 750–759.

Carter, D.P. (1984), A nonlinear soil model for predicting lateral pile response, M.Sc. thesis, University of Aukland, Aukland, New Zealand.

Cheng, Zh.and Jeremic, B.(2009). Numerical modeling and simulation of pile in liquefiable soil, *Soil Dynamics and Earthquake Engineering*,**29**:1404–16.

Dafalias, Y. F. and Manzari, M. T. (2004). Simple plasticity sand model accounting for fabric change effects. *Journal of Engineering Mechanics*, **130(6)**, 622–634.

El Naggar MH. and Novak M.(1996). Nonlinear analysis for dynamic lateral pile response. *Soil Dynamics and Earthquake* Engineering, **15**:233–44.

Finn, W.D.L. and Fujita, N. (2002). Pile in liquefiable soils: seismic analysis and design issues. *Soil Dynamics and Earthquake Engineering*,**22**:731–42.

Finn, W.D.L. (2005), A study of piles during earthquakes: issue of design and analysis. *Bulletin of earthquake engineering*, **3**:141-234.

Gerolymos, N. and Gazetas, G. (2005). Phenomenological model applied to inelastic response of soil-pile interaction systems. *Soils and Foundations*, **45**:119-132.

Gohl, B. (1991). Response of pile foundations to simulated earthquake loading: experimental and analytical results. Ph.D. Dissertation, University of British Columbia.

Jeremic, B., Cheng, Z., Taiebat, M. and Dafalias, Y. F. (2008). Numerical simulation of fully saturated porous materials. *International Journal for Numerical and Analytical Methods in Geomechanics*, **32(13)**, 1635–1660.

McClelland, B. and J.A. Focht Jr.(1958). Soil Modulus for laterally loaded piles. *Transactions, American Society of Civil Engineers*, **123**:1049-1063.

McKenna, F., Fenves, G. and Scott, M. (2000). Open system for earthquake engineering simulation. University of California, Berkeley, CA. (http://opensees.berkeley.edu).

Rahmani A. and Pak A., (2012). Dynamic behavior of pile foundations under cyclic loading in liquefiable soils, *Journal of Computers and Geotechnics*, **40**, 114-126.

Rahmani, A., Taiebat, M., Finn, W. D. L. and Ventura, C. E. (2012). Evaluation of p-y curves used in practice for seismic analysis of soil-pile interaction. State of the Art and Practice in Geotechnical Engineering (GSP 225), Proceedings of GeoCongress 2012, R. D. Hryciw, A. Athanasopoulos-Zekkos, and N. Yesiller, eds., ASCE, 1780-1788.

Taciroglu, E., Rha, C. and Wallace, J. (2006). A robust macroelement model for soil–pile interaction under cyclic loading. *Journal of Geotechnical and Geoenvironmental Engineering*, **132(10),** 1304–1314.

Taiebat, M., Jeremic, B.,Dafalias, Y. F., Kaynia, A. M. and Cheng, Z. (2010). Propagation of seismic waves through liquefied soils. *Soil Dynamics and Earthquake Engineering*, **30(4),**236–257.

Wang, S., Kutter, B. L., Chacko, J. M., Wilson, D. W., Boulanger, R. W., and Abghari, A. (1998). Nonlinear seismic soil-pile-structure interaction. *Earthquake Spectra*, **14(2),** 377–396.

Wilson, D.W. (1998), Soil-pile-superstructure interaction in liquefying sand and soft clay. Ph.D. Dissertation, University of California at Davis.

Yan, L. and Byrne, P.M. (1992). Lateral pile response to monotonic loads. *Canadian Geotechnical Journal*, **29(2),**955–970.