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PILE FOUNDATION MODELLING FOR SEISMIC ANALYSIS OF HIGHWAY BRIDGES USING BEAM ON A NON-LINEAR WINKLER FOUNDATION

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Abstract: This paper presents a study of bridge behavior under seismic loading while comparing two types of modelling techniques to simulate the soil conditions; the fixed base model and the beam on non-linear Winkler foundation model (BNWF). A two-span bridge on pile foundations is analyzed with two types of soils profiles corresponding to type I & type III as defined in the Canadian Highway Bridge Design Code (CSA, 2006). The objective of the study is to present a modelling technique for pile group foundations to be used by practicing engineers to model soil-structure interaction efficiently. For the BNWF model, springs are assigned stiffness' corresponding to load-displacement (p-y) soil curves and these springs are placed at specific intervals along the pile. Deconvolution - convolution of records is used to provide ground motion displacement data at each spring. Non-linear time histories analyses are run for the two soil types and these results are compared to the fixed base study. It is shown that piles in soft soil are quite flexible and larger total pier displacements occur when the soil properties are modelled. Stresses in structural components are reduced as more flexibility is introduced to the bridge. The BNWF technique allows for better material optimization for the earthquake resisting system, provides a more accurate estimation for total pier displacement and directly accounts for pile internal forces.

Keywords: bridge foundations; dynamic analysis; p-y curve; non-linear soil; seismic response;

1- INTRODUCTION

Many earthquakes, such as those made famous in California, Taiwan, Japan and Mexico, have brought attention to the potential dangers of soft soil profiles on earthquake motions that impact all civil engineering structures including bridge structures. These horizontal ground motions which propagate from bedrock to the soil surface must be taken into account during the design process of a bridge.

Many bridges are supported on pile foundations which are located on soft soil deposits. During an earthquake, the ground motion begins at the bedrock and propagates horizontal shear waves upwards through the soil profile while interacting with each pile, which in turn influences the bridge foundation and the superstructure (Schnabel, 1972). The bridge, subjected to these ground motions, is affected by its own mass which causes inertial forces to act on its entire structure, inducing cyclic stresses during a shaking event. To capture some of these soil-pile interaction features in a structural model, practicing engineers could build representations using a wide spectrum of complexities. On the simplest side, a fixed-base model is often used. To capture the flexibility of the foundation system, an equivalent foundation spring model is also often employed. A beam on a non-linear Winkler foundation (BNWF) also captures the flexibility of the foundation along the pile by utilizing distributed springs characterized with a lateral load (p) – displacement (y) curve for each of the springs. A more complex version of the BNWF model (Figure 1) comprises of non-linear springs and dashpots specifically arranged to capture not only the flexibility of the foundation but the damping characteristics of the soil as well. A Penzien model adds inertial effects to the soil system along with the non-linear spring and dashpot arrangement, while a detailed finite-element model can take into account inertia, damping and kinematic interactions with greater precision (Sun et al., 2001).

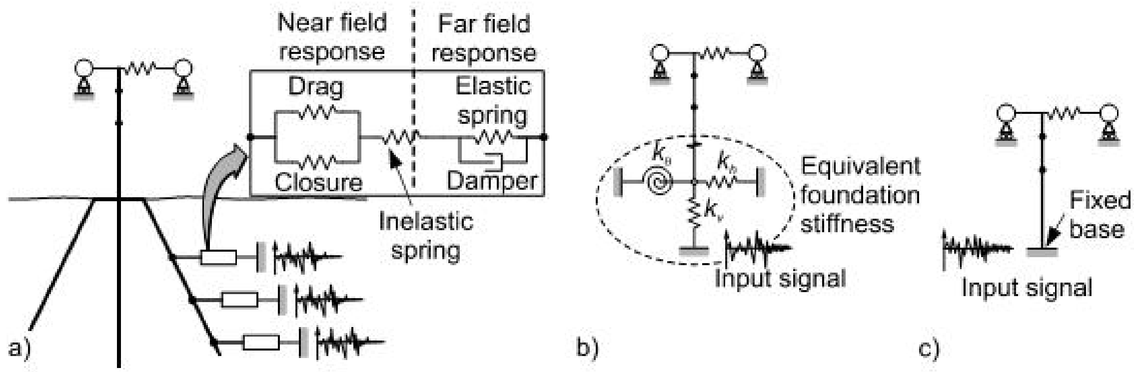
This paper compares two modeling alternatives for an existing highway bridge supported on pile foundation in Eastern North America under typical seismic events: a fixed base model versus a BNWF model. In the BNWF model (an accessible method for practicing engineers) distributed non-linear soil springs are placed along the pile. The effects of pile foundation modeling on elastic displacements and elastic forces are quantified through linear and nonlinear time-history analyses. A pushover analysis using non-linear column and non-linear soil properties is also presented .

There are several advantages to using a BNWF modelling approach. Soil non-linearity is easily taken into account using force-deflection curves in the horizontal and vertical directions (named p-y, t-z, q-z soil springs). Site effects can be taken into account by imposing independent time-history displacement into each soil spring. However, the BNWF modelling approach also has its limitations. The soil springs do not interact with each other, which raises questions about the dependability of pile group analysis and force-deflection curves (Wang et al., 1998). Furthermore, the mass of the soil around the structure is omitted, while the presence of soil around the piles would increase the mass effect of the structural system (Sun et al., 2001).

Nevertheless, many researchers (Wang et al., 1998, Boulanger et al., 1999, El Naggar et al., 2008) have proven the effectiveness of the BNWF model by comparing it to centrifuge test results or to instrumented bridges under seismic events. Therefore, the BNWF approach merits consideration for seismic soil-structure interaction problems for highway bridges supported by pile foundations.

2- SOIL-PILE-BRIDGE INTERACTION USING BNWF

A basic BNWF model only requires a soil force vs. lateral deflection curve (p-y) for each soil spring. More advanced models include an arrangement of elements representing the near-field plastic response and the far-field elastic response with a series of gap, drag, plastic and elastic springs and dashpots (Wang et al, 1998). For advanced models, it is widely accepted that soil around the pile should be divided into different zones; the near-field, where plastic behavior occurs and the far-field, where elastic behavior occurs (Wang et al., 1998). Radiation damping (Gazetas, 1984), which takes place when energy is lost during the motion of a pile against the surrounding soil should be taken account using a dashpot in the elastic region (far-field). Placing this radiation damper in series with the inelastic soil spring was preferred by Wang et al. (1998), as it avoided unrealistic radiation damping forces compared to centrifuge test results. Hysteric damping, otherwise known as the energy lost in materials due to friction between molecules, should be taken into account using a dashpot in the plastic region (near-field). The non-linear p-y soil curves are decomposed into elastic and plastic components. Finally, the springs and dashpots are assembled using the series radiation damping arrangement as shown in Figure 1 (Wang et al., 1998). In comparison, the cyclic p-y formulation by El Naggar et al. (2008) requires only a non-linear spring in parallel with a radiation damper.



a) BNWF model with series radiation damping and drag-closure elements
b) Equivalent lumped spring model
c) Fixed base model

Figure 1- Modelling options
(Adapted from Gagnon et al., 2010)

3- BRIDGE MODEL

A fixed base model and a BNWF model are chosen to represent a continuous two-span bridge with a two-column pier (Figures 2, 3). SAP2000 is used as the modelling software (CSI, 2012). Only the longitudinal earthquake effects are studied and due to symmetry, only one column and half of the longitudinal beams forming the super structure are needed for the model. The bearings at the abutments allow for movement in the longitudinal direction while the bearings at the pier provide a fixed shear connection. The bridge is classified with an importance factor of 1 as described by the Canadian Highway Bridge Design Code (CSA, 2006).

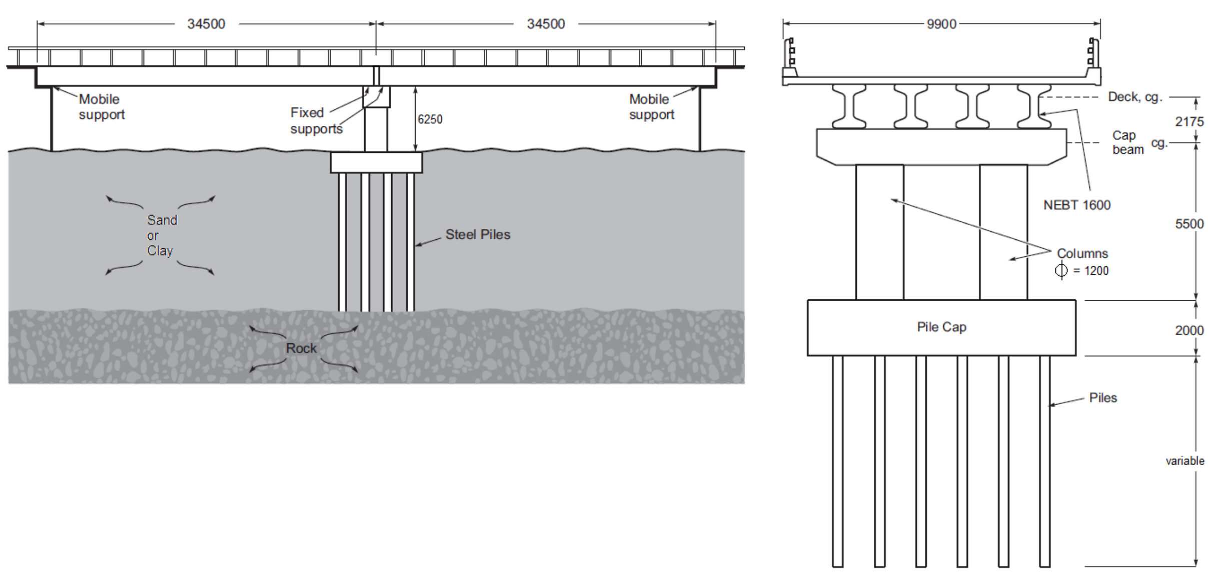


Figure 2- Bridge under study
(Adapted from Gagnon et al., 2010)

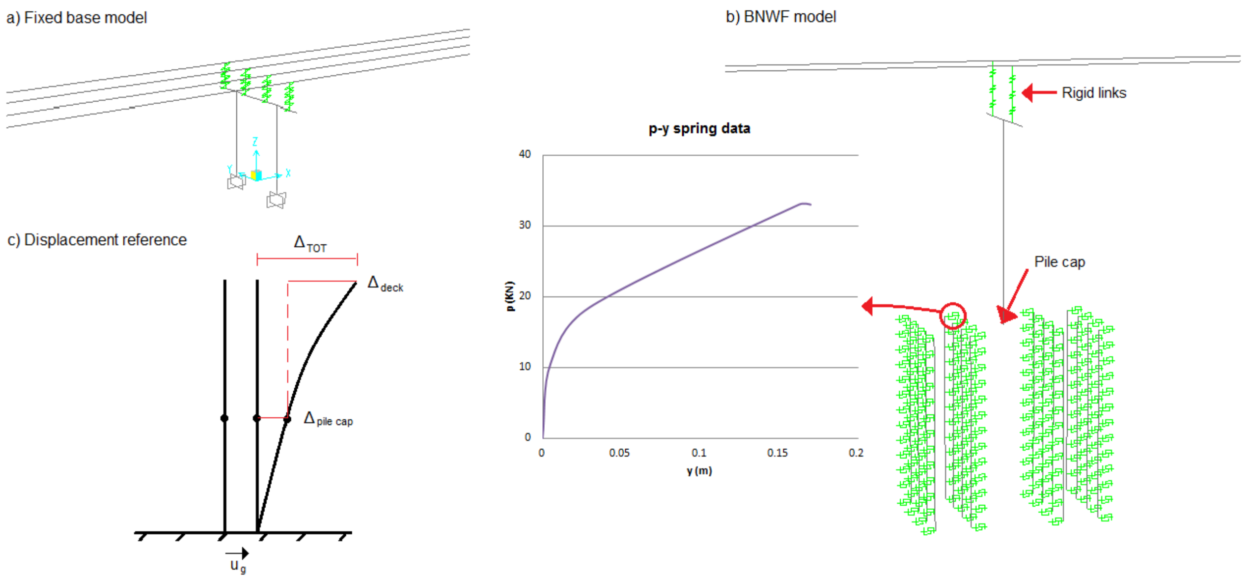


Figure 3- Numerical models and displacement reference

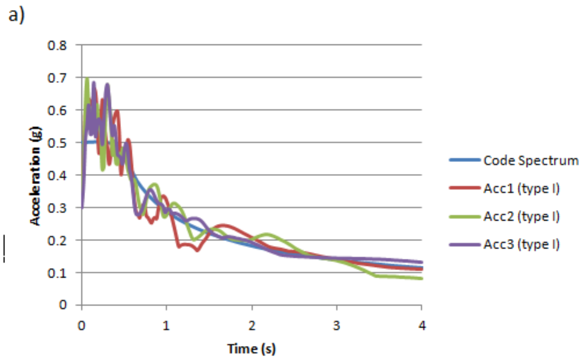
4- SOIL & PILE PROPERTIES

The geometric properties of the end-bearing piles are held constant in each model; an HSS 406x9.5 pile is chosen. The type I soil profile, with an average shear wave velocity of 625 m/s, corresponds to a layer of sand (3.4 m deep - 300 m/s) and a layer of sand with gravel (4.7 m - 900 m/s) over a rock base, while the type III soil profile, with a constant shear wave velocity of 125 m/s, corresponds to a layer of soft clay overlaying rock. Type I soil matches to a stiff soil profile and the type III soil matches to a soft soil profile as defined in the Canadian Highway Bridge Design Code (CSA, 2006). The type I soil profile has a total height of 8.1 meters, while the type III soil profile has a total height of 11 meters.

Lateral load - displacement (p-y) curves at specific depths are necessary to develop the BNWF model. A p-y curve relates the total static soil reaction to the pile displacement. The p-y curves are found for each soil profile using LPile software (Ensoft, 2012). This program provides p-y curves using published recommendations for various types of soils and this method is accepted by the Canadian Highway Bridge Design Code (CSA, 2006). Lateral load - displacement curves offer several advantages over simplified methods: non-linear soil properties are taken into account, soil resistance may vary with respect to depth, soil type variations may be included and changes in pile diameter are taken into account. A p-y multiplier of 0.5 is applied for deflection calculations to take group effects into account (Po Lam et al., 2007). A p-y multiplier of 1 is used to conservatively calculate base shear. The p-y data at a specific depth is entered as a non-linear spring property in SAP2000.

5- ACCELEROGRAM DATA AND SITE EFFECTS

Three accelerograms are chosen to match the S6-06 design spectrum for each soil profile (Figure 4). These accelerograms are applied directly to the fixed base model. For the BNWF model, the possibility of imposing depth-dependent seismic inputs in each non-linear soil spring is studied.



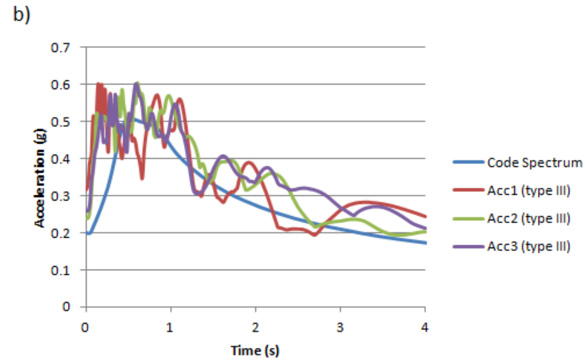


Figure 4- Design response spectrum and spectral accelerations of input signals with 5% damping
a) type I soil ; b) type III soil

As seismic waves arrive at a site, the ground motions produced are affected by the geometry and geology at that site. A soil deposit acts as a filter, amplifying response at some frequencies and attenuating it at others. A simplified site response procedure is undertaken using soil density, shear modulus, damping ratio and soil thickness properties; the soil resting on rigid bedrock and is subjected to vertically propagating horizontal shear waves. Because accelerogram data is recorded at the surface, deconvolution is performed with the Shake2000 computer program (Geomotions LLC, 2010) using an equivalent linear procedure to provide the accelerogram data for the rock layer. This equivalent linear procedure takes into account soil nonlinearity indirectly through an iterative process. Soil stiffness decreases and material damping increases with increasing strain amplitude, thus modulus reduction curves and damping curves are taken into account using program-defined values available within Shake2000. A decrease in the maximum acceleration amplitude was noted with increasing soil depth for both soil profiles.

Convolution using an equivalent linear procedure is performed with the Deepsoil computer program (Hashash, 2012) to verify the results of the deconvolution performed with Shake2000. The acceleration time history records varying through each soil profile is obtained. Figure 5 describes the site response procedure implemented to find the time history data at various soil depths. Implementation of multi-support excitation along the piles in SAP2000 requires displacement time histories at each soil spring. Thus, acceleration time history results through the soil layers are converted to displacement time histories by double integration and applying a baseline correction. Seismosignal software is chosen for this time history conversion (Seismosoft SRL, 2012). Results show that, for both soil profiles, the displacement time histories differences through the soil profiles were negligible. Therefore, no variations in seismic input are applied to the soil springs.

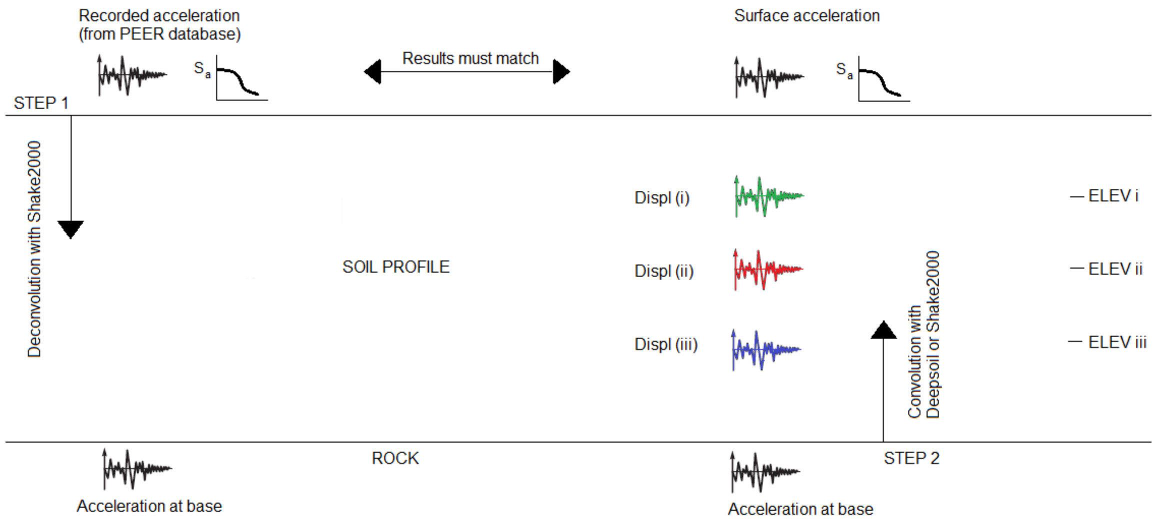


Figure 5- Procedure to find displacement time histories for entry into SAP2000 model

Very small differences within the displacement time histories through two soil profiles studied were attributed to the short fundamental period of the profiles. A characteristic site period could be approximated by $T_s = 4H/v_s$ where H is the height of the soil column and v_s is the average shear wave velocity of the soil column (CRC Press, 2000). The periods of the soil columns, using this approach, were calculated at 0.05 s and 0.35 s for soil type I and soil type III, respectively. The greatest degree of amplification occurs at frequencies corresponding to the characteristic site period; however the amplification for the two soil profiles studied was not substantial enough to merit consideration in the structural model (Figure 6).

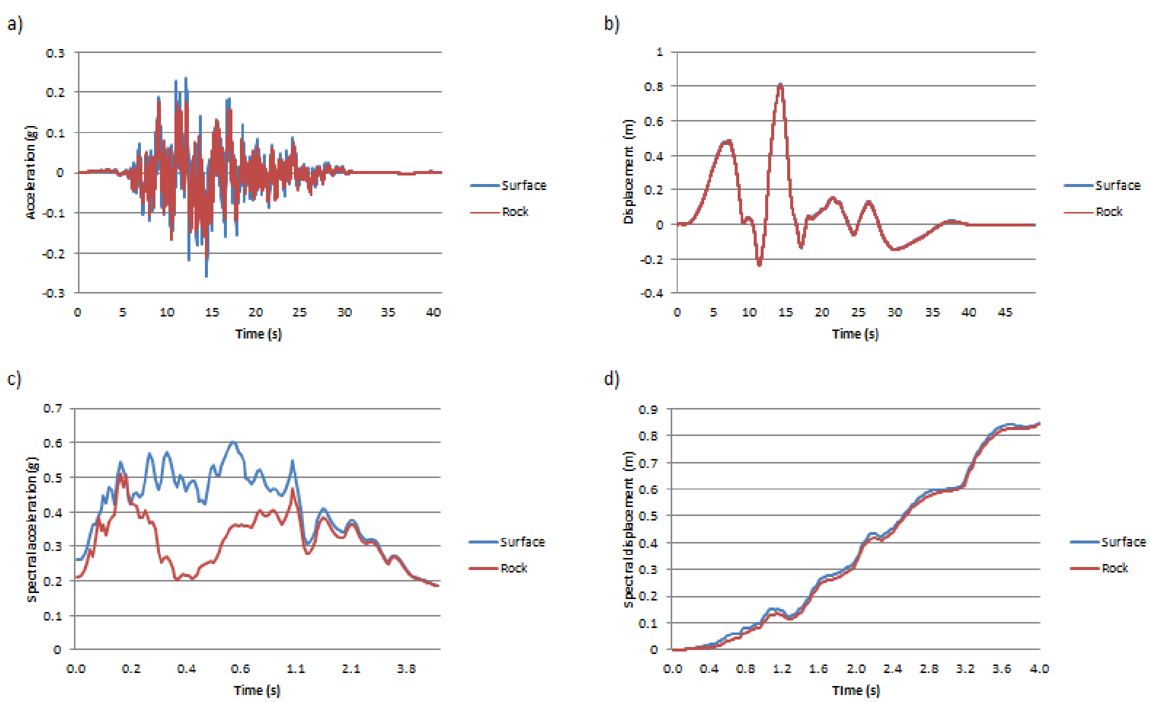


Figure 6- Surface-rock outputs for accelerogram 3 - type III soil
a) acceleration time history (g vs sec) ; b) displacement time history (m vs sec) ; c) spectral acceleration with 5% damping (g vs sec); d) spectral displacement with 5% damping (m vs sec)

6-ANALYSIS

Response spectrum analyses were performed for the fixed base model only because modal superposition techniques do not apply for non-linear models. The fundamental period, elastic base shear and elastic deck displacement for soil types I & III are shown in Table 1. The elastic base shear and the elastic deck displacement results agree with the time history analysis for the fixed base model.

Table 1- Response spectrum analyses for the fixed base model

Soil type	Period (s)	V_e (kN)	Δ_{deck} (mm)
Type I soil	0.85	1612	58
Type III soil	0.85	2407	87

The non-linear soil springs were linearized using initial soil stiffness parameters and the fundamental periods for soil types I & III are shown in Table 2. By modelling the soil properties, the type I soil elongated by 13% for its fundamental period while the type III soil elongated by 18%. These results should be analyzed with caution as initial soil stiffness parameters where chosen.

Table 2- Fundamental periods for a linear soil spring model using initial soil stiffness

Soil type	Period (s)
Type I soil	0.96
Type III soil	1.00

Time history analyses were undertaken for both model types. Linear time history analysis was chosen for the fixed base model, while non-linear time history analysis was used for the BNWF model due to the non-linear soil springs. Linear properties were assigned to the steel piles and concrete column. The elastic base shear and elastic deck displacement and the elastic pile cap displacement for soil types I and III are shown in Table 3. Displacement results from SAP2000 for the deck, pile cap and signal input are shown as an example in Figure 7. The true deck displacement & pile cap displacement values at every time interval are found using these relations:

$$\Delta_{pile\ cap} = \text{total } \Delta_{pile\ cap} - \Delta_{signal\ input}$$

$$\Delta_{deck} = \text{total } \Delta_{deck} - \text{total } \Delta_{pile\ cap}$$

$$\Delta_{TOT} = \Delta_{pile\ cap} + \Delta_{deck}$$

This calculation yields the concomitant pile cap ($\Delta_{pile\ cap}$) and deck displacement values (Δ_{deck}) as well as the maximum total pier displacement values (Δ_{TOT}) as shown in Table 3.

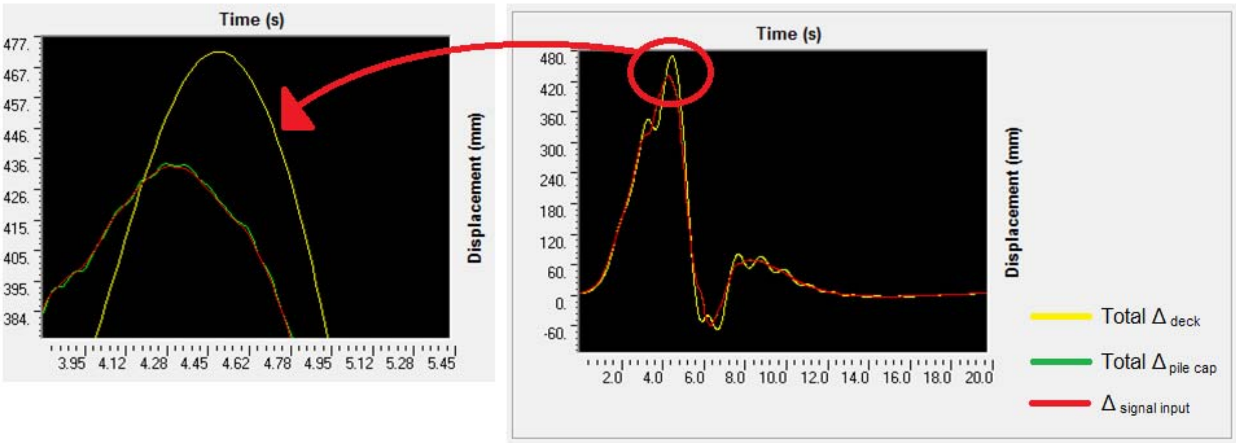


Figure 7- Generalised displacement time history results at the input, pile cap and deck locations (mm vs s)

Time history analyses where undertaken for both model types. Linear time history analysis was chosen for the fixed base model, while non-linear time history analysis was used for the BNWF model due to the non-linear soil springs. Only linear properties were assigned to the steel piles and concrete column. The elastic base shear and elastic deck displacement and the elastic pile cap displacement for soil types I and III are shown in Table 3.

Table 3- Time history analyses for fixed base and BNWF model

Soil type	Model type	Accelerogram 1				Accelerogram 2			
		V_e (kN)	**$\Delta_{pile\ cap}$ (mm)**	**Δ_{deck} (mm)**	**Δ_{TOT} (mm)**	**V_e (kN)**	**$\Delta_{pile\ cap}$ (mm)**	**Δ_{deck} (mm)**	**Δ_{TOT} (mm)**
Type I soil	Fixed base	1386	0	50	50	1933	0	70	70
	BNWF	1649	6	77	83	1307	5	63	68
Type III soil	Fixed base	2980	0	107	107	2621	0	94	94
	BNWF	2238	29	93	122	2502	43	94	137

Soil type	Model type	Accelerogram 3				Average values			
		V_e (kN)	**$\Delta_{pile\ cap}$ (mm)**	**Δ_{deck} (mm)**	**Δ_{TOT} (mm)**	**V_e (kN)**	**$\Delta_{pile\ cap}$ (mm)**	**Δ_{deck} (mm)**	**Δ_{TOT} (mm)**
Type I soil	Fixed base	1782	0	64	64	1700	0	61	61
	BNWF	1379	3	64	67	1445	5	68	73
Type III soil	Fixed base	2443	0	87	87	2681	0	96	96
	BNWF	2139	34.5	85	119.5	2293	36	91	127

Note: p-y multiplier = 0.5 for displacements, p-y multiplier = 1 for base shear

Using the column base bending moments and axial loads for type I and type III soil from response spectrum analyses, two column sections (column A and column B) are created for non-linear analyses. Their flexural properties are summarized as follows:

Column A : 1200 mm diameter; 18-30M rebar; clear cover of 90 mm
Column B : 1200 mm diameter; 24-30M rebar; clear cover of 90 mm

A response modification factor of 5 is used, based on the S6-06 code (CSA, 2006), to find the longitudinal rebar distribution for each column section. Fiber models with an axial force - bending moment interaction are created with SAP2000 for each column and pushover analyses is performed. For type I soil, a fixed base pushover model is compared to a BNWF pushover model using column A. For the type III soil, a fixed base pushover model is compared to a BNWF pushover model using column B.

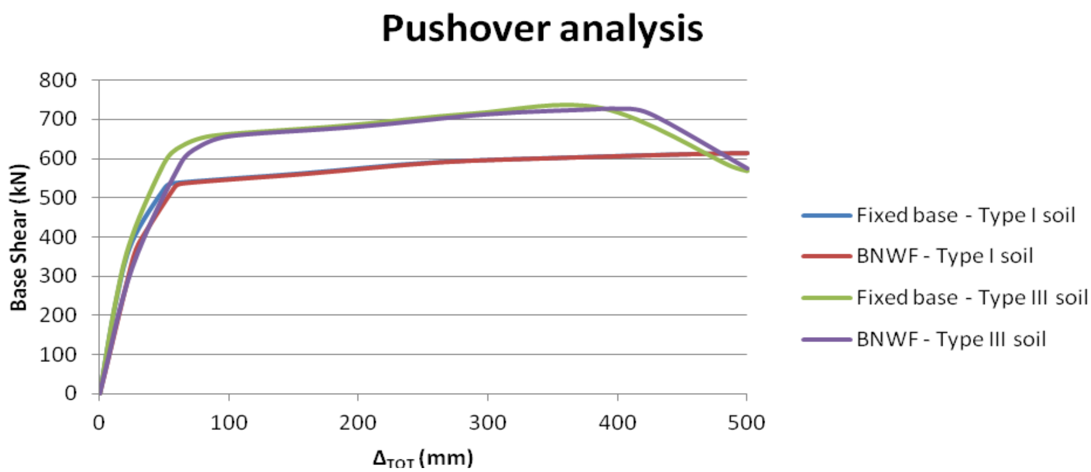


Figure 8 - Pushover results for Column A (type I soil) and Column B (type III soil)

The results shown in Table 3 revealed that elastic column base shear generally decreases using the BNWF model as compared with the fixed model; a decrease of 15% for type I and type III soil profiles was found in this study. Elastic deck displacements are similar for the fixed base and BNWF models. However, pile-soil interaction allows for the quantification of pile cap displacements which are found to be 5 mm for the stiffer soil and 36 mm for the softer soil. For a BNWF model, the total movement (Δ_{TOT}) required at the deck-abutment expansion joint can be found by adding the pile cap displacement to the deck displacement. The lower base shear values and higher pier displacement values for the BNWF model can be attributed to the elongated periods of vibration.

For the pushover analyses, there was an increase in flexibility for a given base shear when pile foundations were considered for both soil types (Figure 8). As expected, the flexibility amplification was more pronounced for the softer soil type. The flexibility increase, for both soil types, was also most notable in the elastic range of the column. As the concrete column yielded under flexure, the additional displacements were caused mostly by the rotation at the column hinge. Table 4 compares the elastic column total pier displacements to the pushover total pier displacements. As expected, total pier displacements were larger for the elastic column analysis cases as the column does not yield. For the pushover displacements, a BNWF model added 12% to the total pier displacement for type I soil and 19% to the total pier displacement for type III soil.

Table 4- Elastic column total pier displacements compared to pushover total pier displacement

Soil type	Model type	Average Displacements – Elastic column	Pushover Displacements At Yield
		Δ_{TOT} (mm)	**Δ_{TOT} (mm)**
Type I soil	Fixed base	61	56
	BNWF	73	63
Type III soil	Fixed base	96	63
	BNWF	127	75

The behavior of the soil springs was also studied. The results show that the non-linear soil springs display plastic behavior closer to surface and more linear behavior near the pile tip for both type I and type III soils (Figure 9).

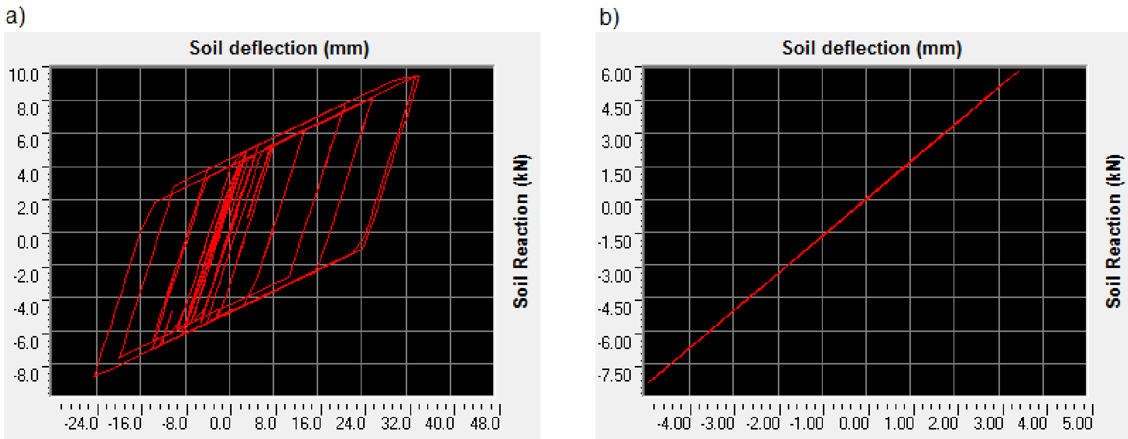


Figure 9 - P-Y curve hysteresis results for soil type III & accelerogram 3 for a) spring near surface b) spring near rock.

7- SUMMARY AND CONCLUSIONS

The seismic response of a two-span continuous highway bridge was studied for ground motions in the longitudinal direction to compare two modelling techniques; a fixed base model and a BWNF model. The pile foundations were modelled directly using non-linear soil springs and a deconvolution was performed, using horizontal shear wave propagation theory from the surface to the rock, to obtain seismic inputs at each spring. Non-linear time history analyses were performed for the BNWF model and compared to response spectrum and linear time history analyses for the fixed base case. A pushover analysis was also performed using non-linear column and non-linear soil properties.

Numerous computer programs where used for the analysis. LPile was used to compute the p-y soil spring data at desired depths. The geotechnical software package, Shake2000, was used for the deconvolution of the acceleration time histories through the soil profiles. Seismosignal was used to convert acceleration time history data to displacement time history entries. Finally, SAP2000 was used to create the structural models.

The analysis showed a decrease of 15% for elastic base shear when soil-structure interaction is taken into account compared to a fixed base model. The selected modelling method (fixed base or BNWF) did not significantly influence deck displacements. However, total pier displacements (Δ_{TOT}) increased by 18% and 31% for the stiff soil profile and soft soil profile as compared to the fixed base model, respectively. This pier displacement increase can be attributed to elongated periods of vibration for the BNWF model. For the pushover analyses, an increase in flexibility due to soil modelling was visible in the elastic range of the column. In the plastic range, the column flexural yielding dominated the total pier displacement.

Based on the study presented, it can be concluded that a fixed base model for highway bridges on pile foundations may underestimate the total pier displacement which increases the risk of unseating at the mobile abutment supports, especially for soft soil conditions. For engineering practice, it is wise to include an estimate of pile cap deflection for the total pier deflection for all fixed base models. In other words, a fixed base model does not give all the information necessary for a pier displacement calculation; the pile cap deflections must also be included. The BNWF model demands much more computations, iterations and discussions between the structural engineer and the geotechnical engineer than the fixed base model. However, for important bridge spans with pile foundations, a BNWF model may better represent the behavior of the highway bridge under seismic loads. Furthermore, the BNWF technique may allow for better material optimization for the earthquake resisting system and can directly account for seismically induced pile internal forces.

The BNWF model constructed was simplified for the scope of this study. In engineering practice, it is important to carefully consider all analysis assumptions to achieve the desired level of modelling accuracy. It is important to note that the effects of column cracking on pier stiffness should always be taken into account for base shear and deflection analyses (CSA, 2006). Column cracking was neglected for this study. For further studies, friction between the soil and concrete at the base of the pile cap may be modelled. Passive earth pressure of the soil surrounding the pile cap should not be ignored for a more accurate analysis. Non-linear soil-pile friction springs and end bearing springs should also be introduced if required. Furthermore, hysteresis models should be chosen to accurately represent the behavior of the soil. Radiation damping may also be included for a more accurate representation of the substructure. In spite of these simplifications, the BNWF model remains an interesting option for a dynamic analysis of a highway bridge structure.

ACKNOWLEDGEMENTS

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8- REFERENCES

Boulanger, R.W., C.J. Curras, B.L. Kutter, D.W. Wilson, and A. Abghari, 1999. *Seismic Soil-Pile-Structure Interaction Experiments and Analyses*, Journal of Geotechnical and Geoevironmental Engineering September 1999, American Society of Civil Engineers.

CRC Press, 2000. *Bridge Engineering Handbook*, CRC Press, USA.

CSA, 2006 (updated in 2010). *CAN/CSA-S6-06, Canadian Highway Bridge Design Code*, CSA, Toronto, ON.

CSA, 2006 (updated in 2010). *CAN/CSA-S6.1-06, Commentary on CAN/CSA-S6-06,* Canadian Highway Bridge Design Code, CSA, Toronto, ON.

CSI, 2012. SAP2000, Version 14.2.4. Computers & Structures Inc., Berkeley, CA. www.csiberkeley.com/

El Naggar, M,H. and N.K. Allotey, 2008. *Generalized Dynamic Winkler Model for Nonlinear Soil–Structure Interaction Analysis*, Canadian Geotechnical Journal 45, NRC Research Press.

Ensoft Inc., 2012, LPile, Version 6.0.28, Austin, TX. www.ensoftinc.com.

Gagnon D., P. Léger, R. Tremblay and V. Latendresse, 2010. *Pile Foundation Modeling for Seismic Analysis of Highway Bridges Located in Eastern North America*, 9th US National and 10th Canadian Conference on Earthquake Engineering.

Gazetas, G., and Dobry, R., 1984. *Simple radiation damping model for piles and footings*, Journal of Engineering Mechanics 110-6, American Society of Civil Engineers.

Geomotions LLC, 2010. SHAKE2000, Version 9.30, www.geomotions.com.

Hashash, Y., 2012. DEEPSOIL, Version 5.0., Department of Civil and Environmental Engineering, University of Illinois, Illinois.

Po Lam, I., H. Law and G.R. Martin, 2007. *Bridge Foundations: Modeling Large Pile Groups and Caissons for Seismic Design*, MCEER, Buffalo, NY.

Seismosoft SRL, 2012, Seismosignal, Version 5.0.0, Pavia, Italy. www. seismosoft.com.

Schnabel, P.B., J. Lysmer, and H.B. Seed, 1972. SHAKE, *A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites*, Report No EERC 72-12, Earthquake Engineering Research Center, University of California, Berkeley.

Sun, L and Y. Goto, 2001. *Nonlinear Analysis on Interaction of Soil-Pile Foundation Under Strong Earthquake*, The 8th East Asia-Pacific Conf. on Struct. Eng. and Construction, Singapore.

Wang, S., B.L. Kutter, J.M. Chacko, D.W. Wilson, R.W. Boulanger, and A. Abghari, 1998. *Nonlinear seismic soil-pile-structure interaction*, Earthquake Spectra 14-2, Earthquake Engineering Research Institute.