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SOIL-STRUCTURE INTERACTION OF SEISMIC ISOLATED BRIDGES

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ABSTRACT: The impact of the Soil-Structure Interaction (SSI) on seismic isolated bridges is investigated. Two stick models for the two seismic isolated bridges of interest are considered and equivalent models of the frequency-dependent impedance functions of the soil and foundation are introduced, with the new elements known as "gyromasses" being involved. Their importance is discussed.

KEY WORDS: Bridges; Earthquakes; Gyromass; Isolation; SSI effects

1 INTRODUCTION

Many design codes state that the SSI effects may be safely ignored during the design process of heavy structures. The myth of the SSI effects being safely neglected stems from the perception that the phenomenon makes the structural system more flexible when subjected to an earthquake and hence it reduces the overall seismic loading. The aforementioned statements have been examined before by many researchers, after considering quite representative models for soil, foundations and superstructures. Spyrakos[1] used simple linear elastic models and concluded that the soil-structure interaction effects make structures more flexible and less seismically affected. In another study, Mylonakis and Gazetas [2] used another simplified elastoplastic model for a bridge and its foundation, which was subjected to a set of actual acceleration time histories recorded on soft soil. Though the lengthening of the period made the structure more flexible, the SSI phenomenon played detrimental role on the seismic performance of the bridge. In fact, damage in structures associated with SSI effects has been proven or suspected in many cases in the past. For instance, the Mexico City earthquake of 1985 was particularly destructive to 10 to 12-story buildings (founded on soft clay) whose period increased from about 1.0 sec (for the fixed-base structure) to nearly 2.0 seconds due to SSI [3]. Other evidence for a detrimental role of SSI has been presented by Meymand [4] and Celebi [5].

2 SOIL- FOUNDATION-BRIDGE SYSTEM MODELING

2.1 Bridge Systems

In this study the role of the soil-structure interaction is investigated on seismic isolated bridges. Two bridge structures are considered: the first (Bridge I) is representative of a typical highway overcrossing with a stiff short pier, while the second one (Bridge II) could be part of a long multispan bridge with flexible tall piers. Figure 1 depicts the geometric characteristics of each bridge. The dynamic impedances of the 5x5 pile groups for both bridges are presented later. The superstructures were chosen to be modeled with the help of the so-called "stick model", as simple and approximate solutions are desired.

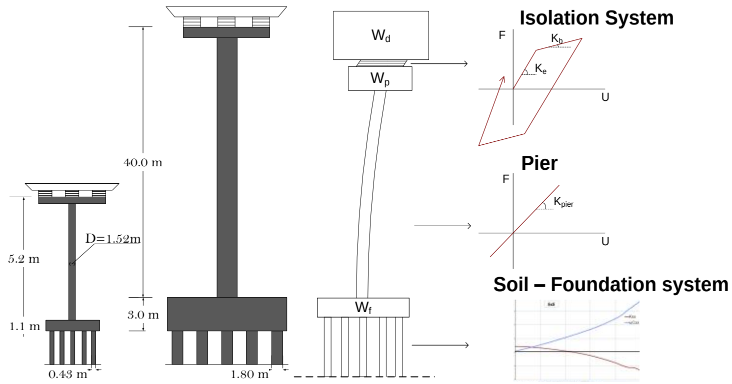

Figure 1. Geometrical representation of Bridge I and Bridge II.

The bridge models consist of a single linear pier (no yield considered), at the top of which the bilinear isolation system carries the deck's weight. At the bottom, the pier is monolithically connected to the pile group cap. The mass of the deck, the pier and the foundation are considered concentrated..

The isolation system is located between the deck and the pier and considered to behave as a bilinear hysteretic spring with smooth elastic to post yielding transition. Such a behavior could be representative of typical lead rubber bearings, as well as of sliding bearings with restoring force capability. Its nonlinear hysteretic behavior was modeled using Ozdemir's model [6]. The variables controlling the systems behavior are the yield strength (F_y), the elastic stiffness (K_e) and the post-yielding stiffness (K_b). There are design philosophies which call for large K_b stiffness, in order to limit the displacement response and minimize potential permanent displacements, while others call for low so as to protect the bridge piers from large shear forces. The values used in this study are presented in Table 1.

Table 1. Properties of the bridge models considered.

Bridge model	Bridge I	Bridge II
Deck seismic weight, m_d (Mg)	265	1440
Isolation system period, T_b (sec)	2	4.5
Isolation strength ratio (F_y / W_d)	0.12	0.04
Pier seismic weight, m_p (Mg)	38.5	620
Pier weight/ Deck weight	0.15	0.43
Pier height, h (m)	5.2	40
Pier elastic stiffness, k_p (kN/m)	1.24E5	1.09E5
Pier damping ratio, ξ	5%	5%
Foundation seismic weight, m_f (Mg)	84	4248
Foundation moment of inertia, I_f (Mg m^2)	173	126200
Pile cup height, H_f (m)	1.1	3

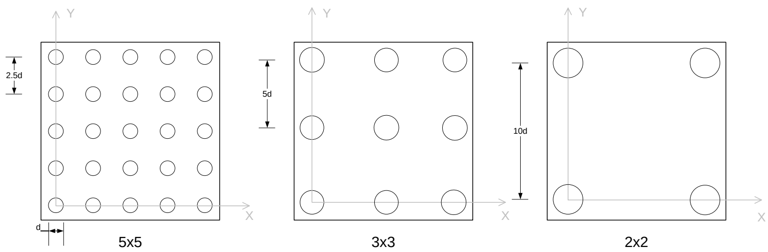

Figure 2. Geometry of 5x5 and equivalent 3x3 and 2x2 pile groups (for both bridges).

Table 2. Properties of the 3 equivalent pile groups of study.

	Bridge I			Bridge II		
Pile Group Label	**5x5**	**3x3**	**2x2**	**5x5**	**3x3**	**2x2**
Number of piles, N	25	9	4	25	9	4
Diameter, d (m)	0.43	0.7	1	1.8	3	4.5
Length, L (m)	21.5	21.5	21.5	21.5	21.5	21.5
Distance, S, from pile to pile (m)	1.08	3.5	10	4.5	9	18
Mass Density, ρ_p (kg/m3)	2500	2500	2500	2500	2500	2500
Modulus of Elasticity, E_p (GPa)	18.5	18.5	18.5	18.5	18.5	18.5
L/d	50	31	21	12	7	5

The pile caps are supported by a 5x5 pile group, with pile spacing (S) 2.5 times the pile diameter (d) (see Figure 2). The pile diameters where considered different for the two bridges, due to the different dimensions of the superstructures (d_1= 0.43 m, d_2 = 1.80 m). The models are also analyzed with the equivalent 2x2 and 3x3 pile groups, with the diameter of each pile in the groups to be adjusted in such a way as the total area of the pile groups (m^2) to be the same in all three cases.

2.2 Dynamic impedances of pile groups

Under lateral loading, the impedances of the foundation are related to: bending (Kxx), rocking (Krr) and coupled bending-rocking effects (Kxr). It is preferable to express impedances as:

$$Kxx = K_{xx} + i\omega C_{xx} \tag{1}$$

where, K_{xx} is the "spring" coefficient modeling the soil and the foundation, C_{xx} is the "dashpot" coefficient, ω is the frequency of the harmonic input (rad/sec) and $i=(-1)^{1/2}$. In this study, for the estimation of the dynamic impedances of pile groups, the boundary element program **PILES** [7] was utilized. This software uses the elastodynamic method which is based on a frequency domain solution of the closed-form Green's function for both the soil and the piles. The soil used in this study is assumed to be a linear, homogeneous half-space, with mass density ρ_s= 1800 kg/m^3, shear wave velocity V_s= 110 m/sec, damping ration ξ=10% and Poisson's ratio ν=0.40 (E_p/E_s=300). The considered value for the shear wave velocity is rather low for typical elastic soil properties. However, it is chosen here in to represent inelastic the soil properties during strong soil motion.

Figure 3 presents the pile group dynamic stiffnesses K_{xx} for lateral and rocking motions as a function of dimensionless wave parameter $a_o=\omega d/V_s$. For a range of excitation's periods T= 0.25-2 secs and a mean value of shear wave velocity V_s=100 m/sec and diameter d=1 m, the a_o parameter takes the values between 0 and 0.25. This study focuses to values up to 1, which is considered as an upper bound for the values of interest. Same results are obtained for rocking impedance and for Bridge II.

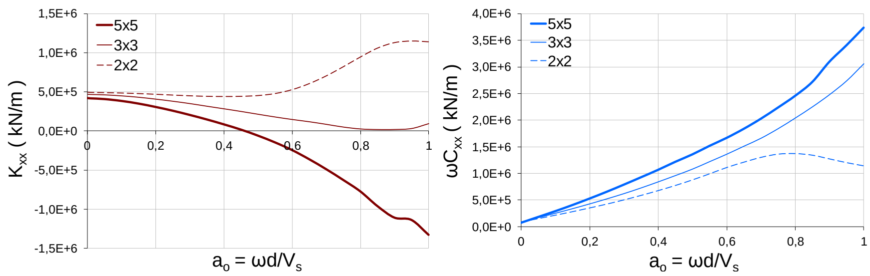

Figure 3. Comparison of the stiffnesses, Kxx(ω) and dampings, ωCxx(ω), of 5x5, 3x3 and 2x2 pile groups, Bridge I.

The inherent nonlinear behavior of the isolation system, does not allow the use of frequency domain analysis techniques to treat the bridge-foundation-soil system. In order to overcome this incompatibility, it is customary to introduce an approximation omitting the frequency dependency of the soil-foundation system considering that the springs and dashpots have constant, frequency–independent, values corresponding to the values that the impedances take for a_o=0. The simple **Voigt model**, consisting of a spring and a dashpot, connected to the foundation mass, is a simple option for modeling the impedance functions, under the aforementioned simplifications. For the capturing the frequency dependent behavior of the soil-foundation system, Saitoh [8] presented a new model consisting of a system of basic mechanical elements (springs and dashpots) together with an element named “gyromass” capable of representing frequency dependent impedance functions while eliminating the shortcomings of the models introduced by De Barros and Luco [9] and Wolf and Somani [10].

The **gyromass** is a mechanical element which has the same dimensions as mass. It is defined as a frequency- independent unit generating a reaction force due to the relative acceleration of the nodes between which the gyromass is placed (Figure 4), but adding no inertial forces. The model introduced by Saitoh containing springs, dashpots, and gyromasses to achieve better fitting of the dynamic impedances in the frequency domain is the **Type II model** (Figure 4). It consists of one *base system*, where the spring-dashpot unit and the gyromass are connected in parallel and two *core systems* where the spring-dashpot unit and the gyromass are connected in series.

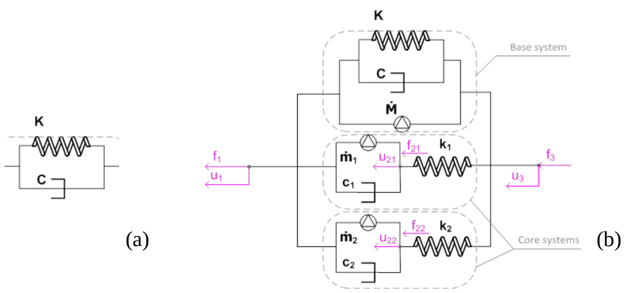

Figure 4. a) Voigt model and b) Saitoh’s Type II model.

With the help of the equilibrium equation of Type II system and the mathematical method of Levenberg Marquadt for achieving an accurate curve fitting, the dynamic impedances are calibrated for each case of soil and foundation and both lateral and rocking components. The values K, C, $\dot{M}$, $\dot{m}_1$, c_1, k_1, $\dot{m}_2$, c_2 and k_2 are now known and the most important, frequency-independent. The time history analysis is performed with 40 appropriate seismic excitations, categorized in two groups: the near fault set of motions [11] and the far field set of motions [12], [13].

3 ANALYSES RESULTS AND DISCUSSION

Non-linear time history analyses of the two bridge models (Bridge I and Bridge II) with different foundations (2x2, 3x3, and 5x5 pile groups) utilizing two different soil-structure interactions models (Voigt, Type II for translation and rotation) subjected to both Far Field (FF) and Near Fault (NF) sets of seismic excitations are performed In the present study the results of the parametric analyses are presented in terms of ratios as isolation drift ratio (IDR) and pier shear ratio (PSR). Thus IDR and PSR are defined as follows:

$$IDR = \frac{\text{Isolation drift}_{\text{TYPE II}}}{\text{Isolation drift}_{\text{VOIGT}}} \tag{2}$$

$$PSR = \frac{\text{Pier shear}_{\text{TYPE II}}}{\text{Pier shear}_{\text{VOIGT}}} \tag{3}$$

Figure 5 summarizes the results of the largest discrepancies between the Voigt and Type II models, which correspond to Bridge II excited by the far field set of motions. The differences in the isolation drift range between -10% and +10% (-2% on the average over all motions. The PSR shows larger differences than IDR: -16% maximum and -10% on the average over all seismic motions. This indicates that using a more accurate SSI model (Type II) than the simple Voigt the shear forces in the pier are on the average 10% smaller over this FF motion set. PSR is sensitive to the pile groups with differences between them ranging from 2% (motion #3) to 10% (motion #9).

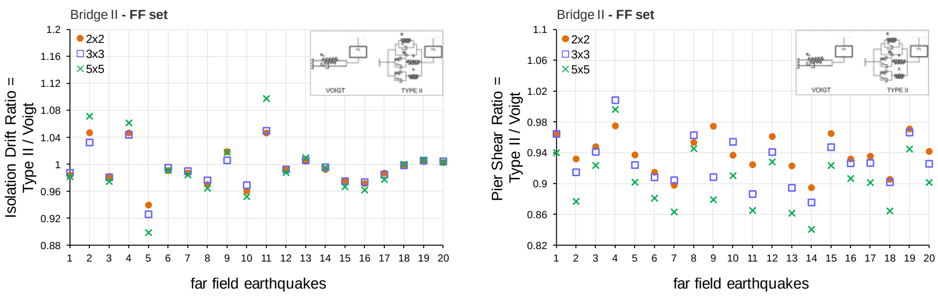

Figure 5. IDR and PSR for Bridge II, E_p/E_s=300 (FF set).

3.1 Effect of E_p/E_s

Considering the results presented in the previous figures, another case of analysis is introduced. The model of Bridge II, founded on a 2x2 pile group with pile diameter d=1.8 m and S/d=10, resting on a much softer soil, so as the value of E_p/E_s to be 1000. The soil of interest now is a linear, homogeneous halfspace, with mass density ρ_s= 1800 kg/m^3, shear wave velocity V_s= 63 m/sec, damping ration ξ=10% and Poisson's ratio ν=0.40. This low value of the shear wave velocity V_s represents the case of soil behaving well into the inelastic range where strong softening behavior in the soil is predominant.

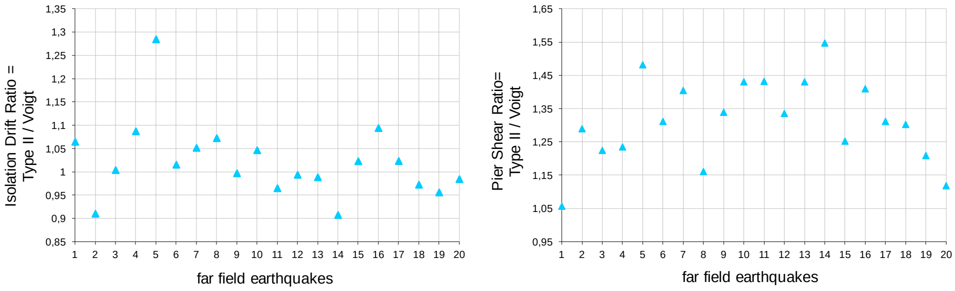

Figure 6. IDR and PSR for Bridge II, E_p/E_s=1000 (FF set).

In Figure 6 the ISR and PSR appear to be much more sensitive to the model used for the SSI effects. There is one excitation where the isolation system reaches 28% larger displacements for Type II (motion #5), while for the rest of the motions the differences on the isolation drift are between +10% and -10%. There are also motions that develop up to 45% larger pier shear forces when Type II model is utilized.

4 CONCLUSIONS

The most important conclusions of this study are:

- Generally, in cases of stiff foundation resting on linear homogeneous soil based on halfspace, the SSI can be satisfactorily modeled with simple Voigt systems. The discrepancies between the Type II (gyromasses) and Voigt models are up to 10-20% for both isolation displacements and pier shear forces.
- A system, with flexible pile group resting on a very soft soil can lead to great loss of accuracy in case Voigt models are used. The differences observed are up to 50% for both isolation displacements and pier shear forces, with the Type II model outweighing the Voigt. This case of such a small value of shear wave velocity could be a simplified approach of the nonlinear behavior of the soil, which is the actual one.

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