THE EFFECT OF SOIL-STRUCTURE INTERACTION ON SEISMIC ASSESSMENTS FOR BRIDGES

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ABSTRACT

The effect of soil–structure interaction on the seismic assessment of bridges is studied in this paper. Nonlinear static pushover analyses and dynamic time history analyses of single column bent bridges are performed using two separate types of finite element models. The first type model assumes that bridge columns are rigidly connected to the foundation without considering soil-structure interactions while the second type model incorporate soil-structure interactions through the use of equivalent springs. In addition, parametric study is carried out to investigate the effects of soil types and buried depth of piles on the seismic assessments for bridges. Numerical results show a markedly different seismic behaviour when the soil-structure interaction is included in such analyses, rather than simply considering a fixed support as usually done in previous studies. Furthermore, for stronger excitations, it is seen that as inelastic mechanisms are introduced and boundary conditions changed, the considerations of the foundation and soil compliance play an increasingly important role that can potentially modify the anticipated failure hierarchy, as well as the ensuing pushover curves in the transverse direction of the bridge.

Keywords: Soil–structure interaction, seismic assessment, pushover analysis, bridge.

1. INTRODUCTION

The seismic design and assessment of structures has become an essential issue for the sustainable development in recent years. A rational seismic design and assessment for a structure has to be based on the accurate prediction of its inelastic displacement demand and capacity. The displacement-based seismic evaluation and design has increasingly become the main stream for the design code of the next generation. The most accurate analytical procedure to estimate the seismic displacement demands of a structure responding in the nonlinear range is to carry out the nonlinear dynamic time-history analyses. However, since this procedure requires the consideration of a large number of earthquakes and is relatively complicated and time-consuming, the use of nonlinear static

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pushover analysis is generally considered to be a more suitable standard practice for the seismic design and evaluation of structures.

The static nonlinear pushover analysis is an analyzing procedure whereby an incremental-iterative process has been carried out to obtain the response of a structure subjected to a specified monotonically increasing lateral load pattern. However, most pushover analyses of structures have been derived based on the assumption that structures are supported on rigid foundations. This assumption could minimize the computation cost, but would result in an overestimation on the foundation stiffness. Today, most building codes adopt idealized envelope response spectra which attain constant acceleration values up to certain period (of order of 0.4 – 1.0 s at most) and then decrease monotonically with period (for example as $T^{-2/3}$). Therefore, the consideration of the soil-structure interaction (SSI) leads to smaller accelerations and stresses in the structure and thereby smaller forces onto the foundation. It is widely believed that the SSI is beneficial to the behavior of the structural system under earthquake loading (Jeremic et al. 2004).

A lot of studies have investigated the influence of the SSI on behaviors of bridges in recent years (Kappos and Sextos 2001, Silva and Manzari 2008, Kwon and Elnashai 2010). The beneficial role of SSI has been essentially turned to dogma for many structural engineers. It is worth to note that the SSI also leads to larger displacements under earthquake loading, which could violate the performance-based design requirements. The object of this paper is to identify the effect of SSI on the seismic assessment of bridges while the SSI is simulated by lump springs in finite element models. In addition, parametric studies are also carried out to discuss the influences of different soil and foundation buried depth on the magnification factor of lateral displacements. Preliminary analytical studies comparing the response of fixed-based models with simplified SSI models are expected to provide important information on the need for considering SSI effects in the design process.

2. SOIL-STRUCTURE INTERACTION

The properties of soil include friction, cohesion, dilation/contraction and buildup/dissipation of pore water pressure. In addition, the randomness of material properties of soil is much higher than that of common structural materials. To avoid the huge computational cost of a full-scale three-dimensional finite element model which consists of solid elements with nonlinear material properties to consider the effect of soil-structure interaction, we adopted a lumped nonlinear soil spring approach to simplify the computation in this paper. Furthermore, plastic hinges, instead of fiber-element, are used to simulate nonlinear deformation of the piers and piles.

2.1. Nonlinear soil spring

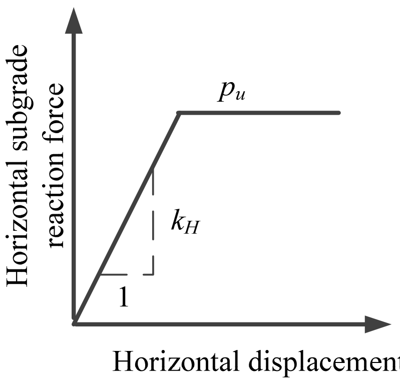
Elastic-perfect plastic Winkler-type soil springs which were attached at the pile nodes were adopted for modeling the soil stiffness. The lateral subgrade coefficient for the piles, k_H, provided by the soil was taken as:

$$k_H = 0.34(\alpha E_0)^{1.10} D^{-0.31} (EI)^{-0.103} \tag{1}$$

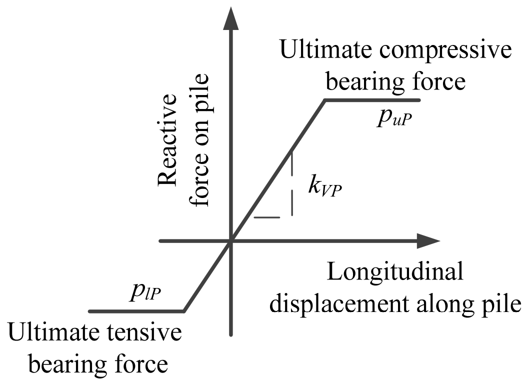
the α and E_0 (kgf/cm^2) denote the coefficient of subgrade reaction under earthquake and soil modulus of elasticity, respectively, and are equal to 2 and 25N where N is the value of the standard penetration test. In addition, D and EI are the diameter (cm) and flexural rigidity (kgf-cm^2) of the pile. The ultimate lateral subgrade reaction force per unit area is given by:

$$p_u = \phi \sigma_p \tag{2}$$

where ϕ is the modification factor and equals 0.9; σ_p is the passive earth pressure in front of piles with unit of tf/m^2. The relationship between horizontal subgrade reaction force and horizontal displacement is shown in Fig. 1(a).



(a) Horizontal soil spring



(b) Vertical soil spring

Figure 1: The nonlinear behaviors of the soil springs

Similarly, the vertical soil model surrounding pile caps is also simulated as springs with stiffness which was computed according to the expression

$$k_{VP} = aA_p E_p / L \tag{3}$$

where A_p and E_p are the cross section area (cm^2) and modulus of elasticity (kgf/cm^2) of the pile cap, respectively. Moreover, L is the pile length and the constant a is equal to 0.031(L/D)-0.15 for case-in-place piles. The ultimate compressive bearing force p_{uP} and tensile bearing force p_{lP} for pile are obtained, respectively, from Eqs. (4a) and (4b):

$$p_{uP} = 0.8(f_s A_s + q_b A_b) \tag{4a}$$

$$p_{lP} = 0.5(f_s A_s + q_b A_b) \tag{4b}$$

where f_s is friction resistance on the pile surface, A_s is the surface area of the pile, q_b is the ultimate bearing pressure of the pile, and A_b is the cross section area of the pile. The relationship between vertical resistance force and vertical displacement of the pile is shown in Fig. 1(a).

2.2. Plastic hinges

Apart from soil flexibility, nonlinear deformation of piers and piles is also considered in terms of the potential plastic hinge development. For piers, the plastic hinges are assumed to be located near the bottom of piers, as shown in Fig. 2(a). The moment-rotational angle curve of the plastic hinge is determined by the envelope of the two different moment-rotational angle curves obtained by considering flexural and shears failure criteria. The equivalent plastic hinge length, L_p, of piers is defined by (Priestley *et al*. 1996):

$$L_p = 0.08L + 0.022 f_{yl} d_{bl} \geq 0.044 f_{yl} d_{bl} \tag{5}$$

where L is the pile length, f_{yl} is the yield strength of the longitudinal bars, and d_{bl} represents the diameter of the longitudinal reinforcement.

On the other hand, when the plastic hinge method is used in the nonlinear analyses of a pile-soil system, difficulties arise from the impossibility of predetermining the location of the plastic zone. In this paper, the distributed plastic hinge model was adopted along an expected plastic zone of a pile (Chiou *et al*. 2009). The tributary length l_{dp} of a plastic hinges is regarded as the plastic hinge length as shown in Fig. 2(b). The yielding plastic hinges define an actual plastic zone. While the definition of plastic curvature in ATC-40 is modified to

$$\phi_{pm} = \phi - \frac{M}{EI_e} \tag{6}$$

where ϕ and M/EI_e represent the total curvature and the elastic curvature, respectively.

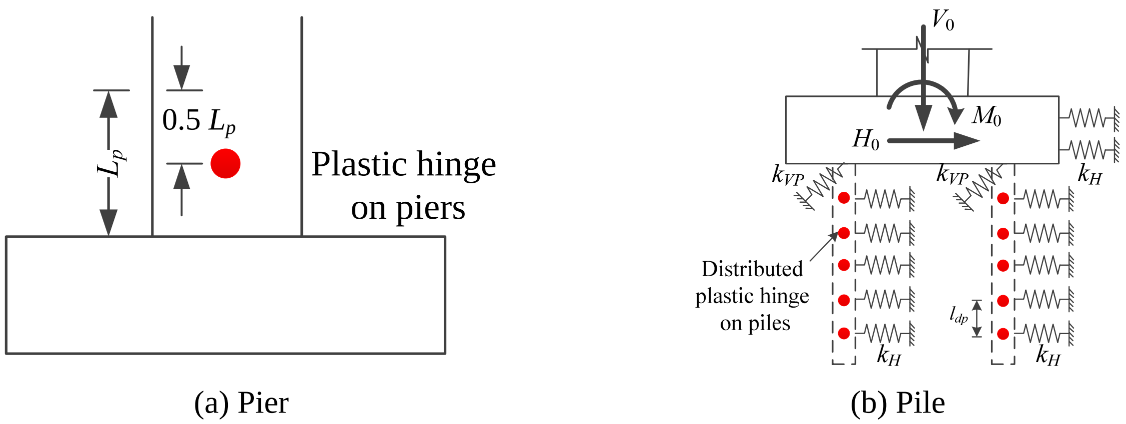


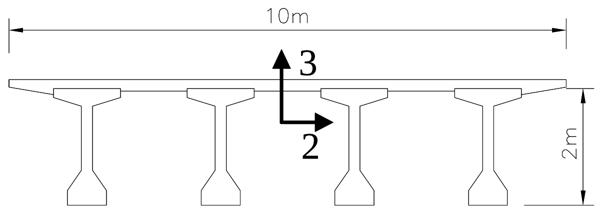
Figure 2: The plastic hinges on RC members

3. NUMERICAL MODLES

Nonlinear dynamic time history analyses and nonlinear static analyses (i.e., pushover) were performed on a continuous bridge with two different boundary conditions of the foundation. The first model assumes the bridge columns to be rigidly connected to the foundation without consideration of the SSI. The second model incorporates SSI using nonlinear soil springs attached on the pile foundation. The soil conditions considered in this paper is assumed to be consisted of uniform deposit sand for simplicity.

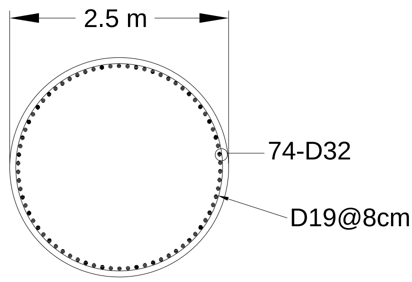
3.1. Description of structure

The selected system is a four-span continuous bridge with equal pier height of 10 m. The length of each span is 40 m and the total length is 160 m. The superstructure of the reference bridge, shown in Fig. 3(a), consists of a concrete deck on top of four continuous steel girders and the superstructure mass was 15 ton/m all throughout. The substructure consists of five piers supported by RC pile foundations. Each pier has six piles which length is 40 m. The bearing systems are pin supports at the intermediate piers P2, P3 and P4, and roller supports at expansion joints P1 and P5. All piers are circular RC columns with a diameter of 2.5 m. Also, all piers have the same reinforcement details of 74-D32 longitudinal reinforcing bars as shown in Fig. 3(b), and are transversely reinforced with 19 mm bar hoop spaced at 8 cm. In the first model, the bridge columns are rigidly connected to the foundation as shown in Fig 4(a). On the other hand, each column of the bridge is supported by six piles which are circular RC columns with a diameter of 0.9 m as shown in Fig. 4(b). All piles have the same reinforcement details of 16-D25 longitudinal reinforcing bars and 13 mm bar hoop spaced at 30 cm for transversely reinforcement.



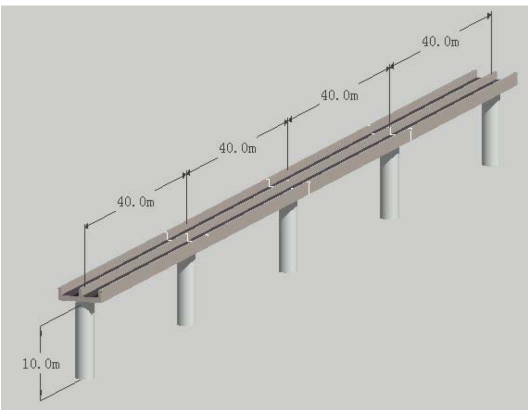
A = 5.27 m^2 ; I_{22} = 3.1394m^4 ; I_{33} = 39.0233m^4
E = 2.49×10^7 kN/m^2 ; m = 15 ton/m

(a) Superstructure

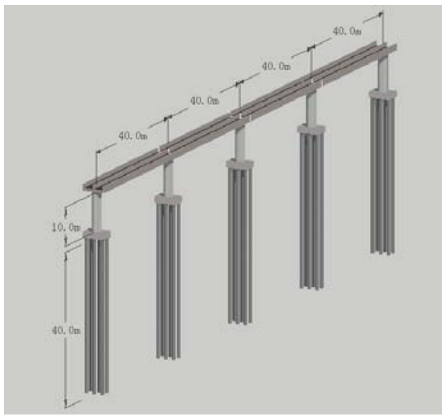


(b) Substructure

Figure 3: The cross section of the reference bridge.



(a) Model 1 (without piles)



(b) Model 2 (with piles)

Figure 4: Two finite element models of the considered bridge.

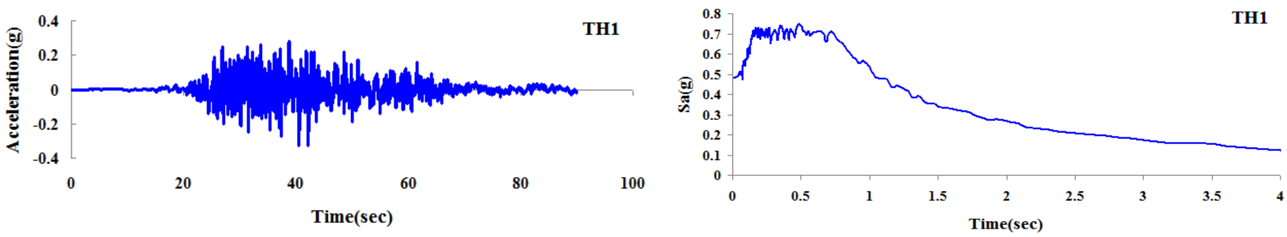
The stiffness of equivalent horizontal and vertical soil springs can be determined using empirical formula with values of the standard penetration tests. In this paper, three different combinations of N values and pile depth are selected to investigate the effects of soil types and buried depth of piles on the seismic responses for the considered bridges. The corresponding fundamental periods of four models are listed in Table 1. From Table 1, one can observe that the fundamental period of the bridge with soil-structure interaction is almost twice of that of the bridge with fixed support. In addition, the fundamental period of the considered bridge decreases with increasing the pile length or the value of the standard penetration test. However, the effects of soil types and pile length on the fundamental period are not obvious.

Table 1: Fundamental period for different finite element modes

Model no.	SPT- N	Pile length (m)	Fundamental period (sec)
Model 1	—	—	0.520
Model 2A	8	40	1.056
Model 2B	15	20	1.061
Model 2C	15	40	1.035

3.2. Nonlinear analyses

The nonlinear static pushover analyses and nonlinear dynamic time history analyses were carried out using the commercial finite element software, SAP 2000N (CSI 2012). Superstructures were simulated by elastic beam elements, and the nonlinear behaviors of piers and piles were modeled by M3 hinge. The fundamental mode shapes in the lateral direction of these finite element models were adopted as the lateral load patterns in static pushover analyses. Since the results of pushover analyses depend on the choice of the monitored point, the concept of system displacement and system mass (Kowalsky 2002) was recommended in the current study to calculate the equivalent capacity curve. In addition, the design based earthquake (DBE with S_{DS} = 0.7 and S_{D1}= 0.52) and maximum possible earthquake (MPE with S_{DS} = 0.9 and S_{D1}= 0.55) are considered in this study to take the effects of earthquake magnitude into account. The results of nonlinear dynamic time history analyses are the average displacements at deck nodes obtained by applying five code-compatible artificial earthquakes on these bridges. A typical time history and the corresponding elastic spectrum of a code-compatible artificial DBE are shown in Fig. 5.



(a) Time history (b) Elastic spectrum

Figure 5: Typical time history and the corresponding elastic spectrum of DBE.

4. RESULTS AND DISCUSSIONS

Figures 6 demonstrated that the pushover curves in the lateral direction form these four models. One can observe that the fixed model (Model 1) has more stiffness than spring models (Model 2A, 2B, and 2C) in the linear range. For bridge with fixed base, the pier response becomes nonlinear early in the response while the yielding taking place with the lateral displacement approaching 0.09 m. On the other hand, for bridges with equivalent soil-structure springs, the pier response is essentially elastic till 0.18 m of lateral displacement. Moreover, the pushover curves of three different combinations of soil types and pile length almost coincide with each other, which implies that the soil types and pile length have little influence on the pushover curves.

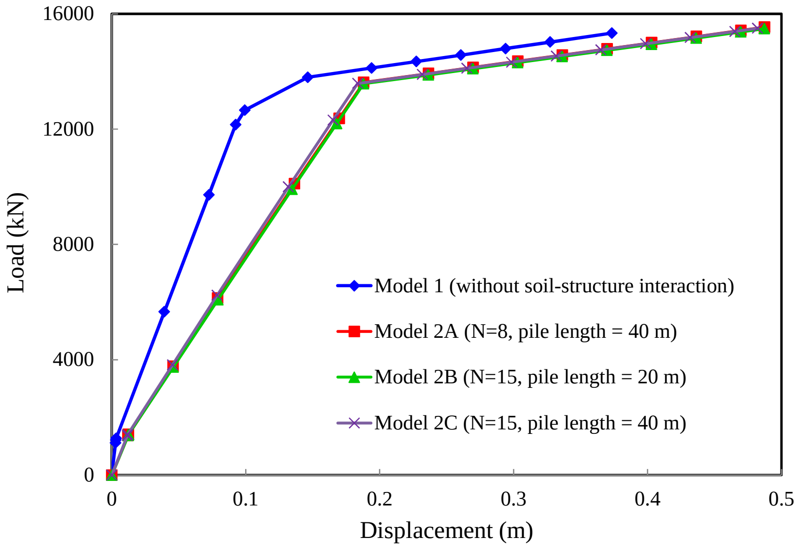
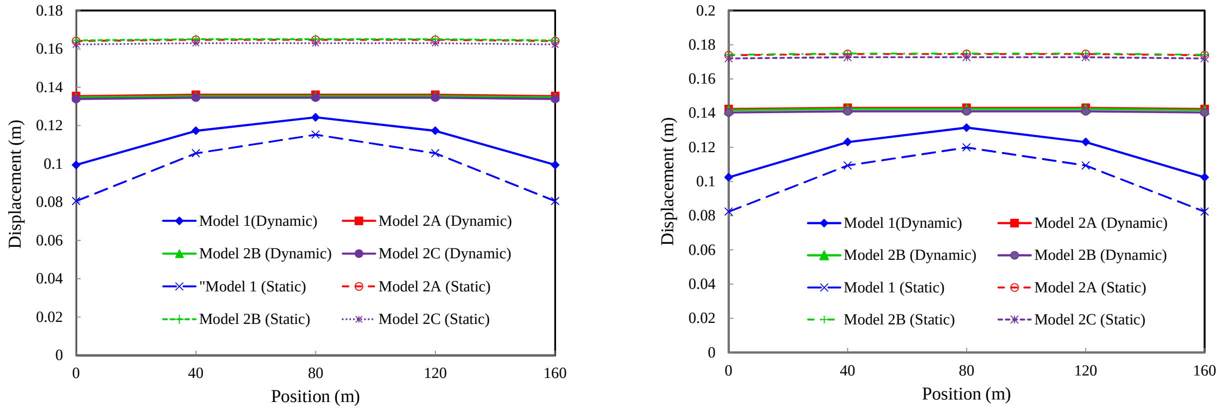


Figure 6: Pushover curves for different finite element models.

Following the procedure of seismic assessment, the capacity spectra developed for different finite element models and the performance points corresponding to the design-based earthquake and maximum possible earthquake can be obtained. Based on the spectral displacement and spectral acceleration at the performance point, the lateral displacements at deck's level under the design-based earthquake and maximum possible earthquake can be obtained for the multiple monitored points, respectively. The calculated transverse displacement profiles from different models are given in Fig. 7. In order to realize the performance of nonlinear static pushover analyses, inelastic dynamic analyses were also performed and the corresponding results were shown in Fig. 7. Considering the average of results of inelastic dynamic analyses as reference values, the transverse displacements from static pushover analyses are smaller than those from dynamic analyses for the fixed base model. However, the transverse displacements from static pushover analyses are larger than those from dynamic analyses for the soil spring models. These trends can be observed both for the design-base earthquake and maximum possible earthquake. These results imply that the static pushover analyses for fixed base models may underestimate the maximum transverse displacements and result in non-conservative designs. On the other hand, the static pushover analyses for models with soil-structure interaction may overestimate the maximum transverse displacements and result in conservative designs.



(a) Design-based earthquake (b) Maximum possible earthquake

Figure 7: Transverse displacement distribution on the desk of the considered bridge.

5. CONCLUSIONS

In this paper, the influence of SSI on a typical regular continuous bridge is evaluated. It is shown that the SSI leads to smaller accelerations and stresses in the structure. For bridge with fixed base, the pier response becomes nonlinear early in the response. In addition, the soil types and pile length have little influence on the pushover curves. However, the transverse displacement would be magnified which could violate the performance-based design requirements. The magnification factors are almost the same for design-based earthquake and maximum possible earthquake. Finally, the static pushover analyses for models with soil-structure interaction may overestimate the maximum transverse displacements and result in conservative designs.

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