Design Guide Lines in the form of Peak Response of Piles in Saturated Sand

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ABSTRACT
A Beam on Nonlinear Winkler Foundation (BNWF) consisting of simple nonlinear springs, dashpots and pipe elements is used to account for nonlinearity of soil-pile relative movement, energy dissipation through radiation damping of soil and material damping of pile. The p-y curves proposed by American Petroleum Institute (2000) are used. The results of the BNWF model are compared with centrifuge test records. Pile peak response curves are obtained by comparing maximum computed and experimental pile peak responses (peak accelerations and bending moments of the pile head and peak structure accelerations) for a range of peak ground accelerations and pile material damping parameters, so the designer can choose peak pile response quantities within the resulting range based on conservativeness of the design.

RÉSUMÉ
Un faisceau est supporté sur une Nonlinear Winkler Foundation (BNWF) qui est composée de ressorts simples non-linéaires, dashpots éléments et des tuyaux qui sont utilisés pour tenir compte de la non linéarité du movement relatif des soils pile , de l'énergie dissipée par le rayonnement du sol, et de l'amortissement du matériel et de la pile. Les courbes p-y proposées par l'American Petroleum Institute (2000) sont utilisées. La performance du modèle BNWF a été comparée aux résultats d'essais centrifugeuses. Le sommet des courbes de réponse des piles sont calculés en comparant la pointe maximale de la pile expérimentale et des réponses (accélérations de la pointe de la pile ainsi que d'accélération et les points de flexions de la pointe de la superstructure) pour une série d'accélérations de la crête à la base et le niveau de paramètres d'amortissement du matériel composant la pile, de sorte que le concepteur/conceptrice peut choisir une réponse des quantités pour la crête de la pile basée sur la conception de la prudence.

1 INTRODUCTION

Earthquake design of pile supported infrastructure in seismically active areas is one of the most important parts of infrastructure design. Seismic soil pile interaction analysis is the main step in the evaluation of seismic behaviour of pile supported structures. Finite Element Methods provide powerful tools for conducting seismic nonlinear soil-pile interaction analyses. Continuum Finite Element Methods treat the soil as a continuum medium. The main advantage of this approach is the capability of performing the soil-pile interaction analysis in a coupled manner (El Naggar & Bentley 2000), without resorting to separate site response analysis. However, this method is not commonly used in design offices mainly due to their excessive computational time and complicated formulation. The dynamic beam on Nonlinear Winkler Foundation (BNWF) method is a simplified approach that can account for nonlinearity in soil-pile interaction analysis. It is commonly used in professional engineering practices as it demands less computation time. El-Naggar et al. (2005) used the BNWF model for piles subjected to lateral seismic excitation. The p-y curves approach is a widely accepted method for predicting pile response under static loads because of its simplicity and practical accuracy. In the BNWF models, soil stiffness is established using p-y curves while damping is established from analytical or empirical solutions to simulate energy dissipation through wave propagation in the soil (El-Naggar et al. 2005). El Naggar et al. (2005) implemented nonlinear springs based on p-y curves and dashpots in parallel to represent radiation damping in BNWF model. The selected soil conditions were such that neither saturated clay nor saturated sand liquefied during seismic excitation. Boulanger et al. (1999) developed a BNWF model utilizing springs in series with dashpots representing radiation damping and used the model to analyze dynamic response of single-pile-supported structures in saturated clay overlying dense sand (relative density, 80%). The results computed by them were in good agreement with experimental centrifuge tests carried out by Wilson et al. (1997a, b).

Talukder et al. (2009b) implemented simplified dynamic BNWF model for piles in saturated sand using the general finite element code ABAQUS (version 6.7).This paper incorporates the numerical BNWF model developed by Talukder et al. (2009b) to predict peak pile responses such as maximum moments in piles, peak displacements, peak superstructure and pile head accelerations observed during dynamic centrifuge tests.

2 DESCRIPTION OF CENTRIFUGE MODEL LAYOUTS

The soil profile in centrifuge models (Wilson et al. 1997a, b) consisted of two horizontal layers of saturated fine graded Nevada sand ($C_u = 1.5, D_{50} = 0.15mm$). The relative density, D_r of the lower layer was about 80%. The upper layer had a relative density of 35% in the Csp2 model (Wilson et al. 1997a) and 55% in the Csp3 model (Wilson et al. 1997b). All tests were performed at a centrifugal acceleration of 30g. The single-pile-supported structure was the most heavily instrumented of the structural models used in Csp2 and Csp3 tests and is the only structure analyzed in this study. The pile is equivalent to a prototype steel pipe pile with outer diameter of 0.67m and a wall thickness of 19mm supporting a superstructure load 49100 kg at about 3.81 m above soil surface. Each model configuration was subjected to a series of shaking events. The earthquake motions (Table 1) used in this study were based on the acceleration time histories recorded at Port Island during the Kobe earthquake and at the University of California at Santa Cruz during the Loma Prieta earthquake.

Table 1. Earthquake Events analyzed in finite element program ABAQUS.

Experiment	Shaking Event	Input	Peak Base Acceleration (g)
Csp2	D	Kobe	0.04
	F	Kobe	0.22
	J	Santa Cruz	0.45
	E	Santa Cruz	0.49
	L	Kobe	0.62
Csp3	G	Santa Cruz	0.025
	D	Santa Cruz	0.04
	J	Kobe	0.22
	M	Santa Cruz	0.41
	I	Santa Cruz	0.49
	O	Kobe	0.6

3 ABAQUS METHODOLOGY FOR PILE-SOIL INTERACTION ANALYSIS WITH BNWF MODEL

BNWF models in dynamic analysis should allow for the variation of soil properties with depth, nonlinear soil behavior and energy dissipation through damping mechanism. Therefore, proper analysis of the seismic response of piles involves modeling the pile and surrounding soil including damping consideration. While performing seismic response analysis, free field ground motion time histories are usually computed in a separate site response analysis (Talukder and Butt, 2009a). The computed ground motion at different depths of sand is then applied to the nodal boundary supports representing the support motions. Figure 1 shows the schematic view of Dynamic BNWF model with its components for simulation of a single pile in saturated sand.

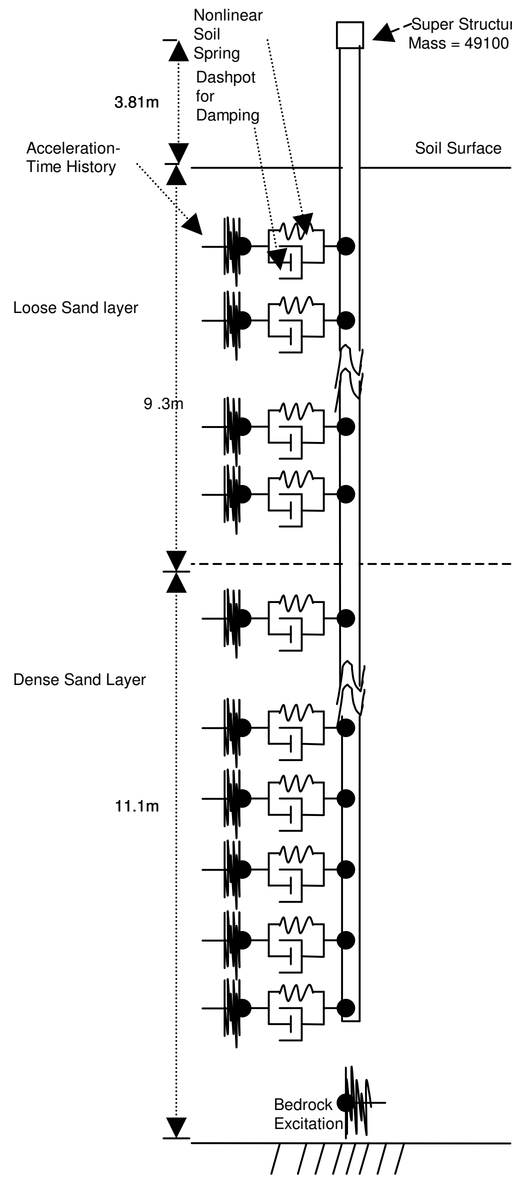


Figure 1. Schematic diagram of BNWF model for simulation of Csp2 and Csp3 tests using ABAQUS.

A "CONN2D2" element of the ABAQUS library adjoins nonlinear springs and linear dashpots in parallel (Figure 1) and is an unidirectional two nodded connector element. One end of each "CONN2D2" element connects a pile node and the other node is assigned with acceleration time histories. Only axial capacity of this element was used for current analysis. P-y curves are assigned to nonlinear springs of "CONN2D2" elements. Details of soil stiffness and pile modeling employed in the paper can be studied from Talukder et al. (2009b). Lumped mass of the superstructure is modeled using element type "MASS" from ABAQUS element library. “MASS” is a point element that is attached to the top of the pile in ABAQUS .

In the present study, two dimensional dynamic response analyses of the pile have been carried out and hence all element capabilities are set to two dimensional modeling. A modal superposition method is used to implement a transient dynamic analysis procedure. Transient modal dynamic analysis gives the response of the pile as a function of time based on a given time-dependent loading. As vibration of a single vertical pile is dominated by the first few modes of the total modes of vibration so the first 100 modes of vibrations are extracted. Each of these modes is assigned with the same pile material damping ratio, ξ in any analysis.

4 ANALYSIS OF ABAQUS RESULTS

Talukder et al. (2009b) showed that by varying the pile material damping ratio, it is possible to achieve good agreement between recorded and computed peak pile bending moment for some important shaking events of Csp2 and Csp3 tests. This paper shows that by varying the pile material damping ratio ξ, we can achieve good agreement between recorded and computed peak superstructure acceleration, as well as peak pile head acceleration for any shaking event of Csp2 and Csp3 tests. Superstructure displacements vs. peak base acceleration curves for different values of ξ are plotted to show the extent of differences between calculated and recorded superstructure displacements. It is also shown that by plotting pile peak bending moment (PPBM) vs. peak base acceleration curves for different values of ξ, predicted PPBMs can lead to better estimation of experimental PPBMs observed over a wide range of shaking events of Csp2 and Csp3 with reasonable agreement.

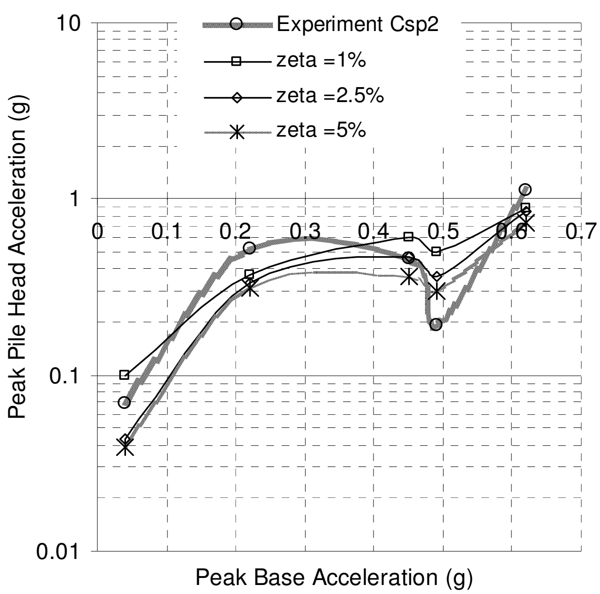


Figure 2. Calculated and Recorded Peak Pile Head Accelerations for Csp2 tests.

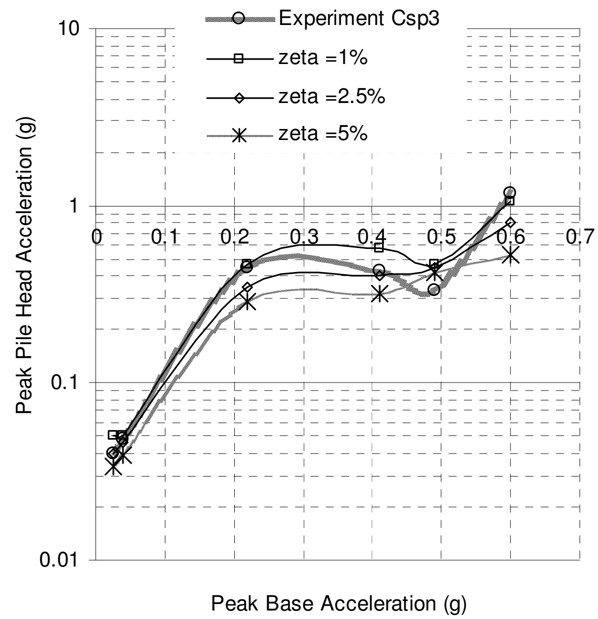


Figure 3. Calculated and Recorded Peak Pile Head Accelerations for Csp3 tests.

4.1 Analysis of Predicted Pile Head Accelerations

Pile head acceleration time histories of the BNWF model were computed for all earthquake events listed in Table 1 for ξ of 1%, 2.5% and 5%. These time histories were compared with experimental pile head acceleration time histories. These comparisons are summarized in Figures 2 and 3 which show both calculated and recorded peak pile head accelerations versus peak base (input) acceleration curves. The calculated responses are in good agreement with the recorded responses for all events of Csp3, with exceptions for events D, F and E of the Csp2 test.

4.2 Analysis of Predicted Superstructure Accelerations

Pile superstructure acceleration time histories of the BNWF model were computed for all earthquake events listed in Table 1 for ξ of 1%, 2.5% and 5%. These time histories were compared with experimental superstructure acceleration time histories. These comparisons are summarized in Figures 4 and 5 which show both calculated and recorded peak superstructure accelerations versus peak base (input) acceleration. The calculated responses are in excellent agreement with the recorded responses for all events of the Csp3 tests, but approximately 250% higher than the recorded responses for events F, J and E of the Csp2 tests.

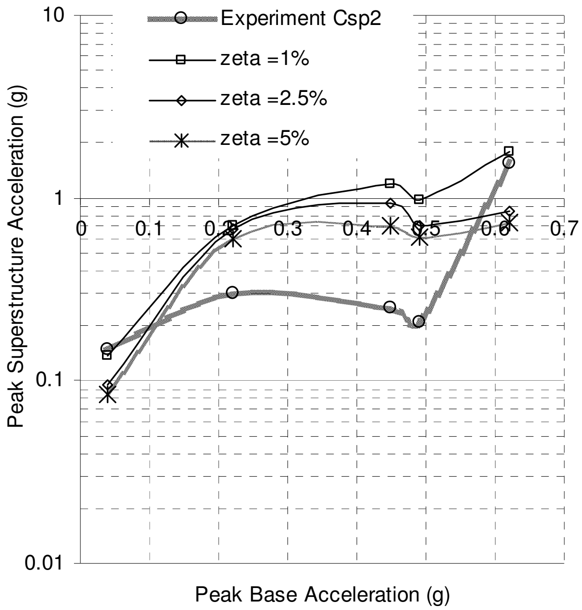


Figure 4: Calculated and Recorded Peak Superstructure Accelerations for Csp2 tests.

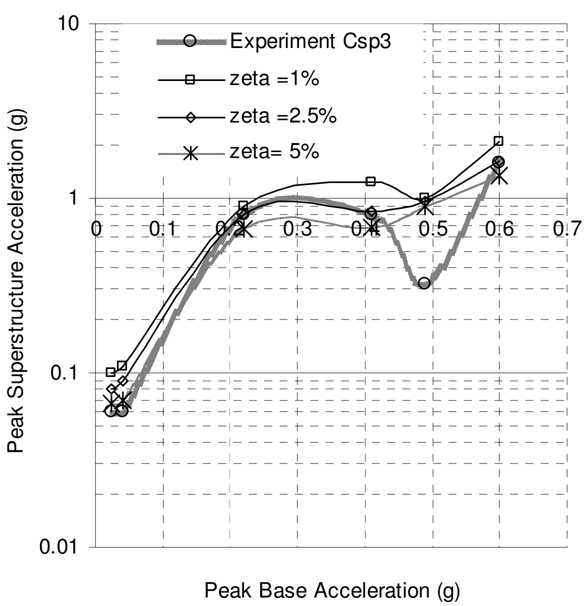


Figure 5: Calculated and Recorded Peak Superstructure Accelerations for Csp3 tests.

4.3 Analysis of Predicted Superstructure Displacements

Pile superstructure displacement time histories of the BNWF model were computed for all earthquake events listed in Table 1 for ξ of 1%, 2.5% and 5%. These time histories were compared with experimental superstructure displacement time histories. These comparisons are summarized in Figures 6 and 7 which show both calculated and recorded peak superstructure displacements versus peak base (input) acceleration curves. The calculated responses resulted in an underestimation (on an average of approximately 400%) of the recorded responses for all events of the Csp2 and Csp3.

Figures 6 and 7 show that superstructure displacements were heavily underestimated by the predictions of the current BNWF model. API recommendations assume that initial p-y stiffness increases linearly with depth. It defines the initial stiffness as the product of depth (z) and the initial modulus of subgrade reaction coefficient (k). API recommendations for k were derived from lateral loading tests that were controlled by drained soil behavior at relatively shallow depths. A constant k value overestimates the p-y stiffness at depths greater than a few pile diameters, because the stiffness of sand generally increases in proportion to the square root of confinement, not in proportion to confinement (Boulanger et al. 1999). Therefore, the initial stiffness of soil springs at all depths along the BNWF model were stiffer than that of measured p-y curves in saturated sand of the Csp2 or Csp3 centrifuge models (Wilson et al. 2000). For the shaking events of Csp2 covered in this study, the current BNWF model underestimated peak superstructure motions (displacements and accelerations) by an average of 400%.

4.4 Analysis of Predicted Bending Moments

Pile peak bending moments (PPBM) within different soil layers could be considered as the main parameter to evaluate the ABAQUS model's ability to conduct seismic response analyses of piles. Talukder et al. (2009b) and El Naggar et al. (2005) used a wide range of pile material damping ratios for achieving good agreement between computed and recorded PPBMs. In line with this approach, different material damping ratios for pile elements were used in the analyses of the present ABAQUS model. Talukder et al. (2009b) reported that an increase in the pile material damping ratio, ξ leads to a decrease in the computed PPBM.

Figure 8 shows the maximum calculated and recorded experimental PPBM along the pile shaft for soil profile Csp2 versus PBAs. It is observed that for a wide range of PBAs (between 0.040 g and 0.62 g) and by using various pile material damping ratios, ξ (between 1% and 15 %), there would be a reasonable estimate of PPBM. This study proposes that by varying ξ between 5% and 12.5%, a foundation engineer can have a safe estimate of PPBM using Figure 8 in response to PBAs between 0.04 g and 0.62 g. For example, the recorded PPBM in event H (0.11 g) of Csp2 was 600 kN.m. This result may be predicted from Figure 8 with a ξ of 5 %.

Figure 9 shows the maximum calculated and recorded experimental Pile Peak Bending Moments (PPBM) along the pile shaft versus PBAs for a soil profile in Csp3. It is observed that for a wide range of PBAs (between .025 g and 0.62 g) while varying ξ between 2.5% and 5%, a foundation engineer can have a safe estimate of PPBM using Figure 9 in response to PBAs between 0.025 g and 0.62 g .

Based on the performance of the present BNWF model, Table 2 summarizes a range of ξ values that may be used by design engineers for the estimation of pile peak responses in saturated sand during seismic events.

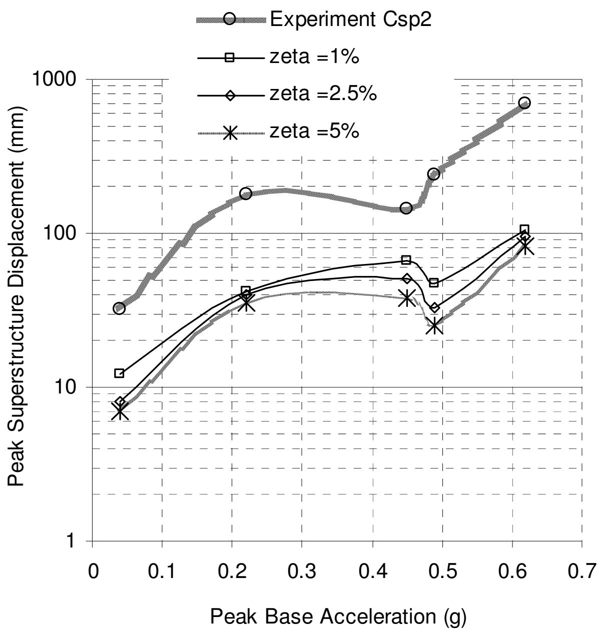


Figure 6: Calculated and Recorded Peak Superstructure Displacements for Csp2 tests.

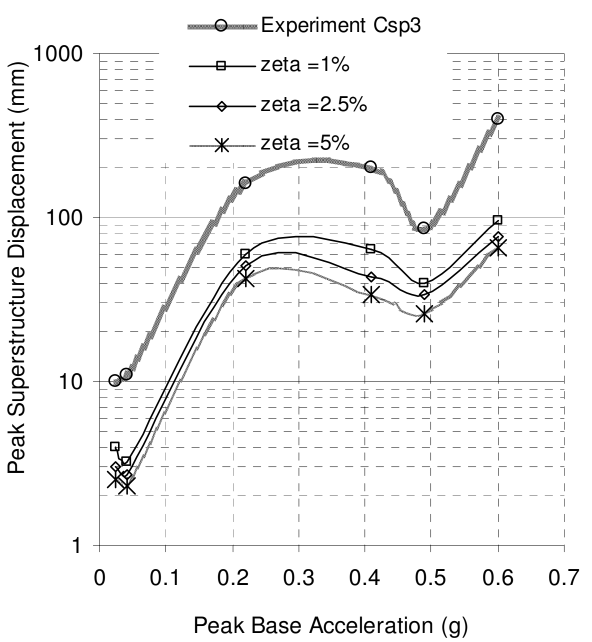


Figure 7: Calculated and Recorded Peak Superstructure Displacements for Csp3 tests.

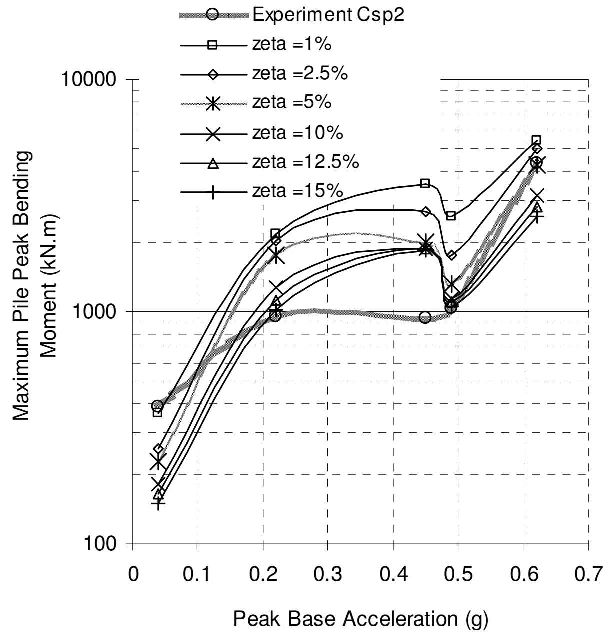


Figure 8: Maximum Pile Peak Bending Moment vs. Peak Base acceleration curves for Csp2 tests.

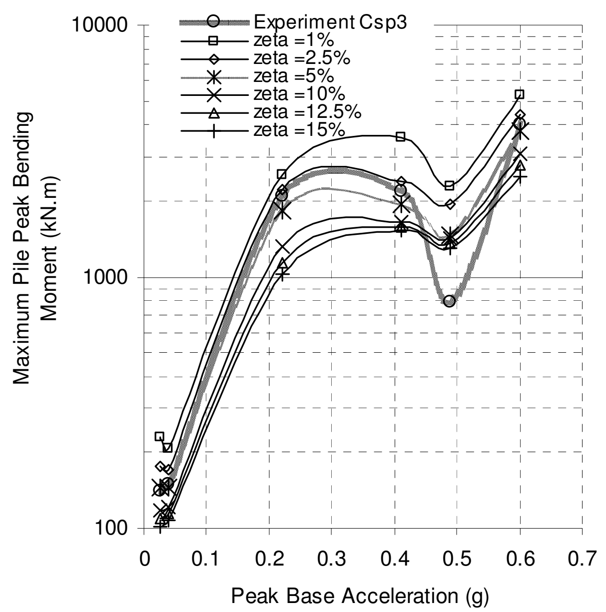


Figure 9: Maximum Pile Peak Bending Moment vs. Peak Base acceleration curves for csp3 tests.

5 DISCUSSION ON THE TREND OF PPBM VS. PBA CURVES

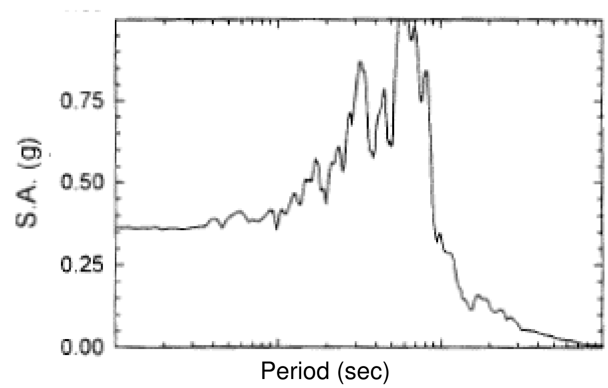
All the experimental peak response vs. PBA curves have a valley near 0.50 g (Figures 2 to 9). The remarkable feature of Figures 2 to 9 is that the numerical peak response curves also capture discontinuity of experimental curves at PBA, 0.50 g. Recurrence of discontinuity in the numerical peak response curves is observed at PBA, 0.50 g for all values of ξ. Spectral acceleration vs. periods curves of the base input motions of the Csp2 and Csp3 tests (Table 1) are shown in Figures 10 and 11. Spectral acceleration of a base input motion near the fundamental period of the pile foundation system indicates the amount of energy content of the base motion to which the structure was subjected (Wilson 1998). In this study, by performing a frequency analysis, the fundamental period of the Finite Element model of BNWF system was obtained to be 0.49 sec. The magnitude of spectral accelerations of base input motions at 0.49 sec are obtained from Figures 10 and 11. We can attempt to analyze the recurrent of discontinuity in numerical PPBM vs. PBA curves near PBA, 0.50g, by referring to Figures 10 and 11.

It is reported by Wilson (1998) that at the time of the occurrence of PPBMs, excess pore pressure ratios were 80% to 100% throughout the upper soil profile in events F (0.22 g), J (0.45 g) and E (0.49 g) of the Csp2 tests. On the other hand, EPWP ratios were 50% to 65% throughout the upper soil profile in events J (0.22 g), and M (0.41 g) of Csp3 tests at the time of occurrence of PPBMs. Wilson (1998) reported that in event L of Csp2 (PBA, 0.62g) and event O of Csp3 (PBA, 0.60 g) EPWP time histories induced dilation in the soil. This dilation occurred in early shaking resulting in the largest lateral resistance from soil to the pile. Consequently, recorded responses of the pile for these two events were at their maximum as can be seen from Figures 8 and 9.

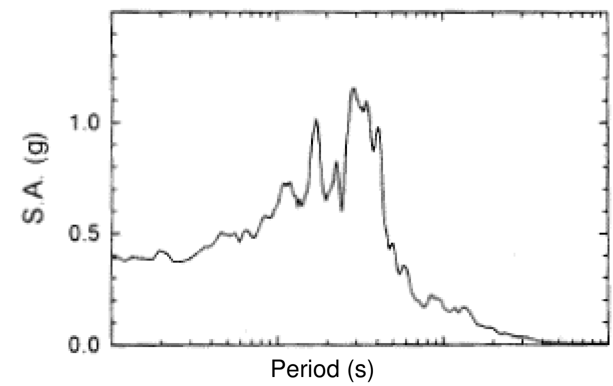
In the Csp2 test, the sand was loose (Dr ≈ 35%), while in Csp3 the sand was medium dense (Dr ≈ 55%). Wilson (1998) observed that liquefaction was more extensive in Csp2 than in Csp3, as evidenced by pore pressure time histories showing that pore pressures increased much more quickly and dissipated more slowly in the relative density Dr ≈ 35% sand layer of Csp2 than in the Dr ≈ 55% sand layer of Csp3. Wilson (1998) reported that the looser condition of the upper layer in Csp2, relative to Csp3, resulted in generally lower ground surface accelerations and lower peak superstructure accelerations. As a result pile peak bending moments (PPBMs) were larger in Csp3 than in Csp2 for nearly similar input base motions (Wilson et al. 1997a, b).

For simulation of Csp3 event I, the BNWF model was excited by peak base motion of 0.49g (Figure 11 b). Spectral acceleration of this base motion near fundamental period of the BNWF model (0.49 sec) was 0.30 g. Csp3 event M was shaken by peak base motion 0.41 g which was lower than that of Csp3 event I. Nevertheless, spectral acceleration of the base input motion for Csp3 event M was 0.85 g (Figure 11a) which is higher than the spectral acceleration of the base input motion for Csp3 event I. Therefore, the base motion applied to event I of Csp3 had lower energy content near the fundamental period of the structure than Event M of the Csp3 test. Thus, the predicted PPBM for event I of Csp3 may have dropped below the predicted PPBM of events M and O of Csp3 (Figure 11a and c).

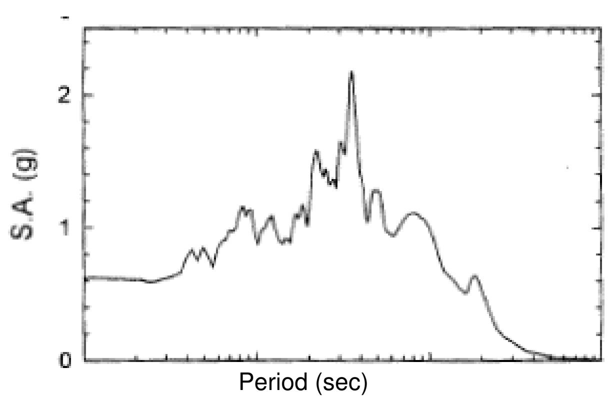
Similarly, the calculated PPBM for 0.49 g event E of the Csp2 test may have fallen below the PPBMs of significant shaking events of Csp2 tests (Figure 10a, c). For simulation of event E (0.49 g) of the Csp2, the BNWF model was excited by peak base motion 0.49 g. Spectral



(a)



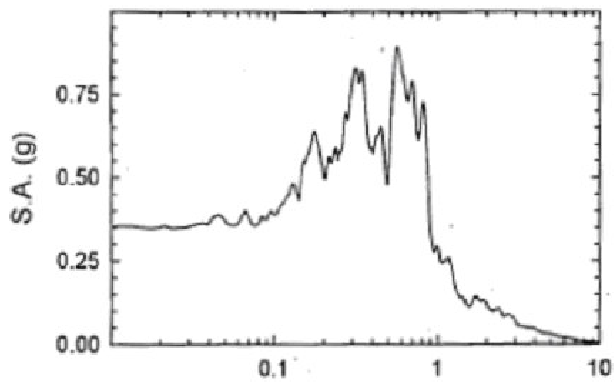
(b)



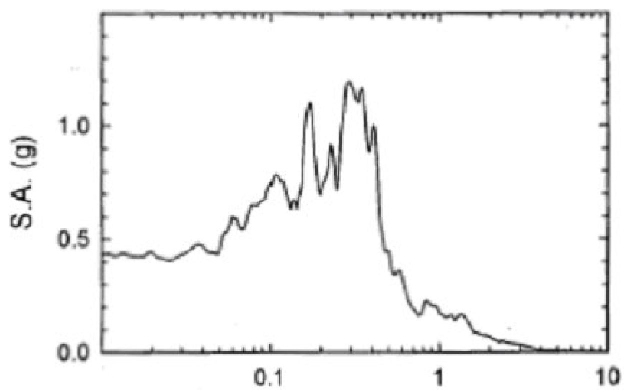
(c)

Figure 10: Spectral Acceleration vs. Period curves for the Base Motions, a) Event J of Csp2 test with PBA, 0.45 g, b) Event E of Csp2 test with PBA, 0.49 g, c) Event L of Csp2 test with PBA, 0.62 g Figure 10(after, Wilson et al. 1997a).

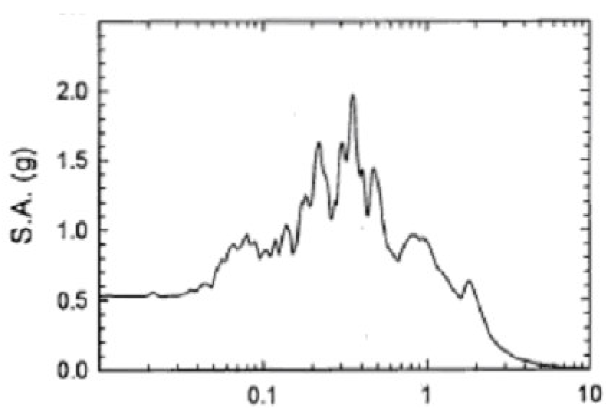
acceleration of this base motion near the fundamental period of the BNWF model (0.49 sec) was 0.35 g (Figure 10b). At the fundamental period of the BNWF model, spectral acceleration of base input motions in 0.45 g and 0.62 g events of the Csp2 test are 0.85 g and 0.90 g(Figure 10a and c) respectively.



(a)



(b)



(c)

Figure 11: Spectral acceleration vs. Period curves for the Base Motions, (a) Event M of Csp3 test with PBA, 0.41 g, b) Event I of Csp3 with PBA, 0.49 g, c) Event O of Csp3 test with PBA, 0.60 g (after, Wilson et al. 1997b).

Hence, the base motion applied to event E of Csp2 had lower energy content near the fundamental period of the structure than 0.45 g and 0.62 g events of the Csp2.

Similar explanation may be drawn for the discontinuity in numerical peak superstructure accelerations and displacements vs. PBAs curves.

6 CONCLUSION

Peak pile response curves were compared with experimental response curves and summarized in Figures 2 to 9. Comparisons of calculated peak pile head accelerations, peak superstructure accelerations and peak pile bending moments with those of Csp3 tests were found to closely agree. It can be noted in Figures 8 and 9 that the BNWF model developed in this study produced close predictions for Csp3 tests, but poor prediction for Csp2 tests. However, overestimation of PPBMs as compared to PPBMs measured in Csp2 tests was similar to the underestimation of superstructure displacements for all shaking events of Csp2 and Csp3 tests. Peak pile bending moments along the pile length were overestimated by about 250% while simulating various shaking events of Csp2. However, peak bending moments predicted for very weak (0.04 g) or very strong(0.62g) base input motion of Csp2 tests were in good agreement with the experimental results. A range of design values for ξ were tabulated in Table 2 for use in the preliminary estimation of peak pile response parameters through BNWF analyses of piles embedded in saturated sand during seismic events. The design recommendations given in the study are based on the soil profiles of Csp2 and Csp3 tests. The results shown in this paper is an outcome of a work in progress report. More numerical studies on various soil profiles other than Csp2 and Csp3 tests are warranted for comprehensive design charts that may be used in design. An extensive parametric study on soil-pile interaction should be conducted involving different pile and soil profile to assess how variations in p-y model parameters affect the response quantities of interest.

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